INVESTIGATION OF THE FEASIBILITY OF AN INLAND HARBOR FOR BEIRUT- LEBANON

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by

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PREFACE

In preparing this thesis the aim of the author has been to treat in systematic manner the design of an Inland Harbor for Beirut. However, he had to resort to a combined harbor with piers projecting into the sea for reasons mentioned in the design.

With regard to the economy of this project, the author found out that such a study will not be reliable because many data are lacking, such as : the nature of the soil, the contours of the area, the procedure of construction, and the machinery to be used. Nevertheless, his conclusion is that this area studied has the potentialities of becoming a modern harbor with reasonable investment of capital due to the following reasons :

a. Excavated earth will be used for regaining lands from the sea.

b. Cost of appropriated lots is relatively cheap.

c. Location of the area is central and affords efficient planning.

d. Possibilities of future extension and development can be planned before hand.

The author wishes to express his thanks to his supervisor, Professor Nicola Manassah, for his helpful comments and suggestions and for his kind assistance in reading most of the manuscript. Also the author's thanks are due to all the personnel of the Civil Engineering Faculty and to Mr. Dregoir, of the Beirut Port, for their kind suggestions.

CONTENTS

INTRODUCTION

The origin and development of harbor engineering depend from their nature on maritime engineering, as cause and effect. That is, with the introduction of vessels demand was created for shelter during stormy weather, and for suitable and safe areas to charge and discharge cargo. Thus, inherently, with their primitive vessels put at work, people, in the past, searched incessantly for natural shelters. Throughout the world such places were scarce, yet, they fulfilled the needs of those generations and afforded the Basis on which harbor engineering had its foundations.

Beirut harbor, as many other harbors all over the world, had its origin as a natural shelter for local boats. However, after the first world war, the old harbor had to develop to what it is now in order to meet the rapid growth of the international trade which is carried mainly by maritime transport.

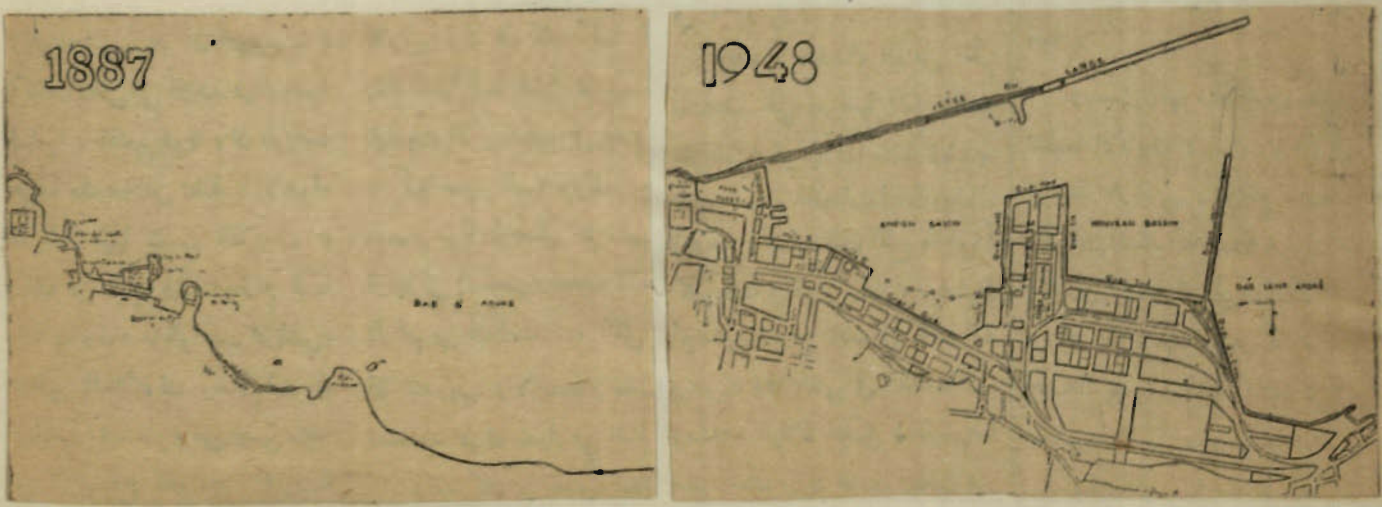

Fig. 1. Ancient and modern harbor of Beirut.

Nevertheless, this increasing demand for growth is impaired at the present time by the encroaching city from all sides; and if any attempt is to be made for a wider expansion of the harbor area, it would mean great expenses should be paid in return for small areas appropriated.

Hence, the idea of having a new harbor which meets the modern requirements in all respects, was open to investigation and awaiting study.

In this attempt of investigating the feasibility of an inland harbor for Beirut, the author wishes that some light is cast on the subject matter to make it worthy of further investigation by parties interested.

CHAPTER I

GENERAL CONSIDERATIONS

1. SYSTEMATIC TREATMENT. In most cases of harbor design, it was found out that experience afforded good and successfull solutions. However, it is not pretended that no case will arise where the most ripe experience may prove to be at fault due to the peculiar characteristics of that place differentiating it from other places. Thus, the author, relying on the established experience and on general requirements, will attempt to tackle this project.

2. DEFINITIONS. Before proceeding further, it is advisable to define some terms which will be used frequently:

Bonded Warehouse:	A warehouse in which goods liable to customs or excise duties may be stored without the duty being paid subject to supervision by revenue officers and to security being given for eventual payment of duty if the goods are not exported.
Dock:	An artificial basin for the construction or repair of ships.
Harbor:	In the past, harbor meant a place for rest and refuge; at present it means a place for rest, refuge and the reception of cargo.

Pier: A structure built out into the water with piles, for use as a landing place.

Quay: A solid artificial structure built into the water for use as a landing place.

Shed: A structure built on a pier or quay for temporary storage of goods.

Waterway: A navigable channel or body of water between two piers, quays or wharves.

Wharf or Bulk-head: A structure built on the shore of a harbor or river so that ships may lie alongside to receive and discharge cargo, passengers etc.

3. REQUIREMENTS OF A MODERN HARBOR. A modern harbor should have the following qualities; if it is to fulfil the functions mentioned in the definition:

a. Ready accessibility, which means that the harbor should have an easily reached location and a good disposition. The entrance should be neither wide nor narrow and conveniently placed to allow for easy access.

b. Safe anchorage against hazards of nature.

c. Facilities for obtaining supplies and for executing repairs in special docks.

d. Presence of piers or quays, sheds andwarehouses for easy transactions of trade.

e. Presence of railroads on the piers, railway yard near the harbor area, and wide distributing streets.

Such harbors may be found in a variety of situations; such as, upon the sea coast, or at the mouth and along the banks of a river. Thus, our investigation of the feasibility of and inland harbor for

Beirut, should show whether these requirements can be satisfied economically.

CHAPTER II

DESIGN OF THE HARBOR

4. WATER-WAY. The width of the waterway is a function of the breadths of vessels and is governed by the following specification: "Where vessels tie up against bulkheads, built parallel with the stream or the shore line, allowance must be made for the vessel to tie against the bulkhead and possibly to accomodate a line of river or canal craft both inside and outside the vessel when so moored. Room should also be available for vessels to pass with ample clearance in the remainder of the waterway. This means that between bulkhead structures a width of about eight times that of the normal vessel should be maintained as the total space available.[1]

Considering carefully this requirement and noting that it will never be expected to have canal crafts in the Beirut River, the author concluded that a width of a waterway equal to five times the normal breadth of a vessel will be ample enough to accomodate two vessels tied against bulk-heads with a third one moving in or out. Thus, this specification leads to the necessity of finding out the normal breadth of a vessel in order to calculate the required width of the waterway.

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(1) J. Nolen City Planning pg. 231

Diagram I, shows the relative growth of vessels up to the year 1911. This and the two following diagrams were plotted by Mr. J.F. King and presented in the XII International Navigation Congress in 1912. It is seen from the diagram that Mr. King divided the growth of cargo vessels into three different classes:

1. Line of progress for extreme cases
2. Line of progress for the main body
3. Line of progress of the lower limit

However, according to the specifications we have to take the "normal width" of vessels and thus we shall take the line corresponding to the "Main body". The attention of the reader is called to the fact that cargo vessels are considered because they constitute the majority of vessels which would be received in such a harbor.

If interpretation for the breadth of vessels be made for fifty years from now, it will be seen that it will amount to:

$$\left[(60-30)/(1910-1860)\right] 90+60 = 114 \text{ ft.},$$

which is not reasonable because, if actual consideration be taken of the growth of modern vessels, it will be noted that development had the following aims:

a. To make light driving machines for vessels with a great efficiency.

b. To have ample access to light and air to most parts of the vessel.

c. To have a great velocity in travel and in loading and unloading ships.

d. To reduce the number of crews to a minimum, as they constitute an expensive element in keeping up the vessel.

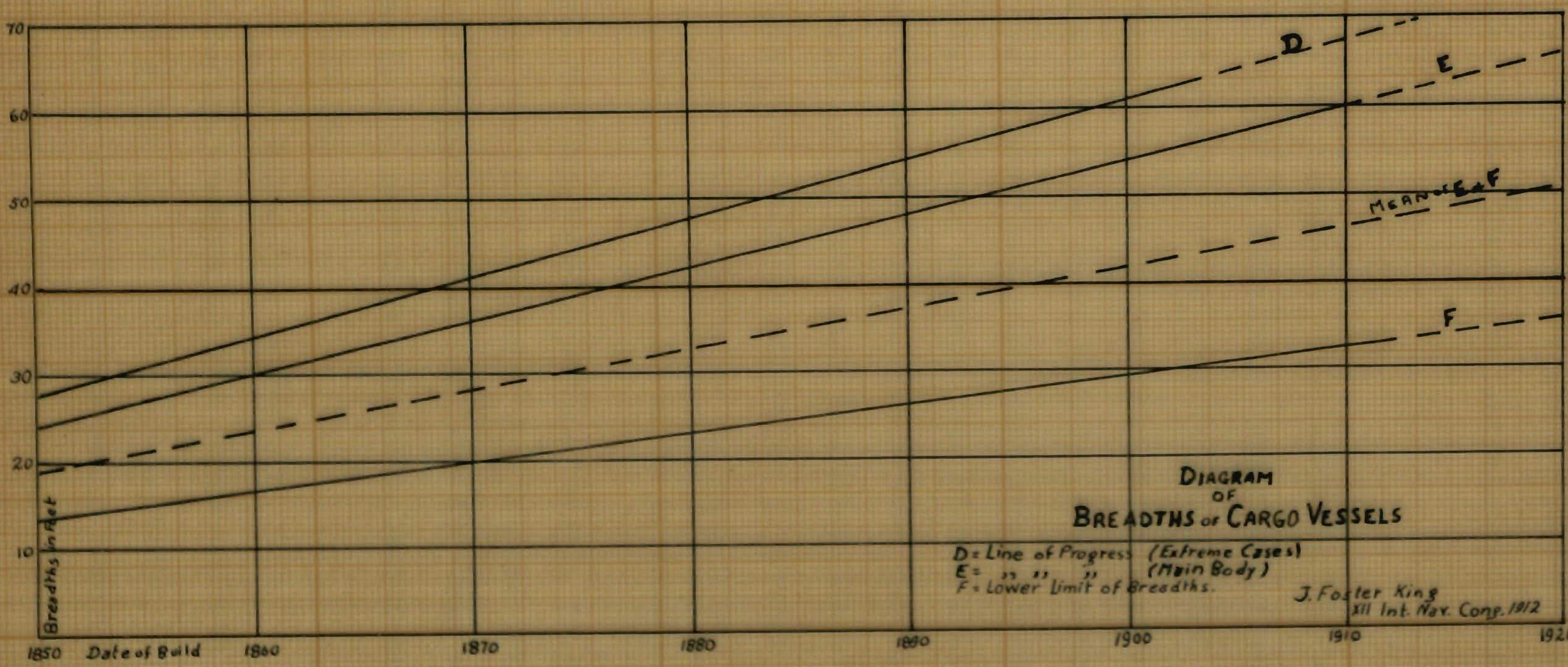

Diagram I

Bredths of Cargo Vessels

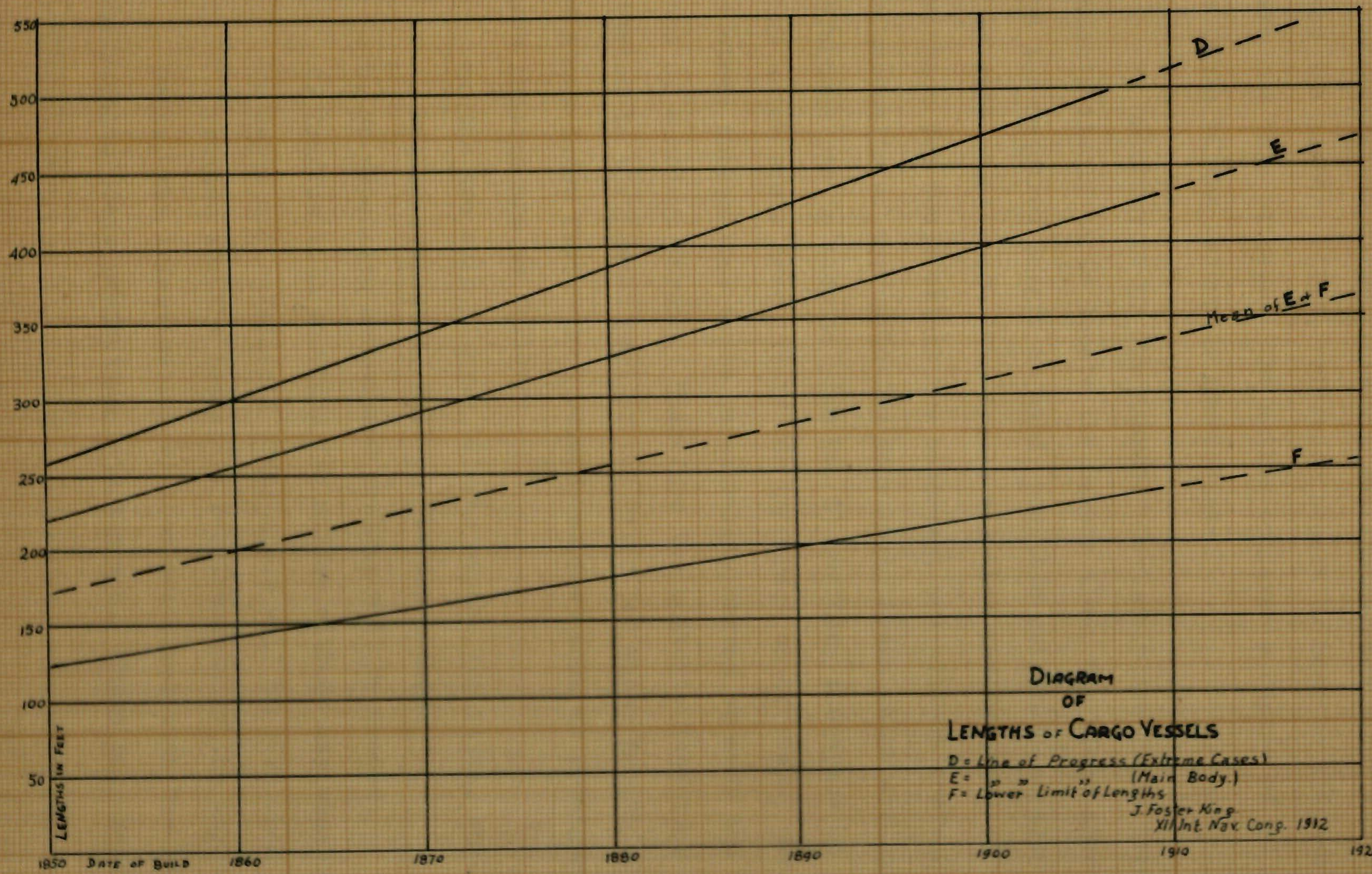

DIAGRAM II LENGTHS OF CARGO VESSELS

6. DEPTH OF THE WATERWAY. The depth of the waterway should be sufficient to allow flotation for the largest ships expected to enter the harbor. Here, reference should be made to diagram III drawn for this purpose. The author believes a choice of 40 feet as the depth of the harbor is reasonable.

7. ORIENTATION OF THE HARBOR. Obviously the first trial which presents itself is to locate the harbor along the river stream. With the required length at hand, possible orientations along the river were considered. The result was that irrespective in which direction you would orient the waterway, you will be confronted with the necessity of appropriating many built areas which are expensive. (Reference can be made to plan No. 1) Besides, the space on both banks which would lend itself for the erection of sheds, warehouses and other necessities would be extremely narrow to meet the requirements of a modern harbor. Therefore the author's attention was diverted to the lowest part of the river bed whose length approximated about 1,500 feet. Good reasons support such an arrangement. These reasons are as follows:

a. The two main highway bridges across the river, which are highly evaluated, will not be demolished.

b. No expensive suspension bridges will be needed.

c. Traffic will continue its motion without any obstruction.

e. Ample and relatively cheap areas will be furnished to provide for the necessary requirements of the harbor.

Hence, the investigation will be continued with the idea of constructing piers projecting into the sea. These will be designed in chapter III.

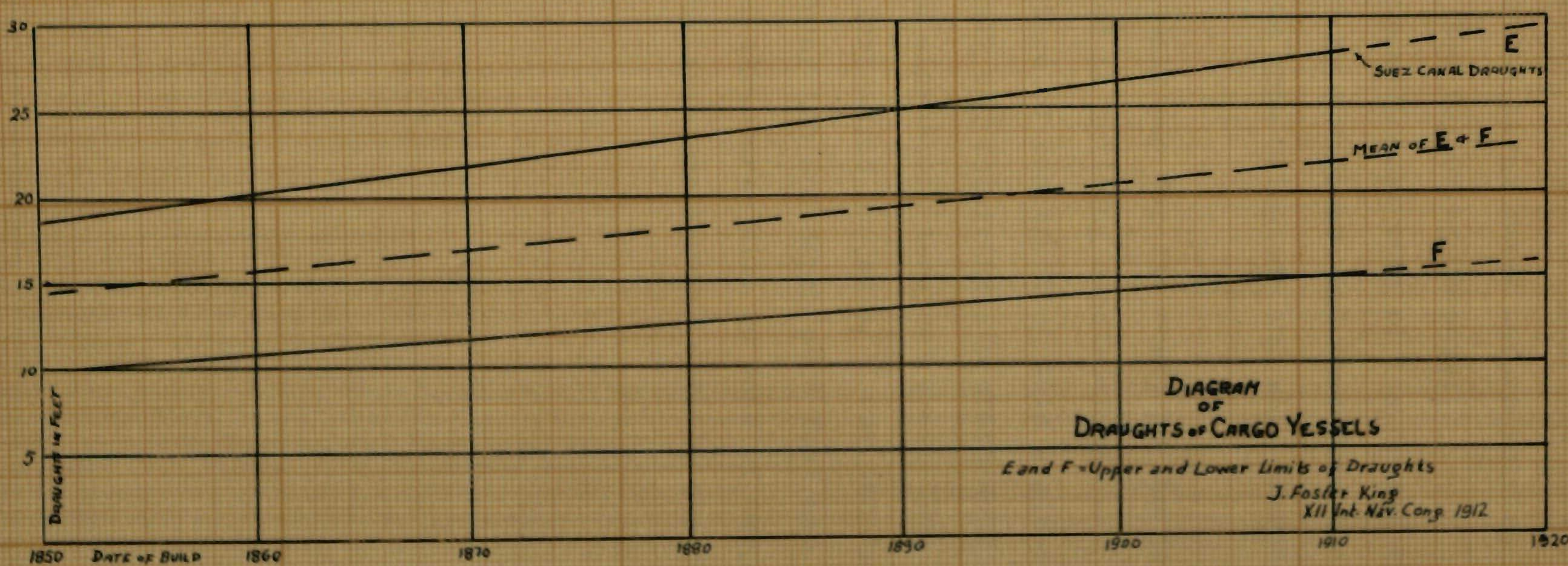

DIAGRAM III

DRAUGHTS OF CARGO VESSELS

8. SETTLING BASIN. In this project, the idea of constructing a settling basin for the river water before pouring into the harbor, presents itself for the following reasons:

a. Such a basin will take care of the coarse and sandy materials carried by the river, and therefore the waterway will be free from such nuisances and will scarcely need any dredging.

b. This basin will pay for its cost, as the deposited materials will be used in building construction.

A suitable method of constructing this basin is by depressing the floor of the canal and enlarging its cross-section. Specifications to govern such dimensions were not available; but from similar cases discussed in irrigation projects,(2) one can deduce that a length of 100, width of 400 and depth of 6 feet are sufficient. Also the inlet and outlet structures will be made of warped surfaces to avoid undesirable eddies. For further detail, one can refer to plan No. 3

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(2) B.A. Etcheverry Irrigation Practice and Engineering

CHAPTER III

DESIGN OF PIERS

9. WIDTH AND LENGTH OF SHEDS. The relation between the pier or quay space and the water area depends on the relationship between the length of vessels and their carrying capacity. This relation between the length and the carrying capacity of vessels is shown in the following table(1):

Length of Vessel in feet	Approximate net registered tonnage per lineal foot
200 - 300	5 - 6
300 - 400	6 - 7
400 - 500	8 - 10
500 - 600	10 - 12
600 - 700	12 - 15

Assuming a cubic equivalent of 40 ft. to the ton, it will be clear that a volume of space of:

$$40 \times 5 = 200 \text{ ft.}^3$$

$$\text{or } 40 \times 15 = 600 \text{ ft.}^3$$

per lineal foot of vessel, will be required for the reception of cargo. This may be provided in either open or covered quay or pier space. But as goods, for a great efficiency in delivery, will rarely be piled to heights more than ten feet, and as allowance of 33% should be made for alley ways and passages, it will probably be reasonable to take an average height of five feet over all the

(1) B. Cunningham Dock Engineering pg. 23

loading or unloading surface of the quay or pier. Hence a width of:

$$200 \div 5 = 40 \text{ ft.}$$

$$\text{or } 600 \div 5 = 120 \text{ ft.}$$

per lineal foot of vessel will be required for the cargo when wholly deposited in open spaces or sheds. The author thinks it is fair then to assume a width of shed of 100 feet per lineal foot of vessel or pier.

As regards the length of sheds, in most specifications they allow a distance of 250 - 1,000 feet. Therefore a length varying between these values will be used, depending upon the space available in each case.

10. ROADS. According to general requirements, two railway tracks should be provided on both sides of sheds, for the fact that cargo might very often be loaded or unloaded directly from trains, without piling them in sheds. Embedding such lines in the road bed, we can use the roads then for ordinary trucks and movable cranes. Thus the apron roads shall be made of a width of 31 feet, to accomodate two train lines with 13 feet between their centers and with the outer side of the train being 5 feet from the edge of the pier or quay wall.

The roads between the sheds will be made 78 feet wide, so as to provide for:

a. Two railway tracks: 26 ft.

b. Four truck lanes : 52 ft.

Then the total width of the pier will be equal to:

$$100 \times 2 + 78 + 31 \times 2 = 340 \text{ ft.}$$

11. DOCKS AND WORKSHOPS. It is emphasized that any harbor should have at least one suitable dock, where vessels undergo examination when necessary, and can have the facilities for painting and repair. Usually it is argued that a dock:

a. Should provide ample access to all parts of the vessel.

b. Should have proper ventilation so that painted vessels will dry easily.

c. Also it should have proper natural lighting for greater efficiency and economy.

Thus, two dry or graving docks will be provided as shown in the general plan. These docks have an important advantage that ships can be towed in and out very conveniently because there is no curve in the basin which obstructs such operation. The sides and floors of these docks should be lined well with watertight material, but their entrance should be provided with a pair of gates or caissons. After the entry of a ship, the entrance is closed and water is pumped out.

The dimensions of these docks were inferred from the next table:

PARTICULARS OF SOME OF THE LARGEST DOCKS[3]

Location	Date of Cons-truc-tion	Available length in ft		Entrance width in ft		Depth at H.W. On Sill	Remarks
		At Block level	At Coping Level	At H.W.	On Sill		
Bania Blanea,	1901	706	728	85	72 1/2	32 3/4	in three parts
Boston Navy Yard - No.2,	1905	729	750	101 1/2	73	30 1/2	
Bremerhaven,	1899	741 1/2	754 1/2	98 1/4	60 1/4	35 1/4	
Brest - Nos.7 and 8,	1902	742	764 1/2	87 3/4	83 1/2	35 1/2	Two docks used jointly
Brest - Commercial,	1909	738	753	92	82	37	
Colombo,	1906	708	711	83 5/6	80	32	
Hong Hong - Quarry Bay,	1908	787	790	86 1/2	81 2/3	34 1/2	
Nagasaki - No.3	1905	714	722	96	88	34	
Newport News - No.2,	1901	804	806 1/2	103	80	30	
Philadelphia - Navy,	1905	707	739	104	86	30	
Philadelphia - League Island, No.2,	1899	716	744	102	...	30	
Portsmouth, N.H	1906	725	750	101 1/2	73	30 1/2	
San Francisco - Hunter's Point,	1902	714	750	103	86	30	
San Francisco Mare Island,	1909	720	742	102	71 1/2	31 1/2	
Singapore - Tanjong Pagar	...	859	865	100	...	34	In course of constru ction

(3) B. Cunningham Principles and Practice of Dock Engineering

It will be seen that it is reasonable to have 900 x 120 ft. as the sides of each dock and say with a wall of 30 ft. between them. This wall should actually be designed. Workshops and stores will be provided near the docks as shown in the general plan.

12. HARBOR STREET. The harbor will be bordered by a wide street all along its boundary, which will separate the harbor area from the city proper and will provide for inter-communication between the different parts of the harbor with the minimum congestion. Its width shall be 100 ft. , According to the requirements. Also a bridge will be provided over the river for the same reasons with a segragation of traffic as follows:

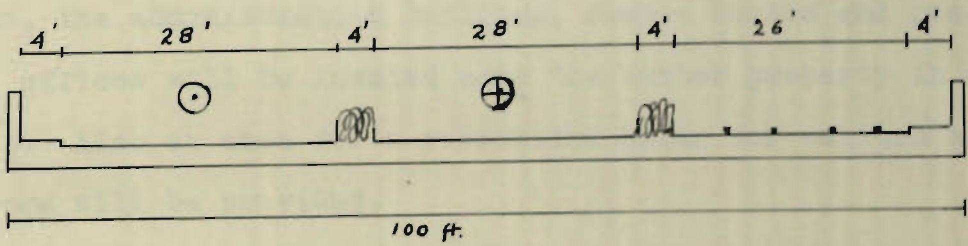

Fig. 2 Harbor Bridge

The main boulevard passing beside the area of the harbor will be continued parallel to the street of the harbor with few entries, say every 1,000 ft., or as the division of the area warrants.

13. MISCELLANEOUS. A. A railway yard will be provided for train interchange and classification, as shown in the general plan. This yard will be connected with the two railway lines of Beirut.

B. Warehouses . Appropriate areas will be provided for the general and bonded warehouses. Their dimensions depend upon the kinds of cargo which will be stored in them and therefore it is not advisable to go into such details. Roads between warehouses will be made 52 ft. wide to avoid any congestion.

C. Factories for the manufacture of raw materials or for other functions, and stores for fuel will be provided in special areas as indicated.

D. As some goods do not need to be put under covered roofs, open spaces will be provided for such a purpose.

E. For an efficient control and for a great advantage in all respects, the administration building, custom houses and passport control offices will be located near the harbor property in a central position. Also an area for a recreation house for sailors and passengers will be provided.

CHAPTER IV

DESIGN OF THE RIVER WALL

14. GENERAL REMARKS. Before proceeding to the design of the river wall, it is necessary to mention that the foundation is assumed, without design, to rest on driven piles of one foot diameter with, say, 3 ft. centre to centre. This foundation is convenient because the bearing power of the soil in that district is small. Also the hydrostatic pressure opposing the earth pressure will be ignored on the ground that at the time of construction and repair, no water will be found to counteract the Earth pressure. Moreover the effect of water getting beneath the wall will not be considered because it will be balanced by the hydrostatic pressure on the face of the wall which was neglected already.

15. SPECIFICATIONS. a. The total height of the wall is 46 ft.
b. Height of water is 40 ft.
c. Angle of friction for clayey earth: 20°
d. Surcharge : 1220 p.s.f.
e. Weight of earth : 100 p.c.f.
f. Weight of Cyclopean concrete : 150 p.c.f.

16. CYCLOPEAN CONCRETE WALL.

Equivalent height of surcharge = 1220 : 100 = 12.2 ft.

A section for the wall is assumed and shown in diagram IV. Here follows the investigation for that section:

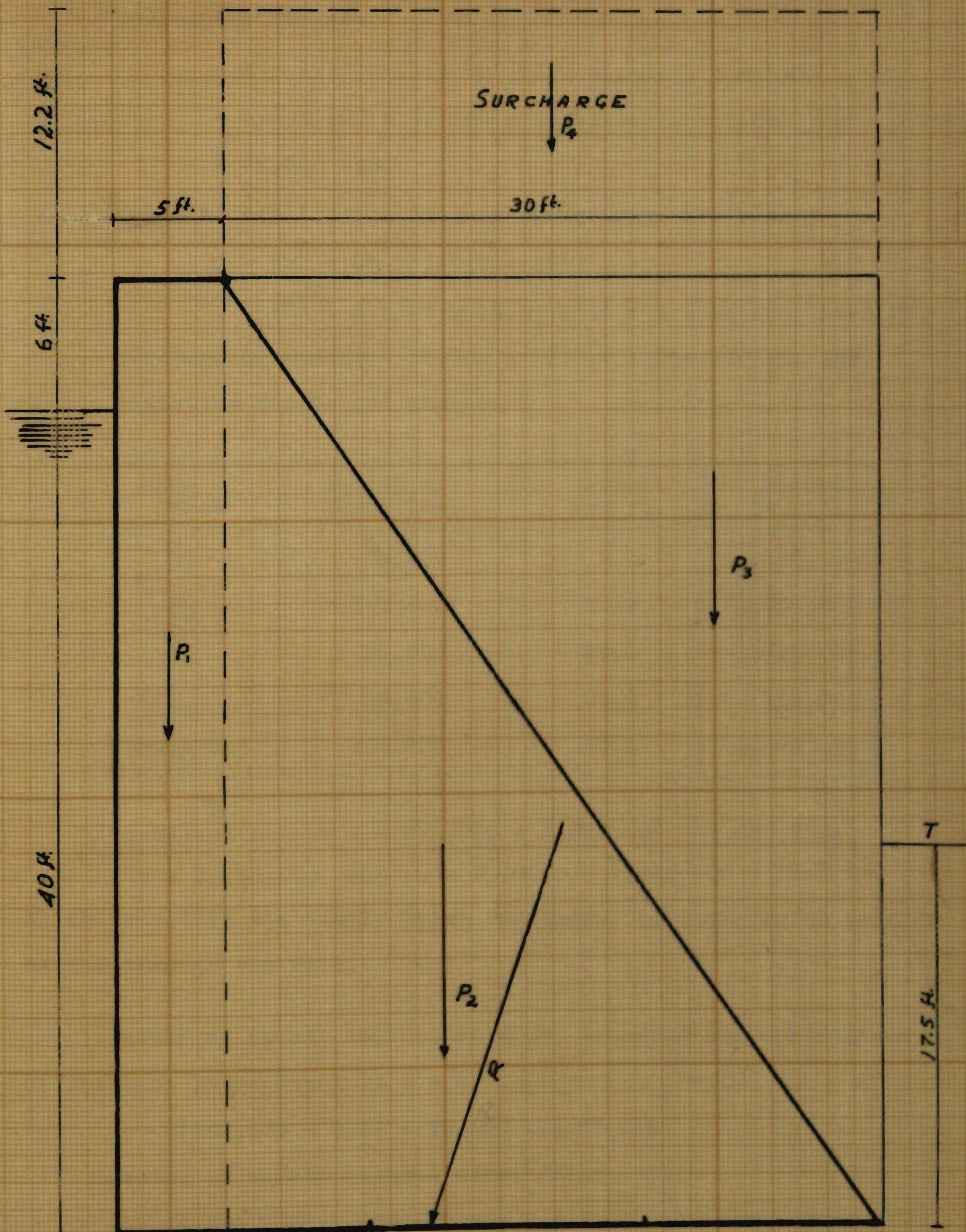

DIAGRAM IV. CYCLOPEAN CONCRETE WALL

Horizontal thrust

$$= F = 1/2\ wh\ (h + 2h')(1 - \sin\theta)/(1 + \sin\theta)$$

$$= 1/2 \times 100 \times 46\ (46 + 2 \times 12.2)(1 - 0.342)/(1 + 0.342)$$

$$= 79{,}500 \text{ lbs.}$$

Point of application of this thrust from the base :

$$y = (h^2 + 3h\ h')/3(h + 2h')$$

$$= (\overline{46}^2 + 3 \times 46 \times 12.2)/3(46 + 2 \times 12.2) = 17.5 \text{ ft.}$$

Overturning moment = 79,500 x 17.5 = 1,390,000 ft. lbs.

Resisting forces and moments :

Name of Force	Amount of Force lbs.	Moment arm ft.	Moment ft. lbs.
P_1	34,500	2.5	865,000
P_2	103,500	15.0	1,550,000
P_3	69,000	25.0	1,725,000
P_4	36,600	20.0	732,000
Resultant	243,600	20.0	4,872,000

Resultant moment = 4,872,000 - 1,330,000 = 3,482,000

a V = 3,482,000 and a = 3,482,000 ÷ 243,600 = 14.3 ft. which is within the middle third and therefore there is no tension on the base of the wall.

Factor of safety against sliding, assuming the coefficient of friction to be equal to 0.6 :

(243,600 x 0.6) ÷ 79,500 = 1.83 O.K.

Factor of safety against overturning :

4,872,00 : 1,330,000 = 3.5 O.K.

17. COUNTERFORTED WALL. Specifications

$f_c = 650$ p.s.i. $f_s = 16,000$ p.s.i. $n = 15$

$v = 120$ p.s.i. v = (without reinforcement) = 40 p.s.i.

$U = 100$ p.s.i.

A. Vertical wall. The usual spacing of counterforts is from 8 - 12 feet. In this case, due to the big height of the wall, a spacing of 8 feet centre to centre is assumed.

The amount of pressure on any given horizontal strip one foot in height, at a distance X ft. below the surface of the earth is given by the formula :

$$P_1 = CWX$$

where C is a constant and equal to $(1-\sin\theta)/(1+\sin\theta)$ in this case; and W the weight of a cubic foot of the earth.

Dividing the vertical slab into strips each one foot in height, then the thrust against the bottom strip is :

$$P_1 = (1-0.342)(100 \times 58.8)/(1+0.342) = 2,880 \text{ p.s.f.}$$

This strip is then a horizontal beam supported at intervals of 8 ft., carrying a uniform load of 2,880 p.s.f. per lineal foot. Considering it to be partly a continuous beam, then :

$$\text{Moment} = wl^2/10 = (2,880 \times 8^2 \times 12)/10 = 220,800 \text{ i.p.}$$

$$\text{and } d = \left[220,800/(108 \times 12)\right]^{1/2} = 13.1 \text{ in.}$$

assuming a thickness of the cantilevers to be 24 in. Then maximum shear = 1/2 x 2,880 x 6 = 8,640 p.

$$d = V/vjb = 8,640/(40 \times 0.875 \times 12) = 20.6 \text{ in.}$$

and a total thickness of 25 in. will be used.

$$A_s = 220,800/(16,000 \times 0.875 \times 21) = 0.75 \text{ s.in.}$$

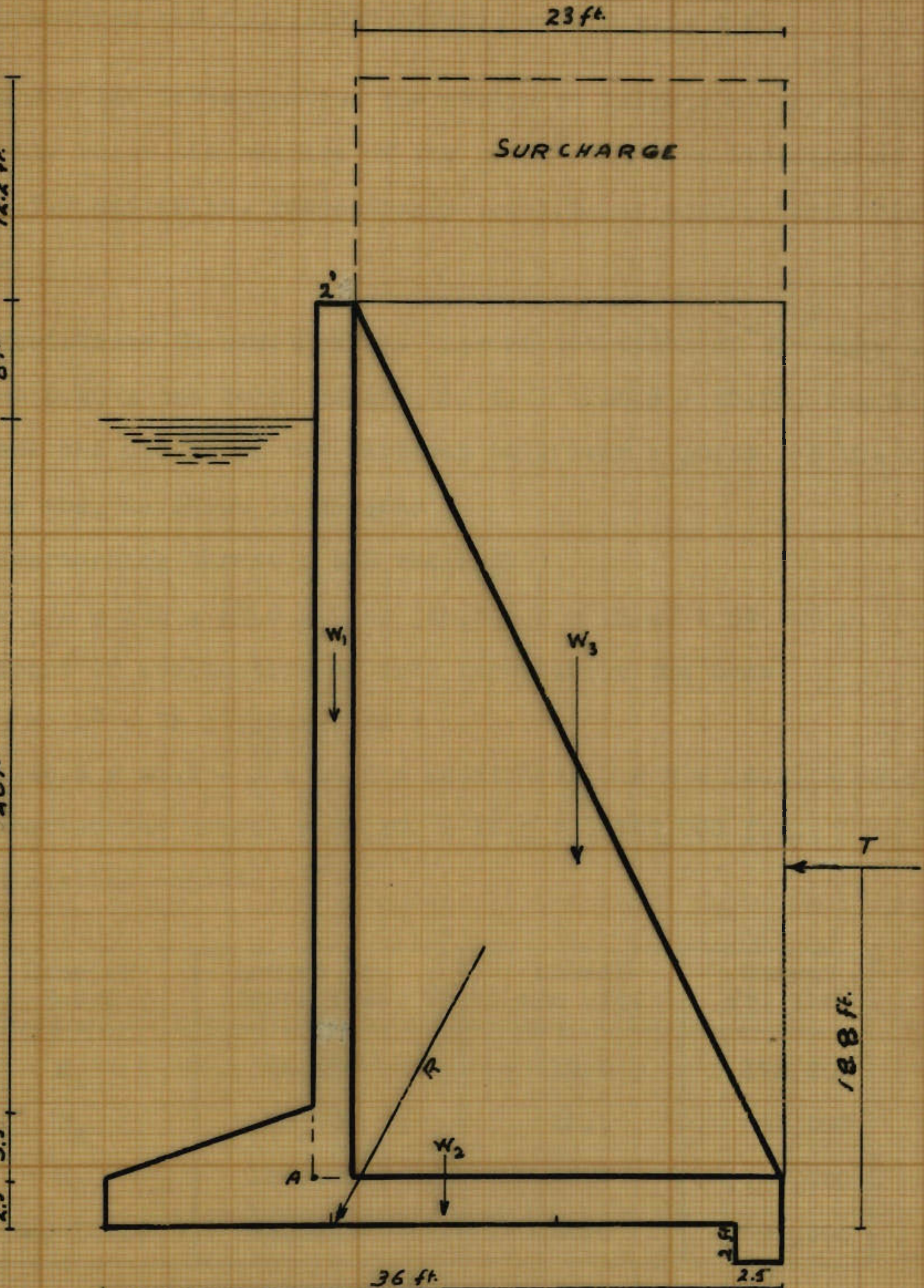

DIAGRAM V

COUNTERFORTED WALL

and 1/2 in. square bars at 4 in c. to c. will be used.

$U = V/\Sigma jd = 8,640/(2x3x0.875x21) = 79$ p.s.i. O.K.

A similar investigation for strips at heights of 40, 30, 20, and 10 ft. below the surface of the earth will give spacings of 4.5, 6, 7.5 and 12 in. respectively for 1/2 in. square bars reinforcement.

Resistance to overturning. Assume the width of the base to be about 0.60 x the total height, or 0.60 x 58.8 = 35.28, say 36 ft. and place the middle of the vertical wall over a point 1/3 the width from the toe, as shown in diagram V

Overturning thrust : 1/2 who(h+2h')

= 1/2 x 100 x 48.5 x 0.49 (48.5 + 2 x 12.2) = 86,200 p.

Point of application of this force above the base:

$y = (h^2 + 3h\,h')/3(h + 2h')$

=(48.5 x 48.5 + 3 x 48.5 x 12.2)/3(48.5 + 2 x 12.2) = 18.8 ft.

Overturning moment : 86,200 x 18.8 = 1,620,000 f.p.

Resisting moment :

Name of Force	Amount of Force lbs.	Moment arm ft.	Moment ft. lbs.
W_1	14,300	12.0	172,000
W_2	13,500	18.0	243,000
W_3	135,000	23.5	3,180,000
Resultant	162,800	22.0	3,595,000

The factor of safety against overturning :

3,595,000 : 1,620,000 = 2.2 O.K.

Resultant moment :

$a\ v = 3,595,000 - 1,620,000 = 1,975,000$

and $a = 1,975,000 : 162,800 = 12.15$ ft.

which is just within the middle third, and therefore this section is O.K.

The pressure on the toe and heel of the wall is:

$p = P/A \pm Mc/I = 16,280/36 \pm 16,280 \times 5.85 \times 6/36 \times 36$

$= 4,530 \pm 4,400 = 8,930$ p.s.f. Compression O.K.

and 130 " "

B. Inner base slab. The loading on the horizontal base slab is the difference between the sum of the weights of earth and of the base acting downward, and the foundation pressure acting upward. The load distribution is shown in Fig. 3.

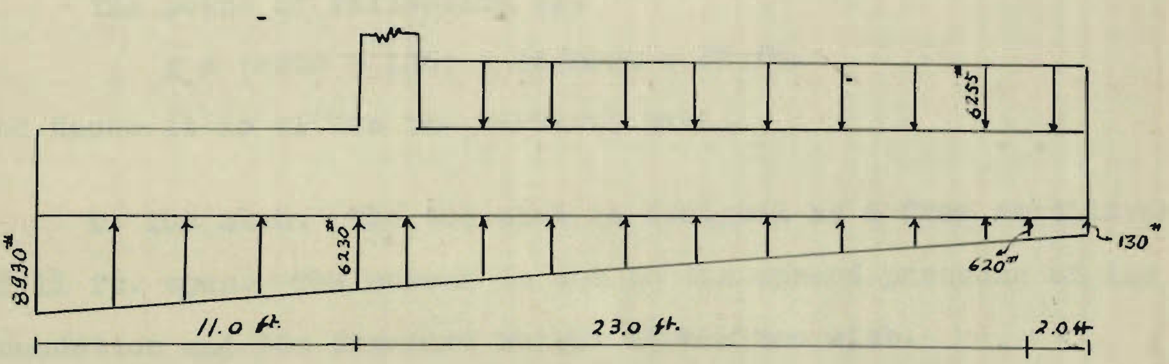

Fig. 3. Distribution of loads.

This slab is designed as a simple beam supported by the counterforts and the strengthening action of its monolithic construction warrants such a procedure. A strip of 2 ft. at the heel is subjected to a load of:

$6,255 \times 2 - (130 + 620)\ 2/2 = 11,760$ p.p.l.f.

Moment $= (1/10)\ 11,760 \times 6 \times 6 \times 12 = 368,000$ in. p.

Maximum shear = (1/2) 11,760 x 6 = 35280 p.

The required depth is obviously governed by shear

$d = 35280/120 \times 0.875 \times 24 = 14$ in.

and a total depth of 30 in, as assumed, will be used.

$As = 1/2\ (368,000/16,000 \times 0.875 \times 28) = 0.49$ sq. in.

Perimeter of steel as determined by bond:

$= 35280/100 \times 0.875 \times 28 = 14.2$ in.

and 1 in. square bars 6 in. c. to c. will be used.

Web reinforcement. Using 1/4 in. round bars for web reinforcement, the required spacing is : $s = (A_v f_v jd)/V'$

$= (2 \times 0.049 \times 16000 \times 0.875 \times 28)/(35280 - 40 \times 24 \times 0.875 \times 28)$

$= 11.2$ in.

and 10 in. spacing for the first 10 ft., and 14 in. for the remainder, will be used.

The point of inflection is:

$x = (6255 - 130) \times 36/8800 = 25$ ft.

and hence it is within the vertical wall.

C. Toe Slab. The toe slab is designed as a free cantiliver of 11 ft. span. The moment is due to the upward pressure of the foundation and the downward weight of the toe slab.

Moment: $= (8930 - 6230) \times (11/2) \times (2 \times 11/3) = 109,000$

$+ (6230 - 375) \times 11 \times 11/2 = \underline{354,000}$

463,000 f.p.

Maximum shear $= 11 \times (8930 + 6230)/2 - 375 \times 11 =$

$= 79,270$ p.

and $d = 79270 / (120 \times 0.875 \times 12) = 63$ in.

and a depth of 72 in. will be provided, which tapers at the end to a depth of 30 in.

As = 463,000 x 12 / (16,000 x 0.875 x 67) = 5.9 sq. in. and two rows of 1 1/4 in. square bars with 6 in. c to c. will be used.

u = 79270/(5x4x0.875x67) = 67.5 p.s.i. O.K.

Web reinforcement. Using 1/2 in. round bars for web reinforcement, the required spacing is

s = (2x0.39x16000x0.875x67)/(79270-40x12x0.875x67) = 14.5 in. and 12 in. spacing for the first 5 ft., and 16 in. for the remainder, will be used.

D. Counterforts. The moment in the counterfort is due to the pressure of the earth on the vertical slab over a length of wall equal to the distance center to center of counterforts.

P = 1/2 x 100 x 46 (46 + 2 x 12.2) x 0.49 x 8 = 636,000 p.

y = (46 x 46 + 3 x 46 x 12.2) / 3 (46 + 2 x 12.2) = 17.2 ft.

M = 636,00 x 17.2 x 12 = 131,500,000 in. p.

The effective depth of the counterfort is the perpendicular distance from the point A. Diag. V to the reinforcing steel

d = 25 x 46 x 12/(46 x 46 + 25 x 25)$^{1/2}$ - 12.5 = 251.5 in.

As= 131,500,000/(16000 x 0.875 x 251.5) = 37.3 sq. in.

And 24 square bars of 1 1/4 in. will be used furnishing an area of 37.5 sq. in.; and these will be put in 4 rows with 5 in. c. to c of row.

u = 636000/(24 x 5 x 0.875 x 251.5) = 24 p.s.i. O.K.

The effective depth to be used in determining the unit shear on the base of the counterfort is equal to the horizontal distance from A, Diag. 5., to the reinforcing steel :

25 x 12 - 12.5 = 287.5 in.

v = 636,000/(24 x 0.875 x 287.5) = 105 p.s.i.

This value is satisfactory because horizontal bars will be provided to anchor the vertical slab and the counterfort together. These will serve the function of the web reinforcement.

Following an investigation of the moment and shear at heights of 12, 24 and 35 ft., the bars of the counterfort are bent as shown in Fig. 4. and anchored to the vertical slab steel.

The curtain wall and base slab must be tied to the counterforts by horizontal and vertical bars capable of carrying the reactions at the points of supports. These will equal the shears on the two sides of the counterfort. The amount of pull at the base vertical slab per foot of height is :

$cwx = 0.49 \times 100 \times 58.2 \times 6.0 = 17{,}500$ p.

and the area of steel required per foot of height is:

$17500 : 16000 = 1.06$ sq. in.

If reinforcing bars are placed in pairs and at the same distance apart as the horizontal bars in the curtain walls, i.e. 4 in. c. to c. 1/2 in. round bars will answer. These must be looped around the steel in the face of the curtain wall and extend to the rear of the counterfort. From similar investigations at various heights, it is shown that a spacing of 8 in. c. to c. above a height of 22 ft. is satisfactory.

For the base slab the load upon the bonding bars per foot of width is:

$= (6255 - 130)\ 6 = 36{,}750$ p.

$A_s = 36750 : 16000 = 2.3$ sq. in.

Pairs of 3/4 in. round bars spaced 4 1/2 in. c. to c. meet this requirement. All of the bars should extend upward and be hooked around the tension steel in the base slab.

The main reinforcing bars must extend into the base slab a distance equal to :

$$l = \frac{si}{4U} = 16000 \times 1.25/(4 \times 100) = 50 \text{ in.}$$

Hence a key wall 25 in. deep and 30 in. wide is constructed under the heel of the wall to provide for the necessary embedment. The main bars are hooked as a further precaution.

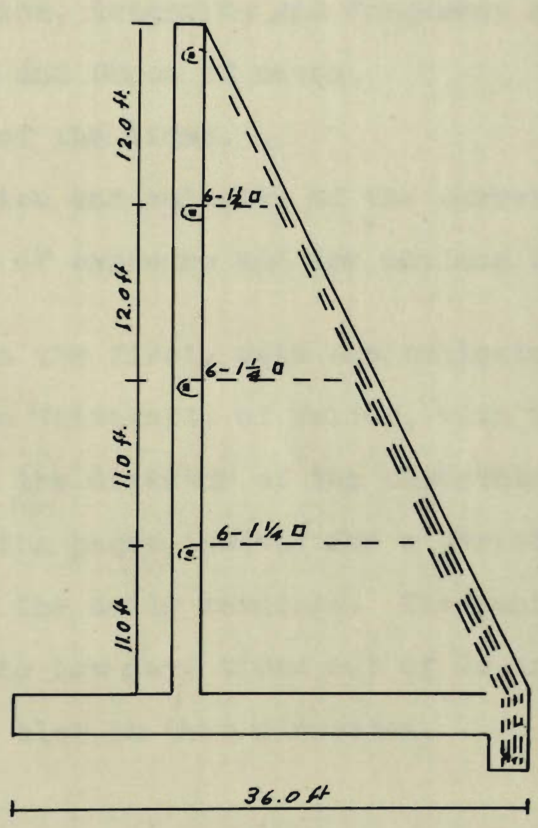

Fig. 4 Reinforcement of the Counterfort

CHAPTER V

BREAKWATER

Naturally, piers projecting into the sea need to be protected by a breakwater whose function is "to break up and disperse the waves, preventing them from exerting their influence on the basin."(1)

18. ORIENTATION OF THE BREAKWATER. The orientation of the breakwater necessitates a study of various natural and meteorological phenomena, mainly :

1. The direction, intensity and frequency of wind.
2. The height and force of waves.
3. The range of the tides.
4. The direction and velocity of the currents.
5. The extent of exposure and the maximum fetch of the place.

With regard to the first, data was collected from the observatory of the American University of Beirut, with the kind permission of Prof. R. Sloane, the director of the observatory. This data tabulated on the following pages, extend for a period of five years with monthly averages of the daily readings. The numbers under the direction of wind indicate how many times out of 90 or 93, depending upon the month, the wind blew in that direction.

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(1) B. Cunningham   Harbor Engineering   pg. 162

WIND OBSERVATIONS FROM THE OBSERVATORY OF THE AMERICAN UNIVERSITY OF BEIRUT

Year	Month	Direction of the Wind									Velocity of Wind in Km/day	
		N	NE	E	SE	S	SW	W	NW	Calm	Average	Maximum
1944	Jan.	2	0	1	3	0	55	4	8	20	497.4	1024.0
	Feb.	4	5	3	4	1	43	0	0	27	490.8	1328.0
	Mar.	0	16	0	1	0	47	1	4	24	434.6	1135.0
	Apr.	1	14	3	1	6	35	3	1	26	357.6	649.0
	May	1	15	0	2	0	60	3	6	16	386.1	805.0
	June	0	13	1	0	2	35	3	7	19	322.6	700.0
	July	0	1	0	0	0	80	0	0	12	505.1	1022.0
	Aug.	0	6	0	0	0	47	4	2	34	333.9	731.0
	Sept.	0	13	3	6	9	19	3	8	29	257.9	660.0
	Oct.	6	10	1	6	3	23	5	19	20	255.3	595.0
	Nov.	5	15	2	3	2	32	3	9	19	312.6	698.0
	Dec.	0	16	5	8	7	32	1	5	19	332.4	950.0
1945	Jan.	3	5	3	4	5	46	5	7	15	381.7	1373.0
	Feb.	2	12	7	2	4	33	2	10	12	305.2	794.0
	Mar.	12	7	3	5	4	27	2	9	24	235.5	716.0
	Apr.	7	5	1	2	8	37	4	8	18	193.7	649.0
	May	2	12	0	0	3	25	2	11	38	141.4	550.0
	June	0	0	0	0	5	59	0	5	21	208.2	402.0
	July	5	0	0	0	6	65	4	3	10	169.7	628.0
	Aug.	0	0	0	0	5	56	6	4	22	251.9	591.0
	Sept.	1	1	3	8	15	28	4	12	18	239.3	382.0
	Oct.	6	10	1	0	7	34	5	3	27	370.3	620.0

Year	Month	Direction of the Wind									Velocity of Wind in Km/day	
		N	NE	E	SE	S	SW	W	NW	Calm	Average	Maximum
1945	Nov.	13	2	3	1	0	24	7	4	36	380.8	1365.0
	Dec.	0	8	2	2	1	50	2	10	18	465.1	966.0
1946	Jan.	14	5	0	2	1	16	12	10	33	441.8	1255.0
	Feb.	1	5	3	1	2	44	9	6	13	652.8	1078.0
	Mar.	0	1	3	3	4	56	11	4	11	467.0	827.0
	Apr.	1	11	4	1	0	37	18	9	9	421.0	760.0
	May	1	10	5	0	0	45	8	6	18	373.7	728.0
	June	1	2	0	0	0	47	12	6	22	343.5	950.0
	July	0	1	0	0	0	55	4	7	26	340.7	562.0
	Aug.	0	0	0	0	0	57	16	3	17	415.2	549.0
	Sept.	1	3	0	1	0	32	21	6	26	280.3	460.0
	Oct.	3	18	0	3	0	41	0	13	15	425.1	750.0
	Nov.	6	16	1	6	3	35	0	2	27	283.4	685.0
	Dec.	2	11	1	17	4	47	2	3	6	414.1	885.0
1947	Jan.	1	1	0	13	7	59	2	1	9	535.8	1122.0
	Feb.	2	18	0	10	0	42	3	7	2	277.2	739.0
	Mar.	1	14	2	9	0	51	1	8	7	330.3	919.0
	Apr.	4	16	0	3	0	45	2	7	13	960.9	974.0
	May	2	24	0	0	0	42	6	10	9	349.7	800.0
	June	0	5	0	0	0	51	3	15	16	324.9	510.0
	July	0	3	0	1	1	65	6	6	11	431.5	855.0
	Aug.	1	0	0	0	0	52	11	11	18	354.9	684.0
	Sept.	12	2	0	0	1	32	18	10	25	316.0	722.0
	Oct.	6	13	0	4	1	19	9	21	20	337.2	472.0
	Nov.	4	20	0	5	5	34	4	6	12	403.0	661.0
	Dec.	0	20	1	11	5	38	1	6	11	397.7	1276.0

Year	Month	Direction of the Wind									Velocity of Wind in Km/day	
		N	NE	E	SE	S	SW	W	NW	Calm	Average	Maximum
1948	Jan.	7	14	0	13	3	30	3	5	18	345.0	1291.0
	Feb.	2	10	0	3	3	49	6	8	6	537.7	1051.0
	Mar.	4	12	0	12	1	43	4	10	7	472.8	1093.0
	Apr.	3	16	0	5	3	46	8	1	8	413.3	958.0
	May	6	18	0	0	0	35	5	11	18	394.5	702.0
	June	0	4	0	0	0	49	14	9	14	330.6	517.0
	July	1	6	0	0	0	48	13	2	23	368.8	917.0
	Aug.	6	0	0	0	0	42	15	5	25	291.3	700.0
	Sept.	0	16	0	0	1	32	10	13	13	301.1	679.0
	Oct.	15	21	0	0	3	21	4	6	23	321.7	837.0
	Nov.	5	5	3	2	7	24	6	8	30	322.2	1072.0
	Dec.	3	9	0	5	11	32	18	9	6	464.1	940.0
	Total	169	538	65	188	159	2495	348	425	998		
Average		3	9	1	3	3	42	6	7	16		

With this data the author was able to draw the wind rose (Diag. VI) for Beirut, which shows that the prevailing wind is South West. It is to be noted that the prevailing wind, with the ebb and flow of tides and the erosive and transportive powers of waves have a great influence upon the coastal contour, which can be shown in the next examples.

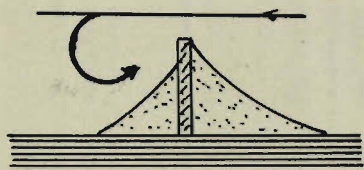

Fig. 5

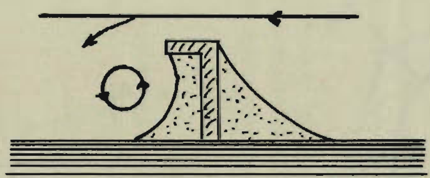

Fig. 6

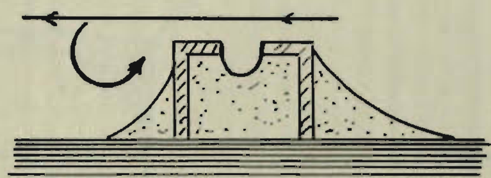

Fig. 7

Variation of sand accretion with shape of a Breakwater [2]

Hence, the orientation of the breakwater is made in such a way so as to keep the entrance of the harbor free from any shoals. A mere glimpse on figure 8. below, shows the effect of the prevailing wind in keeping the main entrance free from silt shoals or bars.

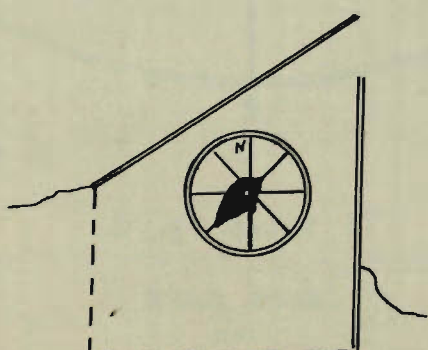

Fig. 8. Relation of the breakwater to the Prevailing wind

(2) B.Cunningham Harbor Engineering pg. 31

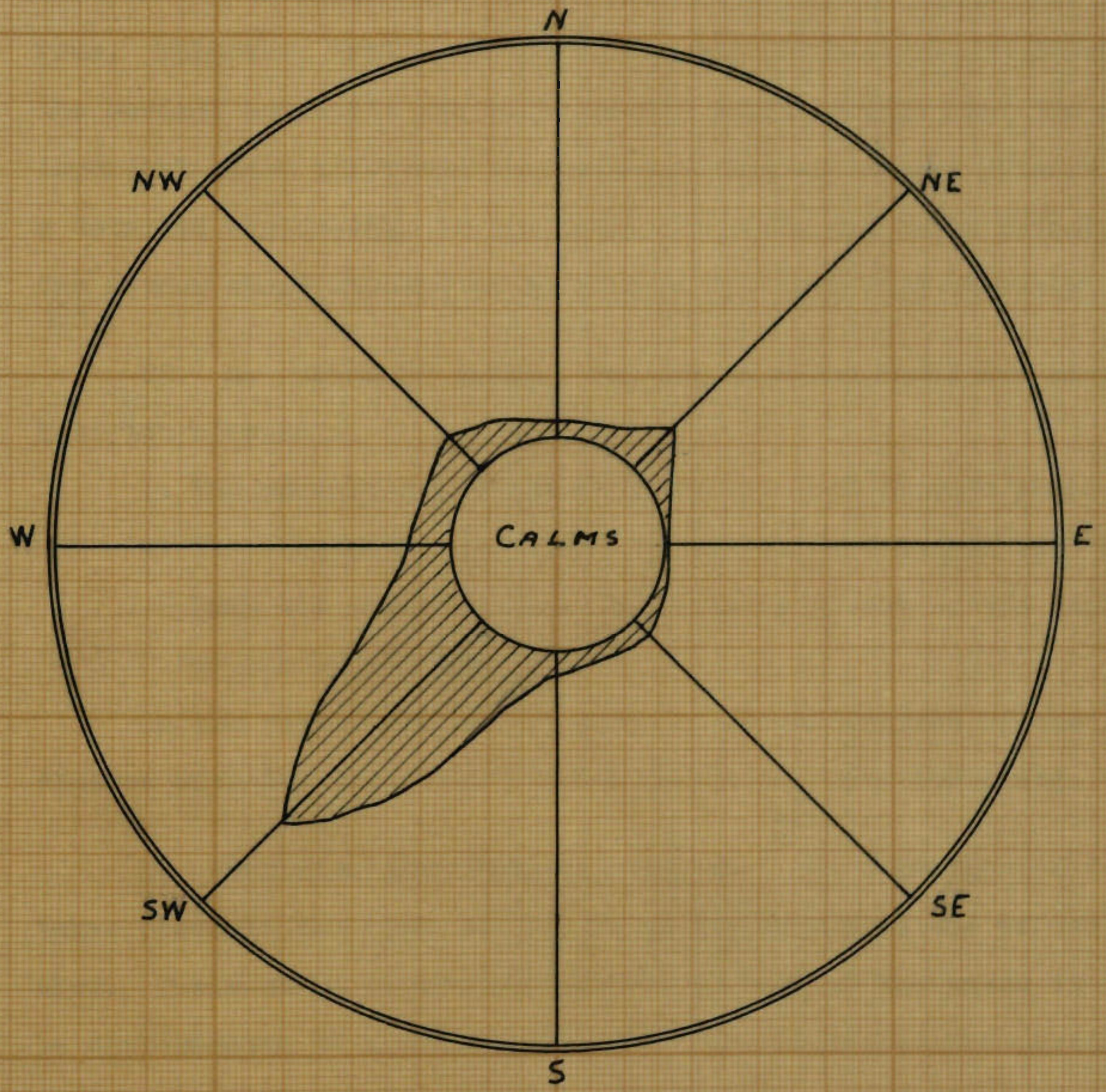

DIAGRAM VI

WIND ROSE
OF
BEIRUT

19. AGENCIES AFFECTING THE BREAKWATER. Before proceeding to the design of the breakwater, it is appropriate to point out the general agencies affecting the design and to dwell more fully on the most important ones. Structures built into the sea have their weight reduced due to immersion, unless the foundation be absolutely impervious. Moreover, small insects of the sea are capable of undermining the hardest materials you will use for construction. But the most important agent affecting the design is the sea wave. This requires a detailed explanation of the nature of waves and their effect. And the author shall attempt to do this in the following articles.

20. A. KINDS OF WAVES. The sea or water waves are divided into two classes :

1. The oscillatory waves which do not have forward motion.
2. The translatory waves which have forward motion and thus have the power to exert appreciable effect on the stability of the breakwater.

On the whole most of the waves are a combination of the two classes with the translatory waves prevailing near the shore.

B. FORMATION AND MOVEMENT OF WAVES. The formation of waves takes place in the open sea and it is due mainly to the effect of wind. Their shape is not uniform but in most of the cases it can be approximated to a cycloidal curve. The motion of the particles depend upon the local conditions of the wave. In a place where the depth of the sea is equal to or more than the length of the wave, the wave is oscillatory and its motion is orbital as shown in Fig. 9 below.

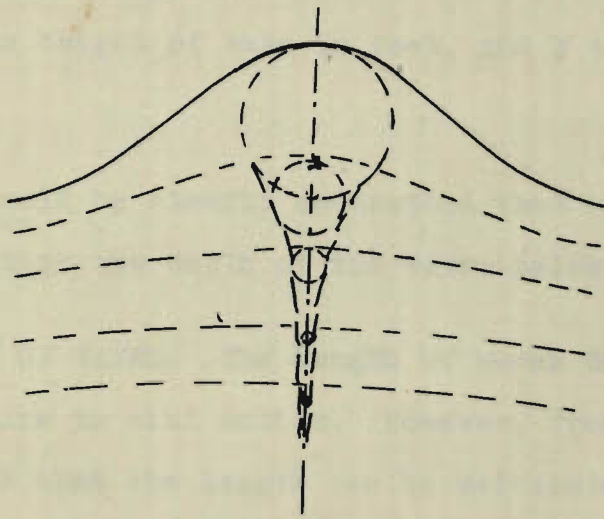

Fig. 9. Wave in deep water. (3)

In shallow water, where the depth of the sea is less than the length of the wave, the orbit of motion is elliptical with the major axis horizontal. But when the depth of the sea decreases, instead of remaining uniform, the orbits of motion become more and more distorted, with the crest gaining upon the trough. Then the orbit breaks and the waves become translatory with a forward velocity equal to the velocity of the wave. Waves in such a state, have their greatest disruptive force on the breakwater.

C. HEIGHT OF WAVES. The height of waves depends on the open length of sea which is acted upon by the wind. The relation was studied and many emperical formulae were devised to calculate the height of the waves as a function of the fetch, or the available clear distance for the purpose of generation of the wave.

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(3) B. Cunningham Harbor Engineering pg. 165

One of the satisfactory conclusions is as follows:

$$H = 1.5\ F^{1/2}$$

Or closer for short fetches : $H = 1.5\ F^{1/2} + (2.5 - F^{1/4})$ where H stands for the height of wave in feet, and F the fetch of the place in miles.

But it should be clearly understood that no wave can have a height greater than the depth of the water below it.

D. LENGTH OF WAVES. The length of waves depends mainly on the amount of exposure to wind action. However, from actual observations, Bertin concluded that the length can be calculated if the period of the wave can be found. His emperical formula is :

$$L = P^2 g/(2\pi)$$

where L stands for the length of the wave in ft., and P its period in seconds.

It is argued that the length of the wave with the depth of the water below it, determine the speed of the wave.

21. BREAKING WAVES. The waves in reaching the breakwater are broken and the motion of the particles will be as shown in Fig. 10.

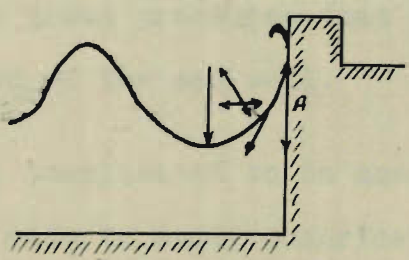

Fig. 10. Breaking Waves (4)

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(4) B. Cunningham Harbor Engineering pg. 171

The particles of water in contact with the wall move up and down along the wall through a height of twice the height of the wave itself. Midway between the wall and the trough, the particles move horizontally back and forth. But in intermediate points the motion of the particles is different and inclined at various angles.

Considering all the phenomena of breaking waves, it is evident that the following forces are generated:

1. A direct horizontal force, exerting compression.
2. A deflected vertical upward force tending to shear off any projection beyond the face of the wall.
3. A vertical downward force due to the collapse of the wave.
4. The suction due to back-draught.

When these facts are applied to the design of the breakwater, it is easily concluded that they give rise to :

1. A powerful momentary impact, combined with:
2. Hydrostatic pressure continuous for some short period after the first shock.
3. A series of impulses imparted to the water contained in the pores and joints of the wall, producing internal pressure in all directions.
4. The alternate contraction and expansion of entrapped volumes of air which cause great pressures that can be avoided by proper construction of the sea wall.

These forces are very complicated to be measured, but in the next article an attempt is made to study numerically the principal effects of the waves.

22. DYNAMICAL VALUE OF WAVE ACTION. Theoretically, breaking waves experience two phases of action as was pointed out, i.e.:

1. A momentary and sharp blow succeeded by
2. A statical pressure during a small interval of time which suffices for the dispersal of the wave.

However, in practice the matter is considered as one of the simple continuous impact for feasibility of calculations. Hence, according to the principles of dynamics, the reaction of a body subjected to continuous impact is measured by the rate at which the momentum is destroyed. Therefore, if we consider "w" as the weight of a unit volume of water, $\frac{wv}{g}$ is the mass which strikes on a unit surface in unit time; v being the velocity per second and g the acceleration of gravity. And the rate at which momentum is consumed equals : $mv = \frac{wv^2}{g}$. Hence the pressure on a unit surface is :

$$P = wv^2/g.$$

Therefore if the velocity is found, the pressure can be calculated very easily from the above relation. In deep water, the velocity of the wave is proved to be equal to the velocity acquired by a freely falling body through a height of one-half the radius of the circle, the circumference of which constitutes the length of the wave.(5) In shallow water the conditions are different and the velocity is proved to be nearly the same as that velocity of a freely falling body from rest, from a height of one-half the depth of water plus three-fourths of the height of the wave.(5) That is:

$$V = \sqrt{2g\left(\frac{d}{2} + \frac{3h}{4}\right)}$$

where d stands for the height of the water which should not exceed the length of the wave.

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(5) Rankine, Civil Engineering 18th edition pg. 754

Such velocities give high pressure intensities which should be accepted with reserve. Nevertheless, experienced authorities found out that a general equation can be written for this pressure:

$$p = kwh$$

where k is given values of fairly wide range, but whose average is: 1.6. That is:

$$p = 1.6\ wh.$$

23. KINDS OF BREAKWATERS. Before proceeding to the design of the breakwater, it is necessary to point out the different kinds of breakwaters and decide on the form which will be adopted.

Breakwaters are classified into three main types:

1. The heap or mound which consists of a heterogeneous assemblage of natural stones deposited pell-mell.
2. The wall type which consists of a regular and systematic construction of a wall, either of masonry or of concrete.
3. A combination of these two, consisting of a foundation mound and a wall above. In modern practice this type constitutes the bulk of all new breakwaters.

A choice between these three types depends upon their particular advantages and disadvantages which may be considered under headings of :

A. Cost of construction

B. Cost of maintenance

C. Efficiency

A. Cost of construction. Of course this will depend on the locality of the breakwater and its coastal environment. Stones, in this special case, whether big or small are expensive both in price and in transportation.

B. Maintenance. A mound is susceptible to constant changes due to waves and thus it should be repaired constantly; but a wall if properly constructed calls for no further attention except in an extraordinary emergency.

C. Efficiency. The efficiency of the breakwater should be of the greatest importance. The waves reach a wall breakwater in an oscillatory-translatory state, where they are deflected upward and falls back on the water without exercising any considerable effect upon the foundation. On the rubble mound, the waves rush up the seaward slope, and falls down over the crest and tend to affect a breach which leads to serious cracks. However, a composite breakwater possesses the property of a wall breakwater if the superstructure commences at a depth at least 30 ft.

Considering all these, the author believes that it is advisable to construct a composite breakwater with a mound foundation and a wall above it.

24. A. DESIGN OF THE WALL OF THE BREAKWATER. A wall breakwater subject to the action of waves may fail in one of the following ways:

a. By sliding, or by shearing

b. By overturning

c. By uplifting

In the first case the dynamic and static horizontal pressures of the waves should be counteracted by the frictional resistance of the wall or by its shearing resistance. In these calculations the buoyant force of the water should be considered.

As concerns overturning, which is also due to the same pressures, it should be counteracted by the weight of the wall and any other force which is made to do so.

The most unfavorable conditions for overturning are when the dynamic pressure is assumed to be uniform on the face of the wall. Although, this condition rarely happens, yet it is considered on the safe side to do the design as such. Also the resultant force should always fall within the middle third of the base and the allowable compressive stress should not be exceeded.

Uplifting which is due to the application of the wave force to the underside of the wall is counteracted by the dead effective weight of the wall itself.

B. DESIGN CALCULATIONS.

Height of the wave $H = 1.5\ F^{1/2}$

where H is the height of wave in feet and F the fetch of the place in miles.

The fetch of Beirut is around 700.0 miles, and therefore the height of the wave is equal to :

$$H = 1.5 \times 700^{1/2} = 39.7 \text{ ft.}$$

But from actual observations carried on the Mediterranean shores, it was found out that the maximum height of waves vary between 15 and 20 ft.[6] This can be explained by the fact that many intervening shoals in the path of the waves reduce their height. Hence it is reasonable to assume a maximum height of 20 ft. in this case.

Length of the wave : $L = \frac{P^2 g}{2\pi}$

where L is the length of wave in ft. and P its period in seconds.

From actual observations in stormy weather, the author found out that a period of 4 - 8 seconds is common.

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(6) B. Cunningham Harbor Engineering pg. 167

Therefore we can assume the length of the wave to be equal to :

$$L = 6 \times 6 \times 32.2/2 \times 3.14 = 184.5 \text{ ft.}$$

The velocity of the breaking waves, where the depth of the water is less than the length of the wave, is:

$$V_2 = \sqrt{2g(d/2 + 3h/4)}$$

$$= \sqrt{2g(40/2 + \frac{3x20}{4})} = 47.5 \text{ ft/sec.}$$

and the pressure : $p = \frac{wv^2}{g} = \frac{64 \times 47.5 \times 47.5}{32.1}$

$$= 4{,}620 \text{ p.s.f.}$$

But from the emperical formula the pressure is:

$$p = 1.6 \text{ wh} = 1.6 \times 64 \times 20 = 2050 \text{ p.s.f.}$$

Although this is less than the first pressure, yet, it is safe to take it as the actual pressure, relying on the conclusions of authorities in this subject.

Having found all the necessary data, we shall now assume a section of the wall, and investigate for its stability. This section is shown in diagram VII. It is to be noted that the seaward face of the wall is curved in order that the resultant force "F" applied on that curve will fall within the base of the wall and will help in counteracting the overturning forces. Also the level of the water on the seaward face is assumed to be 10 ft. higher than the level of the basin water.

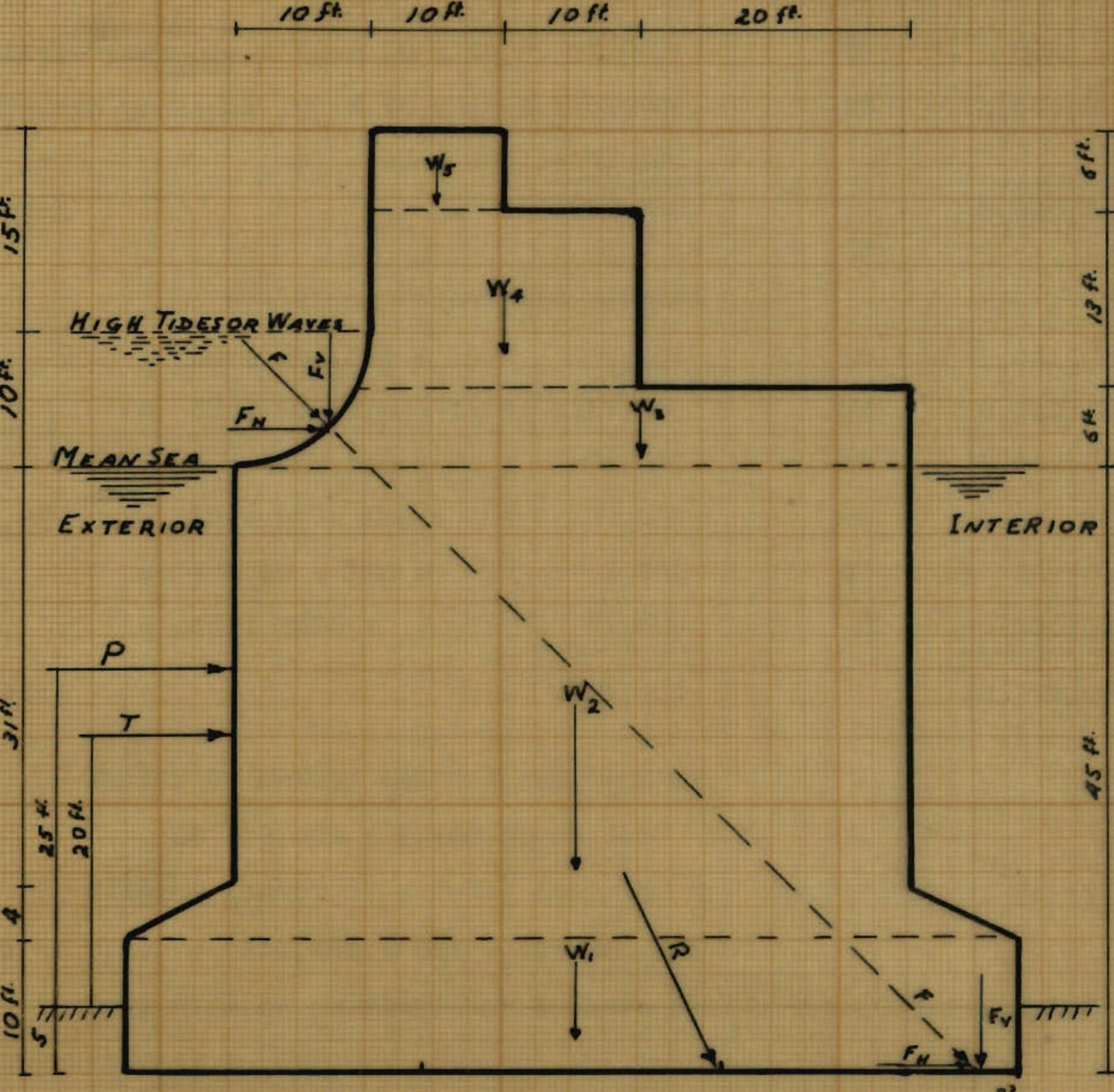

DIAGRAM VII

WALL OF THE BREAKWATER

Overturning forces and moments :

Force in lbs.		Moment arm in ft.	Moment in ft.lbs.
Static pressure = 50 x 10 x 64 =	32,000	25	800,000
Dynamic pressure, T = 40 x 2050 =	82,000	20	1,640,000
" " , F_1 = 25 x 2050 =	51,300	0	
Total	165,300		2,440,000

Resisting forces and Moments

W_1 = 10 x 66 (2.4 - 1) =	59,100	33	1,951,000
W_2 = 35 x 50 (2.4 - 1) 64 =	157,000	33	5,190,000
W_3 = 6 x 40 x 150 =	36,000	28	1,008,000
W_4 = 13 x 20 x 150 =	39,000	38	1,482,000
W_5 = 6 x 10 x 150 =	9,000	43	387,000
F_1 = 25 x 2050 =	51,300	3	154,000
Total	351,400		10,172,000

Factor of safety against overturning:

10,172,000 : 2,440,000 = 4.17 O.K.

Sliding factor of safety, assuming the coefficient of friction to be 0.7. : 351,400 x 0.7 : 165,300 = 1.48

Keys will be provided for anchorage with the foundation and hence it will be O.K.

Point of application of the resultant force :
(10,172,000 - 2,440,000) : 351,400 = 22,003 ft. which is just within the middle third of the base and it is O.K.

Maximum compressive stress :

2 (351,400 : 66 x 144) = 74 p.s.i. O.K.

Uplifting force = 2050 x 66 = 135,300 lbs.

Uplifting factor of safety : 300,100 : 135,300 = 2.15 O.K.

Investigation of the stability of this wall when there is a back-draught of 10 ft. shows that it is safe in all respects.

As regards the foundation of this wall, we need a special study of the earth to be able to design it. However, a rough approximated foundation inferred from available data about Beirut Port, is shown in the section of the breakwater.

BIBLIOGRAPHY

Books.

1. Principles and Practice of Dock Engineering

by

Cunningham, Brysson.

2. Principles and Practice of Harbor Engineering

by

Cunningham, Brysson.

3. Elementary Meteorology

by

Davis, W.M.

4. Regional Planning

by

Esetitt, L.B.

5. Irrigation Practice and Engineering Vol. III

by

Etcheverry, B.A.

6. Principles of City Planning

by

Lohman, Karl B.

7. City Planning

by

Nolen, John

8. Masonry Structures

by

Spalding, F.P.

9. Design of Concrete Structures

by

Urquhart, L.C. and O'Rourke, C.E.

Reports and Newspapers

10. An-Nahar Special Issue Jan. 1, 1949

11. L.16 Port de Beyrouth

Jetée du Large