CHAPTER 130

WAVE ACTION ON LARGE OFF-SHORE STRUCTURES

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SYNOPSIS

The paper describes investigations carried out in order to design for the wave action, both wave force and scour, on large off-shore berthing structures sited approximately 1.3 miles (2.1 km) off-shore near Hay Point, North Queensland, in 56 feet (17 m) of water at low tide, the tidal range being 20 feet (6 m). The region is a cyclone area and the structures must be capable of withstanding attack from maximum predicted waves with period of 8.25 seconds and amplitude of 24 feet (7.3 m).

The main units in the berthing structures are concrete caissons sunk on to the ocean bed and the largest of these have plan dimensions of approximately 150 feet (46.7 m) by 135 feet (41.4 m) with four columns approximately 40 feet (12.2 m) square projecting through the water surface. No theoretical method available at the time of the investigation was capable of accurate calculation of wave forces on these structures. A scale model was tested to obtain wave forces and the paper compares results from the model with those of numerical methods and discusses the application of the results to the design functions. Scour effects were also modelled and the results used as the basis for design of scour protection.

1. INTRODUCTION

Hay Point is an exposed rocky headland on the northern Queensland coast approximately 750 miles north of Brisbane. The Great Barrier Reef offers a degree of protection but fetch distances within the Barrier still allow the development of large wave heights during cyclonic disturbances. The port at Hay Point consists of two berths to cater for 100,000 dwt bulk carriers exporting coal to Japan; the annual throughput being some 20 million tons per annum (mtpa). The berths are sited 1.3 miles (2.1 km) offshore where 56 feet (17 m) of water is provided at low tide. The first berth, a conventional piled structure, was commissioned in 1971 and has a throughput capacity of approximately 10 mtpa. Studies of ship turn round times, tidal restriction and other factors showed that upgrading of the first berth, even with the provision of 2 shiploaders, would still leave it incapable of handling the output from the one or two additional mines proposed. Accordingly the decision was made to construct a second berth.

Once the location and alignment of the second berth had been chosen, several types of construction were considered. In the event, concrete caissons which could be constructed at a remote site, floated to and grounded at Hay Point were selected since they provided, inter alia, the following advantages:-

* the maximum of work could be carried out in the sheltered conditions of the construction site, leaving a minimum to be completed at the exposed site.
* minimisation of risk to shipping and of disruption of port operations during the construction period.

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Figure 1 illustrates the site and the alignment of the two berths. The layout and structures of the new berth are illustrated in Figure 2, which also indicates the scour protection. The cross sections of a concrete berth caisson and of the large columns penetrating the water surface are shown in Figure 3, which also provides details of the scour protection system and of the foundation preparation adopted. Figure 4 shows a completed berth caisson grounded at Hay Point.

2. WAVE CLIMATE

Hay Point is located on latitude 21°S and is subject to cyclonic activity concentrating primarily through the months of December to April. The Great Barrier Reef is located to the East and is an effective barrier to storm waves originating in the Coral Sea. The longest uninterrupted fetch lies to the East with water depths varying between 6 and 30 fathoms. To the North, East and S.E. of Hay Point, fetch distances are largely broken up by islands and by shallow water.

Characteristically the wave climate predominates from the S.E., being influenced largely by storm activity in the Tasman Sea, but interposed with random and irregular cyclonic activity usually generated in the Coral Sea. Refraction of waves arriving from S.E. tends to concentrate long period waves from an East and E.S.E. direction, periods of at least 20 seconds having been recorded from the East. Extensive investigations into the wave climate were conducted and these are summarised in Figure 5. Figure 5 is a plot of measured data on wave height, and from this the following probabilities for deep water waves were obtained:-

Occurrence interval (years)	1	10	100
Significant wave height (feet)	10.5	13.5	17.5

The steepness of the waves is such that breaking conditions would not be expected. Directional distributions showed that most waves approached the site from the S.E. quadrant, the directions E.S.E. and S.E. being particularly dominant.

A special examination of potential cyclone activity was undertaken using hindcast techniques and 4 possible cyclones were simulated. Potential deep water waves of greater than 20 ft were generated with periods of the order of 8 seconds. The determination of the probability of such a cyclone occurring is quite difficult and even more so at a given location. However, at Hay Point the probability was considered to be of the order of 1 in 300 to 1 in 500 years and for the waves generated to occur at high water the probability is 1 in 600 to 1 in 1000 years. In general, most of the higher waves consistent with a prediction of a significant wave height of 20 feet would either be unstable because of large steepness or would have shoaled and, therefore, only a few could be expected to reach the structure in a 6 hour period and, then, only at high tide. The probability of a cyclone occurring with a maximum wave of 24 feet generated is estimated as approximately 1 in 50 years and, at high tide, 1 in 100 years. It was estimated that such waves could be expected to reach the structure 40 to 60 times in any 6 hour period of wave persistence. It was considered reasonable therefore to adopt a maximum design wave of 24 feet with a period of 8.25 seconds from which to determine the forces on the structure due to waves, the forces being subject to load factors as discussed below.

FIG. 1 Hay Point bulk coal loading Berths

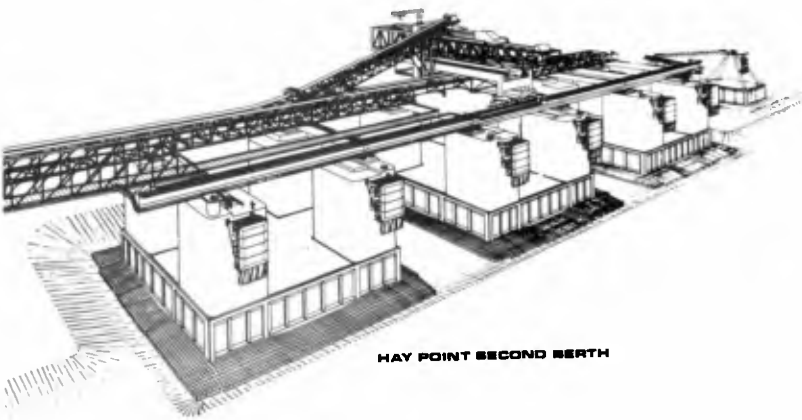

FIG. 2 New Berth structures at Hay Point

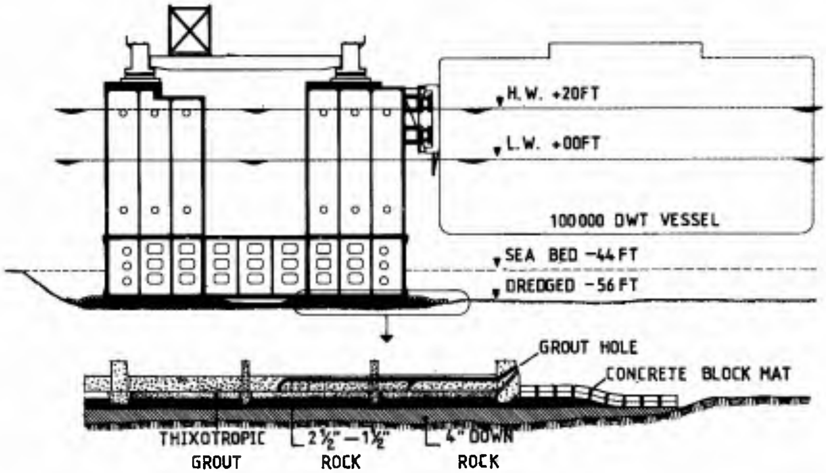

FIG. 3 Main Berth Caisson; structure and foundations

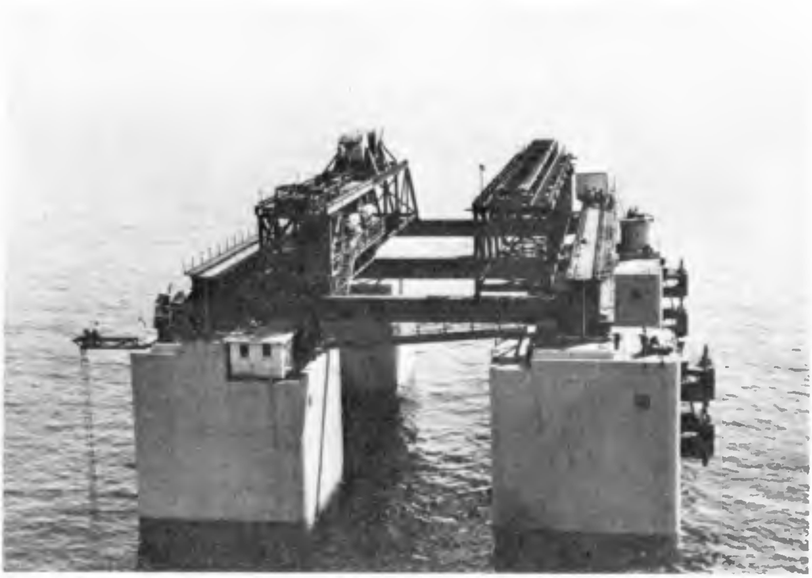

FIG. 4 Main Berth Caisson grounded at Hay Point

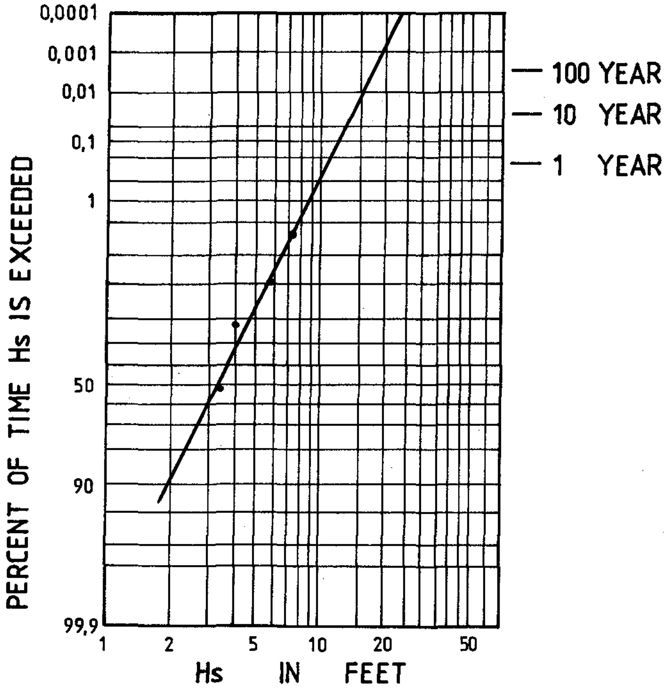

FIG. 5 DEEP WATER SIGNIFICANT WAVE HEIGHT.

3. PRINCIPAL DESIGN REQUIREMENTS

The size of an individual column on one of the caissons is approximately 40 feet (12.2 m) square, the theoretical wave length of the design wave is approximately 350 feet (107 m) and for the more frequent waves the wave length is about 200 feet (61 m). Thus, methods of calculating wave forces such as those developed by Morrison (Ref. 1) or by Sainflou (Ref. 2) are inappropriate since they are based on the assumption that the characteristic structure dimension is either very small or very large (i.e. a bluff body) compared to the incident wave length. In the first instance, therefore, a numerical method based on linear diffraction theory (Ref. 3) was used to give estimates of wave forces to allow the design process to commence and proceed. Simultaneously, measurements of wave pressures were made with a physical model of a complete caisson with its four columns, to include interaction effects from adjacent columns, the influence of a porous foundation and of the geometry of the bottom of the caissons which formed a cruciform gap under the central part of the base.

It was the intention that the caissons should be a gravity design and that they should be stable when grounded without additional securing or ballast. Therefore, an accurate picture of variations with time of wave forces on the columns and on the overall structure was essential. In addition to the overall stability requirements, the value of the moment at the base of the column together with the positive and negative pressure history on a column during the passage of a wave were required to ensure the competence of the structural elements forming the columns.

In the case of scour protection the quantitative assessment of requirements could be achieved only by model analysis. The two principal types of caissons, the large berth caissons and the smaller approach caissons, both required modelling. The chief problem in the case of the berth caisson was to derive a suitable weight for the type of protection considered the most suitable from a construction viewpoint. In the case of the approach caissons it was not clear at the design stage whether or not the foundation should be recessed below the level of the surrounding sea bed or whether the foundation could be constructed directly on it and, so, both these arrangements were modelled.

4. WAVE FORCE ANALYSIS

The numerical calculations of wave forces were carried out by Dr. C.J. Garrison (Ref. 3). Since the dimensions of the structures are not small compared to the wave length, linear diffraction theory was used. All the cases analysed with this numerical model were for incident waves of height 23.6 feet (7.2 m) and period 8.25 seconds at high tide. Wave forces on the exposed surfaces of a main berth caisson of plan dimensions 150 feet (46.7 m) by 135 feet (41.4 m) were computed for incident wave directions normal to the longer and to the shorter faces, in a mean water depth of 78 feet (23.7 m). Wave forces were also computed for a single approach caisson in mean water depth of 70 feet (21.3 m), for a wave direction normal to one face. The total forces and moments on the exposed surfaces of the main berth caisson calculated for the case of waves approaching in the direction normal to the longer face are shown in Figure 6. Some results from the physical model are included for comparison. These are discussed in Section 6.

The wave forces calculated by linear diffraction theory gave estimates which enabled design to commence and proceed. However, the theoretical model is not accurately applicable in this case since it implies that all motions are very small, which is not a good approximation for the large waves involved, for which the ratio of wave height to water depth is approximately 1:3. Further, the theoretical calculations do not take into account effects due to separation of flow past the vertical columns nor can they predict the pressure fluctuations on the underside of the caisson. These limitations of the numerical model made it necessary to conduct extensive measurements of wave effects on a scale model.

5. WAVE FORCE MODEL

Transient pressures on the surfaces of a scale model of a berth caisson were measured under the action of both "cyclone" waves and of "design waves" at High Water and at Low Water. The incident wave characteristics adopted for the model tests on the basis of the data available on wave climate were as follows:-

Cyclone Wave:- Height (H_{max}), 24 feet (7.3 m); Period, 8.25 secs.
Design Wave:- Height ($H_{1/3}$), 12 feet (3.7 m); Period, 6.0 secs.

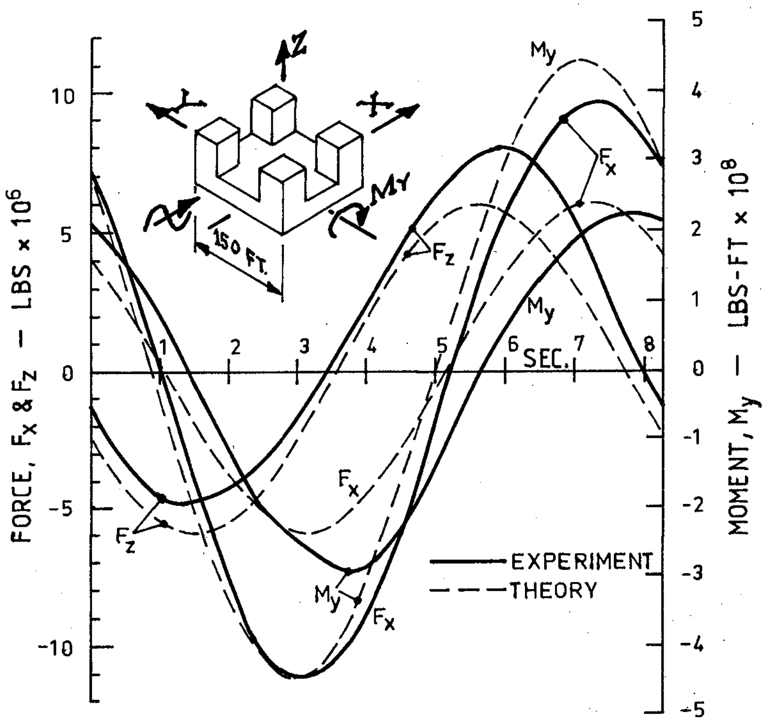

FIG. 6 WAVE FORCES FROM EXPERIMENT AND THEORY.

Wave roses obtained at the site indicated that the dominant direction for incident waves is from ESE, but that waves could be expected from virtually any direction within the range SE to NW. The compressed time period in which the model studies had to be carried out made it impossible to conduct tests for all possible directions of incident wave. Consequently, the tests were limited to the study of waves approaching normally to the berth line (wave crests parallel to the longer side of the caissons) and of waves approaching from the dominant ESE direction (wave crests at an angle of 52½° to the longer side of the caissons). The tide levels adopted gave water depths above the foundation level of the caisson of 76 feet (23 m) at High Water and of 56 feet (17 m) at Low Water.

The berth caissons are founded on four pads, one at each corner, 40 feet x 40 feet in plan. The foundation pads are set on prepared base material, as shown in Figure 3, and it is considered that the water pressures applied to the under side of the foundation pads will be those corresponding to the current tide level without any significant effect from wave action. However, the cruciform area on the under side of the caisson between the foundation pads will be subjected to pressure variations caused by wave action because it communicates directly to the sea through the gaps between the foundation pads along each side of the caisson. The effects of wave pressures on this region of the underside of the caisson were found to be very significant in the overall stability of the caisson. Consequently, wave pressures on the underside of the caisson were measured for two conditions, viz:- (i) the gaps along each side of the base were unobstructed, (ii) 90 percent of each gap was closed, the remaining 10 percent of opening being distributed uniformly over the length of the gap.

5.1 Experimental details

Wave Basin: All of the tests were carried out in a wave basin 10 feet (3 m) wide and 48 feet (15 m) long at the University of Queensland. The wave generator was of paddle type, the throw at top and bottom of the paddle being capable of independent adjustment. A spending beach of crushed rock at a slope of 1 in 10 was placed at the opposite end of the basin from the wave generator in order to eliminate wave reflection. The models were located approximately half way along the basin. The incident waves from the wave generator, as measured at this location, were free of any significant higher harmonics and there was no significant reflection off the spending beach.

Model: The model of the main berth caisson was constructed to a scale of 1:60 from clear perspex (lucite) sheet. It can be seen in Figure 7. A total of 135 pressure tapping points was built into the model for measurement of pressures on all external surfaces, including the under-side of the caisson. This large number of tapping points was dictated by the need to measure the pressure distribution in sufficient detail for the purposes of structural design. The pressure tappings were 0.062 inch (1.57 mm) in diameter and they were connected to the pressure recording system through rigid nylon tubing of 0.127 inch (3.22 mm) internal diameter. It was not possible to locate the available pressure transducers inside the model; instead it was necessary to connect the pressure tappings to the externally located transducers by carrying the connecting tubing out from the model through the tops of the four columns and it was physically impossible to fit more tubes into the columns. Because of this restraint, only one column was fitted with tappings, ten in each vertical face. For tests in which the waves were approaching normally to the structure, it was possible to measure the complete pressure distribution over the base caisson in one set-up, since measurements were needed over only one half of the base on account of symmetry. For this same set-up the pressures were measured on the one instrumented column in the front position and then the model was rotated through 180 degrees so that pressures could be measured on the column in the rear position. For tests in which the waves approached the caisson other than normally, it was necessary to set the model in four orientations in order to obtain the complete pressure distribution over the base caisson and over the four columns.

Pressure Measurement: The pressure tappings at the surface of the model were connected through rigid nylon tubing to pressure transducers with a range ±10 inches (254 mm) of water. These consist essentially of a metal bellows which deflects under pressure and displaces an armature. The movement of the armature induces strain in unbonded strain gauge elements which form the arms of a Wheatstone bridge. The voltage output from the Wheatstone bridge was amplified by carrier amplifiers and the amplified signal was recorded on a pen recorder. The model wave periods were 0.775 sec. and 1.065 sec. for the Design

FIG. 7 Model Berth Caisson and instrumentation

wave and Cyclone wave respectively. The natural frequency of the pressure transducers (dry) is in excess of 300 Hz and the response of the pen recorders is flat up to 40 Hz. The pressure transducers were of differential type and the case reference pressure used was the ambient atmospheric pressure in order to prevent the occurrence of zero shift, which would otherwise be caused by changes in atmospheric conditions. In order to achieve consistent, repeatable performance over long periods of time, it was found necessary to ensure that the air in the cases of the transducers was completely dry. All of the elements of the instrumentation system can be seen in Figure 7. Full details are given by Apelt (Ref. 4).

Wave Profile Measurement: The wave profile was recorded by means of a capacitance type wave probe, the electrical output from the probe being recorded on the pen recorder.

Experimental Procedure: The instrumentation available for the tests comprised three pressure transducers and associated carrier amplifier systems, one wave recorder and two twin channel pen recorders. One recorder channel was used always to register the incident wave profile and the other three channels were used to record the outputs from the three pressure transducers which were connected to pressure tapping points in groups of three until all had been monitored. Synchronisation of the four separate signals was achieved by activation of the event marker pen on each recorder with a signal generated from a relay on the wave generator. The signal was pulsed once for each period of the wave generator and this provided an accurate time base as well as synchronisation. The pressure history at each tapping point was recorded for at least three successive wave periods in order to average out the effects of any small variations in the incident waves. Provision was made in the circuitry to permit purging of the tubing under pressure in order to clear any air or blockage from the tubes at the beginning of each test and at any other time, if the need arose. The pressure transducers were also connected to a calibrating chamber so that static calibrations of the total pressure recording system could be carried out at the beginning and end of each measurement session.

Dynamic Calibration of Pressure Recording System: The inertia of the large volume of water in the tube connecting to the pressure transducer greatly modified the frequency response of the pressure measuring system. A preliminary series of tests established that it was necessary to adjust the length of the connecting tube to achieve satisfactory response characteristics at the different wave periods. For a wave period of 0.775 seconds the optimum length of the connecting tube was found to be 9 feet and for a wave period of 1.065 seconds it was found to be 15 feet. Even with this arrangement it was not possible to achieve a response which was completely free of amplitude modulation. Consequently, the pressure recording system was calibrated dynamically at the frequencies corresponding to the design wave and to the cyclone wave so that the appropriate conversion factor could be applied to the static calibrations carried out regularly throughout the programme of measurement. Details of the procedure used for dynamic calibration are given by Apelt (Ref. 4). At a wave period of 1.065 seconds the dynamic response was found to be amplified 1.13 times compared to the static response and at a wave period of 0.775 seconds the dynamic response was found to be attenuated by a factor of 0.80 compared to the static response.

6.0 RESULTS OF WAVE PRESSURE MEASUREMENTS

Only selected results which are thought to have some general interest beyond the specific design study are presented here. The full set of wave pressures and of wave forces are given in Apelt (Ref. 4). Forces and moments on elements of the caisson and on the caisson as a whole due to wave action were computed by numerical integration of the measured wave-induced pressures. The coordinate and sign conventions used are the standard conventions of calculus, as illustrated in Figure 6. The origin of coordinates is located at the centre of the caisson base at foundation level and moments have generally been computed with reference to this origin. However, moments on the columns have been computed about axes in the base of the column at the level of the upper surface of the caisson base.

In some tests the gaps along the lower edges of the Caisson base were left unobstructed, and this condition is referred to as "FULL OPENINGS", while for cases in which these gaps were closed for 90 percent of their length, the condition is referred to as "TEN PERCENT OPENINGS". In all of the model tests, water was allowed to enter the space inside each column through two holes, located one in each inward-facing side of the column. Scaled to prototype dimensions, each hole was 0.75 feet in diameter and was located 5 feet above the top surface of the Caisson base, on the centre-line of the face of the column. The depths of water inside the columns were inferred from the pressures recorded at the tappings inside the columns in the top surface of the Caisson base, and the water surface inside each column varied significantly from the still water level throughout the wave period. The contributions to vertical components of forces and to moments due to these variations in water levels within the columns are included in all of the results, except where otherwise noted. Since it should be possible to achieve a virtually constant water level within the columns, corresponding to still water level, by reduction of the size of the openings through the sides of the columns, the effects of such a modification on maxima of forces and moments have been computed. Quantities calculated for the conditions when the water level inside each column is constant at still water level are indicated by a prime, i.e., $\Sigma Fz'$ and $\Sigma My'$.

6.1 Maximum forces and moments due to waves approaching in direction normal to berthline: The maximum forces and moments on the Berth Caisson due to waves approaching in the direction normal to the berthline are set out in Table 1. The largest horizontal and vertical forces are caused by 24 feet waves at High Water but the largest moments, in most cases, are caused by 12 feet high waves at Low Water. For the latter case the vertical forces are relatively small and the maximum variations in foundation stresses, both positive and negative, are caused by 24 feet high waves at High Water.

The effects produced by maintaining the water level inside the columns constant at still water level can be assessed from the data in Table 1. For example, this modification causes reductions in vertical forces but increases in moments for the case of 24 feet high waves at High Water, both for FULL OPENINGS and for TEN PERCENT OPENINGS.

The benefits derived from restriction of the openings along the bottom edges of the Caisson are clear from the data in Table 1. For three of the four test conditions the case of TEN PERCENT OPENINGS has smaller maxima for all forces and moments than does the case of FULL OPENINGS UNDER.

6.2 High Water: 24 feet high waves approaching in direction normal to berthline: The details of forces and moments experienced by a Berth Caisson and by components of it, for the case of 24 feet waves approaching normally to the longer face at High Water are shown in Figures 8a to 8d. The total forces and moments for the whole Caisson are shown in Figure 8a and the separate contributions of the base and columns to horizontal and vertical components of force are shown in Figures 8b and 8c respectively. The significant effects on vertical forces caused by partial closure of the gaps along the bottom edges of the Caisson are evident in Figures 8a and 8c. The columns contribute the largest component to the horizontal force on the Caisson, as can be seen from Figure 8b.

The details of moments experienced by the columns are shown in Figure 8d; it can be seen that the columns are subjected to very large moments and, in fact, the columns make the largest contribution to the moments experienced by the Caisson as a whole.

The time histories of forces and moments in Figures 8a to 8d display marked departures from symmetry which are due to a number of effects, including the large wave height (the ratio of wave height to water depth is approximately 1:3), separation of flow past the columns and interactions between the columns. It is of interest to note that, whereas the maximum total horizontal force is larger than the maximum

TABLE 1 - RANGES OF FORCES AND MOMENTS ON BERTH CAISSON FOR WAVES APPROACHING IN DIRECTION NORMAL TO BERTH-LINE

Units of Force:- lbs x 10^6;Units of Moment:- lbs-feet x 10^6

WAVE HEIGHT:- TIDE STATE:-	24 feet HIGH WATER	24 feet LOW WATER	12 feet HIGH WATER	12 feet LOW WATER
ΣFx; MAX +ve	9.53	7.10	1.28	4.91
ΣFx; MAX -ve	-11.07	-4.94	-1.01	-4.78
FULL OPENINGS:-				
ΣFz; MAX +ve	7.75	4.59	2.16	2.76
ΣFz; MAX -ve	-7.93	-4.88	-2.26	-2.86
ΣMy; MAX +ve	242	224	163	363
ΣMy; MAX -ve	-321	-97	-141	-342
$\Sigma F_z'$; MAX +ve	7.09	4.59	1.66	2.88
$\Sigma F_z'$; MAX -ve	-6.03	-4.77	-1.88	-2.98
$\Sigma M_y'$; MAX +ve	265	219	148	343
$\Sigma M_y'$; MAX -ve	-371	-92	-115	-328
TEN PERCENT OPENINGS:-				
ΣFz; MAX +ve	5.58	3.63	1.71	0.24
ΣFz; MAX -ve	-4.43	-3.91	-2.38	-1.33
ΣMy; MAX +ve	233	112	153	353
ΣMy; MAX -ve	-271	NO -ve	-136	-326
$\Sigma F_z'$; MAX +ve	5.11	3.53	1.22	0.32
$\Sigma F_z'$; MAX -ve	-2.72	-3.81	-1.99	-1.20
$\Sigma M_y'$; MAX +ve	256	112	137	333
$\Sigma M_y'$; MAX -ve	-321	NO -ve	-110	-306

of any partial contribution to it, the maximum of total moment is less than the maximum contribution from the columns, a result of the complex phase relationships between the several contributions to the total effect. The maximum of total vertical force is approximately the same as that on the top surface alone for the case of FULL OPENINGS, but is smaller for the case of TEN PERCENT OPENINGS.

6.3 Low Water: 12 feet high waves approaching in direction normal to berthline: The total forces and moments experienced by a Berth Caisson under these conditions are shown in Figure 9. This case is included for comparison with the results for the 24 feet wave at High Water and it can be seen that the maximum moments in Figure 9 are larger than those in Figure 8a. The large total moments in Figure 9 are almost entirely the consequence of large moments on the base of the Caisson which, in turn, result from the fact that the contributions from the top and underneath surfaces of the Caisson are nearly in phase. In contrast, the large total moments in Figure 8a arise from the very large moments experienced by the columns.

6.4 Effects of Wave direction on Column Moments: The tests with waves approaching in the direction normal to the berthline showed that the columns would be subjected to very large forces and moments under the

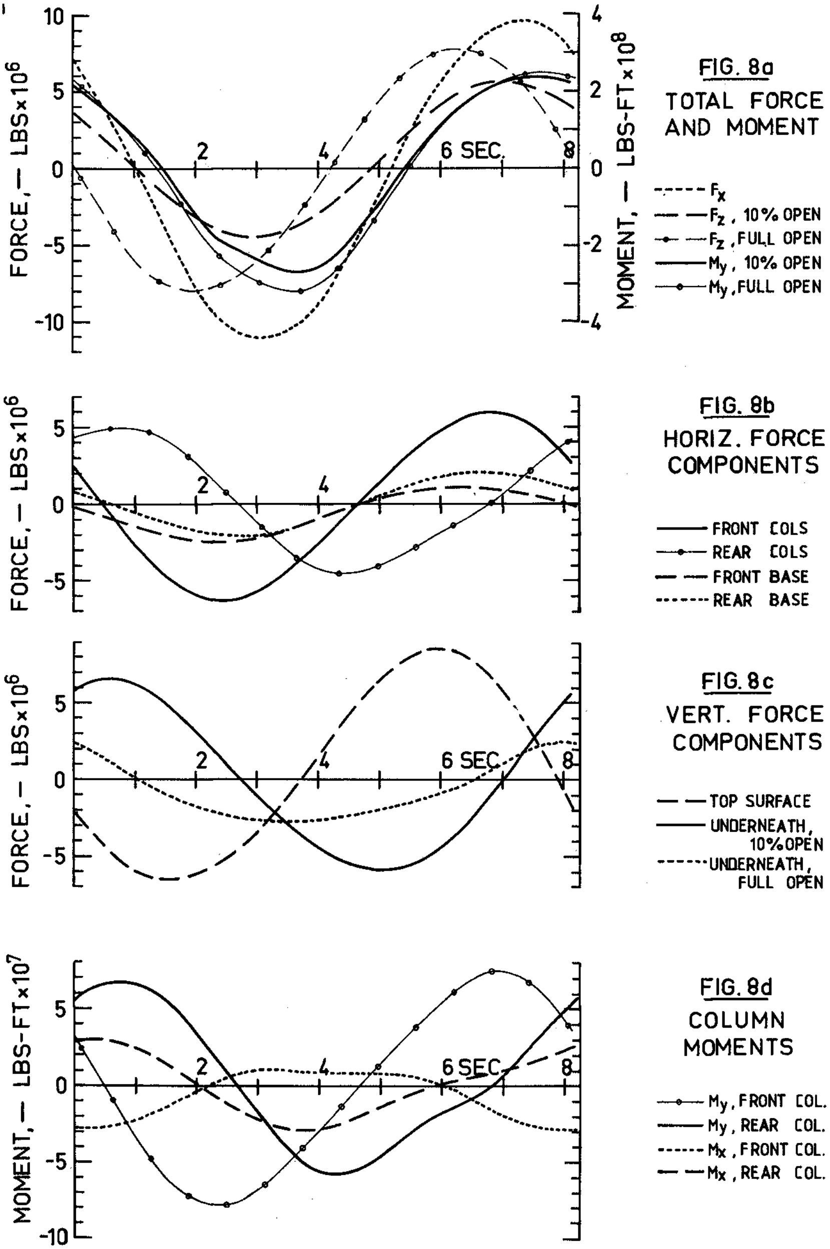

FIG. 8 WAVE FORCES ON BERTH CAISSON; 24 FT WAVES AT H.W.

action of 24 feet high waves at High Water. A series of tests was completed to determine the forces and moments experienced by the columns for the same wave and tide conditions except that the waves were approaching from the dominant ESE direction. It was found that the maximum bending stresses due to moment under these conditions will be developed in the rear leading column in the presence of neighbouring caissons. This also proved to be the worst of all cases tested, giving bending stresses 22 percent higher than for the worst case for waves approaching normal to the berthline. The moments experienced by the rear leading column in these angled waves are shown in Figure 10, which includes results obtained with and without neighbouring caissons. It can be seen that the presence of neighbouring caissons causes the maximum moments on the column to be increased almost twofold. The presence of neighbouring caissons also resulted in increases of approximately 25 percent in the maxima of moments on the front leading column. However, in the case of both "down-wave" columns the presence of neighbouring caissons had only small effects on the maxima of the moments.

7. COMPARISON BETWEEN EXPERIMENT AND THEORY

The forces and moments produced by calculations of Garrison (Ref.3) based on linear diffraction theory, are compared with the results derived from experimental pressure measurements in Figure 6. The conditions for which the comparison is possible are those of High Water with cyclone waves approaching in the direction normal to the longer side of the Berth Caisson and, although the conditions used for theoretical calculations and experimental studies are very closely similar, they are not identical. The depth of water in the experimental studies was 76 feet (23 m) and the wave height was 24 feet (7.3 m). The corresponding values in the theoretical calculations were 78 feet (23.7 m) and 23.6 feet (7.2 m) respectively. The wave period was 8.25 seconds in each case.

The quantities compared in Figure 6 are total Moment, Horizontal and Vertical forces due to wave action on the exposed surfaces of the Caisson. The effects of wave induced pressure variations on the underside of the Caisson and inside the columns are not included in the comparison. The results obtained from the two approaches display significant differences. Whereas all three curves derived from linear diffraction theory are nearly symmetrical, all curves obtained from measured pressure distributions display large departures from symmetry. These departures from symmetry are considered to be due to the significant non-linearity of the incident wave and to the effects of flow separation about the vertical columns. The lack of symmetry in the curves derived from experimental results shows most clearly in the different magnitudes for positive and negative peak values. The two sets of results also show significant and complex differences in the phase relationships between the three quantities, Fx, Fz and My. The ratios of the peak values of the three quantities are given in Table 2.

TABLE 2 - RATIOS OF EXPERIMENTAL MAXIMA TO THEORETICAL MAXIMA

Fx		Fz		My	
+ve	-ve	+ve	-ve	+ve	-ve
1.60	1.85	1.36	0.81	0.50	0.64

The differences are least for vertical forces, Fz. The experimental

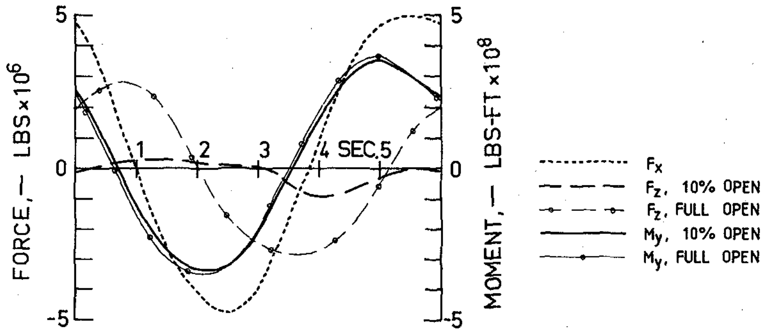

FIG.9 WAVE FORCES ON BERTH CAISSON; 12FT WAVES AT L.W.

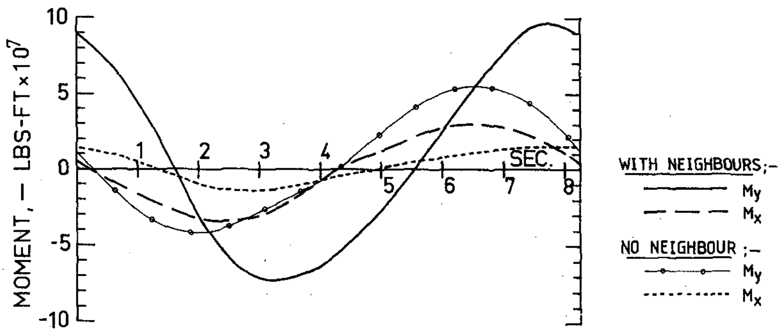

FIG. 10 COLUMN MOMENTS; 24FT WAVES AT H.W. FROM E.S.E.

values of horizontal force Fx are much larger than those predicted by linear diffraction theory but, in spite of this, the experimental values of moments are only approximately one half those predicted by the theory. The explanation of the latter result is that the total moment is the summation of contributions from the four columns, from the vertical faces of the base of the caisson and from its top surface; the combination is considerably smaller in magnitude than some of the individual contributions and the phase relationships between the individual contributions are of very great significance in determining whether effects are additive or whether they cancel each other.

8. APPLICATION OF RESULTS

Pressure readings derived from the model study were converted directly to the equivalent head of water acting on the surface. The maximum loading condition on the column faces for the passage of waves was extracted from the readings and applied directly as a loading to the column for bending moment and shear force calculation on the column as a whole.

The exterior walls of the columns were designed for two methods of behaviour, viz:- as two dimensional panels and as horizontally spanning beams of unit width. In the latter case it was necessary to analyse the entire circumference of the column as a continuous member and to apply the instantaneous pressure loading on the circumference. This loading varied from a maximum positive pressure of 25 feet of water to a maximum negative pressure of 15 feet. The method used ensured that the axial loading in the walls from the pressure distribution was also determined. The water level inside the columns was tidal and found to vary during the passage of the waves. The loading on the walls could vary from positive (inwards) to negative (outwards) as the water level rose and fell above tide level. This introduced a reversing load of compression and tension into the interior walls. The walls were designed for the worst case of tension and compression combined with bending moment induced by uneven external wall loading together with fatigue conditions.

The overall stability criteria were established prior to the present formal recommendations of Federation Internationale de la Precontrainte and Det Norske Veritas and were as follows:

(i) For loadings derived from numerical analysis

* horizontal and vertical wave loads - load factor 1.4
* wind loads load factor 1.4
* dead loads load factor 1.0

In this case the worse combination of horizontal and vertical wave loads was adopted.

(ii) For loadings derived from physical modelling, the same load factors were used and, in this case, the combination of vertical and horizontal loads used was that conforming with the observed phase difference.

The somewhat unusual feature of taking the dead load factor as 1.0 in lieu of 0.9 is justified since the caissons, in floating, were effectively weighed to within 1 percent.

Stability criteria of 1.0 on overturning and 0.7 on sliding were adopted for both these load factors. In the event, the stability criteria adopted conformed to both Federation Internationale de la Precontrainte and Det Norske Veritas requirements.

9. STUDY OF SCOUR NEAR CAISSONS DUE TO WAVE ACTION

The ocean bed at the site of the berth is covered with fine sand. The berth area is dredged to a general depth of 56 feet below low water and the berth caissons are founded on an area of crushed rock, consisting of sizes passing a 4 inch screen, which has been levelled off to R.L. -56 feet (See Fig. 3). Scour tests were carried out at a linear scale of 1:60 in order to safeguard the structures against erosion resulting from wave action. The model of the main berth caisson which was used for pressure measurements was also used in the scour tests. At a scale of 1:60 the corresponding maximum size of foundation material for the model is 0.07 inch. The material used to represent the prototype foundation material in the scour studies was prepared from finely crushed rock, by rejection of all sizes not passing a No. 7 sieve and by washing from the remainder the sizes passing a No. 25 sieve.

Interaction between the incident waves and the berth structures will produce significant velocities in the vicinity of the caissons and these induced velocities provide the most important mechanism for erosion of the foundation material. It is very difficult to assess the relative importance of viscous action in the complex phenomena involved in generation of the currents by interaction between incident waves and structures and in movement of the foundation material by these currents. However, the induced velocities are of the same order as the maximum orbital velocities in the incident wave and the Reynolds number based on bed material size is of order 1000 in the model. It is considered that this Reynolds number is large enough to ensure that scale effects in the model will not be so large as to invalidate the general results of the scour studies. The results of the scour studies are considered to be qualitatively correct but no estimate can be made concerning quantitive accuracy.

9.1 Wave scour near main berth caisson: For studies of wave scour near the main berth caisson, the model of the caisson was mounted on a 2 inch thick layer of the crushed rock fines. All wave scour tests were run continuously for one hour, which corresponds to 7.75 hours of prototype time reckoned according to Froude number scaling. However, as noted above, the uncertainty associated with different viscous effects in model and prototype makes scaling between elapsed times for the model and prototype approximate only. Further, the tide level varies continuously so that a continuous test at low water imposes conditions which are equivalent to a much longer interval of elapsed time with regard to scour development in the prototype.

No Protection: The first studies were carried out with no scour protection in place in order to determine the extent of the problem. The worst scour developed under the action of 24 feet high waves at Low Water. Deep scour and undermining developed along each edge of the caisson for waves approaching normally to the berthline; deep scour holes developed at each corner of the caisson for waves approaching from E.S.E. The scour hole developed at one of the "side" corners in this latter test is shown in Figure 11. Similar scour patterns were observed for 24 feet high waves at High Water and, although they were not as deep or extensive as those observed at Low Water, they were still quite significant. Waves 12 feet high caused some small localised scour when approaching normally to the berthline at Low Water but not otherwise.

Scour Protection: The protection developed during the model tests consisted of a skirt of woven plastic fabric, fixed to the bottom edges of the caisson and extending horizontally for a distance of 15 feet to provide a barrier between the erosive water currents and the foundation material. The fabric was to be held in place by concrete slabs, 7'6" square and having a net weight under water of 50-60 pounds per square foot. The slabs were to cover the fabric completely, and it was planned that they would be linked together at their edges by simple hinged connections. The protective fabric was to be of sufficiently open weave to permit rapid equalization of pressures above and below it, but the weave was to be close enough to prevent loss of any but the finest rock particles through it. In the tests on the model of the main berth caisson, an open weave soft plastic fabric was used to simulate the protective fabric and the slabs were simulated with square sheets of aluminium, 1.5 inch x 1.5 inch x 0.1 inch thick. The weight under water of the corresponding full size slabs is 56.8 lbs per sq.ft and, if concrete slabs, the required thickness is 8.25 inches. This small discrepancy in thickness of the slabs is of no significance in the context of these studies. In the model studies the model slabs were not linked together so that any tendencies for the slabs to be moved about would be shown up more readily.

The series of tests with the scour protection in place showed that it was completely satisfactory for protection against 12 feet high waves. Waves 24 feet high caused varying amounts of movement of the protective slabs. The greatest movements of slabs were observed when these large waves approached from E.S.E. at low water and they occurred at the "side" corners of the caisson. The conditions at the same side corner as shown in Figure 11 after one hour of testing at these most severe conditions can be seen in Figure 12. Pumping action through the woven fabric resulted in some slight displacement of the foundation material.

9.2 Wave Scour at Approach Caisson: The approach structures linking berth to shore are supported on caissons which are 55 feet (17 m) square in plan. Two possible treatments of the foundations for the approach caissons were considered. In the "lowered" foundation a hole 90 feet (28 m) square is dredged to a depth approximately 5 feet (1.5 m) below the general bed level and partly back-filled with a layer of crushed rock, approximately 2 feet (0.6 m) thick. In the "raised" foundation the layer of crushed rock is placed directly on the sea-bed and the caisson founded on this raised layer. The same scour protection as had been developed forthe main berth caissons was proposed for the approach caissons.

The scour studies on the approach caisson demonstrated clearly that the "lowered" foundation is very much to be preferred over the "raised" foundation. In the tests on the "raised" foundation, the wave-induced currents were so strong near the base of the caisson that the scour protection was attacked violently and its effectiveness was completely destroyed. On the other hand, in the case of the "lowered" foundation, the scour protection was found to be satisfactory even under the worst conditions but only if the cruciform-shaped gap under the caisson between its foundation pads is closed off. When this gap was left open a very strong pumping action with significant scour capacity developed beneath the caisson under wave action.

The conditions after one hour of testing on the "lowered" foundation with the gaps closed, under attack from 24 feet high waves at Low Water can be seen in Figure 13. The foundations of the caisson have been protected satisfactorily. The general erosion of the sea-bed down-wave from the caisson is caused by a strong streaming motion in this region which results from the interaction between the incident waves and the caisson.

10. CHOSEN METHOD OF SCOUR PROTECTION

The simulated scour protection devised for the tests was later considered too involved for actual construction and a form of flexible matting was devised as indicated on Figures 2 and 3. This consisted of a mat of concrete blocks approximately 15 feet square, each block being approximately 1'7" square in plan and 1'3" thick and cast on wire ropes running in two directions which allow flexibility and ready handling. The gaps between the blocks were maintained at 1½" to act as a filter

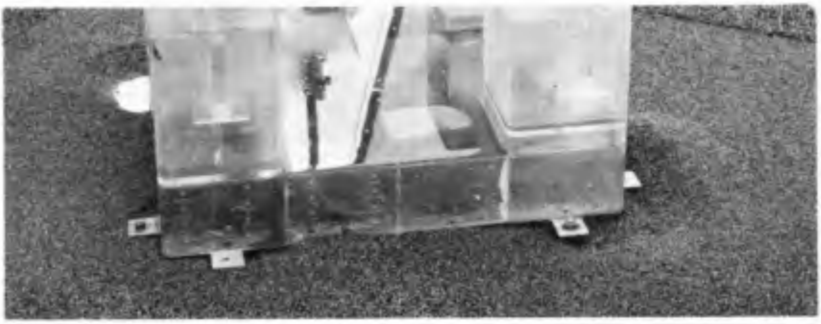

FIG. 11 Scour near Berth Caisson - 24 ft Waves from E.S.E. at L.W.

FIG. 12 Scour protection - 24 ft Waves from E.S.E. at L.W.

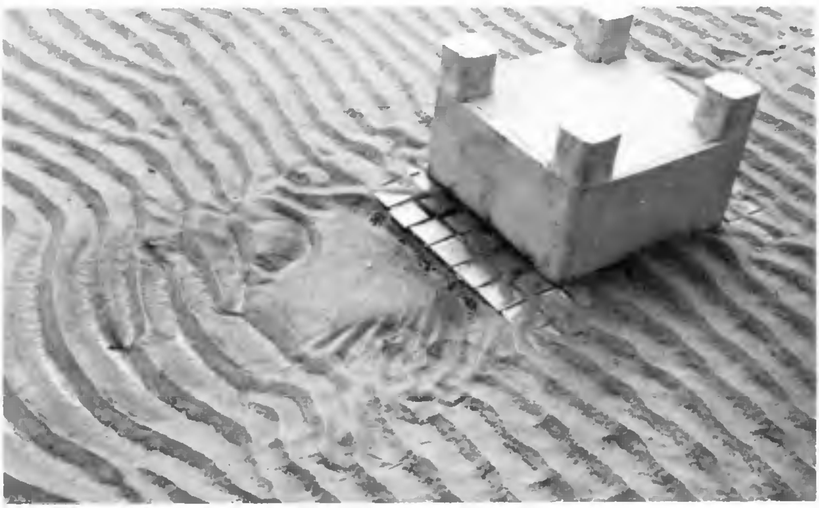

FIG. 13 Scour near Approach Caisson - 24 ft Waves at Low Water.

to prevent the removal of foundation rock during periods of intense wave action. The weight of the matting chosen was equivalent to 90 lbs/sq ft under water, which is nearly twice the model weight, but it was considered that the remedial work that might be required after severe scour could be very serious and therefore a conservative approach was adopted. Figure 14 illustrates the operation of laying scour mats and shows their inherent flexibility. Laying of the units was controlled by divers.

11. PERFORMANCE OF PROTOTYPE

The prototype has been in operation since July 1975 and the Queensland coast experienced 8 cyclones in the 1975-6 cyclone season. A wave-rider buoy has recently been installed by the Department of Harbours and Marine, approximately 19 miles (30 km) to the N.E. of Hay Point in 84 feet (26 m) of water, and a maximum wave height of 14.5 feet (4.4 m) was recorded during the most severe of these cyclones. Only one cyclone passed near Hay Point. This cyclone (code name David) necessitated full emergency precautions at the berth. When operations were closed down, the sea had risen to about 12 feet waves. Recording devices have not yet been placed on the structure and no further observations were possible. Examination of the structure and of the foundation after the cyclone revealed no signs of damage.

12. ACKNOWLEDGEMENT

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FIG. 14 Laying of scour mat