CHAPTER 138

CYLINDRICAL CAISSON BREAKWATER: STRAIN MODEL TESTS

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ABSTRACT

The distribution of *shock pressures on a vertical face breakwater* is so random from one shock to the next that the determination of the *internal forces in a thin-walled reinforced-concrete structure* is difficult even when a large number of pressure cells is used in a model test. Therefore a technique has been developed that allows the direct determination of the internal stresses by means of strain gauges.

The new technique has been applied to a breakwater consisting of a series of cylindrical caissons. The results show that a strain gauge model is more advantageous than a pressure cell model because of its simpler data analysis and better determination of the quantities required for design.

1. INTRODUCTION

It is well known that *vertical face breakwaters* may be exposed to heavy shock forces of short duration. Such impacts have often resulted in the *sliding* of individual breakwater caissons, see for example Refs. 2 and 8, where the slidings/nonslidings of breakwaters in Japan have been used for the evaluation of various wave pressure formulae. Because of the large inertia of the masses involved in rotation, it is much less frequent that breakwaters have been damaged due to insufficient stability against *overturning*. A third effect of the shock pressures is the production of *internal stresses* in thin-walled caissons of reinforced concrete.

A distinction may be made between *ventilated*, *hammer* and *compression shocks*, see Ref. 5, 6 or 7. While it is advisable by the design of the cross section to avoid hammer and compression shocks, there are many cases where ventilated shocks are inevitable. The development of a ventilated shock is described in Refs. 6 and 7.

The influence of the *geometry of the front face* on the magnitude of the shock forces is described in Refs. 6 and 7, the latter giving ranges of dimensionless coefficients in the formulae for forces and pressures.

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The various *types of caissons* (rectangular, cylindrical, diaphragm, multicellular) are discussed in Ref. 7.

In comparison with traditional rectangular caissons, breakwaters of caissons containing *cylindrical, reinforced-concrete shells* are highly economical structures for three reasons:

(a) The cylindrical shell is better capable of resisting the wave forces and the pressure from sand fill, resulting in smaller thickness of the outer walls, less cracking, less weight and less steel.

(b) Transverse walls can be given a larger spacing (diaphragm type) or completely eliminated (cylindrical type), depending upon the method of construction (Ref. 7), thus resulting in smaller weight of the caisson, as well as less concrete and steel.

(c) The stability is improved because of the delay in development of shock pressures from the front generatrix to the reëntrant corner, as described in Refs. 6 and 7. This effect is pronounced in the cylindrical type, but of little importance in the diaphragm type.

Investigations during the last 16 years of breakwaters containing cylindrical shells have also led to a series of *developments in model testing techniques*:

(1) The first application of large-diameter cylindrical shells to breakwaters was for the Hanstholm harbour, Denmark, for which the model tests took place in 1960 (Ref. 4). While, previously, the forces on vertical face breakwaters had been measured by pressure cells only, the stability of the Hanstholm breakwater was investigated by measuring on the model caisson the *total horizontal and vertical forces*, as well as the overturning moments. In subsequent investigations a three-component strain gauge dynamometer has been used. For the Hanstholm breakwater shock forces could be almost entirely eliminated by the introduction of a *top face sloping* 30° with the horizontal, starting from about still water level (Ref. 4), because at Hanstholm there is little variation of the water level for all western gales.

(2) The next step in the development of testing technique was a method for *generation of wave trains of natural shape* directly from wave records, thus producing shock pressures in a flume only 15 m long. At the same time a method was devised for the determination of the *re-reflection from the generator* by measuring the energy in the flume during the test. These methods, which are described in Ref. 3, were first used in 1971 in tests for the breakwater of the Brighton marina, U.K. (Refs. 1 and 7).

(3) Because of the random and complex distribution of shock pressures on cylindrical caisson breakwaters, as much as *20 pressure cells* have been applied in model tests (Refs. 6 and 7), with a view to the determination by computer programs of the internal forces in the shells.

(4) The stability and strength designs require that the forces and pressures measured in model tests be extrapolated to rare situations as statistical parameters. A method of *statistical analysis* combining the statistics of the wave climate with the statistical distributions of the model data has been developed.

Because of the difficulties of extrapolating complex and random pressure distributions to rare situations, Danish Hydraulic Institute felt the necessity of developing a model technique by which the *strains* (and hence also the stresses), induced in the cylindrical shell by the wave pressures, are *directly determined by means of strain gauges.*

This technique was developed in cooperation with the Institute of Hydrodynamics and Hydraulic Engineering and the Structural Research Laboratory, Technical University of Denmark.

The present paper gives a description of this technique, as well as a comparison between the directly measured internal forces and the forces calculated by a computer program from pressure cell recordings.

2. STRAIN GAUGE MODEL

The model (Figs. 1 and 2) is geometrically similar to the prototype. The model material is *araldite* with E = 3.2 GN/m^2, which is about twice as much as required for dynamic similarity at a model scale 1:20. This deviation from dynamic similarity is without significance, however, because the inertial forces of the prototype concrete shell are negligible even for the fastest shocks.

Fig. 1 Strain gauge model placed in a 600 mm wide flume

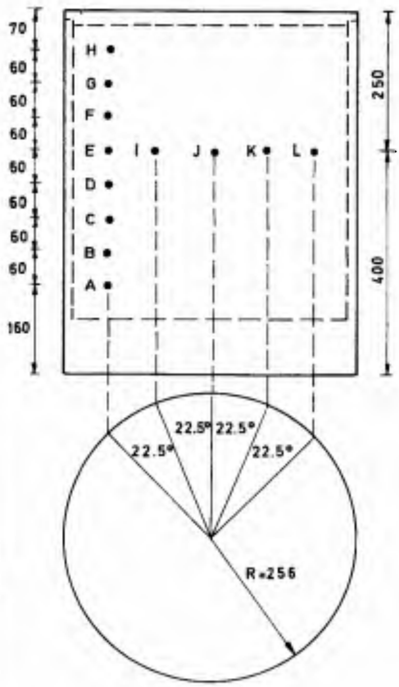

Fig. 2 Strain gauge model (dimensions in mm)

The determination of the *complete stress situation at one point* of the shell requires two rosettes (outside and inside), each containing 3 strain gauges. The rosettes on the outside are embedded so as to give a minimum insulation of 100 MΩ.

The rosettes A-H (Fig. 2) are placed along one vertical line on the cylinder. In order to determine the full stress distribution in the shell, the cylinder is rotated 22.5° between one test series and the next one. For this reason rosettes I-L are mounted at the same level as E in order to give reference time values for the initiation of a shock pressure at the front generatrix.

Normal forces are considered positive as tension in the shell. Bending moments are positive when there is tension on the inside.

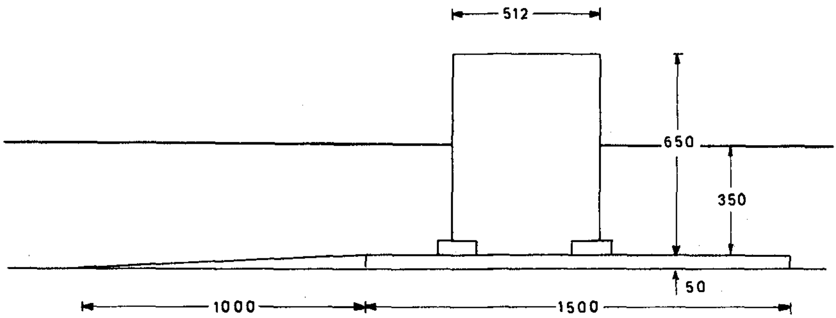

Fig. 3 Longitudinal section of the cylinder in the flume

The model has a diameter of 512 mm, a height of 650 mm, and a thickness of 15 mm. It is placed in 350 mm of water (Fig. 3) inside a 600 mm wide flume that is part of a 4 m wide flume (Fig. 4), in which the undisturbed generated waves can be recorded.

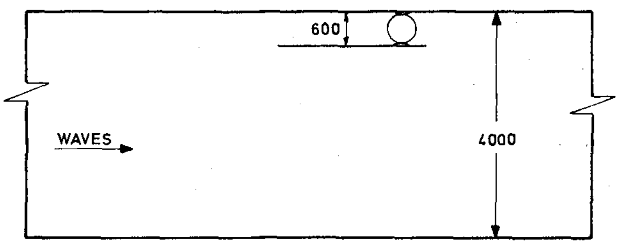

Fig. 4 Plan showing cylinder in the 600 mm flume inside the 4 m flume

The median values of the strains recorded in 10 repeated tests at a specified time for a specific shock wave have been calculated for comparison with the stresses computed from the pressure cell model mentioned below. The *standard deviation* is estimated to be about 10%.

3. PRESSURE CELL MODEL

The pressure cell model has the same diameter and height as the strain gauge model. 16 or 14 pressure cells are used. Fig. 5 shows to the left the arrangement of cells for the measurement of shock pressures around the front generatrix and, to the right, the arrangement determining shock pressures in the corner between two caissons.

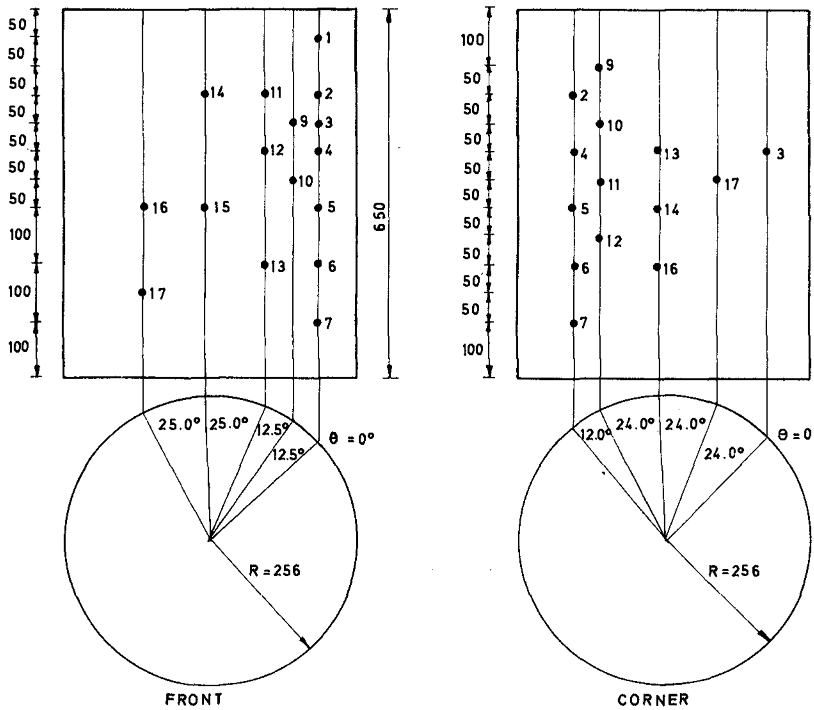

Fig. 5 Pressure cell model

For 10 repeated tests median values of the pressures have been used as input in a *computer program* for the calculation of the internal forces in a circular cylindrical shell of constant wall thickness and elastic material.

The program BAC, developed at the Structural Research Laboratory of the Technical University of Denmark, has been chosen. This program is based upon the development of the load in fourier terms $\cos n\theta$ and $\sin n\theta$ with $n \leq 9$. In the vertical direction the shell is divided into a large number of equidistant horizontal rings. With the use of finite elements in the vertical direction, the program calculates the internal forces (and the deformations) at these equidistant levels.

Because of the limited number of pressure cells and the large gradients exhibited by shock pressures, the actual pressure distribution is poorly defined. In addition, the limited number of terms in the fourier series are unable to give an exact representation of the recorded pressures.

As an illustration of the *approximations involved*, the dotted line in Fig. 6 shows the pressure distribution as defined by pressure cells spaced 22.5° along the most exposed horizontal section in the model tests for the Brighton marina breakwater (scale 1:22), at a moment where the shock pressure on the front generatrix is particularly high. It will be seen that the maximum fourier series load (the full line) is much lower than the recorded pressure. It is estimated that this discrepancy results in a standard deviation of about 10% for the internal forces.

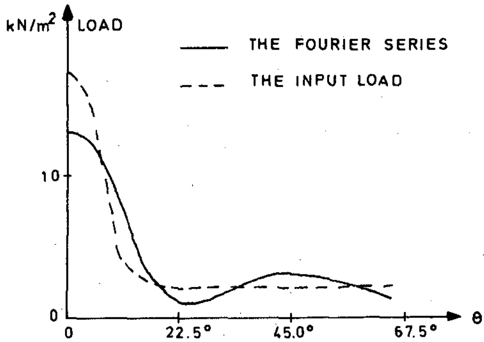

Fig. 6 Shock pressure distribution along horizontal section

Including the large scatter of the pressure distributions in 10 repeated tests, the total *standard deviation* is estimated to be 15% on the median values of bending moments, with a maximum scatter of about 30%.

The fourier series in Fig. 6 represents 10 terms, n = 0-9, which give the contributions shown in Figs. 7 and 8, respectively, to the normal force N_θ and the bending moment M_θ in the vertical section through the front generatrix, as well as to the normal force N_x and the bending moment M_x in the horizontal section.

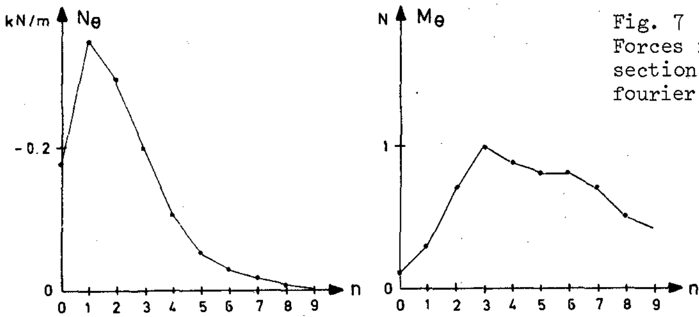

Fig. 7
Forces in vertical section from fourier terms cos nθ

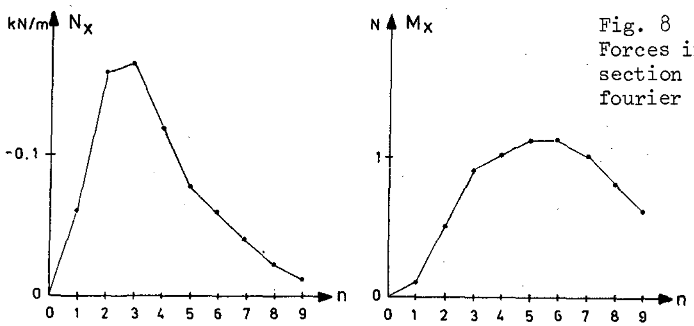

Fig. 8
Forces in horizontal section from fourier terms cos nθ

It appears that the values for n > 9 would have contributed essentially to the bending moments had the computer program been extended to higher terms. (It may be mentioned that the computational error increases with the order of the term.)

4. COMPARISON OF THE RESULTS FROM THE TWO MODELS

The median values of the normal forces and bending moments are presented in Fig. 9 at the instant when a large shock wave hits the front of the caisson, and in Fig. 10 at the slightly later instant when the same wave hits the corner between two caissons. The results from the strain gauge model are shown as dots, while the forces computed from the pressure cell model appear as full curves.

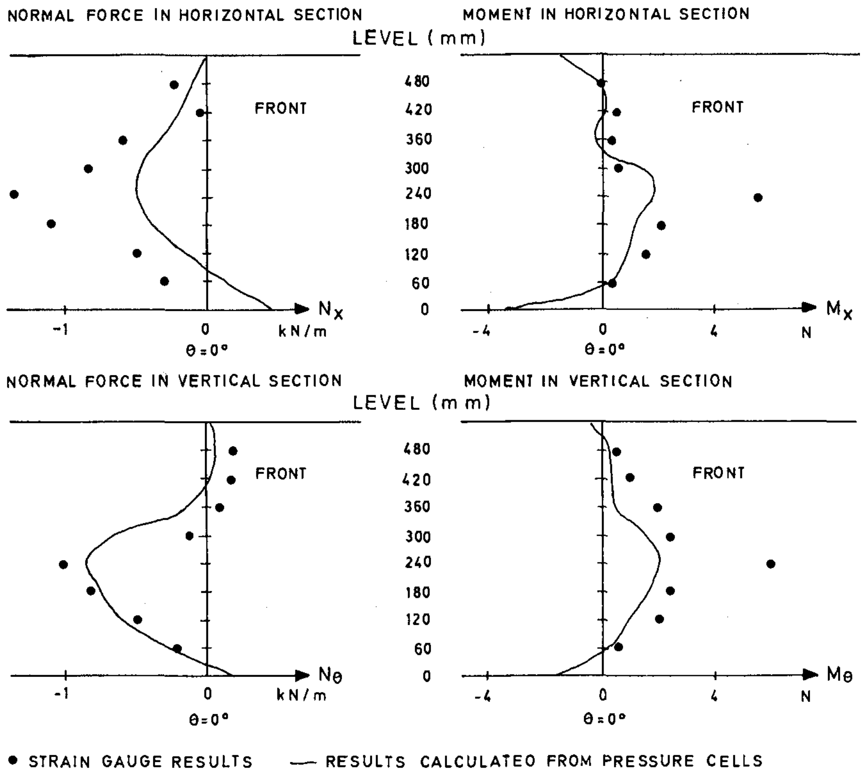

Fig. 9 Internal forces along the front generatrix

When the two sets of results are compared, the following circumstances should be borne in mind:

(a) For the pressure cell model the load is defined by a number of cells that is small compared to the irregular variation of the shock load. Between the cells the pressures have been estimated by linear interpolation.

(b) For the pressure cell model it has been assumed that the load is symmetrical around the plane through the middle of the flume. Because of the sensitivity of the shock pressures to small disturbances, the full curve in Fig. 9 may be slightly incorrect.

(c) The times chosen for the analysis of data from the two models may not correspond exactly to each other.

The comparison shows large differences between the two models at the most exposed points of the cylinder (level 240 mm in Fig. 9 and level 360 mm in Fig. 10). The discrepancy in the bending moments might be the result of a smaller extent of the highest pressures than inherent in the linear interpolation mentioned under (a) above.

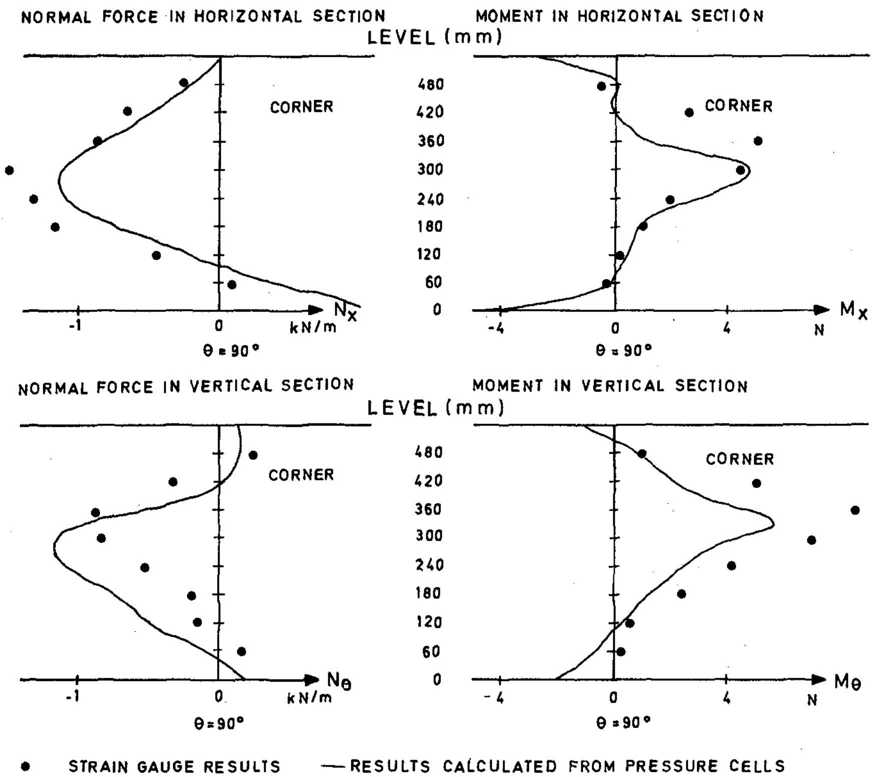

Fig. 10 Internal forces along the corner generatrix

5. COMPARISON OF PRESSURE CELL AND STRAIN GAUGE MODELS

As a result of this investigation it is concluded that a strain gauge model (SGM) is more advantageous than a pressure cell model (PCM). This conclusion is based upon the following comparison of the two techniques:

(a) Construction of Model

A PCM is less expensive than an SGM because the former need to have only the correct outside shape while the latter must be moulded out of a special material of good elastic properties. In addition, the wall thickness of the SGM should not deviate too much from geometric similarity with the final design.

(b) Instrumentation of Model

The acquisition of a large number of pressure cells with amplifiers and equipment for simultaneous recording of many channels is expensive. On the other hand, the same instrumentation may be used for several breakwater projects.

The number of rosette gauges required for an SGM is considerably larger than in the investigation reported in this paper, because the cylinder cannot be rotated due to its structural connection with the neighbouring caissons. Qualified workmanship is a prerequisite for the installation of the gauges. However, the costs of the gauges and their installation add little to the costs of the investigation. Simultaneous recording of 6 channels (two rosettes) is an absolute minimum, because two rosettes are required for the determination of the complete stress situation at one point.

(c) Test Runs

If sufficient pressure cells are available the PCM requires only one test run for a given wave train, while the SGM requires many runs to give records from, say, 20-30 rosettes. The repetitive runs, however, is a matter of routine that can be performed by one technician.

(d) Adequacy of Information

While the SGM gives complete information for the design of the reinforcement at each rosette point, the information from the PCM is insufficient because of the difficulties of defining the shock pressure distributions in connection with the large local pressure gradients. As a consequence of this, the results from the PCM may also depend upon the shell computer program applied, cf. Figs. 7 - 10.

(e) Data Analysis

For the SGM the analysis of the distribution functions for the strain maxima under the heaviest shocks is relatively simple, cf. Fig. 11 below. Also, the extrapolation to rare situations with due consideration of the wave climate can be done with reasonable accuracy, see Figs. 12 - 15 below.

For the PCM it does not seem feasible to carry out the stress analysis by computer program for so many load situations that the distribution functions for the stresses at the various points can be determined with sufficient accuracy. Hence, it is necessary to extrapolate the pressure distributions to rare situations and to apply the computer program for a few loads only. Thus, it is hardly possible to take sufficient regard to the wave climate, and the indirect determination of the design stresses may become somewhat problematic.

6. STRAIN DISTRIBUTION FUNCTIONS

For each zero-crossing period T_z and significant wave height H_s, the strains may be assumed to be *exponentially distributed*. The distribution functions have been determined for a number of combinations of T_z and H_s by means of trains of 1000 waves reproduced as described in Ref. 3.

As an example, Fig. 11 shows the distribution functions from one test run for point D on the front generatrix of the cylinder (Fig. 2). The abscissa PN is the socalled *probability number*, i.e. the relative occurrence per wave for which the strains shown are exceeded. The ordinate is the strain with the unit microstrain μS, where 1 μS = 10^{-6}.

For each value of PN the 6 strains plotted are simultaneous values from the records. The strains measured are arranged after the peak order of ε_{ho}, i.e. the horizontal strain on the outside of the shell. Thus, the extreme value ε_{ho} = − 22 μS recorded for 1000 waves is exceeded with a probability number of PN = 0.55 • 10^{-3} per wave, while the next value ε_{ho} = − 21 μS is exceeded 1.5 • 10^{-3} times per wave.

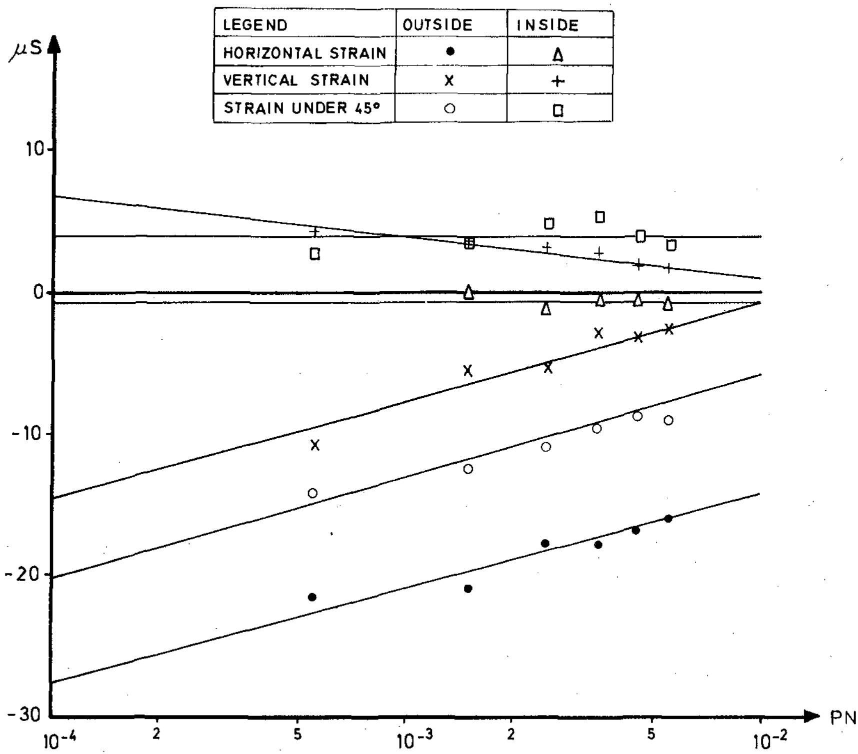

Fig. 11 Strain distribution functions at point D for T_z = 1.52 s and H_s = .134 m

Except for the inside diagonal strain (under 45°), the arrangement after the peak order of ε_{ho} also gives nearly monotonous arrangements of the other simultaneous strains. Thus, the result of this test run may be expressed by the following linear relationships:

Outside horizontal strain = ε_{ho}

Outside vertical strain = $\varepsilon_{vo} = \varepsilon_{ho} + 13\ \mu S$ (1)

Outside diagonal strain = $\varepsilon_{do} = \varepsilon_{ho} + 8\ \mu S$ (2)

Inside horizontal strain = $\varepsilon_{hi} = -1\ \mu S$ (3)

Inside vertical strain = $\varepsilon_{vi} = (\varepsilon_{ho} + 11\ \mu S) \cdot (-0.4)$ (4)

Inside diagonal strain = $\varepsilon_{di} = +4\ \mu S$ (5)

The fact that the rare strains are not proportional indicate that the shell carries the high shock pressures in different ways, with little variation of the strains on the inside.

7. STATISTICAL STRAIN ANALYSIS

For the design of the shell it is necessary to know the *stress conditions* that are exceeded only once in, say, 100 years, with appropriate choice of the factors of safety for steel and concrete.

The *purpose* of the statistical analysis of the strains is to find the annual number of waves, ANWS, that exceed various strain values S. If the distribution function ANWS is plotted against S and extrapolated to the value ANWS = 10^{-2}, the corresponding value of S will occur once in 100 years.

In principle, the statistical analysis should be based upon the *probability density function* $p(T_z,H_s,WL,WD)$, i.e. the wave climate expressed as a function of zero-crossing period T_z, significant wave height H_s, water level WL and wave direction WD. Evidently, it is necessary to know the density function only for the larger values of H_s. In addition, if the wave direction deviates more than, say, 20° from the normal to the breakwater alignment, the shock forces are much reduced. Thus, in practice it is usually possible to introduce several simplifications in the complete statistical analysis.

As an *illustration* of the application of the results of the strain gauge model tests, Fig. 12 shows, for PN = $0.55 \cdot 10^{-3}$ per wave, the outside horizontal strain ε_{ho} at point D along the front generatrix (Fig. 2) as a function of T_z and H_s. (The dotted straight line represents the limitation of wave conditions that could be generated in the flume.)

In order to find $ANWS_{ho}$ as a function of ε_{ho} one could consider the areas within the curves where ε_{ho} takes the values $-40\ \mu S$, $-50\ \mu S$ and $-60\ \mu S$, respectively. For each of these areas a summation is performed of the density function $p(T_z,H_s)$ for the determination of $ANWS_{ho}$ that exceed the corresponding strains. For this calculation it is useful to know the logarithmic slope of the exponential distribution of ε_{ho}, which is plotted in Fig. 13.

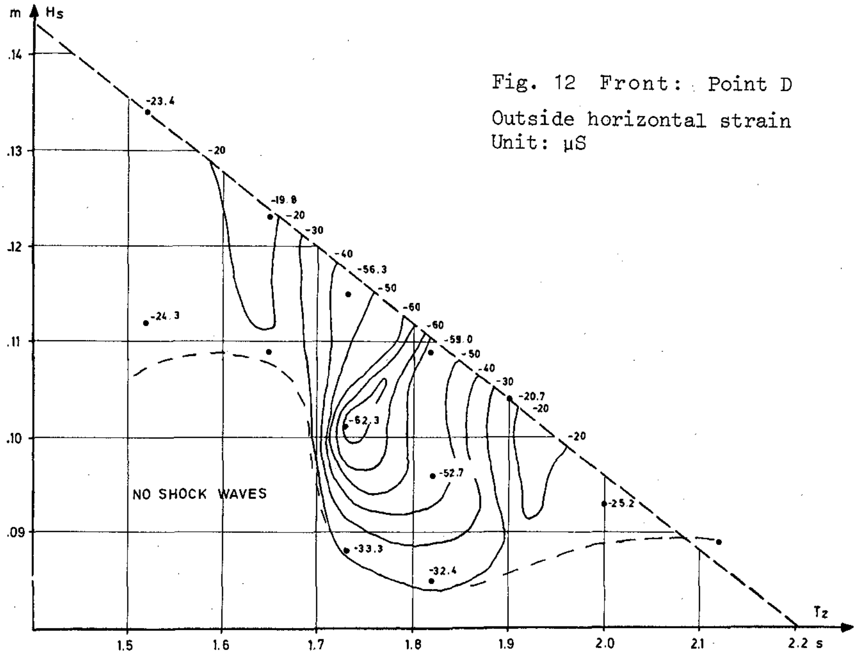

Fig. 12 Front: Point D

Outside horizontal strain
Unit: μS

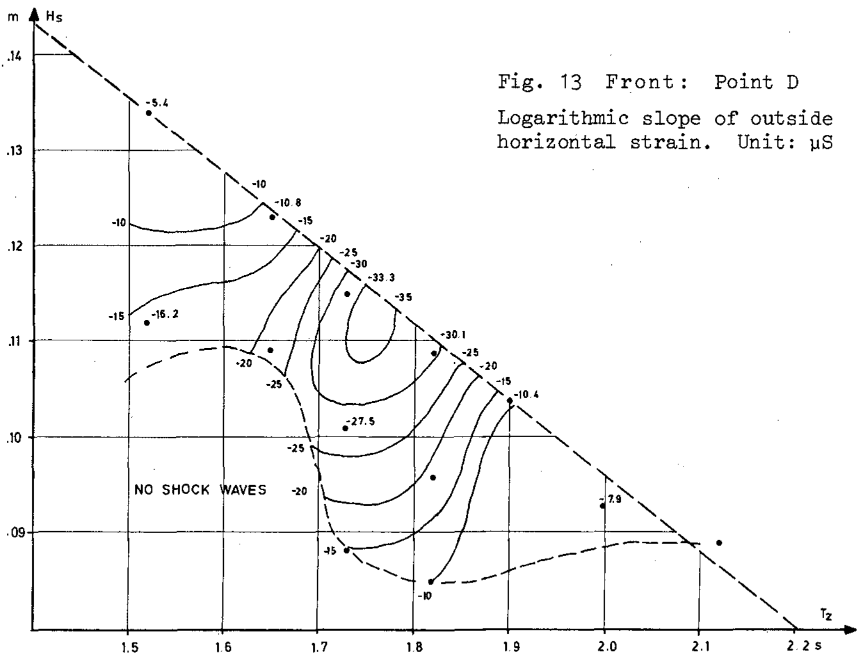

Fig. 13 Front: Point D

Logarithmic slope of outside
horizontal strain. Unit: μS

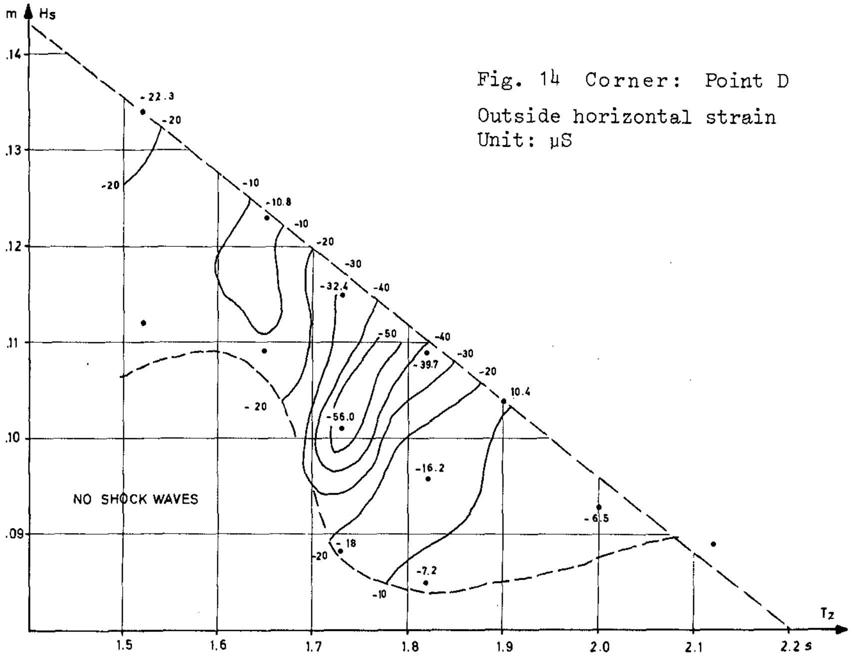

Fig. 14 Corner: Point D

Outside horizontal strain
Unit: μS

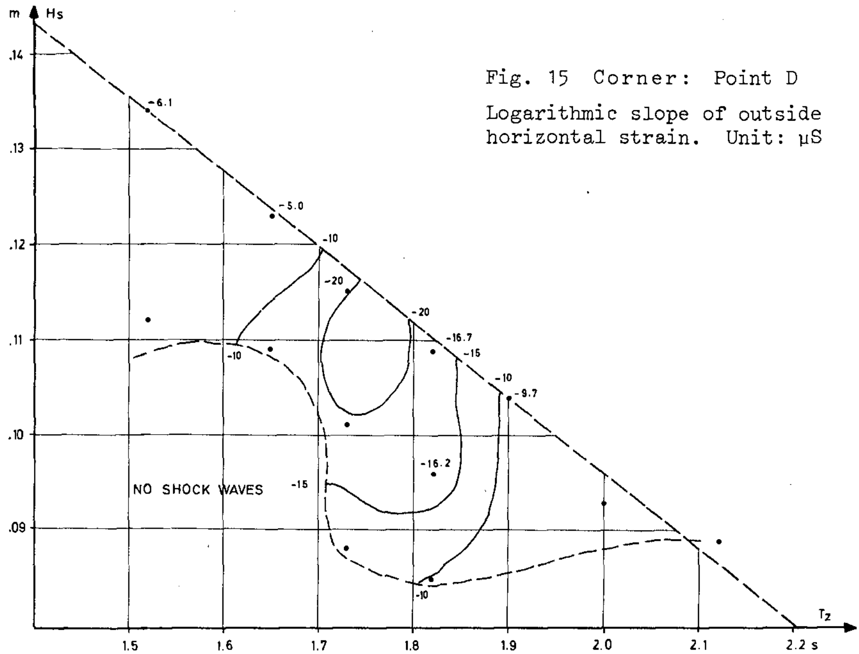

Fig. 15 Corner: Point D

Logarithmic slope of outside
horizontal strain. Unit: μS

After $ANWS_{ho}$ has been found for the three values of ε_{ho} mentioned, the approximately exponential distribution function $ANWS_{ho}$ is extrapolated to the value 10^{-2}. The corresponding value of ε_{ho} is used for the calculation of the remaining 5 strains by means of linear relationships such as (1) - (5) above.

Figs. 14 - 15 give the statistics of ε_{ho} for a corner point, in analogy to Figs. 12 - 13 for a front point.

It will be seen from Figs. 12 and 14 that the highest strain values occur within a rather *narrow interval of mean period* T_z. Hence, it is an *important conclusion* of the present investigation that the shock loads seem to depend considerably on the wave period. This result will have a significant influence on the test programs for future projects.

8. DESIGN OF REINFORCEMENT

During heavy shock loads the fine cracks always occurring in reinforced concrete will open (and close again). This phenomenon cannot be represented in a purely elastic strain model or in the calculation by a computer program. Because of the plasticity inherent in the opening of cracks, it is permissible to distribute the reinforcement much more uniformly than indicated by force diagrams such as Figs. 9 - 10.

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