Teil 1: Behaviour of Floating Breakwaters under Wave Action

Teil 2: Multifunctional Wave Absorbing Breakwaters with Extreme Force Dissipation

Teil 3: Physikalische und mathematische Bestimmung der Energieanteile unterschiedlich erzeugter Schwerewellen

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Vorwort

In dieser Mitteilung berichten wir über die Tätigkeiten von zwei ausländischen Gastwissenschaftlern sowie über ein von der Deutschen Forschungsgemeinschaft finanziell unterstütztes Forschungsprojekt (Gr1000/3-2).


Der Gastaufenthalt von Prof. Koola ist in eine lange Tradition deutsch-indischer Forschungsprojekte zwischen der Bergischen Universität und dem Indian Institute of Technology in Madras (= Chennai) integriert, die weiter beliebt werden soll. Im März 1998 wurde hierfür ein Kooperationsvertrag zwischen beiden Einrichtungen unterzeichnet, der den Austausch der Zusammenarbeit hinsichtlich gemeinsamer Forschungsaktivitäten, den Austausch von Studierenden und wissenschaftlichem Personal sowie von Information und Publikationen vorsieht.


Die von beiden Wissenschaftlern am IGAW erarbeiteten Forschungsergebnisse wurden bereits auf internationalen Kongressen einer breiten Fachöffentlichkeit vorgestellt.

Der dritte Teil dieses Mitteilungsheftes ist eine Zusammenfassung des von der DFG finanzierten Forschungsprojektes "Physikalische und mathematische Bestimmung der Energieanteile unterschiedlich erzeugter Scherwellen" (Gr1000/3-2) und steht in direktem Zusammenhang mit zwei bereits zuvor überaus erfolgreich abgeschlossenen Forschungsvorhaben.

Hans Kalenderhoff
Preface

This publication reports on the work of two visiting researchers from other countries and on a research project (Gr10003-2) funded by the German Research Foundation (DFG = Deutsche Forschungsgemeinschaft).

With financial support from the Alexander von Humboldt Foundation, Prof. Koola spent twelve months doing research at the Institute of Geotechnics, Waste Management and Hydro Sciences, and he conducted numerous experiments in the institute’s wave flume. The aim of his research was to develop new types of breakwaters that could ensure the safety of expensive coastal protection buildings, even against "Freak Waves". Although Freak Waves occur very rarely, their effects are always catastrophic. The basic idea of the concept developed by Prof. Koola is the dissipation of that part of the wave with the most energy, using high-momentum jets of water to "cut off" the wave crest and counteract the destructive force of the Freak Wave. The energy needed to produce the jets of water can, if necessary, be obtained from the movement of the sea, using an energy absorbing system.

Prof. Koola has already been involved in important work on projects for the construction and management of coastal protection buildings and wave-energy converters as part of India’s wave-energy program. During his stay in Germany, he was able to pursue his ideas on this innovative approach and on the development of alternative sources of energy and to work on a scientific basis for their realization.

Prof. Koola’s stay is part of a long tradition of German-Indian research projects involving the University of Wuppertal and the Indian Institute of Technology in Madras (= Chennai), a tradition that is to be intensified. In March 1998 these two institutes signed a cooperation agreement that provides for joint research activities, exchanges at the student and research-staff level, as well as the exchange of information and publications.

Mr. Ebah Rashad Abdel Salam Tolba of the Suez Canal University in Port Said, Egypt, worked at the Institute of Geotechnics, Waste Management and Hydro Sciences from April 1996 until May 1998 as a Channel System scholarship holder. In the two years he spent here, Mr. Tolba carried out a research program on the "Behaviour of Floating Breakwaters under Wave Action", a program that included intensive study of the literature, a large number of hydraulic experiments using models, and detailed physical and mathematical analysis. He integrated the results of his studies into a thesis, which he submitted for a PhD in Civil Engineering after he returned to the Faculty of Engineering at the Suez Canal University. He has since been awarded a very good PhD.

The Channel System, under which young scientists from other countries carry out their research at German universities and then complete a doctorate at their home university, is an important part of cooperation agreements between universities in Germany and other countries. An agreement made between the University of Wuppertal and the Suez Canal University on March 31st 1998 involves hydraulic engineering to begin with, but is planned to be extended to other areas of civil engineering.

The research findings obtained at the Institute of Geotechnics, Waste Management and Hydro Sciences by the two scientists referred to in this publication have already been presented to a wider audience of experts at international conferences.

The third part of this publication is a summary of the research project "Physical and mathematical investigations on the energy distribution of regular gravity waves generated by different wavemaker mechanisms" (Gr10003-2). This project is funded by the DFG, and it is closely connected with two very successful research projects that have already been completed.

Hans Kudlauhoff

INHALTSVERZEICHNIS

TEIL 1: BEHAVIOUR OF FLOATING BREAKWATERS UNDER WAVE ACTION

1 INTRODUCTION ................................................................. 2
   1.1 Wave protection .................................................. 2
   1.2 Floating breakwaters ............................................ 3
   1.3 Purpose of the study ............................................. 4

2 LITERATURE SURVEY.......................................................... 6
   2.1 Introduction .......................................................... 6
   2.1.1 Breakwaters groups ............................................ 8
   2.1.2 Theory of breakwater action .................................. 10
   2.1.3 Transmission characteristics .................................. 12
   2.2 Wave theory selection for the investigation ................. 15
   2.2.1 Limiting conditions, linear theory: ....................... 15
   2.3 Floating structure motion and forces ......................... 16
   2.3.1 Freely floating structures .................................. 16
   2.3.2 Moored floating structures ................................. 17
   2.3.2.1 Anchoring ................................................ 19

3 THEORETICAL CONSIDERATIONS ........................................... 21
   3.1 Dimensional considerations ..................................... 21
   3.1.1 Wave attenuation ............................................. 21
   3.2 Analytical considerations ....................................... 24
   3.2.1 Equation of the heave motion of the floating breakwater 24

4 LABORATORY EQUIPMENT AND PROCEDURES ........................... 29
   4.1 The wave flume .................................................... 30
   4.1.1 General remarks when using wave flume ................. 30
   4.1.2 Generator ..................................................... 35
   4.1.3 Beaches ....................................................... 35
   4.2 Measurement of the water surface profile ................... 35
   4.2.1 Wave probes ................................................ 35
   4.2.2 Measurement of incident and reflected waves .......... 37
TEIL 3: PHYSikalische UND MATHematische BESTIMMUNG DER ENERGIEANTEILE UNTERSCHIEDLICH ERZEUGTER SCHWEREWELLEN

1 Arbeits- und Ergebnisbericht .................................................................................................................. 130
1.1 Ausgangsfragen und Zielsetzung des Projekts .................................................................................. 130
1.2 Entwicklung des Arbeits- und Auswertungsprogramms .................................................................. 132
1.2.1 Probleme bei der Umsetzung des Arbeitsprogramms .................................................................. 133
1.2.2 Datenauswertung .......................................................................................................................... 135
1.3 Darstellung der Ergebnisse .................................................................................................................. 136
1.3.1 Auslenkung der Wasseroberfläche und Orbitalgeschwindigkeiten unterhalb der Wellen ........ 136
1.4 Ergebnisanalyse ................................................................................................................................... 137
1.4.1 Auslenkung der Wasseroberflächen .............................................................................................. 137
1.4.2 Orbitalgeschwindigkeiten ............................................................................................................. 137
1.5 Zusammenfassung ................................................................................................................................ 145
1.6 Literaturverzeichnis .............................................................................................................................. 146

BEHAVIOUR OF FLOATING BREAKWATERS UNDER WAVE ACTION

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1 INTRODUCTION

1.1 Wave protection

The development of coastal or inland waters may often depend on the anticipated wind-generated wave climate at a specific site. Breakwaters of various dimensions and design have been widely employed to increase the use of locations exposed to wave attack. The purpose of installing a breakwater is to reduce the height of the incident water waves to a level compatible with the intended use of the site on their leeward side. Economics and the required degree of wave protection will often dictate possible breakwater alternatives. The increase in the number of private pleasure craft and small commercial vessels has generated a demand for additional sheltered mooring. Many naturally sheltered locations along coastlines have been developed to provide mooring, yet do not have the additional capacity to accommodate the influx of vessels. Assuming the owners of small craft reside in established population centers, their demand for convenient and accessible mooring often dictates where and therefore how such shelter can be provided. It is for these reasons that man-made structures are installed to protect personnel from incident waves where nature offers no protection.

This research evaluates the performance of a floating pontoon breakwater with respect to the anticipated wave climate and permissible disturbance levels at small boat marinas. There are no universal standards to define the maximum acceptable wave height within a marina. The degree of wave protection provided will probably depend on the boat owner’s perception of acceptable costs and damage risks. For small craft these considerations often limit marina feasibility to naturally sheltered or semi-sheltered waters.

The rubble mound breakwater has been widely used to attenuate surface waves. The breakwater is a fixed, pervious gravity structure constructed of graded rock. It has found frequent application in coastal regions due to its durability and the high degree of wave protection it can provide. The structure reduces incident wave heights by reflection and by inducing wave breaking and viscous losses as water particles interact with the breakwater. Though it has proved an effective wave attenuator, the rubble mound breakwater is limited in its potential application for marina use. As the typical cross section is trapezoidal, fig. 1.1, the ultimate breakwater cost will increase significantly with the depth of water at a given site. Due to the mass of the rubble mound breakwater, a firm subsoil capable of providing adequate foundations is necessary. As the breakwater extends to the full depth of water, it acts as a pervious vertical barrier to shoreline processes. This may interrupt littoral transport including local silt or scour problems or may disrupt circulation and cause water quality deterioration within a marina. It is for these reasons that alternative breakwater designs are of interest for small boat marina application.

![Typical cross section of a rubble mound breakwater](image)

### 1.2 Floating breakwaters

An alternative wave attenuator is the floating breakwater. The structures are typically of limited draft and rely on the interaction of structure and wave in only the upper portion of the water column. The floating breakwater typically consists of two modules anchored by chains or cables to the bed of the sea. The floating breakwater, traditionally abbreviated FBW, studied in three-dimensional scales has six degrees of freedom. When this FBW is subjected to the attack of incident waves it will experience the rectilinear motions of heave, sway, and surge, and the angular motions of yaw, pitch, and roll. These potential motions represent the structure’s six degrees of freedom as illustrated in fig. 1.2.

The relative magnitudes of these motions will depend on the incident wave climate, the dynamic properties of the pontoon, the mooring arrangement, and the connection detail between pontoon modules. A two-dimensional model test will limit the possible degrees of freedom to only the motions of heave, sway and roll. It is assumed that this two-dimensional analysis may represent the worst case for the breakwater and that the heave, sway and roll motions will dominate three-dimensional performance of a prototype pontoon in the same manner as in two-dimensions.

Floating breakwaters attenuate surface water waves through the mechanisms of reflection, out of phase damping, destruction of water particle orbital motions, and viscous damping. As a wave attacks the structure, some energy will be reflected, some dissipated, some will induce breakwater motions and some will pass beneath the structure. The induced body motions will subsequently generate waves and the restraint of body motion will be provided by the mooring system. In theory, the structure should provide greater wave attenuation in deep water when $(d/L > 1/2)$ as a greater percentage of the wave’s average kinetic energy is located in the upper region of the water column with which the breakwater interacts.

For wave attenuation purposes concrete pontoon floating breakwaters are typically massive with large mass moments of inertia. This provides relatively long natural periods of structure motion. Floating breakwaters are generally ineffective in wave attenuators in long wave length climates as the structures will tend to ride with the wave profile rather than interact with the wave. It is in the shorter wavelength climates, characteristics of sheltered or semi-sheltered waters that floating breakwaters are capable of providing significant wave attenuation.

Thus the selection of a floating breakwater must be determined after the analysis of the anticipated wave climate at a specific location. An attractive benefit of floating breakwaters is that their cost is relatively insensitive to water depth at a site. As the structures are buoyant, the breakwater is mobile that make facilitates realignment or removal if desired. Of importance, however, is the dual use potential of a floating breakwater. This may permit the structure to act as both pier and breakwater. As the breakwater does not extend to the full depth of the water, interruption of littoral processes and local circulation would not be anticipated. On the other hand, floating breakwaters anchored with chains or cables may resist some disadvantages. Some of these disadvantages are as follows:

1. Large roll motion may affect the performance of the FBW to be used as a pier.
2. The horizontal motion of the pontoon (sway) generates waves that are not recommended on the leeward side of the structure. In addition, the sway motions of the pontoon allow the body to impact with the boat when it is used as a pier.
3. The pontoon moves upward and downward with the water surface according to the tide. When it moves upward with the rising tide, the mooring lines may reach its maximum length, in this case the designed draft may increase and over toppling may occur as shown in fig. 1.3. On the other hand, when the pontoon moves downward with the falling tide, the mooring lines will be deflected. This deflection increases the sway motion and may affect the performance of the FBW to be used.
4. If the FBW is exposed to strong waves, cracks will appear at the connection between the chains and the pontoon and failure may occur at this zone as shown in fig. 1.3.

The floating breakwater is however not a panacea for wave protection. The degree of protection provided is not as substantial as that provided by a rubble mound breakwater. The structures are generally ineffective wave attenuators when subjected to incident waves of extreme height or long periods. Due to the induced motions of the breakwater, the design must allow for fatigue forces in the floating members as well as in the connection detail between the members.

Recognition of floating breakwater limitation is essential. Site conditions and economics will dictate appropriate wave protection measures incorporated in the design of small boat marinas. The floating breakwater is a feasible and effective alternative when installed at sites within its capabilities.

1.3 Purpose of the study

The purpose of this research is to study a suggested solution to overcome the above mentioned disadvantages of the FBW moored with chains or cables. The basic idea of the suggested solution is to replace the mooring system with piles instead of mooring lines (chains and cables). Details of the suggested solution are given in fig. 1.5. Such a system may overcome the problem of sway motion, which is prevented in this case, and in addition the roll motion is limited due to the existence of the piles. The pontoon is allowed to move freely (up and down) with the tides with the same designed

draft, and avoids the problems of the wave over topping and sway motions due to the effect of tides on the mooring lines. The suggested solution also helps to solve the problems of failure at the connection between the pontoon and the mooring lines as the body moves freely in heave motion through the piles. This suggested system also has the same advantages of the floating breakwaters moored with chains which are mentioned above.

The problem posed for study concerns the motion of the body through this suggested system of floating breakwater. As the body is prevented to move in sway motion, the only motions possible for the pontoon are free heave with limited roll. The study also determines the effect of each individual motion on the efficiency of the FBW as a function of transmission and reflection coefficients under different wave conditions.
The purpose of this research may be summarized in the follows:
1. To study the effect of heave motion only on the efficiency of the FBW under different wave conditions and body parameters.
2. To investigate the effect of limited roll motion only on the efficiency of the FBW as a function of transmission and reflection coefficients.
3. To determine the characteristics of heave motion such as, the height of the heave and its phase angle with the standing waves under different wave conditions.
4. To determine the net horizontal force on the pontoon due to wave attack.

Fig. 15: Definition sketch of the suggested floating breakwater

2 LITERATURE SURVEY
2.1 Introduction
A floating breakwater may be defined as a structure or device which combines the ability to appreciably reduce the height of ocean waves in its lee with a degree of mobility sufficient to permit ready transportation over considerable distances and speed installation at the site. Such a device would find application wherever wave protection is necessary for limited a period, as in marinas, offshore drilling operations, or where an installation is required to be completed in a very short time, e.g. for amphibious military operations.

Military needs provided an early incentive for the deployment of floating breakwaters. The Bombardon breakwater (26), employed as one of the methods of building instant harbors for the invasion forces on the Normandy beaches in World War II, represent the first major utilization of such structures. The potential military uses for mobile harbors was the incentive for extensive work in the postwar years on concepts, theories, and experimentation with configurations which could be towed to a site, either anchored or sunk in place, with the possibility of later removal and reuse. Representative studies of the postwar period are those by Minikin (34), who discussed floating breakwaters in general terms, Carr (2), who set out some of the principles governing the characteristics of "mobile" breakwaters, and the review of the performance of the Bombardon by Lochner, Faber, and Penn (26).

Several attempts have been made to achieve the requirements of a mobile breakwater, some with a fair degree of success. A considerable achievement in this direction was the general study of the mobile breakwater problem by the Hydromechanics Laboratory, California Institute of Technology. This study has resulted in a better understanding of the principles which must govern any mobile breakwater, and provides the systematic analysis of various projected mobile breakwater schemes. In 1957, the Naval Civil Engineering Laboratory (in the USA) began a useful survey of the existing knowledge of transportable units that could serve as breakwaters or piers.

The results of the survey are summarized as Technical Report 127 (35), which is an invaluable state of the assessment up to that time, with particular emphasis on military use under rather severe wave conditions. A sequel to this report was issued as Technical Report 727 (48), which is a survey of concepts for transportable breakwaters, including over 60 in the "floating" category. It contains the statement: "Recurring efforts, spanning 125 years or more, have not produced a breakwater for temporary installations which are easily transported, effective over a broad range of wave conditions, and able to endure very high seas."

Further study is warranted in view of potential uses where criteria for the protection of a given area do not entail the transportable or temporary breakwaters, and where there is a natural exposure limit to wave conditions and high seas. Applications even in the extreme conditions have been presented in the form of a new concept of tethered floats proposed by Seymour (41) and with a breakwater form presented by Harris (11).

Most of the naturally protected harbors near the maximum use centers have been developed already, so artificial protection will be required in most expansions. Many lakes and reservoirs undergo large seasonal fluctuations, a condition to which the floating breakwater is uniquely adaptable. There are many places in the marine setting where the traditional fixed structure is not suitable; the floating one appears to be a competitive alternative where exposure to extreme wave activity is limited for some natural or operational reasons. Fixed breakwaters of rubble, timber, piling, or fill, are quite expensive if the water depth is very great and if foundation conditions are not suitable.

Fixed structures may also interfere adversely with the balance of littoral drift, shellfish habitats, the paths of migratory fish, and restrain water circulation vital to the maintenance of water quality. It is natural to look to floating structures as possible alternatives in the development of small boat harbors, but there are other important uses for them as well.

The shutdown time of weather dependent, water-borne activities like construction, logging, and cargo handling, could be reduced by floating transportable breakwaters.

The erosion of shorelines in certain critical areas could be controlled by floating breakwaters designed to alter the eroding "winter" wave characteristics. As waterfront property becomes increasingly scarce and expensive, transfer of some of the conventional shore-based functions to floating facilities will become economically feasible. Among these functions are processing plants for bulk cargo, fish products, water supply and waste treatment facilities, parking and storage facilities and recreation and living accommodation.

In view of the fact that several floating breakwaters are in operation and that there is considerable potential for additional sites and uses, it seems timely to reexamine them from today's perspective. Some floating breakwaters have been installed, and these provide information on initial and maintenance costs and the general operational problems that cannot be studied in the laboratory.
2.1.1 Breakwaters groups

A floating breakwater is a displacement vessel at anchor, its motions can be expressed in components relative either to the water or to the structure. It seems desirable to adopt naval architectural terminology and describe the motions in the translational components of surge (longitudinal) motion, sway (beam) motion, and heave (vertical) motion, and the rotational components of roll (about the longitudinal axis), pitch (about the beam axis), and yaw (about the vertical axis). The parameter in common use to describe the performance characterisitc is the transmission coefficient, which can be defined as either the ratio of transmitted wave height to incident wave height, or the ratio of transmitted to incident energy. This characteristic is dealt with in more detail later. McCartney (32), studied the various types of floating breakwaters, investigating the advantages and the disadvantages of the different types. Alternative mooring systems and anchorage methods are summarized. Some geometric similarities of floating breakwater configurations allow arbitrary grouping into six basic forms, as shown in figures 2.1 - 2.3.

Group I: Single prism

This group contains the simplest forms and those which attract the first attentions of the theorist and experimentalist alike. The prismic form also offers the best prospects for multiple use, i.e. for walkways, storage, boot slips, etc. Kato, Norita, Uekita and Hagino (33) reported an interesting comparison of four basic shapes: inverse trapezoid, trapezoid, rectangular, and triangle sections. The inverse trapezoid shape yielded lower transmission coefficients, but developed higher anchor forces. Jarlan and Marks (18) introduced the concept of perforating the walls of breakwater to increase energy dissipation through turbulent jet action, and thus, reduce the amount of energy available for transmission.

Group II: Rafts

The geometric dimension dominant in this group is the beam width, or horizontal extension, with individual members having a small draft, so their attenuating characteristics rely largely upon suppressing the vertical barrier to transmission. The log boom is probably one of the most widely used forms. It is effective for waves of short periods, but becomes ineffective for longer waves. Kennedy and Marsalek (24) conclude that "A flexible porous floating breakwater extending over two or more wave lengths can greatly attenuate waves of moderate length ". Logs, however, are vulnerable to marine borers, lose flotation with time, are difficult to contain, so their use is limited to places where the suppression of surface "chop" is the primary concern.

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![Group I: Single Prism](image1)

![Group II: Rafts](image2)

Two fabricated forms which are designed to suppress vertical motion as a major attenuating mechanism are the breakwaters presented by Hasler (13) and Harris (11). Experimental studies of these forms have been carried out under natural wave environments as well as under conventional laboratory conditions. Of particular significance in the assessments of these two forms are the relatively small anchor forces encountered. This response is due in part to the small vertical projection these forms provide to the waves; other likely factors are the randomness of the wave input, the short-crestedness of the waves, and variable directions.

Group III: Catamaran

The Catamaran pontoon illustrates one way of distributing a given mass to achieve a longer roll period and potentially a more stable platform than would be achieved with the same mass in the single prism group. The extra corners provide additional zones of energy loss, and the water mass between the hulls may add damping actions, especially to sway (beamwise) excitation. The Tenakee-Stiklo breakwater design (Miller, 33) is of the twin-hull type; the Friday Harbor one has two closely spaced pontoons forming each hull with a wooden deck so it can serve as a dock as well as a breakwater (Pacific Northwest Sea, Vol.5, No.3, 1972).

Group IV: A-frame

The various members of this group utilize combinations of vertical walls for reflecting surfaces and outriggers for stability and to develop a large roll period for a given weight. Anchor forces can be minimized by dimensioning the wall so that the motion at the point of anchor attachment due to roll compensates the motion due to sway.

![Group III: Catamaran](image3)

![Group IV: A-Frame](image4)

Fig. 2.2: Definition sketch for catamaran and A-frame breakwaters

Group V: Flexible assemblies

The members of this group could be classified in the "raft" group, but are somewhat different because of the degree of flexibility involved, which ranges from thin membranes, through fluid-filled bags, to used tires bundled together in a variety of patterns. The performance of the bags filled with air, water, or viscous liquids have been measured with the bags floating, restrained at an intermediate depth, and lying on the bottom. The problems of design and operation associated with them tends to discourage their use.
Group VI: Tethered floats

This breakwater system consisting of an array of individual floating geometric forms, small relative to the wave length, anchored, but not tied to each other, as proposed by Seymour (41), represents a new floating breakwater class that promises versatile design capabilities to meet a wide range of wave conditions. The two systems of groups can be placed in the same geometric class, although they are much less alike in the dynamic sense. The design of the tethered floats and the attendant anchor system is linked analytically very closely to the wave dynamics and kinematics, whereas the wave/barrier is a more empirical solution to attenuating waves with an array of discrete objects.

Fig. 2.3: Definition sketch for flexible assemblies and tethered floats breakwaters

2.1.2 Theory of breakwater action

The action of any breakwater may be considered in terms of basic wave processes. The theory of breakwater action therefore includes all wave processes which may result in wave height attenuation. These include the processes of wave refraction, wave interference, wave dissipation, wave reflection and wave transmission. These processes may be briefly described as follows:

1. Wave refraction is a process resulting from changes in wave velocity, and can result in either an increase or decrease in wave steepness. In the first case, wave breaking with resulting energy loss may occur, and in the second case the desired decrease in wave disturbance is obtained directly.

2. Wave interference is a process involving the vector addition of the partial velocities of two or more superimposed wave trains. The wave height may be increased or decreased, depending on the direction and phase of the interfering wave trains.

3. Wave dissipation is the conversion of wave energy into heat energy through the action of the frictional forces.

4. Wave reflection, in the ideal case, is a change in wave direction without energy loss, hence may be likened to perfectly elastic impact.

1. Wave refraction

The familiar case of water wave refraction is that due to changes in bottom topography, the non-uniform depth corresponding to non-uniform wave velocity in the area considered. The basic method of analysis of such problems consists of plotting the position of successive wave crests by consideration of the local wave velocity at each point, followed by construction of orthogonals to the wave crests. The energy transmitted per unit time, or wave power:

\[ P = EC \]  \hspace{1cm} (2.1)

is assumed constant between any pair of orthogonals, and using of this relationship the change in wave height can be determined.

A different case of wave refraction occurs when waves advance into a region where a current exists. In this case, the wave velocity with respect to the fixed frame of reference is equal to the vector sum of the wave velocity with respect to the water and of the water velocity (current) with respect to the fixed frames of reference. For the case of an opposing current and deep-water waves, an analytical expression for the change in wave steepness can be obtained; Taylor (46), shows that an opposing current of velocity one-fourth the wave velocity will cause a deep-water wave of any initial steepness to build up a steepness of 1/7 and so presumably break and dissipate its energy. Such a current could be considered a mobile breakwater.

2. Wave interference

A progressive wave train traveling in the x-positive direction may be represented by the equation for the surface elevation at any point x and time t:

\[ \eta = \frac{H}{2} \sin \left( \frac{2\pi}{L} \left( \frac{t}{T} - x \right) \right) \]  \hspace{1cm} (2.2)

If a second wave train of identical period and amplitude, but with phase difference \( \phi \) travels in the same directions, its surface elevation is:

\[ \eta_s = \frac{H}{2} \sin \left( \frac{2\pi}{T} \left( \frac{t}{T} + \frac{x}{L} \right) + \phi \right) \]  \hspace{1cm} (2.3)

and the net motion becomes:

\[ \eta = \eta_s + \eta = \frac{H}{2} \sqrt{2} \sin \left[ 2\pi \left( \frac{t}{T} + \frac{x}{L} \right) + \phi \right] \]  \hspace{1cm} (2.4)

Thus the resultant wave height will vary from twice the original one down to zero as the phase difference varies from zero to 180 (one half wave length). These principles of wave interference could be applied as a mobile breakwater if some means could be devised for producing the secondary wave.

Interference can also be considered as occurring when waves advance into a region characterized by a vertical stratified current. Thus, if a current exists in a surface layer, the wave motion in this layer will be advanced or retarded with respect to the wave motion in the undisturbed deeper regions, and destructive interference can occur. It has been shown by Taylor (45) that this mechanism is responsible for the (limited) performance of the pneumatic breakwater, a device often proposed as a mobile breakwater. In 1943, Taylor (44) offered a complete analysis of the problem and derived a relationship between the length of wave which can be damped and the magnitude of the horizontal surface current resulting from the vertical current. Calculations by Carr (2) based on Taylor's theory, show the current required to damp very short period waves to be moderate, but for wave lengths or periods of the order to be expected in typical coastal environment, the current values and corresponding power requirements to generate these currents become enormous.
3- Wave dissipation

Wave energy can be dissipated in the form of heat through the mechanism of fluid turbulence. For the dissipation to proceed at a high level, the turbulence must be general and violent; this is the wave breaking process. Wave breaking occurs naturally on shelving coasts, where most wind-generated wave energy is finally dissipated. The process of wave breaking on such shorelines is preceded by an increase of wave steepness (due to shoaling) to the point of instability.

Artificial offshore bars or reefs to induce wave breaking are very limited forms of mobile breakwater. Anchor possibility for including wave breaking is a submerged vertical barrier that comes close to the still water level. Morison (37) showed that such barriers can cause wave breaking under some conditions. However, such a barrier is also a fairly efficient reflector and must be designed to withstand rather large forces.

4- Wave reflection

An important consideration in the reflection process is the question of the magnitude of the force acting on the reflecting barrier. These forces can be computed in terms of wave heights by the use of various formulae, such as that of Saint-Venant (fixed vertical barrier). When a two-dimensional structure is tested using a train of waves in a wave channel, a multi-reflected wave system is developed between the structure and the wave generator. The decomposition of incident and reflected wave form the multi-reflected random wave system is essential for such experiments to be useful. Decomposition techniques using a simultaneous recording of wave profiles at two adjacent stations in a wave flume were first used by Kajima (21) and Thornton and Callou (47). Analysis using these investigations requires cross-correlation of the two time series.

A similar method was also later used by Morden (36), he presented a technique similar to the previous one but which directly uses the Fourier coefficients of all the frequency resolutions instead of the cross-spectra.

2.1.3 Transmission characteristics

The wave field, or amount of energy contained in the wave behind a floating breakwater represents the residual of the incident wave field that was acted upon by a combination of the following mechanisms attributable to the structure:

1. Reflection,
2. Energy loss through turbulence produced by wave breaking conditions, by hydraulic damping of the structure, by wave drag,
3. Interference with internal orbital motions and dynamic pressure fields,
4. Deformations of the structure and its moorings,
5. Superimposition of wave components in the incident field on those generated by motion of the structure itself, or by the motion of water masses coupled to the structure in some way.

The traditional way of quantifying the transmission characteristics of a floating breakwater has been through a "transmission coefficient", defined as the ratio of the transmitted wave height to the incident wave height. For completeness, a wave length (or period or frequency) should be specified for each value of transmission coefficient. The wave energy that appears in the lee of the breakwater is a combination of fractions which:

1- Pass the structure as if it were fixed,
2- Are generated by the translation and / or rotational motion imparted to the structure by the incident wave.

3- Are due to any coupled motion between the structure and the wave.

Waves generated by the structure may or may not be in the phase with any components in the incident field, and the out of phase conditions often account for a minimum of the total transmitted energy.

Suirko and Haden (43) studied experimentally the effects of sway, roll and heave motions on the performance of a prismatic section (Group I), and Carr (2) assumed that the transmission coefficients for a rectangular cross section in shallow water of depth 'd' could be predicted from linear wave theory, hydrostatic pressure distribution, linear damping and obtaining:

\[ C_t = \frac{H_t}{H_i} = \frac{1}{\sqrt{1 + \left( \frac{\pi W}{4d} \right)^2}} \]  \hspace{2cm} (2.5)

and

\[ C_r = \frac{H_r}{H_i} = \frac{1}{\sqrt{1 + \left( \frac{\pi d}{W} \right)^2}} \]  \hspace{2cm} (2.6)

where:

- \( C_t \) is the transmission coefficient,
- \( C_r \) is the reflection coefficient,
- \( W \) is the weight of the structure per unit length of wave crest,
- \( \gamma \) is the specific weight of the water.

Equations (2.5, 2.6) apply to a freely floating structure. Where some restraint is placed on the motion of the structure, the combined system has a natural period of oscillation:

\[ T_s = 2\pi \sqrt{\frac{K_g}{W}} \]  \hspace{2cm} (2.7)

where: \( K \) is the spring constant of an elastic mooring system. This provides a new transmission coefficient:

\[ C_t = \sqrt{\frac{1}{1 + \left( \frac{\pi W}{4d} \right)^2 \left( \frac{T_s}{T} \right)^2}} \]  \hspace{2cm} (2.8)

From equations (2.7 and 2.8) the values of \( C_r / C_t \) varies with \( T_s / T \), and when this is unity \( C_r / C_t \) is optimum. Ursell (49) and Weigell (52) have derived theoretical relationships of \( C_r \) for deep-water and transitional depths respectively. For the partial transmission and partial reflection of uniform periodic water waves in deep water for a fixed vertical, infinitely thin barrier extending from the water surface to some depth below this surface, the solution of Ursell (42) was:

\[ \frac{H_r}{H_i} = \frac{k}{\sqrt{\pi^2 + k^2}} \left( \frac{2\pi / L}{1 + k^2 (2\pi / L)} \right) \]  \hspace{2cm} (2.9)

\[ -12- \]
where:

\[ I_1^2(2\pi d / L) \text{ and } k_0^2(2\pi d / L) \text{ are modified Bessel functions; } S \text{ is the distance from the water surface to the bottom of the structure. For water of any depth, Wiegell (52) developed a theory based on the assumption that the power being transmitted by a wave between the bottom of the vertical barrier and the sea bottom, assuming the structure is not there, will be the power transmitted past the structure. His solution was:} \]

\[ \frac{H_s}{H_i} = \sqrt{\frac{P_s}{P_i}} \sqrt{\frac{4\pi (y + d) L \sinh 4\pi (y + d) L}{\sinh 4\pi L \sinh 4\pi L}} \left( \frac{4\pi L}{\sinh 4\pi L} \right). \tag{2.10} \]

Stoker (42) has treated the case of a rigid board of length 2B, fixed at the still water surface in shallow water, using linear wave theory. It was found that:

\[ \frac{H_s}{H_i} = \frac{1}{\sqrt{1 + (2\pi B / L)^2}} \tag{2.11} \]

\[ \frac{H_s}{H_i} = \frac{2\pi B / L}{\sqrt{1 + (2\pi B / L)^2}} \tag{2.12} \]

Carr, Healy and Stelterg (50) developed an equation for shallow water which is similar to equation (2.11), but includes a term to express the effect of the finite draft:

\[ \frac{H_s}{H_i} = \frac{1}{\left(1 + \frac{2\pi B}{L} (1 + S) \right)^{1/2}} \tag{2.13} \]

where:

\( S \) is the ratio of the depth of the immersed body below the undisturbed water level to the vertical distance from the bottom of the body to the sea bed. \( H_s / H_i \) can be obtained from:

\[ \left( \frac{H_s}{H_i} \right)^2 + \left( \frac{H_s}{H_i} \right)^2 = 1 \tag{2.14} \]

which holds, provided that no energy is lost or transferred to other frequencies in the process. John (19) treated the case of cylinders floating at the surface under the action of waves represented by linear theory. The case of a flat structure was taken to be a section of cylinder of infinite diameter. In shallow water, the coefficient for an infinitely thin structure is:

\[ \frac{H_s}{H_i} = \frac{\left(2\pi B / L\right)^2}{\left[1 - 0.4 \left(2\pi B / L\right)^2 \right] + \left(2\pi B / L\right)^2 \left(2\pi B / L\right) \left(2\pi B / L\right) - \frac{1}{15} \left(2\pi B / L\right)^2 + \left(2\pi B / L\right)^2 \left(2\pi B / L\right)^2} \tag{2.15} \]

\( H_s / H_i \) can be obtained from (2.14).

Model tests have been conducted by Ross (40) including the measurement of forces in the mooring system. The case of a beam of finite specific weight and finite elasticity in shallow water has been studied mathematically by Stoker (42). Linear wave theory was used. The resulting equations were solved numerically for a number of conditions so that some information could be obtained on the effect of the various parameters on the wave field.

Martin and Addo (31) developed a linear, two dimensional model from ship-motion theory for sway, heave and roll motions, and transmitted and reflected wave fields, and apply it to a rectangular breakwater with comparison to experimental data.

2.2 Wave theory selection for the investigation

To apply the theories of water-particle motion and other phenomena to the wave, simple models must be used. The following general assumptions are made: The wave is a simple component of the spectrum; the wave is part of a train has constant height and period; the height is considered to be very small compared to the length, infinitesimally small; the wave form is sinusoidal, with crest and trough of equal vertical distances from the mean water level; the wave is long-crested and is propagating in a straight line; the depth of water is constant or changes very slowly; the water is considered to be nonviscous.

The main concerns of the coastal engineer in respect to wave theory are the motions of water particles (amplitudes, velocities and acceleration), pressure fluctuations, wave energy and the water surface profiles. Equations for these take the form of exponential series. Theory which incorporates only the first term of the series is called linear, and is the simplest to treat. Fortunately, most engineering problems can be handled with sufficient accuracy by linear theory. Other forms commonly used by the mathematician are second, third, and fifth-order forms, some of which have been shown to be less accurate than first-order when compared to certain theoretical criteria and flume tests. It is only when waves travel in the shallowing zone, and wave steepness increases, that the second, and higher-order theorems are required.

2.2.1 Limiting conditions, linear theory:

Stokes pointed out that for the linear theory to be valid, the wave steepness must be small, and in addition \( L^2 H / 2d^3 \) must be small. The limiting value of \( L^2 H / 2d^3 \) was given by Longuet-Higgins (27) as:

\[ L^2 H / 2d^3 \ll 16\pi^2 / 3 \tag{2.16} \]

The relatively reasonable approximation to regular waves given by the sinusoidal, first order approximation, forms the basis for much of the wave force work in ocean engineering. There are two reasons for this. The most important one is the fact that the first order theory is wholly linear so any number of waves can be simply added together to represent a realistic sea condition. These waves can have any heights, lengths and direction of propagation. Likewise, any wave forces that obtained as linear operation on the waves will be given by the sum of the forces associated with each component used in the sum that represent the sea state.

The second reason is that the mathematical formulations for the linearized problem and the solution are relatively simple and correspondingly much easier to understand and use than non-linear theories.

Wright and Yamamoto (57) compared the kinematics predicted by the Airy theory to those of the Stream function theory of Dean (6), and found that the maximum values of horizontal velocities and
accelerations in the Airy theory are nearly equal to their respective values in the stream function theory under the condition tested. Specifically, the maximum horizontal velocities are 16% greater than the predicted by Airy theory and the horizontal acceleration are 8% greater. These results are justified since the stream function theory evaluates the total acceleration while the Airy theory evaluates only the temporal acceleration.

2.3 Floating structure motion and forces

2.3.1 Freely floating structures

The motion of a freely-floating body in a seaway is extremely complex. First, it is necessary to measure the surface of the sea with all its irregularities, and as waves are of a short-crested, the problem must be considered in three-dimensions.

Second, it is necessary to determine how a freely floating body, having six degrees of freedom, will act in seaway. Following the usual methods of mechanics, a complicated set of floating body motions can be considered as being composed of six simple components. Three of these are translational: The sway along the x-axis taken in the plane of symmetry in the force and aft direction, the surge along the y-axis taken laterally, and the heave along z-axis taken as vertical in the plane of symmetry. Three other components of the motion are rotational: pitching about the x-axis, rolling about the y-axis, and yawing about the z-axis.

To define the motion of a floating body, one must have a knowledge of all six of its components. The determination of the displacements and accelerations requires generally a knowledge of all six components of motion, in regard to both the amplitudes and the phase relationship between these components.

Simultaneous information on the motions of a floating body and on the sea waves which excite these motions are therefore required. The most direct method of obtaining the above information would appear to be by observations made on the body at sea. However, numerous practical difficulties immediately arise. These are:

1. The changeable nature of the sea, which prevents an observed condition from strongly ever occurring again for the repeated observation which is usually desired in any investigation.
2. Only three angular motions are readily observed: roll and pitch can be observed with respect to the horizon and yaw with respect to a direction gyro. Direct observation of the translatory components, that is of surge, sway and heave, is not possible because of the absence of any visible datum from which they can be measured. Apparently the only way to obtain these components of motion is by the double integration of the accelerations, and finally.
3. Even if a complete record were available, an analysis of the six-component motion to obtain the effects of the particular parameters of the body form would be very difficult.

The solution of this problem may be approached in the following manner:

a. Studying the problem theoretically.
b. Using experimental results as obtained under controlled wave and model conditions.
c. Making direct measurements on a prototype under actual conditions.
d. Statistical observations.

If possible, all four methods should be used and the results compared. Studies have been made in the past (Krylov, (19)) of the motion of rolling and pitching of vessels. Many have been studied by Huskind (12). Other investigations have also studied the problem; good summaries are given by Weinblum and St. Denis (51). They present equations and graphs of various ship motion functions, among which are the magnification (response) factors which are shown to be functions of the ratio of the wave period to the natural period of the vessel, and the exciting factors which were shown to be functions of the ratio of the wave length to ship length. Both of these sets of factors were dependent upon several waves and vessel characteristics.

The development of a comprehensive theory of body motions in regular waves is therefore necessary in order to provide knowledge of the hydrodynamic forces involved and their dependence on the form of the body. The development of such a theory has been progressing now step by step for about 80 years, and a certain amount of improvement has been achieved with each step.

The initial formulation of the problem is due to Krylov (25), who set up the classic differential equation of the motion of a rigid body subjected to the action of periodic exciting force and moment. The exciting forces were thought to be due to the hydrostatic water pressure only, the pressure being such as is shown by the classical theory to exist in regular wave.

Weinblum and St. Denis (51) in treating the linear equations of motion added the virtual mass of water to ship masses; evaluating these masses on the basis of comparison of the ship with an ellipsoid for which exact solutions are available, and they have also indicated a more inclusive method of evaluating damping on the basis of the work of Huskind (12).

2.3.2 Moored floating structures

The problem of mooring a floating structure such as a ship in the open sea where it is subjected to wave action, is analogous to the problem of a mass restrained by springs and acted upon by some force which varies with time. However, due to the many degrees of freedom and the non-linearities of the mooring system, no general theoretical solution has been formulated for the problem of a ship moored at any heading. Neither have there been any theoretical solutions to the relatively simple problem of a ship in head or beam seas.

A few studies have been made of the motion of a moored vessel (Wilson (56); O'Brien (38); Wilson and Abramson (55), Wiegell and Dilly (53)). Some of these studies were of the simplified problem of the longitudinal motion of a vessel moored along side a pier with the motion being induced by relatively long period harbor seiches of small amplitude, while some of the studies were of the problem of a ship moored at various headings to waves of the wind waves and swell type. Equations were set up describing the generalized geometry of a stockside mooring system. Then the relationships between cable tensions and elongation were determined for several types and sizes of standard mooring cables (Cort rope and steel wire rope). In order to utilize these data it was necessary for additional simplifications to be made in order to develop the relationships between the components of cable tension and the components of ship movements.

A harbor seiche has a relatively long period (usually in the range of a minute or more) and usually has a low amplitude. Wilson's analysis is considerably simpler than that which would be necessary for a vessel moored in normal sea waves, since the particle motion of a seiche is more easily described than of a wind wave or swell and also since he only considered longitudinal motion. A ship moored in a particular heading to the waves is not limited to motion in only one direction.

These approximations were used by Wilson (54), together with an approximation by Havelock (14) which expressed the wave force on a rigid vessel which extends to the bottom. In the original differential equation, the phase angle was neglected and the damping was shown as being proportional to the first power of the velocity rather than the square of the velocity. However, certain relationships were developed by Wilson which yield some valuable qualitative results.
An approximate solution to this differential equation showed that the natural frequency of the system depends upon the amplitude of the forcing function as well as upon the spring and mass characteristics of the system. As would be expected, the resonant period of oscillation varies inversely with the initial tension in the mooring lines. The effect of cable tension on the periodicity of the system was apparent as the higher the tension, the lower is the resonant period. The effect of the amplitude is relatively unimportant for cables with high tension, but very important for cables with low tension; the greater the seismic amplitude the lower the resonant period of the system.

Wilson and Abramson (55), have used the Ritz method to treat the Wilson equation and have shown some results for ship displacement as a function of various seismic amplitudes. Although the above mentioned solution is a gross simplification of the problem at hand, it does show the effects of mooring line tension and possibly wave amplitude.

The experiments of Wiegel and his co-workers have dealt with the simultaneous heaving, swaying, pitching, yawing and rolling motions which have not received much attention in association with surge. Carr (3), linked the moored ship oscillating against the restoring force of its lines to that of a mass oscillating with an initial velocity against the restoring action of two linear springs with a clearance or gap, where the period of oscillation depends not only on the spring factor but also on the amplitude of the gap and the initial conditions. Wilson's expression for the periodicity, $T_s$, of the longitudinal oscillation of a moored ship of mass $M$ at resonance with the exciting wave, appears to be that for the natural period of oscillation of a non-linear spring-suspended mass $(T_s = 2\pi \sqrt{M/K})$ where the spring factor $K$ for the lines collectively is equal to $DN/4$ where $D$ is the spring factor for a representative line and $N$ is the total number of lines. The $K$ has the unusual dimensions of force per movement multiplied by a factor that expresses the non-linearity of the system and the amplitude of the wave, $a$, $(Ma^2/Kd)^{1/2}$ where $c = (1-n)/2(1+n)$). When the lines become significantly taut $(n = 1$ and $K$ has usual dimensions), the second factor becomes equal to unity, the spring factor becomes linear, and the general expression of Wilson becomes that for a simple system. Like Wilson, they cite the desirability of tight cables as a means of reducing the mooring forces. Joosting (20) states that while the forces which excite a moored object can be considered sinusoidal, the mooring system generally does not have linear extension characteristics, so that no actual resonance can be expected, and the general behavior should be examined on the basis of forced oscillations. On the basis of the equation for the forced vibration of a damped spring-suspended mass, he develops an expression for the longitudinal movement of a moored object in which the inertia force on the object is taken to be the product of the mass of the object and the difference between the acceleration of the water and the acceleration of the object.

Joosting (20) has pointed out an omission in Wilson's original theory in respect of the influence of wave steepness, but has retained Wilson’s concept of an acceleration force due to the relative acceleration of the water moving past the object. It is not necessary here to go through the various steps taken by Wilson, but merely to present the final results:

$$
\frac{d^2x}{dt^2} + \frac{N_s}{M_i} \frac{dx}{dt} + \frac{C}{M_i} x = \frac{Ag}{D} \frac{\sinh kd - \sinh ks \sin kB}{\cosh kd + \sinh kd} \frac{\sin kB}{kB} \left( \sin \phi - \frac{M_i}{N_s} \cos \phi \right)
$$

Equation (2.17) has been treated by Abramson and Wilson (55), and solutions were obtained. When a body is moored to buoys which are anchored, consideration must be given to the natural periods of the moored buoy system. Model studies have shown that something close to impact forces has occurred in the lines between the buoys and the body when the buoys were undergoing certain resonant motions. The performance of a box-type floating breakwater was studied by Drimmer et al (8). The implementation of simplifying assumptions concerning the flow beneath a pontoon-type floating breakwater leads to an analytical solution of the two-dimensional linearized hydrodynamic problem. Comparison of the analytical results with a numerical solution of the full linear problem showed good agreement over a wide range of parameters.

Another study for the Trece floating breakwater dock had been made by Jamison and Mogridge (17). This study described wave flume tests of a floating breakwater dock constructed from two steel pontoons, a wooden deck, and two rows of vertical perforated plates for wave absorption. Test results of full-scale and half-scale models were compared. To quantify the effectiveness of this breakwater, transmission and reflection coefficients were determined for regular waves. The performance is compared to an A-frame floating breakwater.

Valencia (50) studied the linked floating breakwater type. A mathematical model has been developed for the motion response and wave attenuation of a two linked floating breakwaters moored to the ocean floor. The two-body interaction problem is divided into hydrodynamic problem and a motion response problem. The hydrodynamic problems deal with the calculation of the wave exciting, added mass, and damping forces on the two bodies induced by incident regular waves and the waves caused by the motion of the bodies. The motion response problem considers a mechanical system subjected to the harmonic hydrodynamic forces and restrained by the mooring lines and the elastic links between the two bodies. The mathematical model has been programmed using a three-dimensional finite element technique. The numerical model accounts for all three-dimensional effects, including the incidence of oblique waves, the interaction of the two breakwaters, their finite length-to-width ratio, the finite water depth, and the proximity of solid boundaries.

2.3.2.1 Anchoring

A floating structure may be moored to a dock or similar structure, to a buoy which is in turn anchored, or it may be connected directly to an anchor by mooring line or chain. For permanent moorings, the anchor may be a pile driven into the bottom which will stand a large horizontal load. A sea anchor can be made of canvas or other material, and is attached to the end of a line running from the object into the water. When the object moves, a force is developed by the resistance of the sea anchor to
movement. There are many types of anchors, such as the concrete clump, the mushroom, the navy stockless anchor, the Dun fold anchor, and the Admiralty anchor. The characteristics of an anchor for general use are given by (Dove, 7; Farrell, 9) which must have:

1. Good resistance,
2. the capability of withstanding all the forces likely to act upon it without permanent set,
3. dead weight enough to be useful in poor holding grounds and heavy enough to drag the chain or cable from the locker or drum when let go.
4. freedom from rolling,
5. no fouling of cable or chain,
6. the capability of digging into hard beds,
7. ease of manufacture.

Most anchors are designed to withstand a considerably greater horizontal force without pulling out, than a vertical force. Tests on one type of anchor (Dove, 7) showed that the maximum vertical pull was only about two-fifths that of the maximum horizontal pull. Nearly all the operators agree that the mooring line should lie flat at the anchor.

One of the criteria in determining the length of cable necessary to assure this condition is the scope, that is, the ratio of length of the mooring line to the water depth. For single anchor, a cable scope of from 5 to 8 is often recommended (Dove, 7); the scope to be used, however, depends upon the water depth and many other factors (7).

Anchor chain is effective as an anchor. Tests made pulling several sizes of chain alone through the holding ground showed that the ratio of holding pull to chain weight ranged from 1.36 to 1.24 (Dove, 1950). Wire rope has been found to have a holding pull of only about 60 percent of its weight (9). There have been several model studies of anchor characteristics and the effects of different types of bottom materials. One series of tests was made with model anchors 1/8, 1/5, 1/3 and 1/2 scale. These tests were made on both sandy and shingle beaches, and on prepared beds of sand and mud in a towing.

3 THEORETICAL CONSIDERATIONS

The FBW considered in this research and shown in Fig. 1.5 is a suggested solution to overcome the problems of a FBW moored with chains or cables to the bottom of the sea and subjected to high tides. Furthermore, the suggested system tries to improve the performance of the FBW when it is used as a pier by reducing the permitted roll motion and preventing sway motion, which in some cases may cause the boats moored to the pier to impact with it. The suggested system consists mainly of a pontoon with rectangular cross section of dimensions $h, B$ floats with draft $D$, connected to two rows of vertical piles with diameter $D_p$ through a number of rings with internal diameter $D_r$. Studying the system in two-dimensions, the pontoon has only two degrees of freedom, which are the heave and limited roll motions. The heave motion occurs freely as the body moves upward and downward unrestrictedly under the influence of the waves. The roll motion in this case is limited due to the existence of the vertical piles, the degree of roll depends mainly on the ratio, $D_p / D_r$, as well as on the vertical distance between the supporting rings, $J$. The sway motion is completely prevented although there is a small sway motion due to the clearance between the piles and the rings, but it must be considered very small when compared with sway in the FBW moored with cables or chains. Fig. 3.1 shows the motion of the pontoon under wave action. As it is suggested that this type of FBW can also act as a pier, more information is required about its motion under wave action. In this chapter an analysis of its motion is given, and an attempt made to find the equation for the heave motion of the pontoon.

3.1 Dimensional considerations

Dimensional analysis is an algebraic theory of dimensionally homogeneous functions. It is based on the principle that any correct mathematical equation of motion must be dimensionally homogeneous. If an equation satisfies this principle, then Buckingham's theorem states we may reduce the equation to a relationship among a complete set of dimensionless products. Thus if an equation has a number $m$ variables composed of $n$ dimensions, $m-n$ dimensionless terms may be derived, often referred to as $\pi$ terms. The use of dimensional analysis can efficiently shorten laboratory procedures and subsequently enhance the presentation and comprehension of the subject data.

3.1.1 Wave attenuation

The objective of a floating breakwater is to minimize the height of waves transmitted ($H_t$) past the structure. For two-dimensional regular wave tests $H_t$ may be assumed to be a function of the following variables:

$$H_t = f(H_i, \alpha, \beta, g, L, B, D, d)$$

(3.1)
Where the above variables are defined in the list of symbols. A common dimensionless parameter used to quantify regular wave attenuation emerges logically as the coefficient of transmission $C_t$ defined as the transmitted wave height divided by the incident wave height ($H_t/H_i$). The above functional relationship may be rewritten as:

$$C_t = f(H_i, g, y, y_{sw}, L, B, D, d)$$

As the remaining 8 variables are composed of 3 dimensions, 5 more dimensionless terms are expected. Selecting $y$, $g$, and $L$ as repeating variables:

$$\pi_1 = (y)^{(y)}(g)^{(y)}(L)^{y_{sw}} = 0$$

$$\left(ML^2T^{-2}\right)^{(y)(y)}(LT^{-2})^y(L)^{mL^2T^{-2}} = 0$$

$$\pi_1 = \frac{1}{y}$$

$$\pi_2 = (y)^{(y)}(L)^yH_i = 0$$

$$\left(ML^{-2}T^{-2}\right)^{(y)(y)}(LT^{-1})^y(L)^{mL^2T^{-2}} = 0$$

$$\pi_2 = \frac{H_i}{L}$$

$$\pi_3 = (y)^{(y)}(L)^yB = 0$$

$$\left(ML^2T^{-2}\right)^{(y)(y)}(LT^{-1})^y(L)^{mL^2T^{-2}} = 0$$
The remaining variables all have the dimension of length and will follow the pattern above.

\[ \pi_a = \frac{D}{L} \]  

Therefore,

\[ C_1 = f \left( \frac{H}{L} \frac{B}{L^2} \frac{D}{L^2} \frac{d}{L^2} \right) \]  

Note that \( \frac{\gamma_a}{\gamma} \) will be constant for a given breakwater. \( B \) and \( D \), will be dimensional constants of the breakwater tested. Therefore only one need be included in a dimensionless term as the remaining three may then be derived through direct scaling. Thus, the final relationship can be expressed as:

\[ C_1 = f \left( \frac{H}{L} \frac{B}{L^2} \frac{d}{L^2} \right) \]  

### 3.2 Analytical considerations

#### 3.2.1 Equation of the heave motion of the floating breakwater

Consider a body of mass \( M \) in the form of a rectangular block of breadth \( B \), height \( h \), draft \( D \), with its length taken to be equal to a strip of width \( b \) in the third dimension, floating through a system of piles as shown in fig. 3.1 with its longitudinal axis normal to the wave crests of a train of regular waves given by \( \eta = a \cos(Kx - \omega t) \).

The waves are of amplitude \( a \), wave length \( L \), wave number \( K = 2\pi / L \) and period \( T \). The origin of the co-ordinate system \( xoz \) is taken at the mid body, mid-beam sections in the still water level, with the \( z \)-axis horizontal in the direction of the longitudinal axis of the body and the \( z \)-axis positive upwards. Since the body is normal to the waves, its motion will include only heave, \( z_t \), and limited roll, \( \theta \), fig. 3.2, considered with reference to its center of mass \( G \). When the body center of mass is displaced from \( G \) to \( G' \), under the actions of the waves, the body is considered to suffer vertical movement of its center of mass along with rotation about the mass center, all of which can be superimposed linearly. For the present, it is assumed that the body has no influence on the waves.

The equation of motion of the vertical component, \( z_t \), (heave motion) for this case is modified from the suggestion of Ohuya (39) and follows the following equation:

\[ Mz_t = \sum \text{External forces} \]

\[ Mz_t = F_{w} + F_{v} + F_{n} - F_{r} - W \]  

Where: \( F_{w} \) is the vertical pressure force at the bottom of the body acting in \( z \)-direction, \( F_{v} \) is the inertia force acting in \( z \)-direction, \( F_{n} \) is the damping force, \( F_{r} \) is the friction force between the piles and the rings acting in \( z \)-direction, \( W \) is the weight of the body acting vertically downward, \( M \) is the mass of the floating body and, \( z_t \) is the vertical displacement of the mass center of the floating body. The average pressure force at the base \( z = -D \) of the rectangular block is taken to be:

\[ F_{w} = bB \left[ \frac{P_{w} + P_{n}}{2} \right] \]  

where:

\[ P_{w} \] is the pressure value at any point in the seaward side,

\[ P_{n} \] is the pressure force at any point in the harbor side,

\( P_{w} \) and \( P_{n} \) are given in the first order approximation, Ippen (16), as:

\[ P_{w} = \gamma \left( \eta K_{p} + \frac{z}{2} \right) \]

\[ P_{n} = \gamma \left( \eta K_{n} + \frac{z}{2} \right) \]  

where:

\( K_{p} \) is the pressure response factor and is given by Ippen (16) as:

\[ K_{p} = \cosh \left( \frac{K \left( d + z \right)}{\cosh Kd} \right) \]

\( \eta \) is the water surface elevation of the standing wave = \( \eta_{s} + \eta_{r} \), \( \eta_{s} \) is the water surface elevation of the standing wave, \( \eta_{r} \) is the water surface elevation of the reflected wave, and \( \eta_{s} \) is the vertical surface elevation of the transmitted wave.

The incident, reflected and transmitted waves are defined respectively as:

\[ \eta_{i} = \frac{H}{2} \sin(Kx - \omega t) \]  

\[ \eta_{r} = \frac{H}{2} \sin(Kx + \omega t + \phi_{i}) \]  

\[ \eta_{t} = \frac{H}{2} \sin(Kx - \omega t + \phi_{i}) \]  

where \( H_i, H_r, \) and \( H_t \) are the incident, reflected and transmitted wave heights respectively and \( \phi_{i}, \phi_{r}, \phi_{t} \) are the phase angles substituting by equations 3.12, 3.13 into equation 3.11 into equation 3.11 yields:

\[ F_{w} = \frac{bB}{2} \gamma \left( \eta_{i} K_{p} + \frac{z}{2} \right) \]  

\[ F_{n} = \frac{bB}{2} \gamma \left( \eta_{r} K_{n} + \frac{z}{2} \right) \]  

\[ F_{w} = \frac{bB}{2} \gamma \left( \eta_{t} K_{t} + \frac{z}{2} \right) \]
The average vertical upward acceleration of the water particles is defined as:

\[ \ddot{W} = \frac{1}{A} \int_{s-t-D}^{s+t+D} \ddot{w} \, dt \]  

(3.24)

where:

- \( A \) is the area of integration, \( \ddot{w} \) is the vertical water particle acceleration, which is defined as (Ippen (16)):

\[ \ddot{w} = -gk \frac{\sinh K(d + z)}{\cosh Kd} \cdot \sinh(Kx - \alpha) \]  

(3.25)

Using equations 3.24, 3.25, the double integration yields:

\[ W = \int_{s-t-D}^{s+t+D} \ddot{w} \, dt = \frac{H g}{2D} \left[ \frac{\cosh Kd - \cosh KS}{\cosh Kd} \right] \sinh \left( \frac{K B}{2} \right) \sin \alpha \]  

(3.26)

Where \( S = d - D \). Using the above equation, the final form of the equation of the inertia force is:

\[ F_{e} = M_{e} \left[ \ddot{W} - \ddot{z} \right] \]  

(3.27)

The component of hydrodynamic force which is 90 deg out of phase with both the relative acceleration and the pressure force is the damping force and can be expressed in terms of relative velocity as:

\[ F_{d} = N_{d} (\dot{W} - \dot{z}) \]  

(3.28)

where:

- \( N_{d} \) is the damping coefficient,
- \( W \) is the average velocity of the water particles in the vertical direction,
- \( \dot{z} \) is the vertical velocity of the floating body

It is here assumed that the damping coefficient is linear, i.e., directly proportional to velocity. In some cases damping is not linear but varies with the square or cube of the velocity as well. However, experience has shown that a linear approximation is often satisfactory, particularly when the motions are not large.

Let the water particle velocity in \( z \)-direction be \( \dot{w} \) which may be defined as (Ippen (16)):

\[ \dot{w} = -gk \frac{\sinh K(d + z)}{\cosh Kd} \cdot \cos (Kx - \alpha) \]  

(3.29)

The average vertical velocity of the water particles is given by:
following the same procedures as in equation 3.26 yields:

\[
W = \frac{H}{2D\omega} \left[ \cosh KS - \cosh Kd \right] \sin \frac{K}{2} \frac{B}{\cos \alpha} \quad (3.31)
\]

and,

\[
F_d = N_{c} \left[ \frac{H}{2D\omega} \left( \cosh KS - \cosh Kd \right) \sin \frac{K}{2} \frac{B}{\cosh Kd} \right] \frac{\sin \frac{K}{2} \frac{B}{\cos \alpha}}{\cos \alpha - \frac{1}{\omega}} \quad (3.32)
\]

The vertical friction forces generated at the connections between the FBW and the piles are considered too small compared with the vertical pressure force and can be neglected in the general equation of motion.

Substituting by equations 3.22, 3.27, and 3.32 into equation 3.10 yields:

\[
M_{d_{1}} + M_{c_{1}} \ddot{z}_{1} + N_{c_{1}} \ddot{z}_{1} + \frac{yBB_{s}}{M_{c_{1}}} \ddot{z}_{1} = a_{2} \sin \alpha + e_{2} \cos \alpha + \frac{yBB}{W} - W_{l} \quad (3.33)
\]

The value \(yBB_{s} - W_{l}\) is nearly zero, this leads to:

\[
\ddot{z}_{1} + 2 \beta \omega_{n} \dot{z}_{1} + \omega_{n}^{2} \ddot{z}_{1} = a_{2} \sin \alpha + e_{2} \cos \alpha \quad (3.34)
\]

where:

\[
a_{2} = \frac{a_{1}}{M + M_{s}} + e_{2} = \frac{e_{2}}{M + M_{s}} \quad (3.35)
\]

\[
a_{2} = \frac{M_{c_{1}} \frac{H}{2D\omega} \left( \cosh Kd - \cosh KS \right) \sin \frac{K}{2} \frac{B}{\cosh Kd}}{\frac{B}{2} \cos \alpha} - \left( \frac{yBB \cosh KS}{2} \right) \left( \frac{H_{1} + H_{2}}{2} \right) \left( \frac{B}{2} \cos \alpha \right) \quad (3.36)
\]

\[
e_{2} = \frac{yBB \cosh KS}{2} \left( \frac{B}{2} \cos \alpha \right) + N_{c_{1}} \frac{H}{2D\omega} \left( \cosh HS - \cosh Kd \right) \sin \frac{K}{2} \frac{B}{\cosh Kd} \quad (3.37)
\]

\[
\frac{N_{c_{1}}}{M + M_{s}} = \left( \frac{2}{B} \right) \omega_{n} - \frac{yBB}{M + M_{s}} = \omega_{n}^{2} \quad (3.38)
\]

Where \(2 \beta\) is a damping factor and \(\omega_{n}\) is the natural angular frequency of the body in heave motion, equation 3.34 can be rewritten as:

\[
\ddot{z}_{1} + 2 \beta \omega_{n} \dot{z}_{1} + \omega_{n}^{2} \ddot{z}_{1} = a_{2} \sin \alpha + e_{2} \cos \alpha \quad (3.39)
\]

The solution of equation 3.39, an equation of forced vibration with damping may be given as:

\[
z_{1} = \frac{1}{\omega_{n}^{2}} \left[ \left( a_{2} \right)^{2} + \left( e_{2} \right)^{2} \right] \frac{1 - \frac{\omega_{n}^{2}}{\omega_{0}^{2}}}{\left( 1 - \frac{\omega_{n}^{2}}{\omega_{0}^{2}} \right)^{2} + \left( 2 \beta \right)^{2} \frac{\omega}{\omega_{0}^{2}}} \cos \left( \omega_{0} \alpha - \phi_{n} \right) \quad (3.40)
\]

Where \(\phi_{n}\) is the phase angle.

\[
\tan \phi_{n} = \frac{a_{2} \left( 1 - \frac{\omega_{n}^{2}}{\omega_{0}^{2}} \right) - e_{2} \left( 2 \beta \right) \frac{a_{2}}{\omega_{0}}} {e_{2} \left( 1 - \frac{\omega_{n}^{2}}{\omega_{0}^{2}} \right) + e_{2} \left( 2 \beta \right) \frac{a_{2}}{\omega_{0}}} \quad (3.41)
\]

The added mass in heave motion can be computed by using the following equation:

\[
M_{c_{1}} = C_{c} M \quad (3.42)
\]

where, \(C_{c}\) is the inertia coefficient which is a function of the body beam-draft ratio B/D and can be obtained from Comstok (5). From equation (3.38), \(\omega_{n}\) may now be computed as:

\[
\omega_{n} = \sqrt{\frac{yBB}{M + M_{s}}} \quad (3.43)
\]

Knowing the damping coefficients (see section 5.3.3.1), the damping factor \(2 \beta\) can be computed and the amplitude of the heave motion of the body \(z_{1}\) can be calculated using equation (3.40) with the known values of the incident wave \(H_{1}\), and the measured values of \(H_{2}\) and \(H_{3}\).

4 LABORATORY EQUIPMENT AND PROCEDURES

The main part of the present study is to investigate the behavior of the suggested FBW experimentally to provide us with the results and information about the ability of this suggested system to attenuate the incident waves and its motion under wave action to be compared with the theoretical work showed in chapter 3. This chapter deals with the description of the wave flume where the present investigations were carried out, and the different models used to study the effect of limited roll and heave motion on wave attenuation. The test conditions and the measuring devices used to measure the water surface profile, the net horizontal force and the pressure distribution at different faces of the FBW are also described in this chapter.
4.1 The wave flume

Experiments were performed in a wave flume, 24.0 meter long, 0.20 meter wide, and 0.50 meter high. Fig. 4.1 shows the general arrangement in the wave flume, with the sides of the channel constructed of glass plates mounted in steel frames as shown in plate 4.1.

For the purposes of this study a wave flume will be considered as a wave study facility in which the length of the flume is many times greater than the width, while the depth is nearly two times the width. These characteristics generally preclude the study of wave refraction and wave diffraction phenomena; problems in which these two phenomena enter are usually studied in wave basins in which the basin width and length are of the same order and both are many times greater than the depth.

4.1.1 General remarks when using wave flume

It is recognized that the quantitative definition of some of the limiting requirements of the wave flume is very difficult; in many instances and only approximations are presently available. The present practice in the application of these limiting factors is given below.

1. Capillarity

Surface tension tends to increase the velocity of propagation of surface waves. It can be seen that water waves less than about 2 inch in length are considerably influenced by capillarity while those greater than about 2 inches in length are not appreciably influenced (see Joseph M. Caldwell, 1955).

2. Viscosity in wave motion

Short-wave hydrodynamic models geometrically scaled according to the Proud equation do not correctly simulate viscous and frictional effects because the Reynolds number of the prototype is different from that of the model. Waves are attenuated by internal friction and by bottom boundary layer friction caused by the water viscosity. In short-wave hydrodynamic models in wave basins, the magnitude of longshore currents and location of rip currents may depend to some extent on friction characteristics of beaches. However, this is usually not important over the short distances modeled in short-wave models.
An expression to estimate wave height attenuation due to internal friction in waves in deep water where boundary shear is negligible is (Hughes (15)):

\[
\frac{d}{dt} \left( \frac{\rho H^2 c}{4L} \right) = -16\pi^2 \rho c^2 \frac{H^2 c}{4L^2} \tag{4.1}
\]

where the left-hand side represents the time rate of change of total wave energy per unit surface area in linear wave, and the right-hand side is the average rate of energy conversion per unit area due to internal shearing stresses. The variables are defined as:

- \( \rho \) = fluid density
- \( v \) = fluid kinematic viscosity
- \( t \) = time
- \( L \) = wave length
- \( C \) = wave celerity
- \( H \) = wave height (decays in time)

By rearranging and canceling variables, equation (4.1) can be integrated, i.e.:

\[
\int_{H_i}^{H_f} \frac{1}{H^2} dH^2 = \int_0^t \frac{16\pi^2 \rho c^2}{L^2} dt \tag{4.2}
\]
to give

\[
H(t) = H_i e^{-\frac{(x-s)\alpha C}{L}} \tag{4.3}
\]

Where \( H(t) \) is the attenuated wave height at time, \( t \). The above formulation assumes uniform regular waves traveling over a horizontal bottom and thus, is not very useful for modern-day laboratories, however, it can be used to examine the range of potential scale effects arising from internal friction.

3. Boundary friction

Over short distances, internal friction is minimal and viscous dissipative effects in nonbreaking waves are limited to the thin boundary layer. An important formula for estimating wave attenuation of regular waves in a rectangular wave channel having a uniform and constant cross-section is found in Hughes (15). This expression for viscous boundary layer damping of a small amplitude linear wave in a wave flume of constant cross-section is defined as:

\[
\frac{H_2}{H_1} = e^{-\alpha \nu} \tag{4.4}
\]

where

\[
\alpha = \frac{2}{3} \sqrt{\frac{\pi v}{B_1 C}} \left[ \frac{\sinh \left( \frac{4\pi d}{L} \right)}{\sinh \left( \frac{4\pi d}{L} \right) + \frac{2\pi d}{L}} \right] \tag{4.5}
\]

and

- \( H_1 \) = wave height at \( x = 0 \)
- \( H_2 \) = wave height after traveling a distance, \( x \)
- \( x \) = horizontal distance in wave flume
- \( B_1 \) = wave flume width
- \( T \) = wave period
- \( d \) = water depth
- \( C \) = wave celerity

For the present work, the effect of the viscosity in wave motion and the boundary friction on the decay of deep water waves were studied by measuring the incident wave height at 4 different positions spaced 2 meters from each other. On the other hand, the decay was also calculated theoretically using equations (4.3) and (4.4), (as in the following example), and compared to the measured experimental results as shown in Fig. 4.2.

Example

Tested wave period \( T = 0.5 \text{ sec (} f = 2.0 \text{ Hz)} \)

Tested wave length \( L = 0.389 \text{ m for water depth } d = 0.3 \text{ m} \)

Wave height at \( (x = 0) = 2.904 \text{ cm} \)

Width of the flume \( B_1 = 0.3 \text{ m} \)

Relative depth ratio \( d / L = 0.77 > 0.5 \) (deep water wave)

a) Decay due to internal friction

\[
C = \frac{L}{T} = 0.389/0.5 = 0.778 \text{ m/sec}
\]

The time required for the wave to travel 1 m = 1.0.778 = 1.285 sec.

Assume the kinematic viscosity equal to \( 1.3 \times (10)^{-6} \text{ m}^2 / \text{s} \). The attenuated wave height can be calculated using equation (4.3). The time \( t \) used in this equation to calculate the attenuated wave height at the second, third, and the fourth position is calculated as:

\[
t_{x = 2d} = 2 \times 1.285 = 2.57 \text{ sec} \quad \text{(for the second position)}
\]

\[
t_{x = 4d} = 4 \times 1.285 = 5.14 \text{ sec} \quad \text{(for the third position)}
\]
Using the time calculated, the attenuated wave height can be calculated at the different positions using equation (3.4) as:

\[
H(t) = 2.904 e^{- \frac{t}{\lambda_1}} \left[\sinh\left(\frac{4\pi(0.3)}{0.3}\right) + \sin\left(\frac{4\pi(0.3)}{0.3}\right)\right]
\]

b) Decay due to boundary wall friction

\[
\alpha = \frac{2}{0.3(0.778)} \left[\frac{\sinh\left(\frac{4\pi(0.3)}{0.3}\right) + \sin\left(\frac{4\pi(0.3)}{0.3}\right)}{\frac{4\pi(0.3)}{0.3}}\right] = 0.0244
\]

and using equation (4.4) yields:

\[
\left(H_1\right)_{t=20} = 2.904 e^{-0.0244(20)} = 2.765 \text{ cm 4.76%}
\]
\[
\left(H_1\right)_{t=40} = 2.904 e^{-0.0244(40)} = 2.633 \text{ cm 9.30%}
\]
\[
\left(H_1\right)_{t=100} = 2.904 e^{-0.0244(100)} = 2.508 \text{ cm 13.61%}
\]

The calculations showed that wave attenuation due to internal friction is very small and nearly negligible while the effects of boundary friction on wave attenuation reach considerable values. Fig. 4.2 shows the plot of the measured and the calculated attenuated wave heights.

4. Wave height, wave length - water depth and generator stroke

Fig 4.3 shows a set of curves giving the measured wave heights for various strokes, which are functions in the amplification button value, and periods. The distance necessary for the wave to travel before becoming stable is governed by the design length of the flume, and the slope of the structures and beaches to be tested. A study of the waves showed that the shorter waves reached stability almost immediately on leaving the generator. The long waves, however, were found to reach stability after three or four waves left the generator. The difficulty with the longer waves appeared as a tendency to break apart into two waves of different heights travelling at different speeds. It was at first thought that this difficulty was due to the time-displacement curve of the wave generator. As a result of these observations, it was then concluded that the instability was due to a combination of wave height-wave length and water depth aspects of the flume.

4.1.2 Generator

The wave generator used in this study is shown in plate 4.1-a. The wave generator can generate the waves by three methods of motion which are the piston type (which is used in the present work), the flap type and a combination of piston and flap types. In all types of generators the aim is to displace the water at a rate matching the requirements of the wave train being generated.

Both the amplitude and the frequency of generator movement are adjustable. The relationship between the generator stroke and its amplification value are measured and shown in Fig. 4.4. The period can be varied between approximately 0.5 seconds to 2.0 seconds and the wave amplitudes between 0 and 0.03 m. The water depth during all the experiments was 0.3 m. At the opposite end of the channel from the wave generator a parabolic beach was installed for the purpose of absorbing the wave energy.

4.1.3 Beaches

In order to make efficient use of the facility in wave tests, it is necessary to prevent reflection of the waves from the far end of the flume. If this is not done, not only do the reflected waves soon induce standing waves throughout the length of the flume, but the time necessary between runs while the water calms becomes very long.

Beaches have taken all forms from a straight, solid inclined surface to complicated systems of slots, wire mesh, bamboo carpets and so on. The one in use in this study is made of plastic with a parabolic shape as shown in Fig 4.1.

4.2 Measurement of the water surface profile

4.2.1 Wave probes

A frame supporting the probe wires was suspended over the channel by means of an adjustable boom. This mounting greatly facilitated the daily calibration of the probe and also helped to maintain the probe in the proper position relative to the static water level. The water profile is measured using a resistance-wire probe as shown in plate 4.2. Two chrome nickel wires 0.12 inch in diameter were used as the probe. The two wires were stretched 0.65 inch apart on the frame and formed one element of a Wheatstone bridge circuit. There were certain disadvantages in the use of the resistance-type probe. The relationship between water surface position and the electrical output is nonlinear. Furthermore, the calibration of the system is dependent upon the conductivity of the water between the two wires. These difficulties were overcome satisfactorily by always adjusting the position of the probes to the static water level existing in the flume. Each day before testing began, the wires were carefully cleaned and the system calibrated at three or four points. Three wave probes were mounted in front of the model used for measuring the reflection. Another wave probe at the lee of the body was used for measuring the transmitted waves. The output being passed through a purpose built amplifier which was connected to a UV oscillograph and a computer, where the output information was stored in.
4.2.2 Measurement of incident and reflected waves

The Mansard and Funk method (30) was used to measure the incident and reflected waves. In fact, this is a least squares method to separate the incident and reflected waves from the co-existing waves in front of the model. This method requires simultaneous measurement of the waves at three positions in the flume which are in reasonable proximity to each other and are on a line parallel to the direction of wave propagation.

Fig. 4.1 shows the position of the three wave probes used for this purpose. Assume that the tested wave length is L. The wave probes were fixed so that the distance between the front face of the model and the third probe (No. 3), T3, was equal to one wave length L. Before starting the experiments, the reflection coefficients for different wave conditions were measured at different distances from the model as shown in Fig. 4.5. The graph shows that the reflection coefficient is affected greatly by the distance L. Thus, the distance L was chosen to minimize the effect of wall friction and the decay on the measured co-existing wave which appear significantly in the case of short crested waves. The distance between the probes itself was chosen following the recommendation given by Mansard and Funk (30) for accurate results. x12 is the distance between the first and the second probe and was taken to be L/10, while x13 is the distance between the first and the third probe and was taken to be L/4, where L is the length of the tested wave. A computer program was devised for this method to give the results very accurately and fast. The input to the program are the three measured signals at the three different positions and the output is the value of the incident and reflected wave heights. Two methods were used to make sure that the results were correct. Firstly, the incident wave height was estimated by averaging the height of the waves measured by the three wave probes in the upwave array before any waves reflect back to the array from the model. Secondly, the incident wave heights were measured without the existence of the model in the flume. The values of the incident waves calculated according to the Mansard and Funk method were compared to those measured by the two methods mentioned above. These values agreed quite closely as shown in Fig. 4.6.
4.2.3 Measurement of transmitted waves

To measure the transmitted wave height, the fourth wave probe was installed (No. 4) at the lee side of the model at a distance L from the rear face of the body, where L is the length of the tested wave as shown in fig. 4.1. The distance L was taken to overcome the problems of wave decay and its effect on the results as mentioned before. To calculate the transmitted wave height, a number of transmitted waves recorded by the fourth wave probe were analyzed and the average of its heights and periods was taken to be the transmitted wave. It should be mentioned here that the first two waves passing the structure were not included in the analysis because of their incomplete stability and energy. Furthermore, the number of waves chosen to determine the incident wave height depends mainly on the tested wave period (length) and must be suitable to avoid the reflection which may occur from the end of the flume and may affect the results of the transmitted wave height.

![Graph showing the effect of position of wave probes on measured reflection and transmission coefficients](image)

**Fig. 4.5:** Effect of the position of wave probes on the measured reflection and transmission coefficients

4.3 Transducers calibration

Wave, force and pressure transducer static calibrations were conducted with the aid of a calibration routine available on the laboratory computer. The routine would print out the digital output values assigned to a given channel on the computer screen. Thus calibration was conducted by altering the transducers by a known amount.

Wave probe calibrations were performed by adjusting the digital value to zero volts at still water level. By raising the probe a known distance above the S.W.L. (2.5 cm), the digital value was adjusted to -1.0 volt. Then lowering the probe the same distance below the S.W.L. the digital value was adjusted to +1.0 volt. This process must be repeated several times until the output of the probe is equal to zero volt at S.W.L. -1.0 volt when it is raised 2.5 cm above S.W.L., and +1.0 volt when it is lowered 2.5 cm below S.W.L. Then the conversion factor in this case can be taken as 1 volt = 2.5 cm with respect to the positive and negative signs.

The pressure transducer calibrations were performed by adjusting its output digital value to zero when the body floated freely with draft D. Then the body was immersed a known distance of 5 cm into the water and the digital output values were recorded. As the pressure of a 10 m head of water is 9800 mbar, the pressure of 5 cm head of water can be calculated and was found to be 4.9 mbar. From this relationship, the conversion factor for each pressure transducer required to convert the recorded pressure signals from volts to mbar can be calculated as shown in table 4.1.
<table>
<thead>
<tr>
<th>Pressure transducer No.</th>
<th>Record in volt at D = 8.5 cm</th>
<th>Record in volt at D = 13.5 cm</th>
<th>Delta D (cm)</th>
<th>Delta V (volts)</th>
<th>Conversion factor (mbar/v)</th>
</tr>
</thead>
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<td>1</td>
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<td>2.5</td>
<td>5</td>
<td>2.5</td>
<td>1.96</td>
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<tr>
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<td>2.5</td>
<td>1.96</td>
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<td>5</td>
<td>2.5</td>
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<td>1.96</td>
</tr>
</tbody>
</table>

Table 4.1: Calibration of the pressure transducers

Force transducer calibration in the horizontal direction was performed by adjusting the output digital value to zero volts when the body was not exposed to any forces. Then by suspending known weights from the gauge and recording the digital output values which appeared on the screen of the computer, the average of the conversion factor was evaluated and found to be equal to 15.20 N/volt.

### 4.4 Natural period tests in heave motion

Tests were conducted to determine the natural period of the breakwater module by observing the period of oscillation of the model structure unrestrained in still water. A means of determining the natural period of structures of this type and shape was desired so that the natural period of similarly-shaped breakwaters designed for other locations where resonance might be a problem could be altered by changing the physical characteristics and dimensions of the breakwater. Concerning the wave damping effect of a floating breakwater, it can be summarized that an effective floating breakwater should have a large natural period compared with that of the incident wave (Wiegard R. L., 1964). This means that its mass should be large, its viscous and form damping large. Fig. 4.7 shows the motion of the body with time for the case of D/D = ½ and D/D = ¼ which shows the natural period of the body to be 0.82 sec.

![Graph showing determination of the natural period of the model in heave motion](image)

Fig. 4.7: Determination of the natural period of the model in heave motion (experimentally)

### 5 EXPERIMENTAL STUDY AND RESULTS

This chapter describes the experimental work which was done in the laboratory to investigate the aims of this research and to test the efficiency of the suggested pile system FBW presented in fig. 1.5 by measuring the transmission and reflection coefficients. This chapter also gives the details of the models used in the experiments, the different conditions of the incident regular waves and body parameters, the methods for analyzing the measured data, and finally the results obtained from the experiments.

The tests were divided into three groups of experiments. In the first group of experiments, the efficiency of the restrained FBW, no motion of the body, was studied by measuring the transmission and reflection coefficients. These transmission and reflection coefficients were obtained to be used as reference values for the results of the suggested pile system FBW studied in this research and to determine the effect of each motion of the body on wave attenuation.

In the second group of experiments, the effect of limited roll motion on the transmission and reflection coefficients were studied. In fact, it was very important to investigate the effect of this motion on the efficiency of the FBW, because in principle, roll motion of the body is not recommended when the FBW is also to be used as a pier. Before starting the tests of this group, there were two expected ways to continue through it:

First, if the results proved that the values of C T and C R change with the angle of the limited roll motion, then this motion will have an effect on the efficiency of the FBW and in this case we would have to know about the relationship between the angle of rotation and the coefficients studied.

Second, if the results proved that the value for the limited roll has no effect on C T and C R, the limited roll motion can be eliminated by improving the design of the FBW to allow the body to move only in heave motion, one degree of freedom. This will improve the stability of the FBW when used as a pier.

In the third group of experiments, the effect of heave motion on wave attenuation was studied by measuring C T and C R under different wave conditions. For each group of experiments, a particular model was used to obtain the required conditions to be tested. The following are the full descriptions of the experiments and the results obtained in each case.

#### 5.1 Restrained body tests (no motion tests)

The first group of experiments in this research studied the behavior of a restrained rectangular floating body prevented from moving. This was to be a reference point for the two tested motions. The experiments were conducted for different wave periods, T, incident wave heights, H, body drafts, D, and breadths, B, with a fixed water depth in the flume, d. The wave periods tested ranged between 0.5 and 2.0 seconds while the corresponding wave heights ranged between 2.0 and 6.0 cm.

Different drafts of the body were tested ranging between 5.0 and 15 cm for a fixed water depth of 30 cm and were expressed as the draft water depth ratio D/d. For the purpose of studying the effect of the breadth B, three different bodies with breadths 15, 20, and 30 cm were used. The details of the model are given in fig. 5.1. The model consists mainly of a horizontal plate fixed to the top of the side walls of the flume. Four steel pipes were fixed at the top to the plate and at the bottom to the box of the required dimensions (see plate 5.1). The required draft D was created by moving the four steel pipes upward and downward to the required draft and fixing them again to the horizontal plate.
The model was installed in the wave flume at a distance of 13 m from the wave generator. The reflection coefficient, $C_r$, was calculated using Mansard and Funke method (30) by installing 3 wave probes in front of the model such that the distance from the upward face of the body to the nearest wave probe was equal to the incident tested wave length, $L$. This distance was chosen to overcome the problems of decay which occur mainly in the case of short waves as mentioned before in section 4.1.2. The distance between the first and the second wave probe was taken to be $x_{12} = L/10$ and between the first and the third wave probe at $x_{13} = L/4$, where $L$ is the incident tested wave length and is given by linear wave theory as:

$$L = \frac{gT}{2\pi} \tanh \frac{2\pi d}{L}$$

(5.1)
The distances $x_{1.2}$ and $x_{1.3}$ were taken following the recommendations of Mansard and Funke (30) as they give the most accurate results. (see the general arrangements in fig. 4.1).

A software program was developed for the Mansard and Funke Method (30) using MATLAB. The method depends chiefly on transforming the three recorded signals of the standing wave formed in front of the model using the Fourier transform and then solving the equations using the Least Square Method. The input to the program are the three recorded signals of the standing wave formed in front of the model after converting it from volts to cm. The zone which is to be analyzed must be defined for the program. It should be mentioned here that the zone analyzed must be chosen carefully to avoid reflection of the wave from the end of the flume and from the wave generator, which can lead to wrong results. The program extracts the incident and the reflected waves from the three measured signals and the output of the program will be the frequency and the height of both the incident and the reflected waves and the calculated reflection coefficient $C_r$.

Figures 5.2.a and 5.2.b give examples of the input and the output of the program using an incident wave with frequency $f = 1.375$ Hz ($T = 0.727$ sec) at draft water depth ratio $d/D = 1/6$. Fig. 5.2.a shows the three measured signals of the standing wave in front of the model (input to the program). It is clear in the figure that, in the period between 0.0 and 19 sec, the wave travels from the wave generator to the location of the three wave probes. Between 19.0 and 22.0 sec, the incident wave passes the three probes and reflects from the upward face of the model and the standing wave begins to build its shape. At the beginning of the formation of the standing wave some disturbance takes place, after that the standing wave tends to be stable for some time which is the period of the time between 22.0 and 40.0 sec. This period of time is the right zone for analysis and gives the right values of the reflection coefficient $C_r$. During the rest of the recorded signals, (from 40.0 to 60.0 sec), the form of the standing wave changes due to the new reflection of the standing wave from the wave generator, which generates a new incident wave with different characteristics. This stage of the recorded signals must not be used in the program. Fig. 5.2.b presents the output of the program, which shows the calculated incident and reflected wave heights with the calculated frequencies and the calculated reflection coefficient $C_r$.

(a) The three signals recorded in front of the model (input of the program)

(b) Calculated values of $H_i$, $H_r$ and $C_r$ (output of the program)

Fig. 5.2: Calculation of the reflection coefficient using the Mansard and Funke method

(restrained body, $T = 0.727$ sec, $d/D = 0.048$, $d/D = 1/6$, $d/D = 1/2$)
On the other hand, the transmission coefficient can be calculated by measuring the transmitted wave height behind the model and dividing this measured wave height by the incident wave height, which is determined as explained above \((C = H/H_0)\); the transmitted wave height is measured using the fourth wave probe installed behind the model (at a distance from the leeward face of the model equal to the incident tested wave length, \(L\)). Fig. 5.3.a shows the recorded signal of the transmitted wave for the previous example \((f = 1.375\, \text{Hz, } D/d = 1/6, \|v\|d = 1/2)\). This signal can also be classified into 4 zones according to time. The first zone is between 0.0 and 22.0 sec, where the wave travels from the generator and crosses the structure until it reaches the fourth probe.

The second zone, is from 22.0 to 24.0 sec, where some disturbances appears in the shape of the wave. The third zone occupies the time from 24.5 to 46.0 sec and the wave seems to be very stable in this zone. The last zone is from 46.0 to 60.0 sec, where the wave shape changes due to the reflection of the standing wave from the wave generator and the partial reflection of the transmitted wave from the end of the flume. Fig. 5.3.b shows the chosen analyzed zone, the length of the time window was taken to be 2.5 sec, which is nearly equal to 4 times the incident wave period. This time window was found to be enough to calculate the average transmitted wave height. The output of the program is shown in fig. 5.3.b, which gives the calculated frequency and height of the transmitted wave. The time of the test run was constant at 60 sec for all the tests. This time was found to be enough for both long and short waves to measure enough zone of the wave to run the required analysis.

To study the effect of the wave period on the reflection and transmission coefficients, several wave periods were tested ranging between 0.5 and 2.0 sec. In each case, the wave height is calculated using equation (5.1), the effect of the wave period and the corresponding wave height is expressed in the form of dimensionless ratios \(B/L\) and \(d/L\). The ratio \(B/L\) represents the body / breadth wave length ratio while \(d/L\) represents the water depth / wave length ratio, which indicates the zone of the experiments (shallow, transitional, or deep water zone).

As the available wave heights in the flume are limited and range between 2.0 and 6.0 cm, the effect of wave steepness \(H/L\) was studied only over range \(B/L\) from 0.156 to 0.394 with three values of \(H/L = 0.042, 0.048\) and 0.058 respectively. For long waves, the wave steepness could not be kept constant and was reduced to 0.014 for the largest tested wave length. All the experiments were conducted for different values of the body / draft water depth ratio \(D/d\). Figures 5.4 - 5.8 show the effect of \(B/L\) and \(d/L\) on \(C\) and \(C\), for different values of \(D/d\). It is clear in these figures that \(C\) and \(C\) are affected strongly by \(B/L\) and \(d/L\) and that \(C\) decreases with the increase of \(B/L\). For example in fig. 5.4 at \(D/d = 1/6, C\) decreases from the value 0.868 to the value 0.053 where as \(B/L\) increases from 0.046 to 0.394. On the other hand and for the same example, \(C\) increases from 0.206 to 0.811 as \(B/L\) increases with the same previous values. These results can also be discussed as a function of the ratio \(d/L\). The figures show that \(C\) decreases with the increase of \(d/L\). (i.e \(C\) decreases as the wave condition changes from shallow to deep water waves) while, \(C\) shows an increase with this change in wave condition.
Fig. 5.4: Variation of $C_r$ and $C_r'$ with $B/L$ and $d/L$.
(restrained body, $D/d = 1/6$, $H_0/L = 0.014 - 0.048$, $B/d = 1/2$)

Fig. 5.5: Variation of $C_t$ and $C_t'$ with $B/L$ and $d/L$.
(restrained body, $D/d = 1/5$, $H_0/L = 0.014 - 0.048$, $B/d = 1/2$)

Fig. 5.6: Variation of $C_l$ and $C_l'$ with $B/L$ and $d/L$.
(restrained body, $D/d = 1/4$, $H_0/L = 0.014 - 0.048$, $B/d = 1/2$)

Fig. 5.7: Variation of $C_t$ and $C_t'$ with $B/L$ and $d/L$.
(restrained body, $D/d = 1/3$, $H_0/L = 0.014 - 0.048$, $B/d = 1/2$)
The results prove that in the case of short-crested waves (deep water zone), the values of $C_r$ do not increase continuously with the increase of $B/L$ and $d/L$, but show some changes in this water zone which give local minimum and local maximum values of $C_r$ beginning at $B/L = 0.25$ as shown in figures 5.4-5.8.

Fig. 5.9 and 5.10 shows the effect of the ratio $D/d$ on $C_r$, and $C_f$, respectively. It is clear in fig. 5.9 that, as the value of $D$ increases for constant water depth $d$, the ratio $D/d$ increases and $C_r$ decreases. For example, at $B/L = 0.156$ the value of $C_r$ decreases from 0.638 at $D/d = 1/6$ to 0.217 at $D/d = 1/2$, while in fig. 5.10 $C_f$ increases with the increase of $D/d$. As for the previous example, at $B/L = 0.156$, $C_f$ increases from 0.638 at $D/d = 1/6$ to 0.93 at $D/d = 1/2$. The figure also shows the disturbance of the reflection coefficient in the short waves zone (deep water zone) for all the values of $D/d$ as discussed before.

From the previous results, it is clear that a FBW with such dimensions is considered effective for the values of $B/L > 0.15$, while below this value of $B/L$ the corresponding transmission coefficient is very high for all the values of $D/d$.  

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**Fig. 5.8:** Variation of $C_r$ and $C_f$ with $B/L$ and $d/L$  
(restrained body, $D/d = 1/2$, $H/L = 0.014 - 0.048$, $B/d = 1/2$)

**Fig. 5.9:** Variation of $C_r$ with $B/L$ and $d/L$ for different values of $D/d$  
(restrained body, $H/L = 0.014 - 0.048$, $B/d = 1/2$)

**Fig. 5.10:** Variation of $C_f$ with $B/L$ and $d/L$ for different values of $D/d$  
(restrained body, $H/L = 0.014 - 0.048$, $B/d = 1/2$)
Another important factor which was also studied is the wave steepness. The results showed that the tested range of wave steepness has no influence on the transmission coefficients for all the values of D/d, while the reflection coefficient showed some changes with wave steepness H/L and this relationship became significant at low values of D/d as shown in figures 5.11 and 5.12.

All the results described were taken from a restrained body of breadth B = 15 cm, which gives a body breadth to water depth ratio B/d = 1/2. To check the influence of changing the ratio B/d, two more bodies were used with breadths B = 20 and 30 cm, giving the values of B/d = 2/3 and 1 respectively. The results showed that as the ratio B/d increases, the transmission coefficient decreases and the reflection coefficient increases for all the values of D/d as shown in fig. 5.13.

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**Fig. 5.11:** The effect of tested wave steepness H/L on Cr and Ct
(restrained body, D/d = 1/6, B/d = 1/2)

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**Fig. 5.12:** The effect of tested wave steepness H/L on Cr and Ct
(restrained body, D/d = 1/4, B/d = 1/2)

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**Fig. 5.13:** Effect of body breadth B on Cr and Ct
(restrained body, D/d = 1/4, H/L = 0.048)
5.2 Limited roll motion tests

The second group of experiments was to study the effect of limited roll motion on the reflection and transmission coefficients under different body parameters and different wave conditions. To study the effect of this motion, a model shown in plate 5.2 was constructed. The model is supported to the side walls of the flume from above by a horizontal plate. The tops of two vertical steel pipes were fixed to the plate while the body was connected to the end of the vertical pipes through a horizontal axle allowing the body to roll with nearly no friction when it was exposed to the incident waves. The amplitude of roll motion was controlled by 4 vertical screws which could be adjusted to any height allowing the body to be fixed, roll to a certain angle, or roll freely without any restriction. The details of this model are given in fig. 5.14.

The model was installed in the same position as the restrained model (see fig. 4.1). The four wave probes used for measuring the reflection and the transmission coefficients were adjusted in front of and behind the model as mentioned above. The results for the restrained body showed that a FBW with such a system of fixation and such dimensions is effective in the zone of Bi/L > 0.15. Thus, for this reason, the tests for checking the limited roll motion were carried out for a range of Bi/L between 0.156 to 0.349.

In nature, the body will roll about its y-axis when it is exposed to the attack of the waves. The roll motion will not be completely as in the case of a free floating body due to the existence of the piles. The value of the angle of roll in this case depends mainly on the ratio of the diameter of the ring to the diameter of the pile, D/R, and the vertical distance between the rings, (see fig. 3.1)). For suitable design, the smaller the angle of permitted rotation, the more suitable of the FBW is as a pier.

In this set of experiments, 2 different allowed angles of rotation were tested, 3 and 6 degrees. The effect of roll motion were studied under different wave steepnesses and draft water depth ratios D/d. The D/d ratios tested were 1/6, 1/4, and 1/2. Changing the value D/d was done by adding weights inside the free floating box which gives the D/d ratio and then connecting the box to the model again.

Figs. 5.15 and 5.16 show the effect of wave steepness H/I on C and C for different roll angles θ. The figures showed that the wave steepness has no effect on C, but has a small effect on C. Figs. 5.17 and 5.18 show the effect of limited roll motion on C and C for different D/d ratios. It is also clear from these figures that this limited roll motion has no effect on C, and its effect on C is very small.
**Fig. 5.14:** Details of the model used for limited roll motion tests.
Fig. 5.17: Effect of the allowed angle of roll on $C_l$ and $C_r$

$(D/d = 1/6, H_d/L = 0.048, B/d = 1/2)$

Fig. 5.18: Effect of the allowed angle of roll on $C_l$ and $C_r$

$(D/d = 1/4, H_d/L = 0.048, B/d = 1/2)$

Fig. 5.19: Effect of the allowed angle of roll on $C_l$

$(D/d = 1/2, H_d/L = 0.0485, B/d = 1/2)$

From the previous results, checking the limited roll motion showed no effect of this motion on wave attenuation. Using this fact, we can eliminate this motion from the primary design of the FBW by allowing the body to move only in heave motion, this will improve the performance of the FBW as a protector. On the other hand, the problem of studying the behavior of this system of floating breakwater will be reduced to a problem of floating body moving with only one degree of freedom.

5.3 Heave motion tests

Studying the heave motion of the pile system FBW was the third group of experiments in the present study. The investigations were conducted to study the effect of heave motion individually on the transmission and reflection coefficients. The motion of the body in heave and its phase angle with the standing wave were also measured.

The model used for this investigation is shown in Fig. 5.20. The model follows the same procedure as before but with four external hollow pipes with an internal diameter of 1.9 cm fixed to the horizontal plate. Four internal pipes with a diameter of 1.8 cm were fixed to the body and extended inside the
external hollow pipes, allowing the body to move only in heave motion when exposed to the attack of the incident regular waves. It is very important to know the effect of the different wave conditions on the amplitude of the heave motion of the FBW, its phase angle with the standing wave formed in front of the body.

Studying these parameters was done by recording the movement of the body using a video camera as shown in fig. 5.21. The motion of the body in time was calculated by watching the movement of a clear point on it moving beside a vertical linear scale mounted on the transparent side walls of the flume. The motion of the body in time and its phase angle with the standing wave were observed using the slow motion video option which shows 25 frames per second. The body was tested under different wave conditions and body parameters.

The draft D was increased by adding uniform weights inside the body. The model and the wave probes used for running the heave motion tests were installed in the position shown in fig. 4.1. The period of the tested waves ranged between 0.5 and 2.0 sec and the corresponding wave lengths from 0.389 to 3.257 m. The wave lengths were calculated using equation (5.1), which represents the linear wave theory. The dimensions of the tested body were B=15 cm and b = 29.5 cm.

The tested body drafts were 5, 6, 7.5 and 10 cm for the fixed water depth in the flume d = 30 cm, which gives D/d = 1/6, 1/5, 1/4, and 1/3 respectively. Figures 5.22-5.25 show the effect of B/L and d/L on C_h and C_v for different ratios of D/d. It is clear from the figures that C_h decreases with the increase of B/L and d/L. For example, in fig. 5.22 where D/d = 1/6, C_h decreases from 0.904 at B/L = 0.044 and d/L = 0.092 (very near from shallow water zone) to 0.953 at B/L = 0.394 and d/L = 0.789 (deep water zone). On the other hand, the figures also shows that C_v increases as B/L and d/L increase.

The increase of C_v in the deep water zone was unsteady due to the existence of local minimum and local maximum values of C_v, as obtained before in the case of the restrained body. For the same values of B/L and d/L mentioned above it was found that C_v increases from 0.074 to 0.721, which means that the reflection coefficient of a certain body is also affected significantly by the incident wave length and the type of the zone (shallow, transitional or deep) at which the incident wave interacts with the structure.
Fig. 5.22: Variation of $C_r$ and $C_t$ with $B/L$ and $d/L$ (only heave motion, $D/d = 1/5$, $H_d/L = 0.014 - 0.048$, $B/d = 1/2$)

Fig. 5.23: Variation of $C_r$ and $C_t$ with $B/L$ and $d/L$ (only heave motion, $D/d = 1/5$, $H_d/L = 0.014 - 0.048$, $B/d = 1/2$)

Fig. 5.24: Variation of $C_r$ and $C_t$ with $B/L$ and $d/L$ (only heave motion, $D/d = 1/4$, $H_d/L = 0.014 - 0.048$, $B/d = 1/2$)

Fig. 5.25: Variation of $C_r$ and $C_t$ with $B/L$ and $d/L$ (only heave motion, $D/d = 1/3$, $H_d/L = 0.014 - 0.048$, $B/d = 1/2$)
Figures 5.26 and 5.27 give the effect of the draft D on C_r and C_t in the form of the dimensionless ratio D/L. It is clear from the figures that, at the same values of B/L and d/L, C_r decreases and C_t increases with the increase of the value of D/L. The figures also show that maximum reflection and minimum transmission happen in the deep water zone (d/L ≥ 0.5), while the maximum transmission and the minimum reflection are found near the shallow water zone (d/L = 0.05). This means that C_r and C_t are strongly affected by d/L.

The explanation of this phenomenon is given in fig. 5.28, where the figure shows the percentage of the kinetic energy of the wave concentrated above any elevation S measured from the bottom of the flume for different values of the ratio D/L. The figure shows that most of the kinetic energy of the wave is concentrated near the water surface in the case of deep water waves (d/L ≥ 0.5) while the K.E. is linearly distributed over the depth d in the case of shallow water waves (d/L = 0.05). In other words and for example, consider the present pile-system FW floating with draft D such as D/d = 0.5. Entering fig. 5.28 at S/d = 0.5, the value of the kinetic energy concentrated above this level in the case of deep water waves is 96%; this energy will be distributed above the level S/d which is occupied by the body over the draft D. Thus, this value of the wave energy may be reflected from the floating body while the rest of the kinetic energy of the wave can be transmitted beneath the structure (only 4%). While, in the case of shallow water (d/L = 0.05), the value of K.E. concentrated above the level S/d obtained from the figure is 54%, this means that 54% of the kinetic energy of the wave will be precluded by the existing floating body while 46% of the energy can transmit beneath it.

![Fig. 5.26: Effect of D/L on the reflection coefficient C_r](image)

*(only heave motion, H/I.L = 0.014 - 0.048, B/L = 1/1)*

![Fig. 5.27: Effect of D/L on the reflection coefficient C_t](image)

*(only heave motion, H/I.L = 0.014 - 0.048, B/L = 1/1)*

![Fig. 5.28: Concentration of kinetic energy above the elevation S/d](image)
5.3.1 Wave-structure interaction and energy loss

When an incident wave with height \( H_i \) and energy \( E_i \) interacts with a floating breakwater, part of this wave will be transmitted through the FBW with height \( H_t \) and energy \( E_t \) and the rest of the incident wave will be reflected from the structure with height \( H_r \) and energy \( E_r \) such that:

\[
E_r = E_i + E_t \tag{5.2}
\]

or

\[
\frac{\rho g H_i^2}{8} = \frac{\rho g H_r^2}{8} + \frac{\rho g H_t^2}{8} \tag{5.3}
\]

Equation (5.3) can be rewritten after dividing both sides by \( H_i^2 \) as:

\[
1 = \left( \frac{H_r}{H_i} \right)^2 + \left( \frac{H_t}{H_i} \right)^2 \tag{5.4}
\]

In practice, wave reflection and transmission in the presence of a floating breakwater would occur with the loss of some portion of the incident wave energy. Hence, if \( H_t \) is the loss in wave height, then according to the law of conservation of energy:

\[
\frac{H_t}{H_i} = \sqrt{1 - \left( \frac{H_r}{H_i} \right)^2 - \left( \frac{H_t}{H_i} \right)^2} \tag{5.5}
\]

in which the ratio \( \frac{H_t}{H_i} \) is called the coefficient of energy loss \( C_t \) and:

\[
C_t = \sqrt{1 - C_i^2 - C_r^2} \tag{5.6}
\]

Using the experimental results for \( C_i \) and \( C_r \), the loss coefficient \( C_t \) was calculated from equation (5.6). Figures 5.29 - 5.31 show the calculated loss coefficients, \( C_t \), for different conditions plotted together with the reflection and the transmission coefficient.
5.3.2 Description of energy loss in heave motion

In this part, a qualitative study was conducted to describe how the incident wave loses some of its energy when it passes through the structure. The study included the description of water particles movement in the zone around the structure by using small artificial particles and watching their motion using the video camera. Before starting the experiments, a strong light was installed below the bed of the flume exactly below the model. The video camera was adjusted horizontally with the still water level as shown in fig. 5.21.

The study showed that some of the incident wave energy is dissipated due to the formation of vortices around the two sharp edges of the body. The study also showed that the area of the vortex formed in front of the body is bigger in size than the second vortex formed at the lee of the body. Furthermore, the position of the vortex changes with the movement of the free surface of the wave as shown in fig. 5.32.a and 5.32.b.

Another kind of energy loss is the destruction of the water particle orbits due to the fact that the floating body is in the domain of the waves. This study was also conducted by recording the motion of water particles using the video camera and observing these motions clearly by replaying the recorded motions, using the slow motion video option. Many photos were taken of the zone around the floating body, the time chosen for taken the photos was equal to the tested wave period.

Observing these motions, it was found that the paths of the water particles near the structure were completely destroyed and that the adjacent particles to the bottom moved in paths parallel to the surface of the body as shown in plate 5.3. The photos also showed that in the case of short waves (deep water zone, d/L = 0.5), the motion of the particles with their circular orbits, is clearly concentrated near the free surface as shown in plate 5.4. (this means that most of the kinetic energy is concentrated near the water surface as mentioned before). In the case of long waves, (shallow water zone, d/L = 0.85), the paths of the water particles are elliptical orbits and distributed over the entire water depth as shown in plate 5.5.
Plate 5.4: Water particles orbits near the structure in deep water waves
(only heave motion, $D/d = 1/6$, $B/d = 1/2$, $B/L = 0.217$, $d/L = 0.434$)

Plate 5.5: Water particles orbits near the structure in shallow water waves
(only heave motion, $D/d = 1/6$, $B/d = 1/2$, $B/L = 0.007$, $d/L = 0.135$)

Plate 5.3: Position of the vortices around the structure in heave motion
5.3.3 Motion of the pile system floating breakwater

5.3.3.1 Heave motion measurements

The heave motion of the floating body was measured using the method shown in fig. 5.21. The characteristics of the heave motion were obtained by recording the movement of the body using a video camera. The motion of the body was calculated by observing the movement of a clear point fixed on it moving beside a vertical linear scale mounted on the transparent side walls of the flume. The movement of the body in time and the phase angle with the standing wave formed in front of the body were calculated using the slow motion video option which shows 25 frames per second. The motion of the body was expressed as the dimensionless ratio \( H_h/H_0 \), where \( H_h \) is the height of the heave motion calculated as the vertical distance between the highest and the lowest positions of the point during one wave period, and \( H_0 \) is the incident wave height.

The effect of the incident wave height, \( H_0 \), on the ratio \( H_h/H_0 \) was investigated by using two different wave heights of 3 and 4 cm respectively, for different wave periods. For each wave height, the wave period was changed to generate wave ranges from deep water waves (short waves) to nearly shallow water waves (long waves) and in each tested wave period the value of \( H_h \) was measured. The effect of \( H_0 \) on the ratio \( H_h/H_0 \) is shown in Fig. 5.33, where the figure shows that \( H_h \) has practically no effect on \( H_0/H_0 \).

![Fig. 5.33: Variation of \( H_h/H_0 \) with \( B/L \) and \( d/L \) for different heights of the incident waves](image)

Fig. 5.34 gives the measured values of \( H_h/H_0 \) using an incident regular wave with constant height, \( H_0 = 4 \) cm, for different values of \( D/d \). The figure shows that in the shallow water zone, the body has the same dimensionless heave motion ratio and that the ratio \( D/d \) has no effect on \( H_h/H_0 \). The figure also shows a maximum value of \( H_h/H_0 \) which is due to the resonance. Resonance normally occurs when the wave period \( T_w \) is equal to the body's natural period \( T_n \) (or when \( T_w/T_n = 1 \)), at which the waves add energy to the body, increasing the amplitude of the heave motion. Fig. 5.35 is a plot of \( T_w/T_n \) versus

\[ H_h/H_0 \] for all the values of \( D/d \). The natural period, \( T_n \), was calculated for each value of \( D/d \) experimentally as shown in fig. 4.11. This plot shows that resonance did not occur at \( T_w/T_n = 1 \) but shifted to the value \( T_w/T_n = 0.75 \) due to the high damping parameter of the present floating body. This fact was noted before by Comstock [5], who gave an example of a floating buoy and proved theoretically that the resonance is shifted to a value of \( T_w/T_n \) or \( \omega_n/\omega \) to 1.0 as the dimensionless damping parameter, \( 20 \), increases as shown in fig. 5.36.
Figures 5.37 and 5.38 show a comparison between the theoretical and the experimental results of the heave motion for different values of B/L and d/L. The heave motion was calculated theoretically using equation (3.40). The natural angular frequency, $\omega_0$, was calculated using equations (3.42) and (3.43). The coefficient of the added mass $C_a$ used in equation (3.42) was taken from Courtois (5) according to the dimensionless ratio $\alpha^2 B/2g$ (i.e., for each case the value $\alpha^2 B/2g$ is calculated and the corresponding added mass coefficient $C_a$ is determined from fig. 5.38 where the following symbols appear in the figure:

- $\omega$ = angular frequency of the incident wave,
- $B$ = breadth of the breakwater,
- $T$ = draft of the floating body,
- $g$ = gravitational acceleration.

It is clear in fig. 5.39 that the added mass is dependent on the damped frequency. On the other hand, the dimensionless parameter (2b) used in equation (3.40) was calculated using equation (3.38) after calculating $\omega_0$. The values for the damping coefficient, $N$, used for the present analysis were compared with those given by Fugazza (10) and showed good agreement as shown in fig. 5.40.
Fig. 5.38: Comparison between the experimental and the theoretical values of \( \frac{H_n}{H_i} \)

(heave motion, \( D/d = 1/4, B/d = 1/2 \))

Fig. 5.39: Hydrodynamic added mass coefficients for two-dimensional floating bodies in heaving motion

(Comstock (3))

Fig. 5.40: Damping coefficient \( N \) versus dimensionless angular frequency \( \omega^2 B/2g \).

Fig. 5.41 and 5.42 show a comparison between the theoretical and the experimentally measured values of heave motion with time for different wave conditions and body parameters. The figures show good agreement between the calculated and the measured motion except that the experimental results are significantly below the theoretical calculations, this may be due to the gravitational effect.

Fig. 5.41: Calculated and measured heave motion versus time

(\( D/d = 1/4, B/d = 1/2, H_i/L = 0.0485, H_i = 3.5 \text{ cm}, T = 0.67 \text{ sec} \))
The phase angles measured for different wave periods and body drafts are given in fig. 5.44. The figure shows that for small values of B/L and d/L (near the shallow water zone), the phase angles are very small (< 30°) and the body moves nearly in phase with the standing wave. As the wave period increases (B/L and d/L increase), the phase angle between the body and the standing wave increases until it reaches the maximum value, which is nearly 180° (case of deep water); in this case the body is completely out of phase with the wave. In fact, this difference in phase between the body and the wave in heave motion plays a considerable part in attenuating the incident wave and this phenomena is known as out-of-phase damping. Furthermore, the figure also shows that for all the values of B/L, the value of the phase angle decreases as the D/d ratio increases (or the weight of the body increases).
5.4 Efficiency of the pile-system floating breakwater

The previous experiments and the results discussed showed that limited roll motion has no effect on the transmission coefficient, especially in the case of small angles of rotation and for D/d ≤ 1/4. This motion was disregarded in the design and the body was tested with only one degree of freedom (moving in heave motion only). In order to establish the efficiency of the new pile-system floating breakwater (the new suggested system of FBW discussed in this research), a comparison was made between the results for the restrained body (fixed breakwater), the new FBW and the results of some other authors.

Figs. 5.45 - 5.48 shows the values of $C_r$ and $C_t$ for both cases plotted versus B/L and d/L for different values of D/d. The figures shows that the values of $C_r$ and $C_t$ the FBW moving only in heave motion and fixed in position using vertical piles are always lower than the results for the restrained structure for all the values of D/d. The explanation of these results is that in the case of the restrained structure the total energy of the incident wave will be divided into three parts, the energy of the transmitted wave, the energy of the reflected wave and the energy lost. While in the case of the FBW there is an additional source of lost energy, which is the energy lost in inducing the heave motion of the body.

This means that designing a FBW to move only in heave motion and fixing it in position by using a system of piles prevents the structure from moving in sway and roll motions, which improves the performance of the FBW as a pier. Furthermore, the free heave motion of the body allows the structure to overcome the problems of tides, which appears mainly in the case of FBW moored with chains to the bottom of the sea.

![Graph](image)

**Fig. 5.45:** Effect of the heave motion on $C_r$ and $C_t$

(D/d = 1/6, B/d = 1/2, H/L = 0.0485)

![Graph](image)

**Fig. 5.46:** Effect of the heave motion on $C_r$ and $C_t$

(D/d = 1/5, B/d = 1/2, H/L = 0.0485)

![Graph](image)

**Fig. 5.47:** Effect of the heave motion on $C_r$ and $C_t$

(D/d = 1/4, B/d = 1/2, H/L = 0.0485)
Another factor of importance, the effect of the mooring system stiffness, is also discussed here. The mooring system suggested in this research was simulated using four vertical rigid pipes, and as mentioned before, this yields a very high rigidity, thus sway motion of the floating structure is completely prevented (sway = 0.0). The present study was compared with the similar work of Fugazza (10). In his work, the special mooring system was simulated by fastening the model to a four-bar linkage with an elastic cantilever. The rigidity of the mooring system to horizontal wave action was modeled by varying the thickness of the steel bar used to connect the model to the supporting frame. The tested stiffness constant, $K_s$, of this work ranged between 650 and 2500 kg/m$^2$. ($K_s$ was given as the stiffness of the fastening bar per unit length of the barrier expressed in kg/m/m).

Fig. 5.49 shows the effect of mooring system rigidity on the body motion by comparing the present work with that of Fugazza (10). The figure shows that with low values of mooring system stiffness (Fugazza (10), $K_s = 650$ kg/m$^2$), the body moves in sway with significant value while with a high rigid mooring system (present work), no sway motion occurs. On the other hand, the figure also shows that the lower the stiffness the higher the heave motion of the body.

Fig. 5.50 shows a comparison between the results of the present work for $D/d = 0.25$ and $B/d=1/2$, with the work of other authors for different types of floating breakwaters. The figure shows that the suggested FBW is efficient compared with the other results and this efficiency appears mainly in the deep water zone. The figure also shows the effect of $B/L$ on $C_s$, where it is clear for all the results that $C_s$ decrease with the increase of $B/L$. 
5.5 Improvement of FBW efficiency using a vertical rigid plate under the floating body

As a way to improve the efficiency of the floating breakwater, a vertical rigid plate was fixed to the bottom of the floating body and extended downward into the water. The main idea was to minimize the cross sectional area of the floating body immersed under water (reducing the construction costs) and to improve the efficiency by reducing the transmission coefficient. One case was chosen for investigation. The tested body dimensions were $B = 15 \text{ cm}$, $D = 5 \text{ cm}$, $d = 30 \text{ cm}$ and $D' = 15 \text{ cm}$. The corresponding dimensionless parameters are $B/d = 1/2, D/d = 1/6$, and $D'/d = 1/2$.

The results of this investigation are given in Fig. 5.51, which shows that using a vertical thin rigid plate under a floating body to increase the draft water depth ratio, $D/d$ from $1/6$ to $D'/d = 1/2$ reduces the transmission coefficient from the results of $D/d = 1/6$ to be between the results of $D/d = 1/4$ and $D/d = 1/3$. In other words, the immersed cross sectional area of a pontoon required for attenuating an incident wave to a certain value can be minimized by using a vertical thin rigid plate of appropriate length fixed to the bottom of the body. This gives the same required attenuation value but at lower construction costs.
6 HORIZONTAL HYDRODYNAMIC WAVE FORCES ON THE PILE - SYSTEM FLOATING BREAKWATER.

In the present study, the system of fixing the floating breakwater in position is modified by using vertical piles instead of the classical types of mooring lines (chains and cables). This modification was done to improve the performance of the FBW by preventing sway motion and enabling it to be used as a pier beside its main function of attenuating the incident waves and creating a calm water area behind it.

As this system of fixation (by using piles and allowing the body to move freely in heave motion) is considered new in this field, it was important to know about the forces affecting the body which are transmitted directly to the fixing system (the vertical piles) under wave action. Knowing about these forces will be helpful in designing the required piles.

When the floating body is exposed to the attack of the incident waves, it will be mainly affected by two types of force. The first force is the net vertical force, which is the sum of all the external forces acting vertically on the body such as the inertia force, the wave pressure force, the viscous force, the weight of the body, etc. This net vertical force is responsible for the heave motion of the body. A small part of this force is transmitted to the piles through the friction between the body and the piles. The second force which affect the body are the horizontal hydrodynamic wave forces, which are transmitted completely in this case to the supporting piles. These forces are the major factor affecting the design of the supporting piles and are the area of interest to be studied in this chapter.

6.1 Test setup
6.1.1 Description and installation of the model

The model used for this investigation is shown in fig. 6.1. The model has the same design as before but with four external hollow pipes with an internal diameter of 1.9 cm fixed to the horizontal plate. Four internal pipes with a diameter of 1.8 cm were fixed to the body and extended inside the internal pipes, allowing the body to move only in heave motion when it is exposed to the incident regular waves. The horizontal plate was supported on top of the flume using four roller supports and connected freely to the force cell, allowing the horizontal force to transmit to the force cell without any moment. Six pressure transducers were fixed to the faces of the body, (four on the bottom and two on the side walls), to measure the pressure affecting the wetted faces of the body as shown in plate 6.1.
Before running the tests, the force cell and the pressure transducers were calibrated as mentioned before in section 4.3. The dimensions of the box used in running the experiments were $B = 30 \text{ cm}$, $b = 20.5 \text{ cm}$ and $D = 8.4 \text{ cm}$. The run time for all the tests was constant $= 60 \text{ sec}$.

### 6.2 Measurement of the horizontal wave forces

The horizontal forces were calculated from the recorded signals after converting them from volts to force units (N/m) using the converting factor estimated in section 4.1. Fig. 6.2 shows an example of the data for one run of time $= 60 \text{ sec}$. The figure shows the variation of $F_x$ and $F_y$ with time, where:

- $F_x$ = the horizontal force on the body in the direction of the incident wave (N/m).
- $F_y$ = the horizontal force on the body opposite to the direction of the incident wave (N/m).

It should be mentioned here that a negative $F_y$ means that the force is acting opposite to the direction of the incident wave. Fig. 6.2 shows that the horizontal forces equal zero in the period between 0.0 and 10.0 sec (the time required for the first wave to travel from the generator to the structure). In the period between 10.0 and 16.0 sec, the recorded signals of the wave forces show some disturbances due to the interaction of the first three waves, (which are not completely formed), with the body. This period of time is followed by the stable recorded signals which occupy the period between 16.0 and 30.0 sec. After that the form of the signals begins to change due to the reflection of the standing wave from the wave generator.

To obtain the correct results, the signal analyzed must be chosen carefully in the time between 16.0 and 30.0 sec as shown in fig. 6.2b. The figure shows that the width of the signal analyzed $= 6 \text{ sec}$ (from 20.0 to 26.0 sec), and it shows the average calculated values of $F_x$ and $F_y$ as well.

#### 6.2.1 Effect of $B/L$ and $d/L$ on the horizontal wave forces affecting the structure

To study the effect of $B/L$ and $d/L$ on the horizontal wave forces affecting the structure, the wave length was changed by changing the incident wave period from 0.5 to 2.0 sec (with fixed values of $D$, $B$ and $d$). The tests over the range of $B/L$ and $d/L$ were repeated three times with different constant incident wave heights of 3.4 and 4.8 cm respectively.
Figs. 6.3 - 6.5 show the variation of the dimensionless absolute values of $F_{max}/\gamma B^2$ and $F_{max}/\gamma B^2$ with $B/L$ and $d/L$. It is clear from the figures that at the low values of $B/L$ (the case of long waves), both the forces are minimum and that the forces increase with the increase of $B/L$ and $d/L$ to a certain value of $B/L$ equal to 0.25 ($d/L = 0.5$), where the forces become maximum. After these maximum values, the forces begin to decrease with the increase of $B/L$ and $d/L$ (the case of short waves). The above mentioned distribution of the forces versus $B/L$ and $d/L$ was found to be similar for the three tested incident wave heights but with different force values. Furthermore, $F_{max}$ was found to be always higher than $F_{max}$.

**Fig. 6.3:** Effect of $B/L$ and $d/L$ on the max. horizontal force on the floating body in heave motion

($H/d = 1/3.6, B/d = 1, H/d = 1/6.25$)

**Fig. 6.4:** Effect of $B/L$ and $d/L$ on the max. horizontal force on the floating body in heave motion

($H/d = 1/3.6, B/d = 1, H/d = 1/7.5$)

**Fig. 6.5:** Effect of $B/L$ and $d/L$ on the max. horizontal force on the floating body in heave motion

6.2.2 Effect of wave steepness $H/L$ on the horizontal wave forces affecting the structure

The second group of force experiments was conducted to study the effect of the incident wave steepness on the horizontal forces affecting the structure. The effect of wave steepness was studied at three different values of $d/L$ which are 0.092, 0.281 and 0.434, representing the different zones of waves (shallow, transitional and deep water waves). For each studied value of $d/L$, the wave height, $H$, was varied using several values to give different values of $H/L$.

Figs. 6.6 - 6.8 show the plot of the dimensionless absolute values of $F_r$ and $F_c$, versus the wave steepness $H/L$. The figures show that for all the values of $d/L$, the higher the incident wave steepness the higher the resultant horizontal wave force on the structure. On the other hand, the figures also show that the horizontal force of the wave affecting the structure in the direction of the incident wave, $F_r^c$, is always higher than the force affecting the structure in the direction opposite to the direction of the incident wave, $F_c^c$. 
6.3 Hydrodynamic wave pressure on the different faces of the floating body

The wave pressure on the different faces of the structure was measured using six pressure transducers distributed over the wetted area of the structure as shown in fig. 6.1. The purpose of running such experiments is to know which area of the body is exposed to the maximum wave pressure. This may help in the construction stage. After the calibration of the pressure transducers (see section 4.3) and before starting the tests, all the pressure transducers were adjusted to read zero volts at rest (no wave action, D = 8.4 cm). This means that the recorded signals of the pressure transducers will be the pure hydrodynamic wave pressure on the structure. The total pressure at any point on the structure located at distance z under the still water level can be calculated by adding the hydrodynamic wave pressure to the static water pressure (p = ρg) at this point. An example of the measured wave pressure at different locations on the body surface is given in fig. 6.9, where, H = 5.5 cm and T = 1.0 sec.

Figs. 6.10 - 6.12 show the effect of wave steepness on the maximum measured wave pressure on the different faces of the structure for different values of d/L. The maximum pressure on the bottom of the body, \( P_{\text{max}} \) was found always to be at gauge No. 2. Furthermore, the maximum pressure on the body was found to be recorded by gauge No. 1, \( P_{\text{max}} \). It is clear from all the figures that the pressure increases with the increase of wave steepness.

As a result, this investigation showed that the zone of maximum pressure is the zone between gauge 1 and gauge 2 and near gauge 3, \( P_{\text{max}} \). One explanation for this high pressure may be due to the formation of vortices in these zones at which the water particle velocities increase; this may increase the impact of these particles with the structure and as a result increase the pressure at this zone of the structure.
Fig. 6.9: The recorded signals of wave pressure on the faces of the structure moving heave motion
(Did = 1/3.6, Bld = 1, d/L = 0.218, H/L = 0.04)

Fig. 6.10: Variation of the wave pressure on the faces of the body with H/L.
(Did = 1/3.6, Bld = 1, d/L = 0.092)
Fig. 6.11: Variation of the wave pressure on the faces of the body with \( \frac{H}{L} \).
(Dld = 1/3.6, B/d = 1, d/L = 0.218)

Fig. 6.12: Variation of the wave pressure on the faces of the body with \( \frac{H}{L} \).
(Dld = 1/3.6, B/d = 1, d/L = 0.434)
7 SUMMARY AND CONCLUSION

7.1 Summary and conclusion

In the present work, a suggested floating breakwater with a new mooring system was studied. The new mooring system using vertical piles is a modification to the classical mooring system using chains and cables. The new system was suggested in order to overcome the problems of mooring lines at high tide and to improve the performance of the floating breakwater as a pier in fishing harbors and marinas. The FBW with the new mooring system studied in 2-D reduces the motion of the body to two degree of freedom; these motions are defined as the limited roll and the heave motions. Both of these motions were studied under the effect of the regular waves by measuring the reflection and the transmission coefficients. The hydrodynamic horizontal wave forces and pressures on the structure were also measured. The ability of the suggested FBW to attenuate incident regular waves was determined by comparing its results with the results for a fixed structure and the following conclusions were drawn:

1. The experiments were carried out in a wave flume with dimensions of 24.0 * 0.30 * 0.50 meters with a fixed water depth of 0.30 m. The regular waves were generated using a piston-type wave generator. Before starting the experiments, the wave generator was calibrated and the efficiency of the wave absorbers was also tested. The tests showed that there is wave reflection at the end of the flume, and the value of this wave reflection increases with the wave length. The tests also showed that the required time for running both short and long wave tests with minimum effect of wave reflection is nearly 60 sec. The decay of the incident wave propagating in the flume was also investigated. The study showed that the decay has a significant value in the case of short waves. The study showed also that the decay of an incident wave of period T = 0.5 sec propagates in a wave flume with such dimensions reaches 15% due to the internal viscosity and boundary wall friction. Due to this fact, the effect of the position of the wave probes required for measuring the transmission and reflection coefficients measured from the vertical faces of the model was also investigated. The investigation showed that the measured reflection and transmission coefficients decrease as the distance of the wave probes increase. The minimum distance of installing the wave probes was taken as equal to the incident wave length, L, measured from the vertical faces of the model to overcome the problems of decay.

2. The Mansard and Funke method (30) was used for measuring the incident and reflected waves. In fact this is a least squares method to separate the incident and the reflected waves from the coexisting waves in the front of the model. This method requires simultaneous measurement of the waves at three positions in the flume which are in a reasonable proximity to each other and are on a line parallel to the direction of wave propagation. A computer program was designed for this method using MATLAB to solve the problem very fast and accurately. The input to the program are the three measured signals at the different positions and the output is the value of the incident and reflected wave heights.

3. The suggested floating breakwater with a rectangular cross section was investigated (as it is restricted from motion) by measuring its transmission and reflection coefficients under the attack of regular waves. The purpose of these experiments was to find out the transmission and reflection coefficients of the body without motion so that these could be used as a reference to determine the effect of each motion of the suggested FBW on wave attenuation. From the study it emerged that C_T and C_R are affected strongly by each of the dimensionless parameters B/L, d/L and D/d individually. In other words, the higher the value of these dimensionless parameters, the lower the value of C_T and the higher the value of C_R. The study also showed that in the range of the wave steepness that was tested, steepness has practically no effect on C_T but has a small effect on C_R, specially in the deep water zone.

4. As the suggested FBW is installed between two rows of piles, the roll motion of the body is limited. To investigate the effect of this motion on wave attenuation, a model was constructed to simulate this limited roll motion with different angles of rotation. The transmission and reflection coefficients were measured under different wave conditions and different body parameters. The results showed that for a floating breakwater with draft up to D/d = 1.14, the effect of the limited roll motion (for small angles of rotation, $6 \leq \phi \leq 10^\circ$) had no effect on C_T, but has a small effect on C_R, especially in the deep water zone. For the values of D/d > 1.14, the higher the angle of rotation, the higher the transmission coefficient C_T. Thus, for such types of breakwater, the limited roll motion can be eliminated from the design and the structure can be designed to allow the body to move only in heave motion. This will increase the efficiency of the FBW used as a pier and furthermore reduce the motion of the structure in the two-dimensional scale to only one degree of freedom.

5. The effect of heave motion on wave attenuation was also studied. A new model was constructed allowing the body to move only in heave motion without roll. The transmission and reflection coefficients were measured under different conditions. The results showed that C_T decreases and C_R increases as B/L and d/L increase. As an example, at D/d = 1.16 and for B/L = 0.046 (d/L = 0.0923 nearly shallow water or long waves conditions), C_T = 0.0404 and C_R = 0.074. As B/L increases to 0.394 (d/L = 0.789, deep water or short wave conditions), C_T decreases to 0.052 and C_R increases to 0.721.

6. For the suggested floating breakwater moving only in heave motion, the results proved that the higher the draft of the floating body the higher the reflection coefficient, C_R, and the lower the transmission coefficient, C_T.

7. The suggested FBW moving only in heave motion showed the same behavior as the restrained structure but with lower values for C_T and C_R. This may be due to the fact that part of the wave energy is dissipated in inducing heave motion. Furthermore, the heave motion of the body generates secondary waves which are out of phase with the transmitted and the reflected waves; this leads to resultant waves smaller in height than that in the case of the restrained body.

8. The results showed that the increase of C_T with B/L and D/d has some disturbance in the deep water zone but showed a local minimum value that depends on the value of B/L and D/d (this starts to happen nearly at B/L ~ 0.25).

9. An analytical solution to the problem of heave motion was given. The equation of heave motion given here is a modification of the work of Ohiyax (39) including the influence of body breadth, B and finding the added mass and damping coefficients of this suggested FBW. The heave motion of the suggested FBW was measured experimentally under different incident waves. The motion of the body was expressed in the dimensionless form of $H_1/H$, the calculated and the measured results were compared and showed good agreement. The results also showed that the maximum value of $H_1/H$ is nearly 1.0, which is due to the resonance and that there is no resonance when the natural period of the body is equal to the wave period but shifted to a ratio of $T_1/T < 1$ due to the high damping of the suggested system.

10. Observation of the heave motion of the suggested FBW showed that in the case of long waves, the body moves easily with the wave such as the phase angle, $\phi \leq 25^\circ$ (nearby phase with the incident wave), while in the case of short waves the phase angle increases and reach $180^\circ$ and the body seems to move opposite to the wave.

11. An attempt was made to increase the efficiency of the suggested FBW which moves only in heave motion. The investigation showed that the efficiency of the FBW can be increased by adding a vertical rigid plate under the floating body. This will cut the costs of construction of the
rectangular cross sectional area required to produce the same wave attenuation.

12. The results proved that there is energy loss in the incident wave energy and that $C_r^2 + C_s^2 < 1$. Observation of the experiments showed that part of the wave energy loss is dissipated in the form of vortices created around the sharp edges of the body. These vortices occupy an area on the seaward side of the body which is larger than that on the harbor side. Furthermore, the position of these vortices changes with the water surface elevation around the floating body.

13. In the present work, the motion of the body was measured using a rigid mooring system (no sway motion allowed). Comparing these results with the work of Fuggazza (10), who tested the effect of mooring system stiffness, showed that the higher the mooring system stiffness the lower the heave motion of the floating body.

14. The horizontal wave forces on the structure were measured as a guide in the design stage of the suggested mooring system (using vertical piles). The experiments showed that the horizontal wave forces on the floating body are affected strongly by B/L and that F_r is always higher than F_t. The study also showed that maximum value of the forces depends on the wave motion system around the structure. The wave steepness showed an increasing proportional relationship to both F_r and F_t.

15. The pressure measurements showed that the pressure on the vertical sides of the body is always higher than that on the bottom (the water particles move parallel to the bottom of the body), in addition, the seaward vertical face of the body is exposed to maximum wave pressure due to the direct impact of the water particles on it and the additional effect of the vortices formed beside this face.

7.2 Recommendations for further work

- Studying the behavior of the pile system floating breakwater under the action of the irregular waves,
- Checking the performance of the floating breakwater using different shapes for the floating pontoon,
- Studying the same problem in 3-D scale to investigate the wave diffraction around the floating body,
- Knowing the effects of water particles turbulence around the floating body on bed scour.

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MULTIFUNCTIONAL WAVE ABSORBING BREAKWATERS
WITH EXTREME FORCE DISSIPATION

A new design concept

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1 SUMMARY

Structures in the ocean are typically designed for wave forces that are likely to be encountered during its operational life. This could be typically 50 years. These design maximum wave heights are very rarely encountered. Hence one designs a structure for a wave load that occurs for a fraction of its lifetime making them very expensive. A new design approach is presented here for caisson type structures, with wave energy absorption.

Work on multifunctional Wave Absorbing Breakwaters has been in progress for more than a decade. The main theme is to develop a multipurpose structure with energy as a by product. The cost of this structure is shared between berthing, storage facilities and the power plant hence is attractive from the cost point of view. The technology of wave power absorption has already been proven.

In principle a wave absorbing structure is a better engineering design as the force induced on it is reduced due to absorption. However in extreme climate it is impossible for the conventional power plant machinery to be operational. Hence in extreme waves when the forces should have been reduced the structure is subjected to extreme forces, due to power plant shut down. Hence the present concept of extreme force dissipation by hydraulic jetting. Chambers in the caisson structure store sea water under pressure from normally occurring waves. These are used as jets to dissipate extreme waves when they occur once in a while. A wave sensor placed sufficiently in front of the device is used as a trigger to activate the jet. The concept is shown in Fig. 1.

![Fig. 1: Concept of force dissipating structures](image)

Experiments have been conducted at the wave laboratory of IGAW at the University of Wuppertal. This involves the generation of extreme waves in the laboratory and the study of their dissipation using jets. A pump was used to deliver water under pressure for the jet. The jet starting and stopping was controlled by the computer. The data acquisition and control software required for wave generation and its measurement and jet control was developed. Various jets with different pressure generation and its measurement and jet control was developed. As a first discharge characteristics were tested using various angles of jetting and duration of jetting. As a first step it was decided to study the effect of jet without any structure. The first step involves the generation of an extreme wave. The IGAW flume is 0.3 m wide and has a water depth of 0.3 m and is about 21 m long. For the frequency ranges of 0.8 Hz to 2.2 Hz that could be generated in this flume a wave maker. This extreme wave with and without jetting is compared for different angles of jetting. Typical pump pressure was of the order of $5 \times 10^7$ Nm$^2$, and jet discharge of about $3 \times 10^4$ m$^3$/s. The jet duration for these experiments was from 5 to 10 seconds. In general a flat angle of around 60-70 degrees from vertical has the maximum efficiency in reducing the wave height. For reference zero degrees corresponds to a jet fired vertically down to sea bed.

The concept looks promising. Further discussions on practical ways to implement this scheme are also discussed.

2 INTRODUCTION

Structures in the ocean are typically designed for wave forces that are likely to be encountered during its operational life. This could be typically 50 years. These design maximum wave heights are very rarely encountered. Hence one designs a structure for a wave load that occurs for a fraction of its lifetime making them very expensive. A new design approach is presented here for caisson type structures, with wave energy absorption.

Work on multifunctional Wave Absorbing Breakwaters has been in progress for more than a decade. The main theme is to develop a multipurpose structure with energy as a by product. The cost of this structure is shared between berthing, storage facilities and the power plant hence is attractive from the cost point of view. The technology of wave power absorption has already been proven.

In principle a wave absorbing structure is a better engineering design as the force induced on it is reduced due to absorption. However in extreme climate it is impossible for the conventional power plant machinery to be operational. Hence in extreme waves when the forces should have been reduced the structure is subjected to extreme forces, due to power plant shut down.

Hence the present concept of extreme force dissipation by hydraulic jetting. Chambers in the caisson structure store sea water under pressure from normally occurring waves. These are used as jets to dissipate extreme waves when they occur once in a while. A wave sensor placed sufficiently in front of the device is used as the trigger to activate the jet.

To prove the proposed concept in the laboratory the work is split into different stages:

1. Generation of extreme waves at a predetermined point in the wave flume.
2. Computer controlled jet triggering to dissipate these extreme waves.
3. Evaluation and discussion of above jet dissipation results.

To achieve the above tasks some development work on the existing hardware and software was necessitated. The development and the problems faced and solved are described in the subsequent sections.
major use of this configuration file is to change the input control signal filename and also the output file names.

A typical listing of this configuration file is shown in Table 1 below:

<table>
<thead>
<tr>
<th>Line 1:Low Channel (0-15)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Line 2:High Channel (0-15)</td>
</tr>
<tr>
<td>Line 3:Gain = 1 (BIP10VOLTS) - see board manual</td>
</tr>
<tr>
<td>Line 4:Points per channel</td>
</tr>
<tr>
<td>Line 5:Digitizing Rate in Hz.</td>
</tr>
<tr>
<td>Line 6:Control signal filename without extension (MATLAB file)</td>
</tr>
<tr>
<td>Line 7:Data output filename without extension (MATLAB file)</td>
</tr>
<tr>
<td>Line 8:Start of trigger (sec)</td>
</tr>
<tr>
<td>Line 9:Stop of trigger (sec)</td>
</tr>
<tr>
<td>Line 10:Calibration constants (decimal values) &amp; Units for corresponding channels with blanka separating each item</td>
</tr>
</tbody>
</table>

Table 1: Configuration file ‘igaw.cfg’

More details and the source code of this software is given in Appendix. However a brief overview of the graphical screens is presented here to give a glimpse of the programming sequence. It is suggested to start the program from within MATLAB Command Window even though it is an independent executable. Such operating system files are triggered from within MATLAB using the ‘!’ punctuation before the filename as shown in Fig. 2 below.
This program on execution will read the 'igaw.cfg' file and display the important parameters in a dialog box Fig. 3 below.

If one is satisfied with these parameters one can proceed further by clicking the 'ok' button. The Control signal file in this example 'chirpR2000.mat' is read and plotted for visual confirmation before proceeding.

(Note: Since the system is built with an open architecture it is the user's responsibility to check if the frequency ranges are suitable to be input to the wave machine. However amplitude violations beyond the capability of the data acquisition board will be taken care of by the software. This does not again imply that the wave machine can handle these voltages. It is the user's responsibility to do so.) The plot is scaled to show AD values which for a 12 bit board range from 0 to 4096 corresponding to the Gain Setting of the board. It has been set to ±10 Volts, but can be changed depending on requirements.

To avoid transients in wave generation a ramp is commonly applied at the start and the end of control signal. This is also the responsibility of the user when creating the control signal file. This allows the data acquisition program to be general so that those wanting to create transients can do so. Moreover the amount of ramp could be under user control during control signal file generation. It also reduces the complexity of the data acquisition program.

As can be seen in the control signal above Fig. 4, the signal varies from 0 to 4096 the full scale output of the 12 bit AD converter. This is an extreme example of all frequencies having the same amplitude. However normally a user would have varying amplitudes for various frequencies. It should be noted that to obtain maximum AD resolution it is best to keep the maximum amplitude in the signal at full scale value of 4096. This again is to be taken care of in the control signal generation program.

The wave maker hardware requires an input of ±1.5 Volts as opposed to the ±10 Volts output from the computerized data acquisition system. Hence an external amplifier with a gain of less than unity was required.

If the signal is ok then further continuation of the program will make ready the computer for real time data acquisition and control signal outputting. The first step towards this is to check for the time duration of wave maker controls and the time for data acquisition as specified by the user in the 'igaw.cfg' file. Since this DAS 1601/12 AD board has only one clock on board the longer of the two is selected. Since the user inputs the no of points to be acquired and the digitizing frequency the program computes the time duration of the experiment and displays it before actual test starts as shown in message box Fig. 5.

Hence if the data acquisition is longer the control signal has to be padded with a zero control signal and if the control signal is longer the data acquisition will continue till the control file signal is over. This is how the software takes care of the limitations of the data acquisition board and the user is transparent to this. Only the data length requested by the user will be saved as the output file.
the experiment is virtually repeated overwriting the old data.

By changing the options in the configuration file the user can change file names and run the experiments for various control signal files or output files with different data acquisition parameters like number of data points or digitizing frequency.

The files are saved in the MATLAB binary format. The structure of a typical file is as shown below. This can be seen by loading the file into the MATLAB workspace and listing the variables using the ‘who’ command as in Table 2 below.

<table>
<thead>
<tr>
<th>Name</th>
<th>Size</th>
<th>Bytes</th>
<th>Class</th>
</tr>
</thead>
<tbody>
<tr>
<td>calibration_m</td>
<td>5x1</td>
<td>40</td>
<td>double array</td>
</tr>
<tr>
<td>data</td>
<td>5x3500</td>
<td>140000</td>
<td>double array</td>
</tr>
<tr>
<td>units_m</td>
<td>5x7</td>
<td>70</td>
<td>char array</td>
</tr>
<tr>
<td>user_settings</td>
<td>1x225</td>
<td>450</td>
<td>char array</td>
</tr>
</tbody>
</table>

Table 2: Saved variables

All output files having the same naming convention for their variables namely ‘calibration_m’, ‘data’, ‘units_m’ and ‘user_settings’. These variables are in general matrices with the number of rows of the variables ‘data’, ‘calibration_m’ & ‘units_m’ designating the number of channels of data acquired. ‘data’ contains the acquired data as AD integer numbers ranging from 0 to 4096 for the 12 bit AD/DA board. Storing the data as integer values saves disk space. In the above example there are 5 channels and 3500 data points.

user_settings =

63-Apr-1998 12:35:41 USER_SETTINGS
IGAW DATA ACQUISITION PROGRAM
No. of Channels = 5
Points per Channel = 3000
Digitizing Frequency (Hz): 100
Control Signal Filename: break slated
Output Data Filename: filter.parm

Table 3: User settings for data acquisition

Fig. 7: Noise signal acquired

After the file save operation another message window Fig. 8 requesting the user to edit the ‘igaw.cfg’ file before rerunning the program again is displayed. If one does not edit and save the ‘igaw.cfg’ file,
Hence the data matrix has a size of 5 rows by 3500 columns. ‘Calibration_m’ contains the conversion constants for all the channels acquired. Once the data is loaded into MATLAB and processed for trend removal etc. it just a simple operation to multiply the two matrices to get the data converted to the desired physical units. The ‘units_m’ variable contains the actual physical units of the data channels as a character array. The ‘user_settings’ gives the parameters used for that particular data acquisition as given in the above example in Table 3 below. Since all the output files have similar format it is easy to write a MATLAB script file to automatically process the data acquired depending on the users needs.

3.1.4 Wave Generation & Control Software

This software consists of different parts and is written as different MATLAB files for different type of waves. The program ‘wg.m’ is for sinusoidal waves. The program ‘wavspec.m’ is for extreme waves like freak wave pulses.

These programs have similar format. They first take the input by the user and compute wave properties depending on wave flume range. After computing the desired wave the stroke of the wave maker is computed using wave theory and then the computer control signal is computed using the transfer function of the electric motor which is determined by system identification techniques. Finally the control signal file is written as a MATLAB file suitable for the data acquisition program ‘igaw.exe’ described earlier.

The wave generation software for sinusoidal waves ‘wg.m’ takes in as input the desired wave height and wave frequency. Parameters like water depth and digitizing frequency and time duration of signal are set from within the program since they are typically constant for a given series of experiments. However they can be edited and changed using any text editor before running the code in M cMATLAB. From these inputs and using wave theory the 1st and 2nd order stroke of the wave paddle is calculated as described in Hughes (1993). The MATLAB code for the same is presented in the script file ‘wg.m’ in Appendix. This stroke signal is then transformed to a computer control signal by the transfer function of the electric motor driving the paddle. The control signal obtained is outputted as ‘sine1.mat’ for 1st order and ‘sine2.mat’ for 2nd order. These files can then be used in ‘igaw.exe’ for wave generation. The program has options in the code to switch between piston and flap type operations. However this provision is by default and are required. This program was the piston or the flap equations have to be commented and run as required. This program was developed more to check the wave generation as a first step. But for the purpose of force dissipating structures the code ‘wavspec.m’ described below is used to create the extreme waves required for testing.

For freak waves the program ‘wavspec.m’ generates a control signal ‘freak.mat’. The principle adopted is that of linear superposition of waves of different frequencies such that they positively interfere at the point of interest. The program inputs are internally set in the code. They mainly consist of the flume water depth (typically 0.3 m), and the frequency range of the wave maker (0.8 Hz to 2.2 Hz). One could generate a time series based on a typical spectrum in heavy seas like JONSWAP but then the limited length of the flume of 21 m and the frequency range, limit the amount of high waves that can be created. Hence in the present experiments a flat spectra of unity is used to demonstrate the generation technique. The point of interest of wave generation is first predetermined. In these sets of experiments it was kept constant at 15.7 m from the wave paddle. It was decided to sum up 30 waves to get some components in the frequency range of 0.8 Hz to 2.2 Hz. (This number 30 was arrived at by trial and error. Too many components tend to flatten out the time series and too less do not contribute to a freak wave by superposition. However at this point the optimization of this number of components has been undertaken experimentally.) This is done with the intention of having all the peaks coincide at this point as shown in Fig. 9 freak wave below. This is only the mathematical creation of a desired wave time series.

Fig. 9: Mathematical freak wave

This freak time series is then back propagated in time using linear theory of water wave dispersion to obtain the ‘Wave @ t’ time series shown in Fig. 10 below. The technique adopted is one of shifting each frequency component by its appropriate wave number and generating the time series at the wave paddle. Since the number of components is less this brute force computational technique was adopted as compared to the mathematically efficient FFT techniques for large number of components.

Fig. 10: Wave elevation close to paddle

Using this wave signal at the paddle and the linear transfer function mentioned in the sine wave generation program, the stroke to the paddle was computed. This is done using the wave number at each frequency. This is shown in Fig. 11 below.
3.2 Jet dissipation of waves

In order to test the present concept of force dissipating structures it was required to reproduce the phenomenon of jet dissipation in the laboratory. In order to generate the hydraulic pressure required for jetting a pump was used in the laboratory. In reality this pressure could be provided by wave energy absorption from normal climates. This energy could be stored in pressure chambers in the device and used when necessary against extreme waves. However in the laboratory the principle of energy absorption was not attempted in these series of experiments as they have already been proven earlier. A wave energy absorbing prototype has already been built and tested in the sea. Koola (1993).

It is assumed here that this energy can be stored in pressure chambers. Hence for the laboratory demonstration of jetting the use of a pump with various jets is justified. This pump was connected to the jets and a throttling valve in series with the jet is used to vary the pressure discharge characteristics of each jet. Fig. 13 shows the pump and its accessories used in the jetting experiments.
3.2.1 Jet variations

In order to dissipate the waves a number of jets spanning the entire width of the flume was necessary. This was achieved by holes drilled on a tube which spanned the width of the flume, namely 300 mm. In order to test jets with different characteristics these tubes had holes with different diameters and spacing. A numbering scheme was adopted to describe each jet geometry as shown in Table 4 below.

<table>
<thead>
<tr>
<th>Mean Discharge (cc/h)</th>
<th>Pump Pressure (bar)</th>
<th>Jet Diameter (mm)</th>
<th>No. of Holes</th>
<th>Velocity (m/s)</th>
<th>Force (N)</th>
<th>Power (W)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Jet 4</td>
<td>101.45</td>
<td>5.70</td>
<td>1</td>
<td>8</td>
<td>16.15</td>
<td>1.64</td>
</tr>
<tr>
<td>Jet 3</td>
<td>273.61</td>
<td>5.10</td>
<td>1</td>
<td>15</td>
<td>23.22</td>
<td>6.35</td>
</tr>
<tr>
<td>Jet 2-2</td>
<td>345.00</td>
<td>4.89</td>
<td>2</td>
<td>8</td>
<td>13.73</td>
<td>4.74</td>
</tr>
<tr>
<td>Jet 2</td>
<td>401.48</td>
<td>4.60</td>
<td>1</td>
<td>8</td>
<td>15.97</td>
<td>6.41</td>
</tr>
<tr>
<td>Jet 1-2</td>
<td>382.40</td>
<td>4.60</td>
<td>2</td>
<td>15</td>
<td>8.11</td>
<td>3.10</td>
</tr>
<tr>
<td>Jet 1</td>
<td>514.07</td>
<td>4.25</td>
<td>2</td>
<td>15</td>
<td>10.91</td>
<td>5.61</td>
</tr>
</tbody>
</table>

Table 4: Jet classification

Even though 6 jets were fabricated the last two with jet diameters 0.5 mm had a very high pressure drop across them that the existing jet had hardly any power to serve the purpose of dissipation. Hence these jets were not used for the experiments. However the jets 1 & 2 were throttled to obtain different pressure drops. Jet 1 throttle is designated as Jet 1-2 and Jet 2 throttled as Jet 2-2 gave rise to pressure drops of 4.60 bar and 4.89 bar respectively at pump exit. Hence a total of 6 jet configurations were used for this testing program. Since the jets were to be fired only for a short interval of time of the order of 1 second to 10 seconds, it was decided to look at transients if any. Using the computer controlled jet firing to be discussed later, the discharge of the various jets for a firing duration of 1 second to 10 seconds was measured. A pre-calibrated measuring vessel was used for the purpose. The plots for the Jet Volume with time is shown in Fig. 14 below. It can be seen that after 1 second the plots are linear indicating negligible influence of transients. The slope of these plots gives the mean discharge of these jets. It can be seen that Jet 1-2 (throttled) has almost the same discharge as Jet 2 namely 382.40 and 401.48 cubic centimeters per second respectively as mentioned in Table 4 above.

The pump pressures to drive this discharge are obviously the same. However it should be noted that the no. of jet holes for both these jets are different. Jet 1-2 has 15 holes for the 300 mm total width whereas Jet 2 has only 8 holes. This indicates that the ejecting velocity from these two jets are different even though they have the same driving pressure and discharge. The velocity of the jet is computed from just the knowledge of the discharge and the jet geometry namely the jet diameter and the no. of holes.

Velocity = Discharge / (no. of holes * Area of one hole)

Area of hole = \( \frac{\pi}{4} \times D^2 \), where D is the diameter of the jet.

The Reaction Force of the Jet = \( \rho \times \text{Discharge} \times \text{Velocity} \), where \( \rho \) is mass density of fluid.

And the Power of Jet = Force * Velocity.

Fig. 15 shows the pump balance points for the various jets tested. These are the pressure and discharge values for each of the jet tested. It should be mentioned here that the jet piping circuit was not optimized for minimum pressure drop and hence when scaling to prototype values this should be kept in mind.

3.2.2 Jet control by computer

The timing of the jet and its duration was computer controlled. This was achieved using the second digital to analog part of the AD board, the last one being used for wave control. A square signal is generated in the control software 'igaw.exe' based on the start time and the stop time of the jet trigger specified in the 'igaw.cfg' configuration file. This square signal activates the relay valve built in series.
3.3 Experimental setup

The IGA Hofau 300 mm wide wave flume 21 m long was used for these experiments. The newly developed data acquisition software mentioned in section 2.1 was used to drive the wave paddle to generate large waves required for jet dissipation. To monitor the waves 3 wave probes at 0 m, 5.5 m and 15 m from the paddle were used. 0 m actually corresponds to 0.7 m from the mean position of the wave paddle. This was to avoid the paddle hitting the wave probe in case of extreme paddle motion. The extreme wave was achieved at the 15 m position which actually corresponded to 15.7 m from the mean position of the wave paddle. Fig. 16 shows the flume layout.

![IGAW Hofau Wave Flume](image)

**Fig. 16: Experimental layout for Jet experiments**

The jets are placed 1.3 m in front of the 15 m wave gauge. The measurements at the wave probe 15 m from the paddle with and without the jet quantify the levels of dissipation achieved with the jet.

4 JET RESULTS & DISCUSSIONS

With the computer aided experimentation system developed experiments have been run to test the effect of jet dissipation. Only the wave at 15 m was used to evaluate the effect of the jet. The quantification of the wave train is done on the basis of the single large wave. The wave height obtained in the flume for this large wave at the point of interest is 10.28 cm and this occurs at around 42 seconds.

It was first decided to look at the effect of jet timing and duration on the effect of dissipation. Since the large wave occurs at 42 seconds 15.7 m from the wave paddle and since the jet is 1.3 m in front of this location the jet firing has to start before this and must continue almost till 42 seconds in order to effectively dissipate the wave. After some trial and error it was decided to start firing the jet at 30, 33, 35 and 37 seconds for duration of 3, 5, 7, 10 seconds. The wave trains measured at the 15 m location in the time range of interest 30 to 50 seconds are presented in the subsequent graphs along with the reference wave without the jet. The maximum wave heights computed are shown in the respective figures. These timing and duration experiments were carried out only on Jet 1. It is assumed that these results will hold good for the other jets also.

Hence these set of experiments actually optimize the timing and duration for the entire set of jets to be tested. Fig. 17 shows the variations for Jet 1 at jet firing start at 30 seconds and duration of 5, 7 & 10 seconds. Jet duration of 10 seconds show maximum wave height reduction from 10.28 cm to 5.83 cm. Fig. 18 similarly shows that a 7 seconds firing duration is the most appropriate for the ranges of 5, 7 & 10 seconds when the start of firing is at 33 seconds. Here it can be seen that firing for a longer duration of 10 seconds is actually waste of jet power for no gain in wave reduction. This highlights the importance of jet timing and duration on this technique of force dissipation. Fig. 19 and Fig. 20 similarly shows these effects for 35 and 37 seconds start of firing respectively. Compiling all these results together it is seen that the best firing start time is 33 seconds and its best duration is 7 seconds. Further optimization was not attempted in these preliminary experiments. Once the jet timing and duration was optimized these set of values were then used to evaluate the effect of jet geometry and jetting angle. Again Jet 1 was used to study the effect of angle and to arrive at suitable values to run the other jet geometry. For this configuration the angles tried out were from 0 degrees to 75 degrees. Zero degrees correspond to a vertically downward fired jet. The angle 0° is marked in Fig. 16 for Jet layout for these experiments. 90 degrees corresponds to a horizontally fired jet against the wave. Since the jet was placed 13 cm above the still water level for all these experiments, firing at 90 does not make much sense as most of the jet would be wasted.

Fig. 21 shows the effect of the large wave due to jet dissipation for Jet 1 for 10 different jet angles. The start timing of jet firing was 33 sec for 7 seconds duration and is adopted for all further experiments. In this particular case it was noticed that the larger the angle the better it is for jet dissipation. Hence in all further experiments only angles above 40 degrees were tested up to 75 degrees. The angles tested are 40, 50, 60, 65, 70, and 75 degrees. Fig. 22 shows the results for Jet 1-2, the throttled jet 1. The best angle for this configurations is 70 degrees. Fig. 23 for Jet 2 shows a wave height of 3.38 cm for a jet angle of 65 degrees and Fig. 24 shows 4.06 cm for 70 degrees for the throttled Jet 2-2. Similarly Fig. 25 & 26 show the results for Jet 3 and 4, the best angle being 75 and 70 degrees respectively.

Compiling the above results Fig. 27 shows that the best timing is at 33 seconds for a duration of 7 seconds and an angle of the order of 70 degrees is best suited when the jet is fired from an elevation of 13 cm above still water level. These plots computes the percentage wave height reduction as compared to the wave without any jetting. Percentage wave height reduction of the order of 15 to 65 have been achieved with these jets.
It is interesting to note that Jet 1-2 and Jet 2 have similar pressure discharge characteristics. However, Jet 2 has 8 jets whereas Jet 1-2 has 15 jets over the wave flume width of 300 mm. Hence, the velocity of Jet 2 was higher. Jet 2 had a velocity of 15.97 m/s as compared to 8.11 m/s for Jet 1-2 as shown in Table 4. It can also be seen from Fig. 27 that Jet 2 is more effective in wave dissipation than Jet 1-2. Hence, for a given pressure and discharge, higher the jet velocity the better. It should be noted that this will affect the spacing of jets and if kept too far apart it would not be so effective. However, this aspect of jet spacing was evaluated only for two values in the present set of tests. More experiments would be required to quantify this.

Fig. 27 shows that Jet 2 was the most effective in wave height reduction. From Table 4 it can be seen that Jet 2 had the highest reaction force of all jets even though Jet 3 had the highest power and velocity. Hence, from the results presented it can be concluded that higher the reaction force of the jet the better. This has very important practical value. The cost of jet firing will be in proportion to the jet energy which is the product of power of jet times the time duration of firing. Since for these series of tests the timing was kept the same one could compare the costs to be proportional to the power. But jet dissipation is more effective based on the jet reaction force which is higher for Jet 2 as compared to Jet 3 with higher power. Hence it is not only cheaper to use Jet 2 but, it is also more effective.
Based on observations of these tests it is felt that the jet acts like a reflecting wall in addition to the dissipation that it produces due to fluid turbulence. Hence the effect in addition to dissipation is to partly reflect the wave thereby altering the wave sequence and preventing it to interfere positively to create the big wave.

The next step was to compute the force on a vertical wall like a caisson. The extreme wave created in the lab had a wave height of 10.28 cm and a time period of 0.86 s. The force was computed based on guidelines in the shore protection manual (1984). For steep waves the procedure by Miche-Rundgren has been adopted.

**Input parameters:**

- Incident wave height $H_i = 10.28$ cm
- Water depth $d = 30$ cm
- Time period $T = 0.86$ s

**Computations:**

$$H_i/d = 10.28/30 = 0.3426$$

$$H_i/(gT^2) = (10.28/100)/(9.81 \times 0.862) = 0.142$$

From design curves for $(H_i/(gT^2)) = 0.142$ and $H_i/d = 0.34; h_i/H_i = 0.34$

Therefore, $h_i = 0.34 \times H_i = 0.34 \times 10.28 = 3.495$ cm

Crest height $y_c = d + h_i + H_i = 30 + 3.495 + 10.28 = 43.775$ cm

This is the height to which the crest of the wave will reach on a vertical fully reflecting barrier. This is 13.775 cm above the mean water level of 30 cm.

The force coefficients from design chart,

Crest force coeff. $(F/wd^2) = 0.12$

Hence $F_L = 0.12 \times 9.8 \times 0.32 = 0.106 \text{ kN/m}$, where $w = 9.8 \text{ kN/m}^2$ the weight density of water

For the flume width of $30 \text{ cm} = 0.3 \text{ m}$, the total crest force works out to

$$F_L = 0.106 \times 1000 \times 0.3 \text{ N} = 31.8 \text{ N}$$

Similarly for the trough $(F/wd^2) = 0.22$

And $F_L = 58.2 \text{ N} $

The maximum reaction force of the Jets tested are of the order of 6 $N$ which is just 1/5th the crest wave force on a vertical wall assuming 90 degree jetting. Since the jet is at an angle only the sine component of the jet angle is in effect transferred to the caisson. Computing the wave force for the jet reduced wave of height 4 cm using the above approach the crest height works out to 21 Newtons. If it is assumed as first approximation that the reaction on the structure with jetting is the same as the reaction due to the jet plus the force on it due to the reduced wave height we have total reaction on structure is $21 + 0 = 27 \text{ N}$, which is less than the force due to a 10.28 cm wave namely 31.8 $N$. Hence
5 CONCLUSIONS

A computerized data acquisition system and necessary hydrodynamic software was developed to generate single large waves and jet triggering.

Jet dissipation of these large waves is technically achievable. Jet reaction is only a fraction of the reaction wave force acting on the structure in order to dissipate the waves to the desired level.

Six jet configurations were tested and in general the higher the jet reaction better is the dissipation. This force determines the penetration of the jet into the wave.

Large firing angles of the order of 60-70 degrees from the vertical and into the wave seems to be most effective in dissipation. This is for the case of jet firing above still water level.

It has been observed that highest jet power need not mean best dissipation. Jet force is the most decisive criteria. This has to be more critically evaluated. In the present tests this has been achieved by controlling jet area to obtain a higher jet reaction with lower jet velocity.

For the range of wave heights tested in the laboratory, for a 102.8 mm wave with time period 0.86 seconds the wave force on a vertical wall 300 mm wide is of the order of 31.8 Newton. The jet reaction to reduce this wave height of 102.8 mm to 40 mm is only 6 Newton. And the resulting wave force due to a 40 mm wave is 21 Newton.

The force reduction achieved with jet reaction included is 15% and without is 34% for the above values tested.

Actual forces on a vertical wall should be measured, rather than computed as done in the present work, to quantify the benefits more accurately.

Based on experience the following suggestions are put forward:

In order to avoid the reaction of the jet on the structure, it can be fired from the sea bed up. However it has to be noted that maximum wave energy is at the surface and hence effective dissipation will occur at the crest of the wave. The benefits have to be worked out.

Pulsed jetting as suggested by Prof. Kaldenhoff will reduce jet discharge, but water hammer effects and their associated reactions will have to be worked out. This could further be combined with alternate jet firing. However it is felt that stationary jets with fluid controls should be used if pulses or swings should be attempted. This is because the sea environment is highly corrosive and motion mechanisms should be avoided.

High pressure storage has advantages in that Jet pumps utilizing this pressure could use the surrounding sea water to dissipate the waves. Hence the volume of water required for jetting need not be stored in the structure.

In the present work only two spacings were tried out. Optimal spacing of jets should be looked into to reduce wastage of power.

Tests at 2 different model scales should be attempted to understand the scaling process better. This is because the wave phenomenon is based on Froude scaling whereas the jet and dissipation are based on the Euler and Reynolds scaling. It is impossible to model all the three simultaneously. Hence scaling to prototype values should be carried out with caution.
6 ACKNOWLEDGEMENTS

My sincere thanks to The Alexander Von Humboldt Foundation for sponsoring my 1 year research stay in Germany and supporting this research work.

Prof. Hans Kaldenhoff, my mentor and Director of IGAW had the faith in me which was my main driving force to get these experiments up and working against all odds. I appreciate the efforts taken by my friend Prof. Kai Graw who came all the way from Leipzig at short notice to help me interface my software to the wave machine at Hofau. Dr. Andreas Schlenkhooff was the key figure in making me comfortable with the laboratory. Teaching assistants Mr. Torsten Schlurmann and Mr. Torsten Dose were always there for me when I got stuck. Of all the students that helped me I must mention Mr. Dirk Hagemann who was my electrical hardware support without whom there would have been only a software code and no interface. Ms. Anke Helfer ran most of the experiments for me so meticulously at a time when I had nearly run out of time. My sincere thanks to Mrs. Anke Cordt our Administrative officer, who made me forget what administrative boredom is, the last one year so that I could concentrate on my work.

7 REFERENCES


PHYSIKALISCHE UND MATHEMATISCHE BESTIMMUNG DER ENERGIEANTEILE UNTERSCHIEDLICH ERZEUGTER SCHWERFELLEN

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### Arbeits- und Ergebnisbericht

1.1 Ausgangsfragen und Zielsetzung des Projekts

Im Rahmen dieses Forschungsvorhabens wurden sehr umfangreiche experimentelle Untersuchungen in der Wellenlänge des Instituts für Grundbau, Abfall- und Wasserwesen durchgeführt. Das dabei in erster Linie angestrebte Ziel bestand darin festzustellen, in welcher Größe die Art der Wellenerzeugung die Entstehung hoch- und niedriger frequenter Wellen beeinflußt. In einem weiteren Schritt sollte untersucht werden, nach welcher Entfernung vom Wellenblatt sich die Art der Wellenerzeugung auf die Energieaufteilung einer Welle nicht mehr merkbar macht.

Um einen natürlichen Seegang, der auf den Ozeanen durch Windfelder erzeugt wird, nachzuvollziehen, bedarf es im kleinenstädtischen Labor mechanisch ungetriebener Wellenmaschinen, die auf unterschiedliche Art und Weise arbeiten können [Biesel, 1954]. Die meisten Institut zur Verfügung stehende Wellenlinie (L = 24 m, B = 30 m, d = 0,20 bis 0,40 m) mit der „Wuppertaler Wellenmaschine“ basiert auf der Methode, daß ein senkrecht zur Wasseroberfläche eingetauchtes Wellenblatt harmonisch mit einer Frequenz f angeregt wird. Die Erregungsfrequenz f und die Auslenkung 5 (Stroke) des Blattes werden dabei auf das Medium übertragen, so daß sich eine fortschreitende Schwerewelle mit derselben Frequenz f, aber nicht zwangsläufig derselben Wellenamplitude 5 des Strokes S, im Wasser breitet. Die „Wuppertaler Wellenmaschine“ ist in diesem Sinne eine Weiterentwicklung der klassischen Wellenmaschinen, da das Wellenblatt in beliebigen Moden angesteuert werden kann. Abbildung 1.1 veranschaulicht diese Ansteuerungsmechanismen schematisch.

![PISTON MODUS, PISTON - FLAP MIX, FLAP MODUS](image)

Abb. 1.1: Schematische Ansteuerungsmechanismen der „Wuppertaler Wellenmaschine“

In Vorfeld der Antragstellung dieses Forschungsvorhabens konnten erste Versuche mit dieser Wellenmaschine gefahren werden. In der Literatur herrschte die einheitliche Meinung [Dalrymple, Doan, 1984], daß eine bestimmte Wellenkategorie vorzugsweise auch nur durch eine zuvor definierte Erregung erzeugt werden kann. Tabelle 1.1 liefert einen Überblick über diesen in der Literatur zu findenden Zusammenhang:

<table>
<thead>
<tr>
<th>rel. Wassertiefe d/L</th>
<th>Urs-Null-Zahl Ur = [H+L]/d’</th>
<th>Bezeichnung (Wellenkategorie)</th>
<th>Ansteuerung der Wellenmaschine</th>
</tr>
</thead>
<tbody>
<tr>
<td>&lt; 1/20</td>
<td>&gt; 400 H</td>
<td>Flachwasserwelle</td>
<td>PISTON MODUS</td>
</tr>
<tr>
<td>1/20 ≤ L ≤ 1/2</td>
<td>&gt; 400 H ≥ Ur ≥ 4 H</td>
<td>Übergangsbereich</td>
<td>PISTON oder FLAP</td>
</tr>
<tr>
<td>&gt; 1/2</td>
<td>&lt; 4 H</td>
<td>Tiefwasserwelle</td>
<td>FLAP MODUS</td>
</tr>
</tbody>
</table>

Tab. 1.1: Wellenkategorien

Das theoretische Kriterium der relativen Wassertiefe d/L, das eine Flachwasserwelle, eine Welle im Übergangsbereich oder eine Tiefwasserwelle beschreibt, kann in der Praxis anscheinend durch die die gesamte Wassertiefe d eine nahezu konstante Horizontalgeschwindigkeit u, während die Geschwindigkeitsverteilung in einer Wellenlinie nachzuvollziehen, liegen (vgl. hierzu die Abb. 1.2a). Eine Flachwasserwelle hat über Vertikalgeschwindigkeit v am Boden Null und an der Oberfläche maximal. Um diese Art der gleichmäßigen Energieeintragen des Wellenblattes verlangsamen, so daß eine translatorische Ansteuerung Tiefwasserwellen in Wellenlinien eine grundsätzlich andere Energieeintragung, als die Horizontal- und Vertikalgeschwindigkeit am Boden beide Null und an der Oberfläche beide maximal sein müssen. ("Flap Modus") werden, indem der Fußpunkt des Wellenblattes am Boden festgehalten wird und an beiden Modus sowie jegliche dazwischen liegende Ansteuerung problemlos nachfahren und somit zunächst das gesamte Spektrum der Schwerewellen in einer Wellenlinie simulieren.

Teil 3: Bestimmung der Energieanteile

1.2 Entwicklung des Arbeits- und Auswertungsprogramms

<table>
<thead>
<tr>
<th>Wassertiefe d = 20 cm</th>
<th>Wassertiefe d = 30 cm</th>
<th>Wassertiefe d = 40 cm</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 (Piston)</td>
<td>1 (Piston)</td>
<td>1 (Piston)</td>
</tr>
<tr>
<td>f = 0,5075/1,015/2,02 Hz</td>
<td>f = 0,5075/1,015/2,02 Hz</td>
<td>f = 0,5075/1,015/2,02 Hz</td>
</tr>
<tr>
<td>H = 10/15/20/25/30/35/40 mm</td>
<td>H = 10/15/20/25/30/35/40 mm</td>
<td>H = 10/15/20/25/30/35/40 mm</td>
</tr>
<tr>
<td>0 (Flap)</td>
<td>0 (Flap)</td>
<td>0 (Flap)</td>
</tr>
<tr>
<td>f = 0,5075/1,015/2,02 Hz</td>
<td>f = 0,5075/1,015/2,02 Hz</td>
<td>f = 0,5075/1,015/2,02 Hz</td>
</tr>
<tr>
<td>H = 10/15/20/25/30/35/40 mm</td>
<td>H = 10/15/20/25/30/35/40 mm</td>
<td>H = 10/15/20/25/30/35/40 mm</td>
</tr>
</tbody>
</table>

Tab. 1.2: Versuchssprogramm

Zu Beginn der experimentellen Ausführungen wurde eine umfangreiche Literaturstudie durchgeführt, die auch schließlich zur Wahl der Meßeinrichtung in der Wellenrinne führte. In einem ersten Schritt diente die von Dalrymple und Dean [1984] entwickelte „Linear WaveMaker Theory“, die theoretisch postuliert, daß in einer Entfernung 3*d vom Wellenblatt (lineare) höherfrequente, sichelförmige Wellen zu 99,9% abgebaut werden müssen und sich die Welle anschließend mit ihrem ursprünglichen Ausbreitungsverhalten fortpflanzt wird. Dieser Ansatz berücksichtigt nicht die nichtlinearen, höherfrequenten harmonischen Anteile, beschränkt sich somit auch auf verschwindend kleine Amplituden. Dennoch wurde die Meßeinrichtung nach diesem Ansatz ausgewählt. Insgesamt wurden vier Wellenpfeile in den Abständen 0,8*d, 2,5*d, 4*d und 10*d vom Wellenblatt installiert. Somit können Datenreihen in der Umgebung der Stelle 3*d, mit 0,8*d unmittelbar hinter dem Wellenblatt bzw. mit einem weiteren Pegel bei 20*d in genügender Entfernung vom Wellenblatt aufgezeichnet werden. Insgesamt umfaßt das Meßprogramm ca. 350 Einzelversuche, die in der Tabelle 1.2 zusammengestellt sind. Es ist auffällig, daß das Meßprogramm für die Wassertiefe d = 30 cm am umfangreichsten ist, was darauf zurückzuführen ist, daß ursprünglich nur für diese Wassertiefe Experimente gefahren werden sollten. Nach ersten Datenanalysen konnte die Abhängigkeit des Wellenstandes für die Wassertiefen Versuche im Pisten und Flap Modus nachgeführt werden, um den vermuteten Einfluß aufzudecken. Zusätzlich muß erwähnt werden, daß zu den WasserstandsMESSungen mit den Wellenpfeilen nur bereichweise Geschwindigkeitsmessungen durchgeführt wurden, um Aufschluß über die sich einstellende Geschwindigkeitsverteilung unter der nichtlinearen Welle zu erhalten und der Differenzierung der Energieanteilbestimmung einer Welle gerecht zu werden.

1.2.1 Probleme bei der Umsetzung des Arbeitsprogramms

Nach der umfangreichen Literaturrecherche konnte zudem ein weiteres Phänomen bei der Erzeugung von Schwierwellen in labotechnischen Einrichtungen beobachtet werden. Durch eine lineare Ansteuerung des Wellenblattes werden speziell im Bereich der Flachwasserwellen (d/L < 1/50) zusätzlich freie, parasitäre Wellen generiert, die sich mit ihnen originären dispersiven Ausbreitungsverhalten in der Wellenrinne fortsetzen und mit der ursprünglichen Welle überlagert werden. Dieses Phänomen existiert nur für mechanisch erzeugte Wellenzüge, ist also unnatürlich und muß infolgedessen bei der Generierung von Wellen verhindert werden.

Abb. 1.4 a: Räumliche Darstellung der Überlagerung

Abb. 1.4 b: Zeittliche Darstellung der Überlagerung

Abb. 1.4 c: Überlagerte Ortilgeschwindigkeiten
Die Abbildung 1.4 a stellt diesen Überlagerungsvorgang exemplarisch für einen Zeitpunkt räumlich dar, während Abb. 1.4 b das Prinzip zeitlich an einem festen Ort beschreibt. Hervorzuheben ist, dass das Phänomen der parasitären Wellen nicht an jedem Ort in einem Wellenkanal sofort eindeutig zu erkennen ist. Es existieren Positionen im Kanal an denen sich die fundamentale und die parasitäre Welle so nahe kommen, dass die 'Verurteilung' zu visuell erfahrbar ist. Im Welle phasengleich überlagern und infolgedessen keine "Verunreinigung" zu visuell erfahrbar ist. Im Gegensatz dazu heben sich parallele und auf den Wellenwellen aufgelesene "Verunreinigungen" der Meßsignale ab. In Abb. 1.4 c ist das sich dadurch "theoretisch" ergebende Geschwindigkeitsprofil als Darstellung aufgrund der unterschiedlichen Phasengeschwindigkeiten der einzelnen Wellen in der Wellenleiterleitung gegeben. Die Wellenlinien bestehen aus konsekutiven Phasenversetzungen und werden in der Literatur als "Second Order Waveform Theory" bezeichnet. Aufgrund der vorgeschlagenen Anlagierung der Wellenleiterleitung wird der Wellenleiter in einer harmonische, x-fach phasenversetzte 1. Oberschwingerung hinzugefügt, die die parasitäre Welle in eine scheinbare Amplitude ausläuft. Die Annahme an dieser Stelle, dass die parasitäre Welle in einer scheinbaren Amplitude ausläuft, ist somit validiert.

Das im nächsten Abschnitt beschriebene Auswertungsverfahren ist sehr genommen für Flachwasserwellen anwendbar, so dass für diese spezielle Kategorie die ermittelten Ergebnisse anwendbar sind. Abbildung 1.5 verdeutlicht den Zusammenhang der ermittelten Ergebnisse. Hervorzuheben ist in diesem Zusammenhang, dass von der gesamten Versuchsausführung dieses Anwendbarkeit der Wellenleiterleitung, die dafür gedacht ist, dass die Wasserspiegelhöhe von der Wellenleiterleitung produziert werden - und diese Versuchskonstellationen bei strenger Beachtung der Anwendungsbestimmungen nicht in die weitere Analyse einbezogen werden dürfen.

**Abb. 1.5: Wellenkategorien**

**Abb. 1.6: Phasendiagramm an zwei Pegelfeststellen**

Welle wieder. In der Wellenlinie setzt sich diese Welle aber aus zwei frequenzgleichen Wellen mit unterschiedlichen, unbekannten Amplituden und Phasen zusammen. Dabei entspricht die geschilderte dargestellte Kreis mit dem Radius $A_0$ und der Phase $\phi_0$ der ersten gebundenen superharmonischen Welle und der punktierte Kreis spiegelt die parastrem, freie Welle mit dem Radius $A_0$ und der Phase $\phi_0$ wider. Um auf die ursprüngliche Größe der freien Welle bzw. der nichtlinearen höher harmonischen gebundenen Welle zurückzuschließen, sind die Informationen an einer Pegelmeßstelle nicht ausreichend, da den insgesamt vier Unbekannten (2 Amplituden, 2 Phasen) nur zwei nichtlineare Gleichungen gegenüberstehen, so daß (mindestens) eine weitere Pegelmeßstelle (z.B. 4 d) in einem definierten Abstand zur Verfügung stehen muß, um zwei weitere Gleichungen zur Lösung dieses Problems hinzuzufügen. Auffällig ist, daß die vektoriell summierten Signale nicht zwangläufig dieselbe Amplitude besitzen. Aufgrund des unterschiedlichen Ausbreitungsverhaltens (Phasengleichung) der beiden voneinander unabhängigen Wellen existiert eine in Abhängigkeit der Entfernung vom Wellenblatt charakteristische Oberflächenkontur (siehe auch Abb. 1.4a).

1.3 Darstellung der Ergebnisse

1.3.1 Auslenkung der Wasserroberfläche und Orbitalgeschwindigkeiten unterhalb der Wellen

Die auf den folgenden Seiten – nach der Wassertiefe $d$ differenzierten - dargestellten Abbildungen 1.7a bis d, 1.8a bis d und 1.9a bis d zeigen in zusammengefaßter Form die Ergebnisse dieses Forschungsvorhabens. Die jeweils vier aufgeführten Einzeldarstellungen der Ergebnisse der Fourier-Analyse an den vier Wellengängen. Jedes zeigt eine logarithmisch aufgetragene Beziehung zwischen der Urselzahl und der berechneten (DFT) fundamentalen Amplitude beziehungsweise der ersten gebundenen Superharmonischen der Welle. Es wird darauf aufmerksam gemacht, daß die Ergebnisse doppelt logarithmisch aufgetragen sind, um die Analyse der fundamentalen und der ersten superharmonischen Frequenzen visuell anschaulicher zu gestalten. In jedem Diagramm werden zu den fünf Erregerfrequenzen $f = 0.5; 0.75; 1.0; 1.5$ und 2.0 Hz und den variierenden Gesamtwellenhöhen die zugehörigen Ursel-Parameter berechnet und anschließend mit der durch die Fourier-Analyse bestimmten Amplituden der fundamentalen Frequenz und der Amplitude der ersten Oberschwingung aufgetragen. Jeder Datensatz in Abbildung repräsentiert jeweils einen Meßwert. Es wird darauf hingewiesen, daß die Information auf welche Art (Erzeugung) ein Meßpunkt zustande gekommen ist nicht mehr zusätzlich angezeigt wird. Um diesen Effekt dennoch zu erfassen, ist in jedem Diagramm eine frequenzabhängige polynomielle Approximationskurve 2. Grades eingetragen, die aus den Meßwerten mittels eines Fehlersquadrativverfahrens bestimmt wurde. Je kleiner die Fehlerabweichung ist – sich die Meßwerte, obgleich der untersuchten Erzeugung, an das Poly nom schmiegen – desto eher kann das Ergebnis gefaßt werden, daß die Art der Erzeugung einer Welle an dieser Stelle in der Rinne keine Rolle mehr spielt.

Im Rahmen dieses Forschungsvorhabens wurde die Untersuchung der Geschwindigkeitseigenschaften unterhalb der Wasserroberfläche nicht mit demselben umfangreichen Versuchsprogramm gefahren. Diese Versuche basieren auf einer Wassertiefe $d = 0.30 m$, Piston und Flap Modus, Frequenzen $f = 0.5; 1.0; 1.5$ und 2.0 Hz, sowie einer Wellenhöhe $H = 4.0 cm \pm 0.5 cm$. Für diese Konstellation wurden Geschwindigkeitsmessungen unter der Welle in den Tiefen 1.0; 10.0; 15.0; 20.0 und 25.0 cm durchgeführt. Die Orbitalgeschwindigkeiten werden hier nur in 5,0 cm Tiefe an den vier Pegelmeßstellen dargestellt. Zusätzlich werden durch die DFT ermittelten Amplituden der fundamentalen Frequenz und der ersten superharmonischen Oberschwingung in Form einer Tabelle angegeben und die daraus berechneten Orbitalgeschwindigkeiten in den Abbildungen 1.10a und b, 1.11a und b, 1.12a und b, 1.13a und b angezeigt.

1.4 Ergebnisanalyse

1.4.1 Auslenkung der Wasserroberflächen

Das in Tab. 1.2 beschriebene Versuchsprogramm deutet an, daß die Randbedingung der Wassertiefe $d = 0.30 m$ die umfangreichen Untersuchungen angestellt wurden. Erst nach Analyse der Ergebnisse dieser Wassertiefen wurden hinsichtlich der vermuteten direkten Abhängigkeit von den mit beispielsweise bei einer hochfrequenten Tiefwasserwelle ($f = 2.0 Hz$) nach einer Entfernung benötigt eine niedrigfrequente Welle ($f = 0.5 Hz$) eine deutlich längere räumliche Entwicklung ($20 d$) unterschiedlicher Erzeugung veranschlagt wird, spiegelt sich in der rezeptiven Aufteilung der Amplituden (Amplituden) der beiden voneinander unabhängigen Wellen manifest eine in Abhängigkeit der Entfernung vom Wellenblatt charakteristische Oberflächenkontur (siehe auch Abb. 1.4a).

1.4.2 Orbitalgeschwindigkeiten

Logarithmischer Vergleich der Amplitude der fundamentalen und der ersten superharmonischen Frequenzen einer jeweiligen polynomialen Approximation 2. Grades in Abhängigkeit der Umsch-Zahl für 100% und 6% an den vier Meßpositionen 0.8 d, 2.5 d, 4 d und 20 d für Wassertiefe d = 0.20 m

Abb. 1 a: Umsch-Zahl zu Amplitude der fundamentalen Frequenz bzw. 1. Superharmonischen für d = 0.20 m und Erregerung 0.4 d vom Wellenbruch

Abb. 1 b: Umsch-Zahl zu Amplitude der fundamentalen Frequenz bzw. 1. Superharmonischen für d = 0.20 m und Erregerung 2.5 d vom Wellenbruch

Abb. 1 c: Umsch-Zahl zu Amplitude der fundamentalen Frequenz bzw. 1. Superharmonischen für d = 0.20 m und Erregerung 4.0 d vom Wellenbruch

Abb. 1 d: Umsch-Zahl zu Amplitude der fundamentalen Frequenz bzw. 1. Superharmonischen für d = 0.20 m und Erregerung 20.0 d vom Wellenbruch

Phasendiagramme der Meßwerte und der nach linearer und nichtlinearer Theorie (2. Ordnung) berechneten Geschwindigkeiten an den vier Meßpositionen 0.8 d, 2.5 d, 4 d und 20 d

ORBITALGESCHWINDIGKEITEN: \( f = 0.5 \text{Hz}, t = 0.05 \text{m}, d = 0.30 \text{m}, \text{PISTON-MODUS (100%)}, H = 4 \text{cm} \)

<table>
<thead>
<tr>
<th>0.8 d</th>
<th>2.5 d</th>
<th>4 d</th>
<th>20 d</th>
</tr>
</thead>
<tbody>
<tr>
<td>( f = 0.5 \text{Hz} )</td>
<td>( f = 1.0 \text{Hz} )</td>
<td>( f = 0.5 \text{Hz} )</td>
<td>( f = 1.0 \text{Hz} )</td>
</tr>
<tr>
<td>( A = 2.137 \text{cm} )</td>
<td>( A = 0.125 \text{cm} )</td>
<td>( A = 2.178 \text{cm} )</td>
<td>( A = 0.218 \text{cm} )</td>
</tr>
</tbody>
</table>

Abb. 1.10 a: Orbitalsgeschwindigkeiten Piston Modus \( f = 0.5 \text{Hz} \)

ORBITALGESCHWINDIGKEITEN: \( f = 0.5 \text{Hz}, t = 0.05 \text{m}, d = 0.30 \text{m}, \text{FLAP-MODUS (90%)}, H = 4 \text{cm} \)

<table>
<thead>
<tr>
<th>0.8 d</th>
<th>2.5 d</th>
<th>4 d</th>
<th>20 d</th>
</tr>
</thead>
<tbody>
<tr>
<td>( f = 0.5 \text{Hz} )</td>
<td>( f = 1.0 \text{Hz} )</td>
<td>( f = 0.5 \text{Hz} )</td>
<td>( f = 1.0 \text{Hz} )</td>
</tr>
<tr>
<td>( A = 1.921 \text{cm} )</td>
<td>( A = 0.302 \text{cm} )</td>
<td>( A = 1.878 \text{cm} )</td>
<td>( A = 0.323 \text{cm} )</td>
</tr>
</tbody>
</table>

Abb. 1.10 b: Orbitalsgeschwindigkeiten Flap Modus \( f = 0.5 \text{Hz} \)
Phasendiagramme der Messwerte und der nach linearer und nichtlinearer Theorie (2. Ordnung) berechneten Geschwindigkeiten an den vier Meßpositionen 0,8 d, 2,5 d, 4 d und 20 d

**ORBITALGESCHWINDIGKEITEN** $f = 1,0$ Hz, $t = 0,05$ m, $d = 0,30$ m, PISTON-MODUS (100%), $H = 4$ cm

<table>
<thead>
<tr>
<th>$0.8$ d</th>
<th>$2.5$ d</th>
<th>$4$ d</th>
<th>$20$ d</th>
</tr>
</thead>
<tbody>
<tr>
<td>$f = 1,0$ Hz</td>
<td>$f = 2,0$ Hz</td>
<td>$f = 1,0$ Hz</td>
<td>$f = 2,0$ Hz</td>
</tr>
<tr>
<td>$A = 1,999$ cm</td>
<td>$A = 0,152$ cm</td>
<td>$A = 2,032$ cm</td>
<td>$A = 0,157$ cm</td>
</tr>
</tbody>
</table>

**Abb. 1.11 a: Orbitalgeschwindigkeiten Piston Modus $f = 1,0$ Hz**

**ORBITALGESCHWINDIGKEITEN** $f = 1,0$ Hz, $t = 0,05$ m, $d = 0,30$ m, FLAP-MODUS (0%), $H = 4$ cm

<table>
<thead>
<tr>
<th>$0.8$ d</th>
<th>$2.5$ d</th>
<th>$4$ d</th>
<th>$20$ d</th>
</tr>
</thead>
<tbody>
<tr>
<td>$f = 1,0$ Hz</td>
<td>$f = 2,0$ Hz</td>
<td>$f = 1,0$ Hz</td>
<td>$f = 2,0$ Hz</td>
</tr>
<tr>
<td>$A = 1,900$ cm</td>
<td>$A = 0,235$ cm</td>
<td>$A = 1,927$ cm</td>
<td>$A = 0,153$ cm</td>
</tr>
</tbody>
</table>

**Abb. 1.11 b: Orbitalgeschwindigkeiten Flap Modus $f = 1,0$ Hz**

Phasendiagramme der Messwerte und der nach linearer und nichtlinearer Theorie (2. Ordnung) berechneten Geschwindigkeiten an den vier Meßpositionen 0,8 d, 2,5 d, 4 d und 20 d

**ORBITALGESCHWINDIGKEITEN** $f = 1,5$ Hz, $t = 0,05$ m, $d = 0,30$ m, PISTON-MODUS (100%), $H = 4$ cm

<table>
<thead>
<tr>
<th>$0.8$ d</th>
<th>$2.5$ d</th>
<th>$4$ d</th>
<th>$20$ d</th>
</tr>
</thead>
<tbody>
<tr>
<td>$f = 1,5$ Hz</td>
<td>$f = 3,0$ Hz</td>
<td>$f = 1,5$ Hz</td>
<td>$f = 3,0$ Hz</td>
</tr>
<tr>
<td>$A = 1,808$ cm</td>
<td>$A = 0,184$ cm</td>
<td>$A = 1,687$ cm</td>
<td>$A = 0,115$ cm</td>
</tr>
</tbody>
</table>

**Abb. 1.12 a: Orbitalgeschwindigkeiten Piston Modus $f = 1,5$ Hz**

**ORBITALGESCHWINDIGKEITEN** $f = 1,5$ Hz, $t = 0,05$ m, $d = 0,30$ m, FLAP-MODUS (0%), $H = 4$ cm

<table>
<thead>
<tr>
<th>$0.8$ d</th>
<th>$2.5$ d</th>
<th>$4$ d</th>
<th>$20$ d</th>
</tr>
</thead>
<tbody>
<tr>
<td>$f = 1,5$ Hz</td>
<td>$f = 3,0$ Hz</td>
<td>$f = 1,5$ Hz</td>
<td>$f = 3,0$ Hz</td>
</tr>
<tr>
<td>$A = 2,055$ cm</td>
<td>$A = 0,200$ cm</td>
<td>$A = 1,999$ cm</td>
<td>$A = 0,149$ cm</td>
</tr>
</tbody>
</table>

**Abb. 1.12 b: Orbitalgeschwindigkeiten Flap Modus $f = 1,5$ Hz**
1.5 Zusammenfassung

Im Rahmen des Forschungsvorhabens „Physikalische und mathematische Bestimmung der Energieanteile unterschiedlich erzeugter Scherwellen“ wurde ein sehr umfangreiches experimentelles Untersuchungsprogramm für die Ermittlung der zu erwartenden Wellenamplituden und Orbitalgeschwindigkeiten durchgeführt und ausgewertet. Die Einflüsse der nichtlinearen Anteile aus den Superharmonischen einerseits, aber auch aus den an den Wellenflansch zusätzlich erzeugten freien Wellen andererseits, wurden theoretisch erörtert und durch die Messungen sowohl der Amplituden als auch der Orbitalschwingungen unter den Wellen verifiziert.

Für die Trennung der einzelnen Amplitudenanteile wurde eine neue sehr aufwendige Analysenmethode entwickelt, die nach Abschluß der noch laufenden Sensitivitätsprüfung gesondert veröffentlicht werden soll. Die Darstellung der Ergebnisse für die Amplituden in den Abb. 1.7 bis 1.9 zeigt, daß nach einer gewissen Entfernung vom Wellenflansch die Amplituden nicht mehr von der Art der Wellenerzeugung (Piston 100%, Flap 90%) bzw. Piston-Flap-Mix) abhängen. Diese Entfernung wird von den Wellenparametern \( f \), \( H \) und \( d \) geprägt, kann aber nicht durch ein konstantes Verhältnis der relativen Wellenhöhe \( H/d \) oder der relativen Wellenlänge \( L/d \) angegeben werden. Für die Orbitalschwingungen ergibt sich eine ähnliche Tendenz (Abb. 1.10 bis 1.13), allerdings ist die Schwankung der Messwerte um eine Zehnerpotenz größer, so daß die Angabe konkreter Werte wie bei den Amplituden nicht eindeutig möglich ist. Der Einfluß der Nichtlinearitäten ist wesentlich höher, wie die Differenz der Messwerte zu den berechneten ausweist. Es handelt sich jedoch nicht um Meßfehler, sondern um tatsächlich vorhandene Geschwindigkeiten, wie aus der Überlagerung der Orbitalschwingungen der Superharmonischen und der parasitären, freien Wellen gezeigt werden kann.

Die Ergebnisse dieses Forschungsvorhabens zeigen erneut deutlich auf, daß mit der Weiterentwicklung der Meßtechniken und der Analysetechniken viele eingesetzte Praktiker bei der Wellenforschung aufgegeben werden sollten, insbesondere das Zureden von gemessenen Wellenbelastungen an Strukturen zu berechneten Orbitalbewegungen; vielmehr sollten diese im ungestörten Zustand am Ort gemessen werden. Gerade die Tendenz bei der Berücksichtigung aller Wellenanteile, daß die maximale horizontale Geschwindigkeit bei vielen Wellenparametern in realität signifikant größer ist als die prognostizierte, sollte zum Nachdenken anregen.
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