

CHAPTER 37

DETERMINATION OF THE WAVE ATTACK ANTICIPATED UPON A STRUCTURE FROM LABORATORY AND FIELD OBSERVATIONS

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1.0 INTRODUCTION.

1.1 The structure. In one of the estuaries of the Rhine-Meuse delta, the Haringvliet, a series of discharge sluices is under construction as part of the Delta works [fig.1 and 2]. The effective aperture of the sluices when completely opened is 6000 m² below N.A.P.*)

Such an opening is necessary in times of high river discharges to enable large quantities of water (up to 20,000 m³/sec) to be drained away and floating ice to be got rid of in severe winters. To meet the latter requirement the spans of the sluice-openings have been fixed at 56,5 m. The sluice consists of 17 identical openings separated by piers about 5 m wide.

Each opening can be closed with two steel gates. Each gate is connected by means of four steel levers to the Nabla-beam, a triangular, hollow girder of prestressed concrete, which also serves as a traffic-bridge [fig.3]. The gates can be moved independently by means of hydraulic lifting mechanism.

In the event of a storm surge the sluices will be closed; then they may have to withstand a maximum head of 4,5 m.

As the Haringvlietsluices are vital to the water economy of the northern delta area, they must be extremely reliable. Consequently when both gates are closed, or while they are being opened or closed, the structure will have to be capable of withstanding wave attack.

In principle two kinds of wave attack can be distinguished in this case:

a. Wave load in a horizontal direction; this may occur when the gates are closed. The magnitude of the loads determines the design of the gates, the levers and the Nabla-beam [fig.3].

b. Wave-load in a vertical direction; this will occur, in combination with the horizontal load, when the gates are lifted during wave motion. The proportioning of the lifting mechanism is largely determined by this particular load.

*) N.A.P. is a datum level, approximately at mean sea level.

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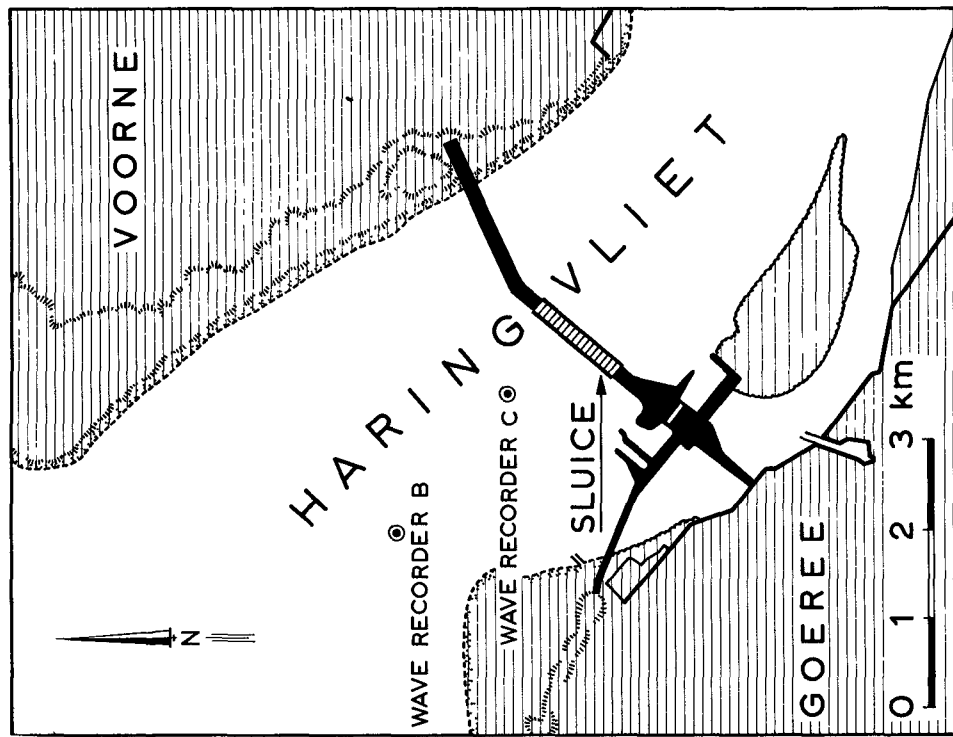


Fig. 2. Situation of the sluices and the wave measuring poles in the Haringvliet.

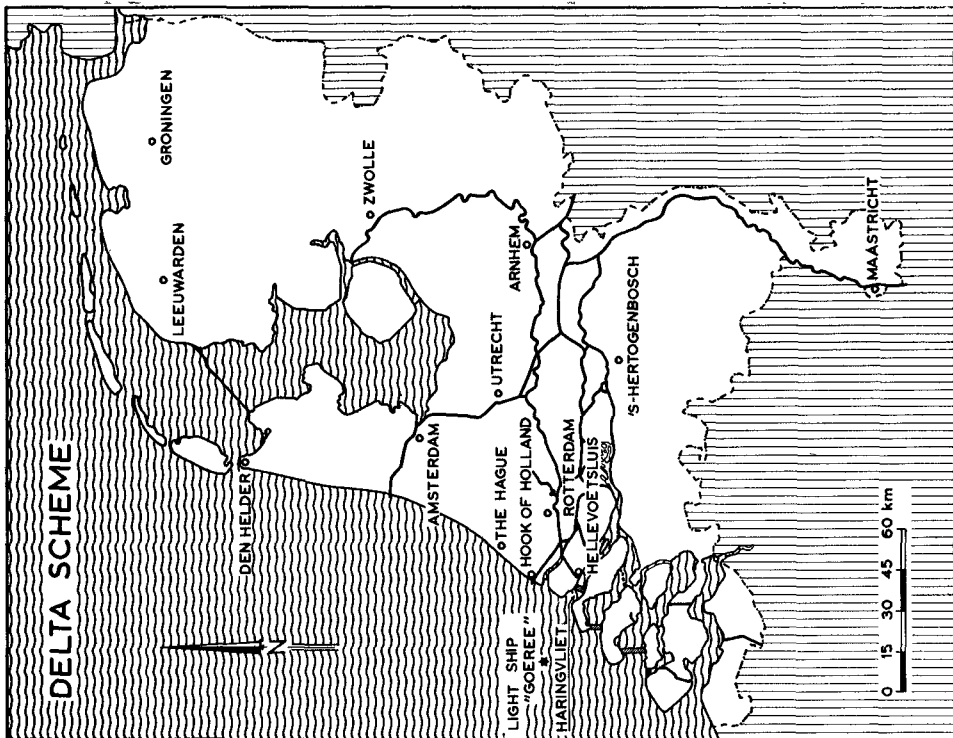


Fig. 1. Delta scheme in the southwest of the Netherlands.

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1.2 Purpose of the study and the data used.

Model tests have been carried out to obtain an insight into the magnitude of the wave-pressures in various situations. These tests showed, that sharp high pressure peaks occur in addition to the pressures caused by the reflecting of the waves, which pressures are quasi-static. As the structure can be compared with a multiple mass-spring system these pressure-peaks may cause the whole construction to vibrate. Wave-attack therefore can be expressed in terms of impact. Moreover, calculations revealed that the impact pressures were critical factors in determining the strength of the structure.

So many model tests were carried out to determine the design and location of the sluices. These tests involved numerous water-levels discharges and waves. Regarding the pressure-peaks a comparative study was made in the model, which led to the structure being designed in such a way that the occurrence of critical impacts was reduced to an acceptable minimum.

As it was impossible to avoid the occurrence of impact pressures entirely it remained necessary to determine a basic load for the structure that takes care of the impact pressures. As it has not yet appeared possible physically to determine a theoretical maximum for the impact pressures, it has to be borne in mind that there is a probability that each pressure measured will be exceeded.

So this paper describes, how the cumulative frequency curve of the impacts for the case mentioned in 1.1 sub a, which served as a basis for determining the basic load was arrived at by a certain combination of laboratory and field observations.

The data used for this purpose were

- a. Results of wave-impact measurements on a model of the sluices. This model, built in accordance with the results of the comparative study, was situated in the wind-flume of the "de Voorst" hydraulic laboratory.
- b. Wave height measurements in the Haringvliet during 1957 and 1958.
- c. Wind-speed measurements on board the lightship Goeree, likewise during 1957 and 1958.
- d. Tidal registrations at Hellevoetsluis from 1920 to 1960.
- e. Wind-force data from the Hook of Holland, likewise from 1920 to 1960.

2.0 THE MODEL.

2.1 Description. Investigation into the reaction of the structure to impact-pressures can be conducted in two ways, dependent on the model chosen:

- a. An elastic, identical scale model, on which deformations are measured directly.
- b. A relatively rigid model on which pressures are measured by means of pressure gauges, the recordings of which are used as a basis for computing the deformation of the

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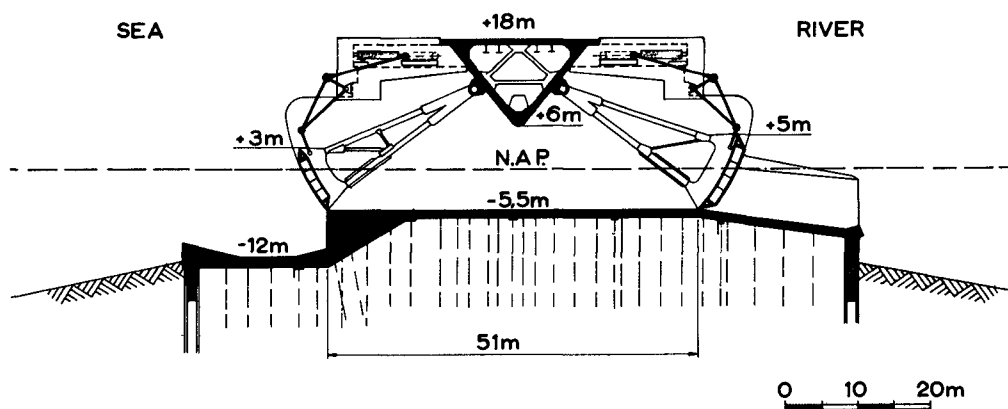


Fig. 3. Cross section of the sluice.

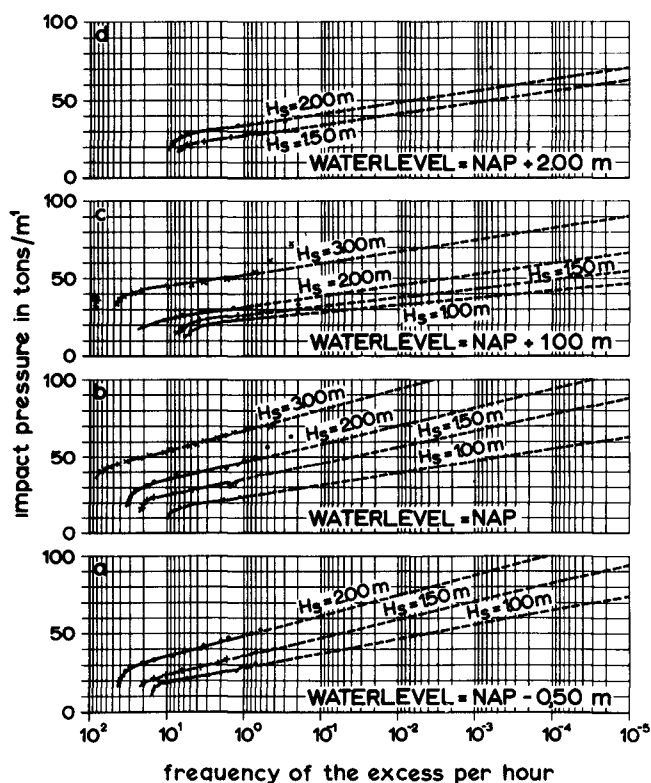


Fig. 4. Frequency curves of the excess per hour of the impact pressures measured in the model for different combinations of water levels and significant wave heights (Wave attack from the sea).

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structure.

Method b was chosen, because the external shape of the construction was already known, but not the exact proportioning of the components. In addition this method linked up with the comparative study that had also been carried out on a rigid model.

The model, which was built of concrete, was on a scale of 1:40. In the model the external geometrical shape of the closed gates, the piers and the floor was reproduced.

Preliminary study showed, that the impact pressures occur simultaneously over the entire height of the gate [lit.2]. The width of the area in which the impacts occurred was variable. A correlation between the breadth of attack and the magnitude of the wave pressure could not be discovered, however. As it is not impossible that the spread found in the breadth of attack depends on the properties of the windflume and because no data were available from the prototype it was assumed, that every wave load would affect the full width of the gate instantaneously. In this connection three pressure gauges of the capacitive type (resonance frequency under water = 1400 Hz) were placed one below the other in the middle of the model gate.

The most important purpose of our study was the determination of the basic load for the Nabla-beam. Because of the special shape of the structure the Nabla-beam only "feels" the natural or forced vibration of the gates caused by a certain mean value of the wave-pressures. To simplify the elaboration, the pressures measured simultaneously were averaged electronically. This average was registered by filming the screen of an oscilloscope connected to the pressure gauges.

2.2 Situations tested in the model. In the model wave pressures were measured in the following situations.

- a. Wave attack from the sea on the closed outer gate.
- b. Wave attack from the sea on the closed inner gate, when the outer gate was open.
- c. Wave attack from the Haringvliet on the closed inner gate.
- d. Wave attack on the closed outer gate, when the inner gate was open.

Cases b and d are not dealt with here, because in them the wave pressures were much lower than in the other situations.

The wave attack from the sea was measured for the following combinations of water-level and significant wave height:

Water-level	Significant wave height in m.		
N.A.P. - 0.50 m	1.00	1.50	and 2.00
N.A.P.	1.00	1.50	2.00 and 3.00
N.A.P. + 1.00 m	1.00	1.50	2.00 and 3.00
N.A.P. + 2.00 m	1.50	and 2.00	

Water-levels, higher than N.A.P. + 2.00 m were not considered, because the comparative study showed that then the pressure

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becomes relatively low.

This is due to a bend in the plating of the gates from N.A.P. + 1.00 m to N.A.P. + 3.00 m, when they are closed. At water-levels lower than N.A.P. - 0.50 m the likelihood that wave motion of any importance will occur is so small that these levels have not been considered, either.

In the model tests for wave attack from the Haringvliet it was assumed, to be unlikely that high water-levels and strong easterly winds would occur simultaneously. Therefore only a water-level of N.A.P. - 0.50 m, combined with significant wave heights of 0.80 m and 1.50 m, was investigated for such a contingency.

In the windflume the waves were generated by means of wind only [lit.3].

2.3 Model results. For each situation tested in the model a frequency curve was obtained, showing how often a certain value of the impact pressure is exceeded on an average in a certain period. The period during which measurements in the model are taken depends on the one hand on the accuracy with which one wishes to determine the frequency curve, but on the other hand on the properties of the windflume and the extent of the investigation. In this case the duration of each measurement was restricted to 40 minutes of model time, corresponding to about 4 hours of prototype time. In this way any pressure, exceeded on an average once per hour, can be determined fairly accurately. Pressures occurring less frequently have been determined by rectilinear extrapolation of the frequency curve of excess on logarithmic paper [fig. 4a t/m d and fig. 5].

In this way the frequencies with which the wave pressures for each combination of water-level with significant wave-height were exceeded, were ascertained.

3.0 FREQUENCIES OF WAVE ATTACK.

3.1 Wave attack from the sea. With the aid of the data obtained from the prototype an attempt has been made to discover at what frequencies certain combinations of water-level and wave height occur. First the registrations of wave recorder C in the Haringvliet [fig.2] were worked out. The period considered ran from 30-1-1957 till 23-1-1959. The recorder measures wave heights as well as water-level variation. So frequency curves of the excess of the significant wave heights could be determined, expressed in percentages of the time the water-level was lower than that stated [fig.6]. The accuracy of these data, however, was considered to be too small because

- a. the recording period of about two years did not form a good enough basis, for extrapolation to very low frequencies.
- b. the registrations of waves generated by winds blowing from directions between west and north could not be separated from the aggregate.

This was necessary, as laboratory tests showed that impulse loads only occur when the waves attack the sluice at

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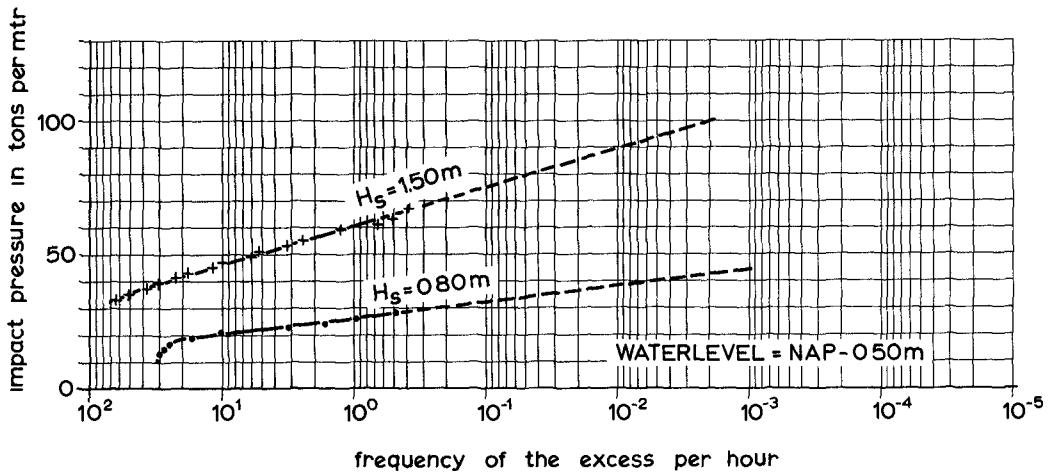


Fig. 5. Frequency curves of the excess per hour of the impact pressures measured in the model for two combinations of water level and significant wave heights (wave attack from the Haringvliet).

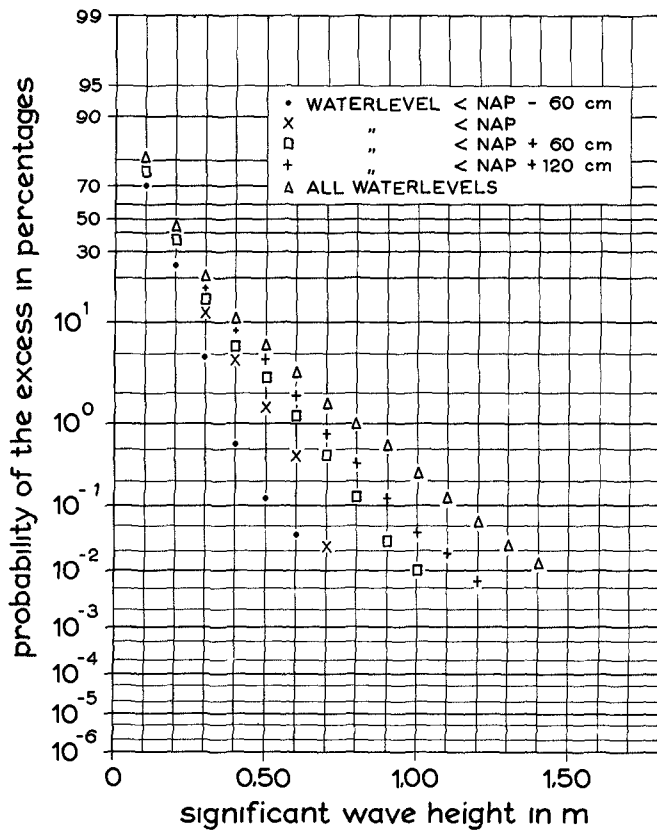


Fig. 6. Frequency curves of the significant wave heights at pole C, in a percentage of the time during which the stated water level is not exceeded.

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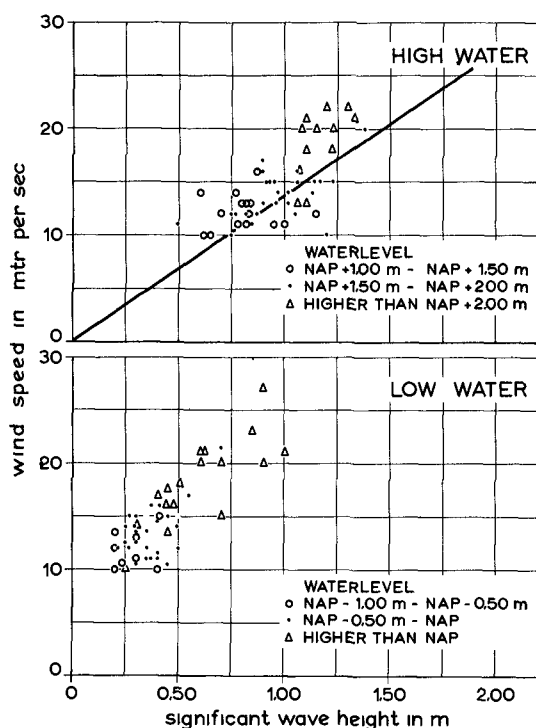


Fig. 7. Correlation between the wind-speeds measured on board the lightship "Goeree" and the significant wave heights measured at pole C.

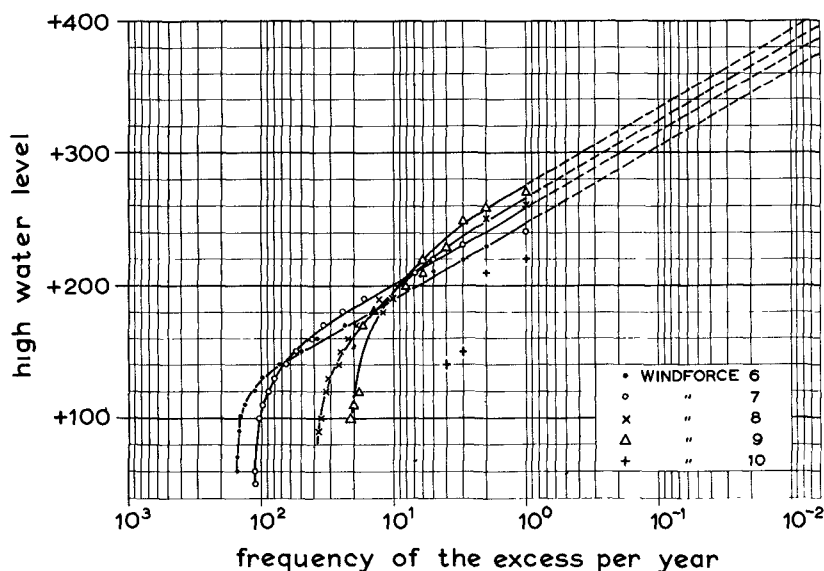


Fig. 8. Frequency curves of the excess per year of the high water levels at Hellevoetsluis, provided the stated windforce occurs.

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right angles. The location of the sluice in the prototype [fig.2] shows that this attack is most likely with north-westerly winds.

With this in mind an attempt was made to obtain the desired frequency-data indirectly.

The wave registration period of two years did appear long enough to establish the relation between the wave heights registered and the corresponding wind-speed and direction. Wind measurements carried out on board the lightship "Goeree" during the two year period were used but only those readings showing

c. wind speeds in excess of 10 m/sec.

d. winds blowing from between west and north.

The wave height and water-level registered at pole C was then checked for each of these readings. The result is given in two diagrams [fig.7a and b], one for the group of water-levels around low water and one for the group of water-levels around high water. The line drawn through the points in the high water diagram, is regarded as the correlation between wind-speed and wave height. This premise is unfavourable for the present situation. In the ultimate situation, however, there may be three reasons for using the line given.

Firstly, the sluices will be closed only during the high water period, secondly, when the sluices are closed tidal currents will cease to influence wave development and thirdly, the Haringvliet will become much deeper locally in future.

Then again, a selection was made from the tides registered at Hellevoetsluis and from the corresponding wind measurements in degrees Beaufort carried out at the Hook of Holland in each of ten years chosen at random from the last forty years. These wind data satisfied the criteria mentioned under c and d. With these figures frequency curves have been drawn from which the frequency of excess of a high water-level, dependent on the wind force could be deduced. It is assumed that these lines will also hold good in the future situation [fig.7].

Now, with the aid of the correlation line for wave height and wind-speed it is possible to determine the frequency with which certain combinations of water-levels and wave heights can be expected on an average. It should be noted, that the data given are the medial values of intervals from a continuous distribution.

The frequencies thus obtained are unfavourable as the whole of the tidal cyclus (approx. $12\frac{1}{2}$ hr) has been considered, even if the wind blew from the northwest for only part of that time.

3.2 Wave attack from the Haringvliet. The wave-measuring poles, in the Haringvliet are all situated on the seaward side of the sluice. Therefore they are on the lee-side of the building-pit for these sluices when the wind is blowing from directions between south and east. In order to get an idea of the wave motion to be expected, ten years of registrations of the wind recorder at Hook of Holland (1949-1958), were selected for directions between 95° and 145° with regard to the north. This selection has been worked out as a frequency curve of the excess of the wind-speeds. From these, with the aid of the Bretschneider

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diagrams for the generation of waves in deep water frequency curves of the excess of the wave heights have been plotted [fig.9]. The results of the wave height calculations, for which a certain fetch was adopted, have been checked by incidental visual wave height observations in the field. Although the visual observations gave values lower than those calculated, the latter have been taken into account, also in view of the future greater depth of the Haringvliet, on this landward side of the sluice as well.

The conditions for the occurrence of high water-levels on the Haringvliet lake are well known. The probability of these high water-levels coinciding with stormy easterly winds is negligible. The average daily level will fluctuate between N.A.P. and N.A.P. + 0,50 m. By using a water-level of N.A.P. - 0.50 m in the model tests an extra margin of safety has been provided.

4.0 THE CUMULATIVE FREQUENCY CURVE OF THE PRESSURES.

4.1 For wave attack from the sea. The results of the model-investigation shown in fig.4a to d combined with the data from the field described in 3.1, give the total average frequency of excess of the wave pressures per year. This is obtained by multiplying the frequencies of excess of the pressures for each combination of wave height and water-level with the frequencies in hours per year of the occurrence of these combinations and adding up of the results. The year-frequency curve of the pressures can be found by carrying out this operation for a number of pressures and plotting the values obtained on probability paper. It will appear as a straight line on logarithmic paper, which has been extrapolated rectilinearly for the higher pressures, viz., the lower frequencies.

The operation described applies to one sluice opening only. For considering any opening, the frequencies found will have to be multiplied by 17, as the whole construction has 17 openings. In this connection it is assumed, that statistically speaking the circumstances are identical for all the sluice openings [fig.10].

The curve thus obtained gives the frequency of excess of the value of the pressure-peak as measured in the rigid model. As already stated these pressure-peaks possess the characteristics of impulses. Therefore an impact-factor will have to be introduced into the static calculation of the stresses in the structure. The magnitude of this factor depends on the part of the structure considered [lit.1 and 2] and other factors. With the results of numerous calculations as a basis this impact factor has been fixed at 2 for the Nabla-beam and 1.35 for the gate and the levers.

4.2. For wave attack from the Haringvliet. This was dealt with in the same way as the foregoing. The data used were those given in fig.5 and fig.9.

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5.0 THE BASIC LOAD.

5.1 The frequency adopted. For partly subjective economic considerations a basic load for the Nabla-beam was chosen having an average probability of excess per year of 5.10^{-5} . The load thus found must be taken as the breaking load of the Nabla-beam. This also means that there is a greater probability of the occurrence of cracks, requiring repair and a still greater probability of cracks occurring that will close again because of prestressing.

If it is assumed, that the construction will be out of date in 200 years and will therefore have to be replaced by a new one, the frequency chosen means a one percent probability that the breaking load of any Nabla-beam will be exceeded during the lifetime of the structure.

The main difficulty was that the phenomena we dealt with were of a statistical nature the laws governing which are as yet obscure. To obtain an insight into the possible spread of pressures and wave height do not follow the logarithmic distribution described here the same process was applied to extrapolations on different kinds of probability paper.

The actual choice of the basic load was made from a comparative study of the frequency curves thus obtained.

6.0 COMMENTARY

6.1 Consideration of the doubtful points. Excessive extrapolation of the frequency curves of the excess of the wave-pressures is required when determining the basic load.

This is sufficient reason for considering the results with a certain reserve. Moreover it should be borne in mind that a satisfactory answer to the following questions has not yet been given:

a. Over what width will the wave attack affect the structure? For safety's sake it was assumed that the wave attack will affect the whole width of one sluice-opening instantaneously. An insight into the degree of safety will only be obtained when enough is known about the breadth and shape of the wave-crests in the prototype.

b. What correlation exists between the magnitude of the pressure-peak and the impact factor. From calculations it is known, that the impact factor depends largely upon the time in which the pressure rises from zero to its maximum [lit.2].

Preliminary tests showed no direct correlation; however, the investigation was limited. For safety's sake therefore the impact factors mentioned in 4.1 were introduced.

c. The shape of the waves in the laboratory and in the prototype. As already mentioned the waves in the laboratory were generated by wind [lit.3]. As the fetch in the windflume was not to scale, the wind-speed was exaggerated. This caused the wave height distribution to show a flattening especially for the higher waves, which has not yet been observed in the prototype.

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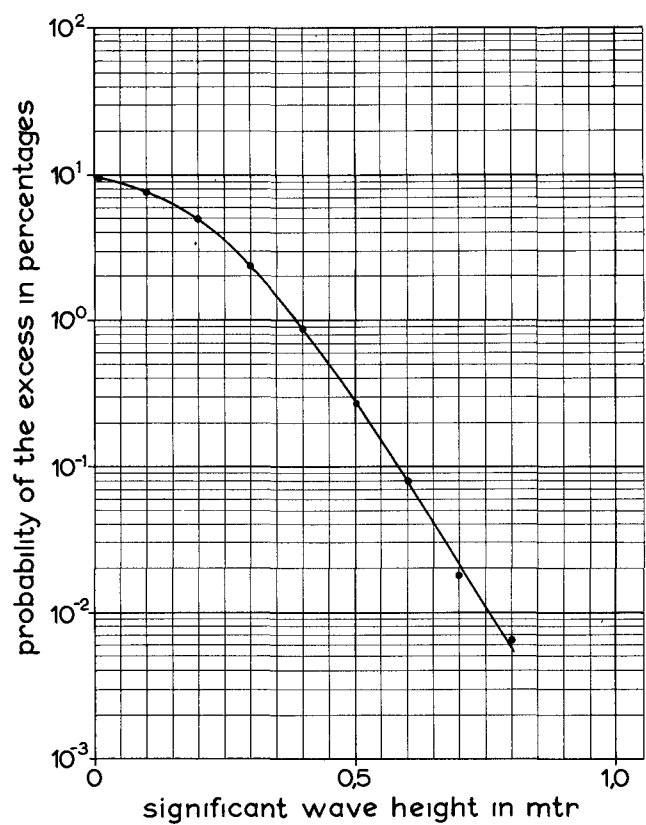


Fig. 9. Probability of the excess of the significant wave heights on the Haringvliet, provided the wind blows from directions between 90° en 145° with respect to the north.

- a WAVE-ATTACK FROM THE SEA
- b WAVE-ATTACK FROM THE HARINGVLIET

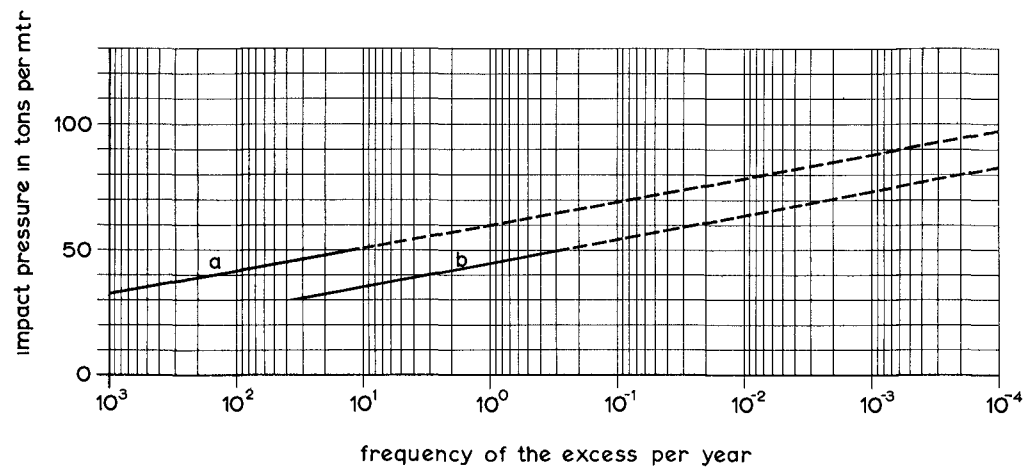


Fig. 10. Cumulative annual frequency-curves of wave-pressures .

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The wave recorders placed on top of the wave-measuring poles, however, do not yet permit the working out of any series longer than 100 to 150 waves, so that information about the "tail" of the distribution is still very scarce. For comparison it is noted, that the distributions in the windflume are determined from series of 500 to 1500 waves.

Neither has it appeared possible so far to get an adequate idea of the extent of the agreement between prototype and wind-flume as regards the shape of the wave. Determination of the shape is very difficult, as the registration is a height/time diagram which has to be transformed to a height/length diagram via an idealised formula for wave celerity. In the windflume this can be done with comparative ease; however, for the prototype, with its freakish bottom configuration no satisfactory method has been found yet.

d. To what extent does the wave registration of pole C represent the ultimate situation and the whole of the Haringvliet. Only the future can answer this question, as the bottom configuration will undergo a considerable change. As already stated it was assumed for the present that the waves will become somewhat higher than they are now. This also depends on the available fetch, which may change because of banks that may be formed in the mouth of the Haringvliet.

6.2 Summary. The basic load of the Haringvliet-sluices was arrived at by accepting a certain probability of excess of this load. The calculation of this probability was based on a combination of observations in the laboratory and in the field. An outline has also been given of the doubtful points, which have been circumvented by introducing a safety factor. The basic load for the lifting mechanism was determined in a similar manner. A difficulty here arose because of our ignorance of the water-level variations in the Haringvliet lake of the future.

This problem was solved by tracing the course of the water-level variations during ten years chosen at random from the last forty years, supposing the sluices had already been present in that period. So the points of time of equal water-level on either side of the sluices (criterion for the opening of the gates) as well as the slack water period (criterion for the closing of the gates) could be determined, together with the corresponding water-levels.

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