MODEL INVESTIGATIONS OF WIND-WAVE FORCES

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OBJECT OF THE INVESTIGATIONS

The load caused by wave attack had to be determined in connection with the design of a steel structure located adjaoent to the open sea. This load had to be expressed in such a way that the designer:-
a) Could base the dimensioning of his structure on a consideration of the probability of failure
b) Would have at his disposal data from wnich could be derived the number of stress alternations together with the distribution of their amplitudes during the lifetime of the structure.

The data required for this is:-
1) The probability of occurrence of significant waves of any given magnitude
2) The load spectra associated with the wave spectra of significant waves of any given magnitude.

With regard to 1) a prognosis was made by the "Rijkswaterstaat" from wave records and meteorological conditions at the prototype location.[*] The load spectra were evaluated in the laboratory "de Voorst" by exposing a small scale model to wind generated waves.

In the following, a review is given of the preliminary studies made to compare the wave characteristics in model and prototype, the investigation of wave form and the forces exerted on the structure, and the representation of the final results.

PRELIMINARY STUDY OF THE WAVE CHARACTERISTICS

The investigations have been carried out in a wind-flume having a length of 100 m, a width of 4 m and a height from bottom to ceiling of 2 m. The maximum waterdepth can be 0.75 m and the maximum wind velocity 12 m/sec. At the end of the flume a wave absorber is available.
From consideration of the prototype dimensions the scale for the model was chosen to be 40. This yielded a water depth in the wind-flume of 0.625 m for which the wave characteristics were of interest. In addition some of the characteristics were also observed for a depth of 0.25 m.
The waves were generated by wind, by a mechanical wave generator or by the combination of the two. The wave absorber was used.
Wave heights were recorded continuously. Irregular waves were characterized by the significant wave height (H_{sign}) which was deduced from a series of 200 waves. The regular waves (i.e. the mechani-

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[*] This subject is covered in Venis (1960).

cally generated waves) were characterized by the average value of the smallest and largest wave height within half a wave length (H_{swell}). The accuracy of the wave heights for this study are

$$H_{sign} \quad 10\%^{*}$$
$$H_{swell} \quad 6\%$$

WIND-GENERATED WAVES

To relate the growth of the wave to the fetch (F) and wind velocity (W_z), the wave height was measured at every 15 m along the flume (figure 1 and 2). From the wave recordings at a fetch of 90 m, wave height distribution curves were made (figure 3 and 4). This data was compared with prototype curves (Table 1) and agreed quite well, taking account of the scatter in the prototype curves themselves (Paape 1960, figure 3). The model results in general show a tendency to flatten the slope of the curve at high wind velocities. The average period ($\bar{T}$) was determined and related to the fetch (figure 5).

To verify whether the variables involved in the phenomenon are reproducable to scale, the data of the model was expressed in the dimensionless quantities:

$$g\,\bar{T}\,/\,2\,\pi\,W_o \; , \; g\,H_{sign}\,/\,W_o^{2} \text{ and } gr\,/\,W_o^{2}$$
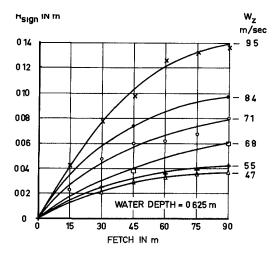
which are plotted against $g\,F\,/\,W_o^{2}$ (figure 6).

The wind velocity W_z, measured at height z (in the flume 0.25 m), is reduced to W_o at height z_o according to $W_z^{2}\,/\,g\,z_o = 5.2$

The measuring points of $g\,H_{sign}\,/\,W_o^{2}$ vs $g\,F\,/\,W_o^{2}$ show a reasonable consistency and the scatter is less than that of prototype data. A comparison has been made with the graphs of Bretschneider (1958) and Thijsse (1948) by superimposing their curves (unfortunately the region concerned is mainly based on model data).

From the overall impression it could be decided that, since the dimensionless parameters $g\,F\,/\,W_o^{2}$ and $g\,H_{sign}\,/\,W_o^{2}$ are linear in fetch and wave height and quadratic in wind velocity, the Froude law was applicable.

Some values of $g\,\bar{T}\,/\,2\,\pi\,W_o$ were also calculated and compared with the graph $g\,\bar{T}\,/\,2\,\pi\,W_o$ vs $g\,F\,/\,W_o^{2}$ of Bretschneider (1958). The model shows a 15% too high value of the period for these points (figure 6). The same points were expressed in terms of $g\,r\,/\,W_o^{2}$, in which $r = L\,/\,2\,\pi$ (L is the wave length), and compared with the Thijsse graph $g\,r\,/\,W_o^{2}$ vs $g\,F\,/\,W_o^{2}$. The curve was not fitted too well (figure 6).

Wind velocity distributions in the vertical were also measured (figure 7) and compared with the prototype data from Abbotts Lagoon given by Johnson (1950). Figure 8 shows a logarithmical plot of this. The curve obtained for a water depth 0.625 m shows a deviation from that for a depth of 0.25 m. The agreement with the distribution of the data of Abbotts Lagoon is good.

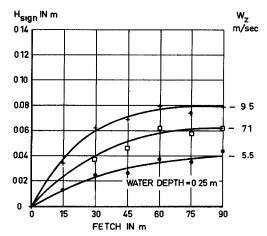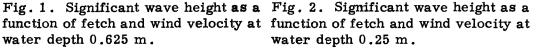[*] H_{sign} deduced from 500 waves: accuracy 5%
1000 waves: 2%.

Fig. 1. Significant wave height as a function of fetch and wind velocity at water depth 0.625 m.

Fig. 2. Significant wave height as a function of fetch and wind velocity at water depth 0.25 m.
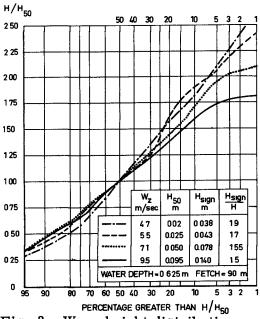

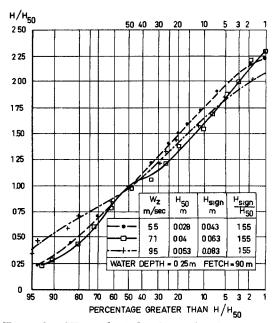Fig. 3. Wave height distribution curves at fetch 90 m and water depth 0.625 m.

Fig. 4. Wave height distribution curves at fetch 90 m and water depth 0.25 m.
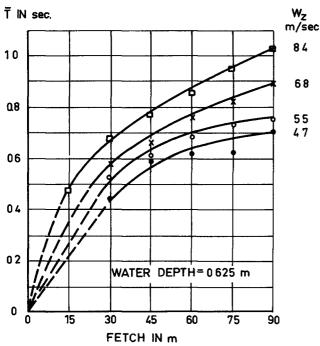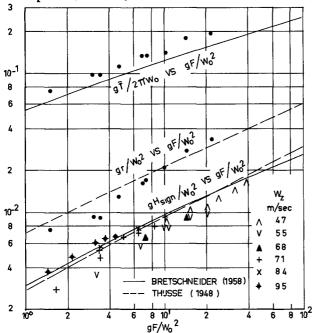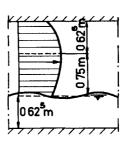
Fig. 5. Average wave period as a function of fetch and wind velocity at water depth 0.625 m.

WIND VELOCITY
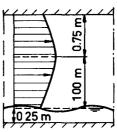DISTRIBUTION
IN WIND FLUME

Fig. 7. Wind velocity distribution in windflume at water depth 0.625 m and 0.25 m.
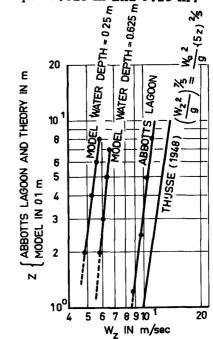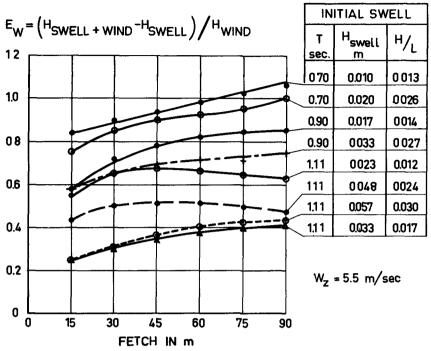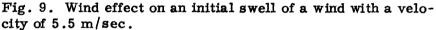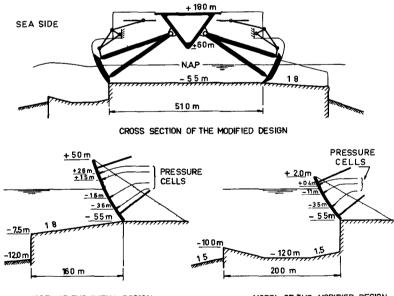

Fig. 6. Relationship between the dimensionless parameters and comparison with curves of Bretschneider and Thijsse at water depth 0.625 m.

Fig. 8. Comparison of wind velocity distribution curves from model and nature.

$$E_W = \left(H_{SWELL + WIND} - H_{SWELL}\right) / H_{WIND}$$

INITIAL SWELL		
T sec.	H_{swell} m	H/L
0.70	0.010	0.013
0.70	0.020	0.026
0.90	0.017	0.014
0.90	0.033	0.027
1.11	0.023	0.012
1.11	0.048	0.024
1.11	0.057	0.030
1.11	0.033	0.017

W_z = 5.5 m/sec

FETCH IN m

Fig. 9. Wind effect on an initial swell of a wind with a velocity of 5.5 m/sec.

SEA SIDE

+180 m

±60m

NAP

-55m   18

510 m

CROSS SECTION OF THE MODIFIED DESIGN

+50 m
+26m
+15m
-16m
-36m
-55m
-7.5m   18
-12.0m
160 m

PRESSURE CELLS

MODEL OF THE INITIAL DESIGN

PRESSURE CELLS
+2.0m
+0.4m
-1.1m
-3.5m
-55m
-10.0m
-120m   1.5
15
200 m

MODEL OF THE MODIFIED DESIGN

Fig. 10. Cross section of the Haringvliet sluice. Models of original and modified designs. (Prototype dimensions).

WIND-STRENGTHENED SWELL.

This study of wave forces required a knowledge of the influence of the wind velocity on an initial swell in the same direction. For this purpose a series of investigations were carried out with a constant wind velocity of 5.5 m/sec on a variable swell of small steepness.

Every 15 m in the flume the significant wave height was determined.

For characterizing the effect of the wind the quantity E_w was introduced and expressed as follows:

$$E_w = (H_{swell + wind} - H_{swell}) / H_{wind}.$$

The parameter E_w is the ratio of the growth of swell due to the wind and the growth of a pure wind-generated wave.

In figure 9 for some periods and wave heights the ratios deduced from adjusted curves, at equal intervals along the flume are given. Although the accuracy of these points is not very great the results show a tendency for longer periods to be less affected by the wind than shorter ones. The results did not show a systematic relationship with respect to the wave height.

Tabel 1 resumes some of the data on the wave height distribution.

As the forces were found to be the most severe for pure wind-generated waves this study was discontinued.

STUDY OF THE WAVE FORCES

This investigation was carried out in the windflume described above. The model of the structure (a gate of the Haringvliet sluice) was built to a scale 40. As the study progressed it proved to be necessary to modify the original design. In figure 10 the cross section of the modified design is given. The preliminary studies described hereafter were made for the original design. The outline of this model is also given in figure 10.

Experiments showed that some of the waves approaching the structure may be reflected smoothly, while others strike against it. The pressures due to the reflection or the impact are measured by pressure cells[*]. By calibrating those instruments the pressure is expressed as a height of water. If it is assumed that Froude law is valid for these phenomena, then the pressure for the prototype can be expressed in ton/m^2.
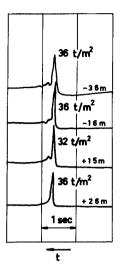
Studies of the mechanism of wave impact and of the wave conditions causing them were made.

PRELIMINARY STUDIES

Extent of the impact forces – In the model of the original design it was found that the impact forces occurred simultaneously over the vertical. Examples are given in figure 11.
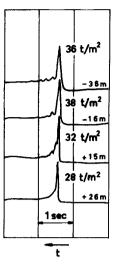
[*] The resonance frequency of these pick-ups had to be high because of the fast pressure rise of the impacts. In submerged condition it amounted 1400 Hz. The shortest time of pressure rise measured in the model was 1/600 sec, which equals 1/100 sec in prototype.

771

GATE HEIGHT : NAP+45 m
WATER LEVEL: NAP

WIND STRENGTHENED
SWELL

H_{sign} = 3.4 m
T = 50 sec
W_{10} = 30 m/sec

Fig. 11. Records of impact forces in a vertical (original design. Prototype quantities).

Fig. 12. Relationship between tan ϕ an probability of the occurrence of an imp

IMPACT   NO IMPACT

$\dfrac{H^2}{d\frac{1}{2}L}$ = 1 2 $\dfrac{H^2}{d\frac{1}{2}L}$ = 0 7

IMPACT   NO IMPACT

$\dfrac{H^2}{d\frac{1}{2}L}$ = 0 9 $\dfrac{H^2}{d\frac{1}{2}L}$ = 0 6

H = WAVE HEIGHT
L = WAVE LENGHT
d = WATER DEPTH IN WAVE TROUGH
TIME INTERVAL BETWEEN THE LINES IS 0.4 sec

Fig. 13. Impact related to wave form (Prototype quantities).

Table 1

Location	initial swell		wind	wind gen. wave wind etr. swell			spectrum			
	$\overline{T}$ sec.	Hswell m	m/sec.	$\overline{T}$ sec.	Hsign m	Ew	H_{50} m	H_2/H_{50}	H_{10}/H_{50}	H_{20}/H_{50}
Haringvliet wave recorder C.	–	–	20 20 18	–	–	–	0.4 0.5 0.5	2.4 2.2 2.2	1.9 1.8 1.7	1.6 1.5 1.5
North Sea wave recorder Katwijk	–	–	–	–	–	–	1.9 1.7 1.5 1.2	1.9 1.9 1.9 2.1	1.6 1.5 1.6 1.7	1.4 1.4 1.4 1.4
wind flume water depth 0.625 m. Fetch 90 m.	– – – 0.70 0.79 0.90 0.90 1.11 1.11	– – – 0.010 0.028 0.033 0.048 0.033 0.055	5.5 7.1 9.5 5.5 5.5 5.5 5.5 5.5 5.5	0.74 0.89 1.11 0.82 0.95 0.97 1.01 1.11 1.11	0.043 0.078 0.140 0.058 0.067 0.065 0.095 0.050 0.092	– – – 1.12 0.94 0.77 1.06 0.41 0.88	0.025 0.050 0.095 0.038 0.045 0.040 0.065 0.045 0.085	2.3 2.1 1.8 1.9 1.9 2.2 1.9 1.3 1.2	1.9 1.7 1.6 1.6 1.7 1.8 1.7 1.2 1.2	1.6 1.4 1.4 1.4 1.4 1.4 1.4 1.2 1.1
wind flume water depth 0.25 m. Fetch 90 m.	– – –	– – –	5.5 7.1 9.5	0.71 0.92 1.03	0.043 0.063 0.080	– – –	0.025 0.043 0.050	2.2 2.1 2.1	1.8 1.7 1.7	1.5 1.4 1.5

Fig. 14. Load as a function of wave height (Prototype quantities).

Fig. 15. Frequency curves of the load, associated with significant wave heights (Prototype quantities).

773

The extent in the width depends on the length of the wave crest and its straightness. In the model for wind-strengthened swell and for wind-generated waves, with wave lengths from 35 to 55 m in prototype, in no case was there found an equally high impact pressure over the full width of the gate (length 58.5 m). From nature no data are available.

Air content of the water – As it was thought that the air content of the water could possibly influence the results, an increase of the air in the water was made artificially.
Records of the pressure history for the conditions of the normal and the high air content did not show a marked difference. This eliminates within a certain range the difficulty of the correct reproduction of the compressibility, due to bubbles in the water, in the model.

Rigidity of the structure – In the first stage of the investigations the measurements of the impacts were carried out in a rigid model representing the structure. To verify whether a receding movement of the structure under influence of the load had a reducing effect on the impact, the model was provided with gates allowing a bending of several decimeters in prototype. In those gates the pressure piok-ups were mounted. The wave impact forces were still measured but the pressure history became more irrigular.
For one gate also the pressure at the inner side was measured. The impact at the outer side was hardly noticable at the inner side and it may be concluded that the water at the inner side does not react noticeably in taking up some of the impact.

Form of the wave – The first measurements did not show a clear correlation between the magnitude of the wave impact pressure and the wave height. When the waves are generated mechanically a clapotis is formed due to the obstruction of the model and no impact phenomenon is observed. When wind is added to the mechanical wave generation, and foaming crests come into being, impacts occur. The deformation of the wave by the wind appeared to be of essential importance. To correlate the wave form and the wave impact pressures, the record of a wave height meter located 0.25 m in front of the model was compared with simultaneously recorded pressures on the structure.
As a criterion the tangent (tan φ) to the angle (φ) between the horizontal and the tangent to the surface at the point of inflection on the front face of the wave was used. A frequency distribution of tan φ was made and the probability of the occurrence of an impact was determined (figure 12). This figure shows clearly that the probability of the occurrence of an impact increases strongly with the increase of tan φ.
No direct relationship between the magnitude of tan φ and the impact pressure was found.
By taking a moving picture, on which the water movement in front of the model was filmed simultaneously with an oscillograph showing the pressures exerted, a relationship could be established for this special case in which the wave height, the wave steepness, and the water depth at the trough of the wave preceeding the impact were in-

corporated (figure 13).

It was found that, when H^2/l d exceeded a certain value a wave impact occurred. There was no direct relationship between this number and the magnitude of the impact pressure.

Impulse measurements - The impact forces can be deduced from the impulse - momentum equation,

$$P.dt = d\ (m\ v).$$

The model investigation showed that the original design gave rise to very high wave impact forces (P) because of:
1) High velocity v: the unstable waves of a wave train tend to break on the shallow sill in front of the gate, causing water velocity to approach the propagation velocity of the wave.
2) Large mass m: the inclined overhanging position of the gate tends to maximise the mass of water which is effective in causing impact.
3) Small interval dt: the form and orientation of the front face of an approaching wave can be such that it becomes sensibly parallel to the surface of the gate so that the full impact occurs within a very short interval.

In order to overcome these difficulties the design was modified as shown in figure 10. From further measurements it appeared that for this new arrangement impact forces due to the instability of some of the waves of a wave train still occurred.

The quantity $\int P$ dt during pressure rise and during the total impact was measured at three places in a vertical line on the gate front. The pressure record of the impact was divided into its dynamic and static parts.

Some typical data (in prototype quantities) from a series of 2740 waves with 174 impacts are presented in the following table.

$H_{swell} = 1.0$ m $T_{swell} = 5.0$ m Water level NAP - 0.5 m	$W_{10} = 36$ m/sec $F^{10} = 3600$ m		$H_{sign} = 3.0$ m $T_{sign} = 5.0$ sec	
measuring point	at NAP 0.4 m	at NAP - 3.5 m	unit	
duration of pressure rise	0.1-0.3	0.8-1.2	sec	
duration of impact	0.8-1.1	1.8-2.3	sec	
$\int P$ dt during pressure rise	0.4-1.6	3-5	t sec/m^2	
$\int P$ dt during impact	2-4	7-11	t sec/m^2	
max. dynamic pressure	19.8	12.0	t/m^2	
max. impact pressure	21.2	16.1	t/m^2	

DETERMINATION OF THE LOAD

These investigations were carried out in a rigid model representing the modified design according to figure 10. In the model three pressure cells were mounted in a vertical line.

The forces exerted by the waves on the structure were recorded as the arithmetic averages of the signals of the three pressure pick-

ups. By applying Froude law and multiplying by the height of the gate, the load for the structure per m' (P_t) was obtained from the record. It was expressed in ton/m'.

Mutual comparison of the varied conditions was made by deriving the load (P_{t2}) from the maximum value of the averaged pressure of a record taken over a length of time of two hours in prototype (approximately 1200 waves).

Load due to swell - The mechanically generated waves experienced a smooth total reflection. No impact forces occurred.

Load due to the wind-generated waves - For this condition impact forces occurred. The load P_{t2} was plotted as a function of the significant wave height* (figure 14). It showed a linear relationship.

In addition to this there have been experiments with gusts of wind. It proved that a gust quickly creates a wave form causing impact pressures. In this respect the initial wind velocity on which the gust is superimposed and the duration of the gust are of importance. The study was not continued, due to lack of prototype data.

Load due to wind-strengthened swell - For this investigation the wind velocity was 35 m/sec and the initial swell was varied from 4 to 10 seconds, with wave heights from 0.4 to 3.0 m. The fetch was 3600 m.

For these conditions also, the load was plotted in figure 14. A direct relationship was not found. It is clear that the significant height in itself is no measure for the force exerted. As was stated before the wave form is very important. When in figure 14 an upper envelope is drawn it shows that the most dangerous combination of swell and wind does not exceed the forces exerted by waves generated by wind alone. It may be concluded from this that the steepest wave fronts or the largest steepness appears to occur with wind generated waves.

Not much is known of the wave form in nature. For this reason the final study has been carried out with wind generated waves only, yielding the most severe conditions.

PRESENTATION OF THE RESULTS

For the design of the structure, based on the probability of failure, the expectation of any load had to be determined.

For this purpose a series of statistical distributions of forces was made, each distribution applying to one value of the significant wave height. The wave conditions were those generated by pure wind, which was found to represent the most unfavorable state. Also the water level in front of the gate was varied and the effect of the height of the gate was included in the investigation.

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* Wave heights are related to measurements with a wave absorber in the flume.

Figure 15 shows the frequency curves of the load at several significant wave heights. Any significant wave height by itself is also associated with a certain frequency of occurrence. The combination of the two yields the probability of occurrence of each load as required by the designer.

Although a physical limit to the phenomenon may be expected it cannot be logically deduced. From a measurement of duration of 40 hours (prototype) linear extrapolation proved to be still possible.

ACKNOWLEDGMENT

The data given above forms part of a study included in the reports "Golfaanval Haringvlietsluizen", Volume I and II, March 1960, which were compiled by M.A. Aartsen, a former engineer of the Delft Hydraulics Laboratory. The work was carried out at the "de Voorst" Laboratory, by order of R.W.S., and was under the guidance of Mr. Aartsen.

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