Development of a Manual for the Design of Floating Breakwaters

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DEVELOPMENT OF A MANUAL FOR THE DESIGN OF FLOATING BREAKWATERS

by

Western Canada Hydraulic Laboratories Ltd.

Port Coquitlam, B.C.

Prepared under DSS Contract No. ISZ79-00268

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ABSTRACT

Western Canada Hydraulic Laboratories Ltd. 1981. Development of a manual for the design of floating breakwaters. Can. MS Rep. Fish Aquat. Sci. 1629: 228 p.

The purpose of this study was to collect available information on floating breakwater design and to develop from this information a manual to assist engineers and marina operators. This report provides both a methodology and technical information which will assist the user to assess the applicability, benefits and limitations of using a floating breakwater design for a given site situation. A comparison is made of the relative performance characteristics of different existing breakwater designs and the feasibility of preparing a comprehensive floating breakwater design manual from the current state of knowledge discussed.

RÉSUMÉ

Western Canada Hydraulic Laboratories Ltd. 1981. Development of a manual for the design of floating breakwaters. Can. MS Rep. Fish. Aquat. Sci. 1629: 228 p.

Le but de la présente étude était de recueillir les informations disponibles sur les divers types de brise-lames flottants et de préparer, à partir de cette information, un manuel à l'intention des ingénieurs et des exploitants de ports de plaisance. Le présent rapport fournit une méthodologie et des informations techniques qui aideront le lecteur à évaluer, pour un endroit donné, les possibilités, les avantages et les limites de l'utilisation d'un brise-lames flottant. On compare l'efficacité des différents types actuels de brise-lames et on étudie la possibilité de préparer un manuel détaillé sur les brise-lames flottants à partir des connaissances actuelles.

2.0 SUMMARY

A study has been undertaken to collect as much published and site information as feasible in order to develop a design manual on floating breakwaters for use by engineers and marina operators. The study involved a computer library search, review of published and unpublished information and visits to harbours protected by floating breakwater installations.

It was found that most breakwaters in current use fell into one of the five following categories:

i. centreboard breakwaters in which a vertical wall reflects back wave energy, Figures 3 and 4;

ii. caisson breakwaters consisting of a box-like structure which transforms wave energy from primary to secondary wave trains, Figure 5;

iii. pontoon breakwaters consisting of several units fastened together to form a hollow-centre caisson, Figure 6;

iv. floating tire breakwaters which dissipate wave energy primarily by turbulence, Figure 7.

v. log bundles or moored ship's hulls.

Of these five main concepts, caisson breakwaters most effectively reduce wave heights for a given width of breakwater. Rubber tire breakwaters are generally the cheapest type of breakwater to construct but are neither as maintenance free nor as long-lived as caisson breakwaters. Depending on the local availability of materials, labour and equipment, any one of the main design concepts may be cheaper at a given site than any of the other concepts.

The selection of a suitable floating breakwater design for a given location should be based on the required reduction in wave heights, cost and longevity of the structure, secondary usage of the breakwater, aesthetic and navigational considerations and risk of damage to the breakwater by storms and ice.

Prior to designing a floating breakwater for a specific site, the site wave climate must be established on a recurrence frequency basis, such as exemplified by Figure 16. For the relatively short fetches facing normal floating breakwater installations, the relationships between wave height and significant period shown in Figures 17 and 18 may be used to establish the return frequency of significant wave periods from the wave height curve. Application of criteria for acceptable wave heights in a harbour, such as recommended in Table II, to the wave period recurrence curve will give the performance characteristics, typified in Figure 20, required from any breakwater suitable for the site.

Methods and guidelines for preliminary design of suitable floating breakwaters are given in Section 8. The guidelines are based on observation of existing installations, Appendix A, and on results of previous model studies, Appendix C. There is at present insufficient knowledge of floating breakwaters available for development of a procedures manual which will produce reliable final floating breakwater designs. Detail designs of significant floating breakwaters should be the subject of model studies to economically optimize the structure and to determine mooring forces.

3.0 CONCLUSIONS

3.1 General

The following general conclusions regarding development of a design manual for floating breakwaters have been drawn from this study:

i. There is insufficient information available at the present time to develop a complete design manual for floating breakwaters. Guidelines on the selection of the most suitable breakwater type for a given location and methods to establish preliminary design dimensions have been developed and are described in sections 7 and 8.

ii. Results of previous hydraulic model tests on floating breakwaters, carried out for site specific conditions under limited ranges of hydrodynamic variables are not always suitable for extrapolation to general conditions. Further systematic testing programs such as outlined in Section 6.3 are required to develop general relationships for predicting the performance of floating breakwaters.

iii. Due to the lack of available analytical methods, reliable final design dimensions for floating breakwaters may only be determined through physical model studies.

3.2 Breakwater Designs

Conculsions on the performance characteristics of specific floating breakwater designs are as follows:

i. Caisson and rubber tire designs are the most common and most economical types of floating breakwaters used to protect harbours and marinas in North America. However, any of the more commonly used design concepts may be cheaper than other concepts for any given site, depending on availability of supplies, equipment and labour.

ii. Caisson breakwaters appear to be the most cost effective design concept for floating breakwaters with life expectancies exceeding 10 years.

iii. Centreboard A-frame breakwaters are less efficient with respect to breakwater width versus transmission coefficient than are caisson breakwaters but they may offer the additional advantage of deflecting wind above moored vessels or stopping overtopping waves from damaging power lines or fuel conduits.

iv. Addition of a centreboard to a caisson breakwater improves the breakwater's efficiency.

v. Pontoon breakwaters may offer some cost savings over caisson designs if fabrication has to take place at some distance from the harbour site but they are less efficient in reducing waves than are caisson breakwaters of equal width.

vi. Rubber tire breakwaters have been found to be the cheapest type of floating breakwaters to construct and install but they have suffered a high failure rate and have an average life expectancy of less than 10 years.

vii. The pole-tire design is a more efficient design with respect to breakwater width versus transmission coefficient than any of the other rubber tire designs studied.

viii. Present rubber tire breakwater designs may be improved at added cost by increasing the size of moorings and improving their connections.

ix. Most floating breakwater problems occur as a result of mooring and breakwater connection failures.

x. Connections between adjacent caisson breakwater sections using a design similar to that shown in Figure 21 have been found to reduce breakwater damages.

4.0 INTRODUCTION

4.1 Study Requirement

Floating breakwaters have been investigated with increasing frequency in recent years as alternatives to conventional gravity structures for protecting harbours against wave action and for reducing levels of wave agitation to acceptable levels. Floating breakwaters are normally considerably less costly than rubble mound or caisson structures, particularly when the structure must be sited in deep water, where the wave climate consists primarily of short choppy seas or where the soil cannot easily support other forms of structures. At some locations floating breakwaters have been proposed, and built, using inexpensive and readily available materials and uitilizing volunteer labour for construction. Breakwaters made of scrap tires or floating logs are examples.

At the present time a manual for design only of rubber tire breakwaters exists (Bishop, 1980). There are no published guidelines available to assist in selecting and preparing any alternative design of floating breakwater without extensive and time consuming research and investigation. In general, in existing reports, each describes only one type of breakwater, exposed to a limited range of wave conditions, designed for one depth of water, and with one mooring system. A study was required to bring previously developed knowledge of floating breakwaters into one compendium and to summarize this knowledge as far as was economically feasible so that it would be of value in the design stage for new structures. This study was undertaken through DSS Contract No. 1SZ79-00268 to establish current levels of information as the first phase in development of a complete design manual.

This report summarizes a history of floating breakwaters, discusses their hydrodynamic properties, reviews available data of prototype and model performance and presents procedures for design of further floating breakwaters. The report is intended to assist govenment and licensing officials, professional engineers, developers and marina operators or users who are concenred with floating breakwaters. The compendium of existing data is presented in the Appendices.

4.2 Historical Review

Documented references to floating breakwaters go back to a paper in the Civil Engineers and Architects Journal in 1842 entitled "Reids Floating Breakwater". In a 1905 presentation to the Royal Dublin Society, J. Joly suggested that a pontoon with the shape of a ship's hull could be used as a floating breakwater. This breakwater was never built, although a number of floating breakwaters of other designs were built in Europe following Joly's paper. These floating breakwaters were constructed at a number of deep harbours where construction techniques for building more conventional bottom-resting structures were not available. Few details are known about these breakwaters which were subsequently destroyed during storms.

Further experience with floating breakwaters was not recorded until the second world war. In 1941, at Lysekil, Sweden, a 120 m long floating concrete breakwater was built for a small boat harbour. The breakwater was constructed from two 4.5 m deep by 4.5 m wide ballasted concrete pontoons. It has been reported that this breakwater is still performing satisfactorily.

In 1944, artificial harbours were required along the exposed coastline of Normandy to support the Allied invasion forces. The Royal Navy developed the Bombardon floating breakwater to protect the artificial harbours. The Bombardon breakwater was designed to withstand waves with significant heights of 3 m and periods of 5 to 6 seconds. The breakwater had a crucifix cross-section, with overall dimensions of 9 m by 9 m, by 61 m long. The arms of the crosses were 1.5 m thick. A trial section was field tested in Weymouth Bay, England. During field tests, the breakwater reduced an incident significant wave height of 2.5 m at 5.8 sec period by approximately 75%. Along with grounded ships and caissons, the Bombardon breakwaters were used to construct approximately 1.5 km of breakwater at each of two harbours in Normandy. They performed satisfactorily until destroyed by an unusually severe storm with wave periods of 7 to 8 seconds. Very large resonant motions, coinciding with the natural period of the breakwater, were observed which led to a failure of the mooring system.

In 1948, three 20 m long reinforced concrete barges left from World War II, were used to construct a small boat harbour at Elsero, near Bergen, Norway. This breakwater is still performing satisfactorily.

Following the war, interest in floating breakwaters subsided until 1957, when the U.S. Navy Civil Engineering Laboratory (NCEL) began an extensive study of transportable floating breakwaters for use during amphibious landings on open coastlines. The objective was to produce a temporary harbour with wave heights of less than 1.2 m which would be suitable for transferring cargo from ocean going ships onto barges or pontoon bridges.

Many concepts were studied in the laboratory and the results suggested that a floating breakwater constructed from standard Navy barges would be the most suitable structure. The barges were ballasted with sea water so that they floated at an angle sloping towards the waves.

In the 1960's, floating breakwaters were constructed for harbours in Canada, Japan, Norway, United States and Great Britain. Many of these breakwaters were built using existing floating structures such as log bundles, log rafts, old barges or old ships.

Between 1963-1968, Public Works Canada developed the A-frame breakwater and installed these in Ontario and British Columbia. Concrete pontoon breakwaters were developed and built in Japan and in Norway.

In 1971, as part of an on-going study of transportable breakwaters by the NCEL, a literature search identified 106 concepts for mobile breakwaters. Not all of these were floating breakwaters. The majority of the floating breakwater designs had not been field tested.

In the 1970's a large number of floating breakwaters were built to protect marinas and small craft facilities. On the Pacific coast of North America, plastic pontoon and/or concrete caisson breakwaters were built at several public and private marinas. In Alaska, a pontoon breakwater was developed which was prefabricated in modules and shipped by barge to be assembled in remote locations. In Japan, member companies of the Japanese Floating Breakwater Association developed floating breakwaters for use in small craft harbours and for the fish farming industry. These breakwaters were extensively tested in models and in the field.

Floating breakwaters built from used and surplus tires were deployed at a number of locations in the United States, primarily as a result of the development encouraged by the Goodyear Tire Company.

In addition, the NCEL, Scripps Institute of Oceanography and the State of California developed, with the assistance of model and field tests, the tethered float breakwater consisting of an array of partially submerged and taut moored floats for use as a transportable breakwater and for use in the open ocean.

By 1980, between 100 and 200 floating breakwaters had been built around the world. Of these breakwaters, many proved very successful, and provided the required protection to the moored vessels. However, there were also many which had not proven successful either in providing the required protection or in maintaining structural integrity during severe storms.

A bibliography listing published information on floating breakwaters is given at the end of this report. Observations of floating breakwater construction details and performance made at 21 sites in North America, are set out in Appendix A along with a brief summary of overseas installations.

5.0 FLOATING BREAKWATER CHARACTERISTICS

5.1 Hydrodynamics of Floating Breakwaters

5.1.1 Characteristic Methods of Reducing Wave Height

Incident wave energy which is transmitted under or past a floating breakwater produces wave action on the lee side. Measurement of wave heights in the protected lee area provides a direct measure of the breakwater's effectiveness in reducing wave action. The efficiency of a floating breakwater in reducing wave heights results from an interaction between the breakwater, and the incident waves. This interaction occurs in three principal ways:

i. Reflection of Energy

Vertical or inclined reflecting surfaces reflect incident wave energy back out to seaward in such design concepts as centreboard, thin-wall, A-frame, or offset breakwaters. The efficiency of such designs are influenced by the depth and angle from the horizontal of the breakwater face and by the stability of the overall structure. For example, an A-frame breakwater is more efficient in a given wave climate than is a thin-wall breakwater having the same vertical face area due to its increased lateral stability.

The amount of energy which is reflected seawards depends upon the wave height, the draft and freeboard of the structure, and the depth of water. The depth of water relative to breakwater draft is important because the distribution of wave energy throughout the water depth changes significantly as a wave approaches shallow water. Therefore, in shallow water, where the draft of a centreboard breakwater may extend through a major portion of the depth, a significant portion of the incident wave energy can be intercepted by the structure.

Figure 1, illustrating typical distributions of wave energy through depth for similar wave conditions in two different water depths, shows that a breakwater face of given draft D will intercept a smaller proportion of the incident wave energy in deep water than if it is placed in shallow water.

ii. Transformation of Energy into Other Wave Trains

Absorption of incident wave energy by the structure and subsequent transformation of the energy back into the water in other wave forms may be achieved through motion induced into the breakwater by the passage of waves. Large displacement caissons or slabs such as the Harris floating breakwater or the zig zag "torsion path" breakwater are most suitable for reducing incident wave heights through this characteristic principle as their large mass enables them to take up more energy from a given wave climate than could a lighter structure. The absorbed energy is transmitted back to the water primarily in the forms of secondary out-of-phase wave trains. The effect produces highest breakwater efficiencies when the secondary wave train is out of phase or at different wave period from the incident waves. The elimination of wave energy by this type of floating breakwater is influenced by its mass, by its moments of inertia, by its relative change in buoyancy during the passage of waves and by the relative width of the structure with respect to the local wave climate.

iii. Dissipation of Energy

Dissipation of wave energy through conversion into heat, sound and turbulence is achieved in the breaking of waves on sloping surfaces or against structural members of the breakwater. The amount of energy dissipated by a floating breakwater is governed primarily by the geometry and mooring restraints of the structure. This has led to the design of waffle-type breakwaters, the wave maze, and the proposal of floating or hinged beach breakwaters.

5.1.2 Breakwater Motions

When a floating breakwater is exposed to wave action it responds with a complex resultant of the six degrees of motion. The six degrees of movement are defined as heave, roll, sway, yaw, pitch and surge. Heave, sway and surge are horizontal and vertical displacements of the floating body. Roll, yaw, and pitch are rotations about the axis of the breakwater. Each movement is illustrated in Figure 2.

The response of the breakwater depends upon the natural periods of the moored breakwater for each motion, and on the incident wave period. The natural periods of the breakwater are determined by the physical characteristics of the breakwater and its moorings. These include:

- breakwater geometry
- breakwater mass
- distribution of mass and conseuquent moments of inertia of the breakwater
- characteristics of the mooring system, such as length of mooring lines, unit weight of lines, elasticity of lines and diameter of lines
- pre-tension in the mooring lines,

The phase relationship between the incident waves and secondary waves generated by breakwater motion depends upon both the period of the incident waves and the natural period of the breakwater. Each degree of breakwater motion has its own natural period which depends upon the hydrodynamic characteristics of the floating breakwater and on its mooring system about the relevant axis.

The motion of the breakwater when the wave direction is perpendicular to the long axis of the breakwater is the most simple case. In these conditions the motion consists ideally of heave, roll and sway. It is this combination of motion which is most often modelled in a physical or numerical model of a floating breakwater. The motion depends on the height and period of the waves, the physical characteristics of the breakwater, and restraints from the mooring system and adjacent breakwater units. When the wave direction is at an oblique angle to the breakwater, the motion is more complex than when the waves are perpendicular to the breakwater. The motions then are the resultant of all six degrees of motion.

Each motion of the floating breakwater generates waves which radiate outwards with relative magnitude and phase relationships dependant upon the dynamic response characteristics of the breakwater. Sway is the predominant breakwater motion for incident wave periods of approximately 2 seconds and less. Secondary waves generated by this motion constitute the major component of the transmitted wave. For incident wave periods between approximately 2 and 4 seconds, secondary waves are generated by the roll, heave, and sway motions of the breakwater, its mooring system and the incident wave height and period.

For incident wave periods much greater than 4 seconds, waves generated behind the breakwater by the breakwater motions are not generally significant in relation to the residual wave heights resulting from wave energy which has been transmitted under or past the breakwater.

6.0 COMPILATION OF KNOWLEDGE

6.1 Prototype Installations

6.1.1 Review of Design Concepts

An extensive literature review was conducted during this study to document and compare the performance records of actual breakwater installations and design concepts. Site visits were made to several floating breakwater installations in North America to evaluate and compare the performance of various designs. Notes were made during the visits describing the facility, use, wave climate, performance and/or maintenance experience and, where available, cost per unit length. The collected data is given in Appendix A and summarized in Table 1.

There are numerous floating breakwaters in existence today. Although many designs are hybrids, the majority fit into one of the following conceptual categories:

a. Centreboard breakwaters in which a relatively thin wall hangs vertically downwards to reflect back incident wave energy. This category includes the A-frame centreboard, thin wall breakwaters and caisson centreboard structures.

b. Displacement breakwaters, of which the caisson concept is most common. These structures generally consist of reinforced concrete or steel box structures of varying shapes which reduce wave heights through both reflection and transformation of energy.

c. Pontoon or catamaran breakwaters, which consist of several individual pontoons connected together to form a larger monolithic structure similar to a hollow-centred caisson.

d. Floating tire breakwaters in which wave energy is dissipated by turbulence in passing through the breakwater. This group includes the Wave Maze, Goodyear and Pole-Tire breakwaters.

e. Log breakwaters, including rafts and bundles of logs.

f. Old ships and barges.

g. Inclined slope breakwaters which both dissipate energy by induced wave breaking on the slope and which also act as sloping centreboards to reflect wave energy seawards.

The four most successful and frequently used design concepts for harbour installations not necessarily in respective order, have been:

i. centreboard A-frame breakwaters
ii. caisson breakwaters
iii. pontoon breakwaters
iv. rubber tire breakwaters

These concepts are briefly discussed below with a summary of their major advantages and disadvantages. Approximate cost ranges for the various design concepts have been determined in terms of 1980 dollars by assuming an average 10% inflation rate per year since 1972. Inflation before 1972 was considered at 5%. The generated costs per metre are therefore, approximate and are for comparative purposes only. Actual construction and installation costs for any specific site should be obtained from a contractor prior to preparing detailed project cost estimates.

Names of some companies, institutions and persons connected with design and installation of floating breakwaters in North America are listed in Appendix B. This list is perforce not complete. However, it supplies initial contacts whereby persons wanting information on floating breakwaters may obtain the names of further sources or references.

6.1.2 Centreboard A-frame Breakwaters

The A-frame centreboard design developed by Public Works Canada is typical of most centreboard designs. It consists of a large vertical plate supported by steel trusswork floating on two cylindrical steel pontoons. Four breakwaters of the centreboard A-frame type were constructed, two in B.C. and two in Ontario. The two breakwaters in Ontario are an inverted version of the B.C. breakwaters and are more suitable for use as a dock. An example of each type is shown in Figures 3 and 4.

The performance of the breakwaters has been very satisfactory. The A-frame breakwater at Lund, B.C. built in 1963, is one of the oldest surviving floating breakwaters. Although in need of major refit, it continues to perform satisfactorily.

Problems experienced with the A-frame structures have been:

i. Corrosion of the steel frames. This has been reduced by use of bituminous epoxy coatings and sacrificial anodes.

ii. End damage due to collision with neighbouring modules and breakage of interconnecting chains. This may be reduced or eliminated through use of connections similar to those developed at Tenakee, Alaska.

At Lund B.C., modules are connected with slack chains with tire fenders slung between the ends of the pontoons. The ends of the cylindrical buoyancy pontoons have been damaged as a result of collisions between modules during storms. The damage caused a loss of buoyancy. Styrofoam has been inserted into the pontoons to keep them afloat. Corrosion of the damaged modules has been severe and a second row of pontoons has been added recently to provide the required buoyancy. The breakwater at Lund is exposed to waves generated over fetches which are larger than at most other existing floating breakwaters and the problems resulting from module collision at the other three A-frame breakwater sites have not been as serious.

The Ontario fresh water breakwaters have experienced less corrosion than the B.C. breakwaters located in seawater. Sacrificial anodes and bituminous epoxy coatings have been used to reduce corrosion.

The major disadvantage of the A-frame breakwater is its relatively high cost. Table I indicates that the A-frame breakwaters built in the 1960s would cost approximately \$2,500 - \$3,000/m length in 1980.

6.1.3 Caisson Breakwaters

Caisson breakwaters are one of the most common type of floating breakwaters in use today at major marinas and harbours. The majority are built of prestressed or post-tensioned reinforced concrete.

Steel caissons are extensively used in Japan but few details of their performance have been published. With few exceptions, these breakwaters all serve some secondary function, such as providing a public wharf, a temporary mooring area for transient boats, or as permanent berthing areas. A caisson breakwater at Maple Bay, B.C. has 6 m long finger floats included as an integral part of the construction. Some examples of caisson breakwaters are shown in Figure 5.

Reinforced concrete caisson breakwaters built in Scandanavia in the 1940's and 1960's have performed satisfactorily, although some breakwaters have experienced problems with mooring and connections. Most North American caisson breakwaters were built after 1974 and although they have experienced relatively few problems compared to other breakwater types, it is too soon to completely evaluate their service history.

Six caisson breakwaters were visited during this study and all were reported to give satisfactory performance with respect to wave damping. These breakwaters cost more to construct and install than did the rubber tire design, Table I, but experienced relatively little damage and none were known to have shifted anchors.

Problems encountered with caisson breakwaters were:

i. Damage to caisson ends when the breakwater modules were not rigidly connected together. This damage occurred most often for breakwaters which were interconnected with slack chains and had tire fenders slung between the breakwater sections. This was the most common and serious of the problems experienced with this type of breakwater.

ii. Fatigue failure of the connections when the breakwaters are rigidly connected. No disastrous failures of this type have occurred. Maintenance of connectors is an ongoing task at several breakwaters.

iii. Loss of buoyancy. A few caisson breakwaters have sunk after puncturing of the caisson walls without any reserve buoyancy available.

Caisson breakwaters should be constructed with positive flotation material; i.e. the central cavity should be filled with light foam to prevent the caisson sinking in the event of fracture or leakage. Precautions should also be taken through design of semi-rigid connections between modules to prevent damage to caisson ends by collision with other caissons during storms. A properly constructed caisson breakwater filled with foam or other positive displacement material should have a life expectancy of from 10 to 20 years.

6.1.4 Pontoon Breakwaters

These breakwaters generally consist of several reinforced concrete pontoons connected together in some fashion to form a larger monolithic structure similar to a hollow-centred caisson. Examples are the reinforced concrete breakwaters at Sitka, Tenakee, and Ketchican in Alaska, Figure 6. The designs have been developed to permit easy and economic assembly of a large structure at a site remote from the point of fabrication. The breakwaters can be disassembled and are more easily transported than normal caisson designs. Timber decking has been used over plastic pontoon floats at Friday Harbour, Washington.

Pontoon breakwaters have generally been reported to perform satisfactorily although no storm wave measurements are yet available. A program of field measurements is being conducted in Alaska. Pontoon breakwaters have experienced the same type of connection failures as caisson breakwaters as well as the following design-associated problems:

i. Fatigue problems with connections between breakwater sections. On the Alaska breakwaters, assembled sections were held together with chains and rubber blocks. Chain links have been worn from constant motion and some rubber blocks lost. At Tenakee, a chain link connecting two sections at a change in the breakwater alignment broke during a storm. The connections at the site were satisfactorily redesigned using rubber ring fenders to hold the pontoons and cushion shock loads coming into the connecting chains.

ii. Fatigue failure of individual pontoons. The polyolefin plastic pontoons used at Friday Harbour had a complex shape and fatigue failure has regularly occurred at points of stress concentration. Replacement pontoons constructed from a new polymer have been more successful but also more expensive.

iii. Structural Failure. A concrete pontoon breakwater which was post-tensioned together in Scotland failed when cracks in the structure developed during a storm which exceeded the design conditions. A similar failure occurred with a pontoon breakwater in Norway.

iv. Assembly. The pontoon breakwaters have generally been assembled while floating in the water. During the assembly, difficulty has been experienced stringing the post-tensioning rods or cables through the ducts. The mass of individual pontoons varied by a small amount and the ducts did not line up.

Total construction, transportation, assembly and mooring costs for pontoon breakwaters are higher than for other types of breakwater. An approximate cost range for pontoon breakwaters is $2,300 to $4,500/m.

6.1.5 Rubber Tire Breakwaters

There are numerous designs of floating breakwaters utilizing the energy dissipative characteristics and frequent low cost availability of scrap rubber tires. Of these, the following three designs have been publicized and have been installed at one or more locations.

i. The Wave Maze breakwater - a patented design
ii. The Goodyear breakwater - no patents held
iii. The Pole-tire breakwater - no patents held

The Wave Maze breakwater is one of the earliest rubber tire breakwater designs proposed. The design consists of used or surplus tires bolted together with galvanized bolts and reinforced backing pads into a patented symmetric pattern. The wave maze design has experienced field failures and has been found to cost more than other rubber tire designs for the same degree of wave protection.A wave maze design tested by the U.S. Corps of Engineers at Pickering Beach appeared to fail due to overstressing of both tires and connecting bolts during a storm through which a nearby Goodyear Module breakwater survived. Failure appeared to be due to bolts pulling through the tire casing. Breakwater sections with strengthened tire casings did not fail. The wave maze breakwaters tested cost about 30% to 40% more per metre length than did Goodyear

Module breakwaters tested with comparable width. Following the severe storm damage, the U.S. Corps of Engineers discontinued tests on the wave maze.

Another breakwater of the wave maze design was installed for pier 39, San Francisco, in 1978. The breakwater did not appear to prove satisfactory in either performance or maintenance costs.

The Goodyear floating tire breakwater, Figure 7, was developed in 1974, primarily through the efforts of the Goodyear Tire Company and the University of Rhode Island. The Goodyear breakwater is formed of modules containing 18 tires each, bound together with flexible belting. It can be assembled by unskilled labour without heavy equipment.

It is estimated that more than 100 of these breakwaters have been built. As these breakwaters were built, problems with the design became apparent and many breakwaters sank, broke up, or were abandoned. Adequate low-cost wave protection has been obtained using 3 module wide Goodyear breakwaters at Catumet and Newington harbours. The breakwater at Catumet prevented extensive marina damage during 1.83 m high waves. Following repairs to the initial construction, the breakwater has provided satisfactory protection to its marina. A 2-module wide breakwater at Diversey Harbour was found to be of insufficient width to provide adequate harbour protection.

Some of the problems which developed with rubber tire breakwaters and the solutions which were proposed are as follows:

a. The material which bound tires together tended to deteriorate with time. Rubber conveyor belting has since been used with considerable success.

b. Bolts used to fasten the conveyor belting tended to pull through the belt, corrode, or if nylon bolts were used, deteriorate from ultra-violet radiation. The present solution is to use black nylon bolts which is thought to alleviate this problem.

c. Tires which have air trapped in the crown of the tire when first immersed in water tend to lose that air and sink from the combined weight of sediments that accumulate in the bottom of the tire and marine growth. It is now recommended that foam be cast in place in the crest of the tires.

d. The breakwaters have had a tendency to drag their anchors and break up on the shore during storms. Full scale model tests have been conducted to measure mooring loads and permit correct design of a mooring system.

The above remedial measures have not fully solved all the problems of the Goodyear breakwaters. There are examples of this design which have incorporated all of the above features, yet have failed for the same reasons. Foam which was cast in place in the crest of tires at Pickering Beach was worn by the constant flexing of the breakwater and the worn pieces escaped from the tires resulting in a loss of the positive reserve buoyancy. Connections using nylon bolts and conveyor belting failed on a Goodyear breakwater in Vancouver harbour in British Columbia. Many Goodyear floating tire breakwaters have been built, largely because of their low cost and relative ease of construction. These factors may play an important role in their largely unsuccessful past; the low cost discourages sufficient engineering effort and/or anchoring design, which can form a substantial part of the cost, and its relative ease of construction encourages the use of volunteer labour. In many cases, the supervision of quality control has been low and once problems occur it is often not possible to isolate the problem and undertake remedial methods.

Many instances have been reported of rubber tire breakwaters shifting their anchors during storms. Low initial costs of these structures may partially result from lack of proper design and/or from insufficiently heavy anchors being used. It is false economy not to spend money to properly design and construct a breakwater.

Rubber tire breakwaters are particularly susceptible to structural damage by ice. In situations where there is likely to be ice movement in the area of the breakwater, it is recommended that the breakwater be removed from its location each autumn and stored in a more protected area until spring.

The pole-tire breakwater design consists of rubber tires being strung side-by-side along lengths of belting strung between parallel floating poles. This design is a recent development and only one such installation at Mamoroneck, N.Y., was visited. The breakwater was constructed in 1980 at an approximate total cost of $700/m. No performance or maintenance data is as yet available for this design. However, the design has been extensively model tested and was shown to be more efficient for a given size than either the Goodyear or Wave Maze breakwater.

Table I shows that construction costs for the rubber tire breakwaters visited were generally below \$1,000/m (1980) including anchorage. A Design and Construction Manual for Floating Tire Breakwaters by the National Water Research Institute (1980) indicates that the pole-tire design was found to require less width to achieve the same wave height reduction as a Goodyear module breakwater but that the Goodyear breakwater cost less to construct per metre of length. No conclusions were drawn as to the relative maintenance costs or service lives of the two designs. Construction and installation costs of between \$350 to \$720/m length were estimated for a properly designed breakwater with a life expectancy up to 10 years in that report. Any persons planning to develop rubber tire breakwaters are recommended to initially consult that report (Bishop, 1980).

6.1.6 Log Bundle and Log Raft Breakwaters

Log bundles and rafts are used extensively in British Columbia and have been used in the past in Washington, Alaska and northern Ontario.

These breakwaters may be constructed using either logs or timber piles. Log breakwaters are basically a specialized class of caisson breakwater in which the design is modified from the normal angular cross-section to a more rounded one due to the nature of the readily available construction material, Figure 8A.

Transmission characteristics of log bundle breakwaters have not been determined in the field and only very limited physical model test results are available. The relatively narrow width to which these breakwaters can be constructed limits their satisfactory use to very short choppy seas such as are generated over short fetches of 1.5 km or less.

Log breakwaters have problems with loss of buoyancy in the logs. The length of service depends on water quality, type of log, and whether a wood preservative has been used. Some logs have become water logged as early as 6 months after installation, while other locations have lasted for up to 15 years. The cost of wood preservatives can make a log bundle breakwater uneconomical.

6.1.7 Ships and Barges

Permanently moored ships and barges as have been used as floating breakwaters at Powell River, B.C., Figure 8B, are basically another form of caisson breakwater in which readily available units of unique design were used. No prototype data is available on specific performance which would set these designs apart from others in the displacement category.

6.1.8 Tethered Float Breakwater

The tethered float breakwater, which consists of an array of taut moored, partly submerged floats, was developed jointly by the NCEL Scripps Institute of Oceanography and the State of California. The breakwater was developed for ocean use as a transportable breakwater for military needs. Two test field installations have been monitored off the California coast and both installations have experienced problems with fatigue failure of the float tethers and their connections. Large amounts of ballast are required to moor the breakwater, presenting a considerable problem in their design and construction.

A tethered float breakwater was installed at Seabeck Bay, Wa., in 1979. However, the submergence of the installed tethered floats was much less than required by the design specifications. Severe wave conditions have been experienced behind the breakwater during at least two storms since the breakwater was built although the floats were reported to have reduced damage to docks in its lee.

Tethered float breakwaters are not considered further in this report for the following reasons:

a) Solutions to the problem of fatigue failure of the tethers and their connections have not yet been proven in field conditions.

b) The cost of the breakwater and difficulty of mooring the floats, particularly in areas where large changes in water elevation occur, is too great for any likely application of this concept to a marina or small craft harbour.

6.1.9 Inclined Float

An inclined float breakwater is currently being developed by the U.S. Navy, NCEL, and U.S. Army Corps of Engineers, Figure 9. This breakwater rests with one edge along the seabottom and the other end floating at sea level but restrained by mooring lines. The structure, which combines the effect of a full-depth centre board with the stability of a wide caisson is being designed to withstand waves with over 7 second incident peak period. The structure will be field tested in 1981.

6.1.10 Mooring Forces

Failure of moorings and connections have been the most frequent problems with floating breakwaters. This factor is of great importance as movement of anchors during storms threatens the safety both of the breakwater and of vessels moored in the harbour.

Rubber tire breakwaters appear to have had a very much higher incidence of anchor movement under wave or strong current action than do the other main design concepts. Anchoring details given in Table I indicated that the anchor weight per metre breakwater length was usually very much less for the rubber tire designs than for other concepts.

The caisson, pontoon and A-frame designs investigated were all anchored either by concrete blocks weighing over 1000 kg/m length of breakwater or by stub piles. None of these breakwaters are known to have had any problems with shifting anchors.

Anchor blocks should be carefully designed with regard to wave climate, currents, dimensions of structure, bottom material and strength of connections. Concrete blocks may be reinforced and should be attached to the breakwater by heavy chain. The 25 mm diameter chain used to moor the Lund A-frame breakwater lasted 15 years before replacement. Lighter chain down to 9.5 mm diameter has been used at other installations but no information is available on its life expectancy.

Normal economic breakwater design is to use less anchoring weights on the leeward side of the breakwater than on the windward side. Crossing anchor chains under the breakwater, as opposed to taking them out to each side, reduces the overall area required to moor the breakwater and has been used for some installations. There is insufficient data to judge whether this anchoring technique signficantly affects breakwater performance.

Use of stub piles cut off shortly above the seabed for anchoring and use of dolphins extending above the water line have both given satisfactory mooring results.

6.2 Physical Model Studies of Floating Breakwaters

6.2.1 General

Physical model studies of many floating breakwater designs have been undertaken to determine breakwater effectiveness and, in some cases, mooring forces. The results of model studies into the following types of floating breakwaters, which have been considered of sufficient value by others that they are at present being used in at least one harbour or marina, are discussed below:

i. A-frame breakwaters;
ii. Caisson breakwaters;
iii. Pontoon breakwaters;
iv. Goodyear floating tire breakwaters;
v. Pole-tire breakwaters.

The model study results, along with test conditions, breakwater dimensions and model scales have been taken from published information, related to a common format and are shown in Appendix C.

Figures C1 to C38, show the study results in the format of the transmission coefficient, C_t, plotted against test wave period.

$C_t = H_t/H_i$

where:

C_t = transmission coefficient
H_t = transmitted wave height
H_i = incident wave height

The breakwater efficiency, describing the percentage reduction of incident wave height, is related to the transmission coefficient by $E_{ff} = 1 - C_t$

The published model study results usually compared the breakwater performance to dimensionless ratios such as L/B or B/L

where:

B = the beam or width of the breakwater, measured in the direction of the incident wave

L = the wave length of the incident wave at the breakwater.

In some cases, data was presented as a function of wave steepness, expressed as the ratio H/L or the relative draft D/d.

H = incident wave height

D = draft of the breakwater

d = depth of water at the breakwater

6.2.2 Comparison of Derived Dimensions

Dimensionless expressions such as L/B, H/L and D/d are useful tools for defining complex phenomena which are valid for the range of values tested for each variable. In most model studies, a single breakwater design was tested over a range of wave periods between 1.5 and 4 seconds at only 1 or 2 water depths. The model study results are therefore limited to scaled up combinations of those conditions.

The response of a floating breakwater depends on its mass, distribution of mass, geometry, and mooring system constraints. Use of the dimensionless parameter B/L to summarize the effect of the above factors on breakwater efficiency assumes simplistically that the physical and dynamic characteristics of the breakwater are directly proportional to the width B and that wave height and steepness have negligible effect on breakwater efficiency. These widely used assumptions should only be used for a preliminary assessment of breakwater performance or dimensions. The comparative results of model investigations into transmission coefficients for caisson, centreboard or rubber tire breakwaters have been shown in Figures 10, 11, and 12 respectively. For clarity, only individual study results from the most efficient designs tested are shown in Figures 10 to 12. Detailed test conditions and results from all model studies considered in deriving Figures 10 to 12 are shown in Appendix C.

The wide spread of results shown in Figures 10 and 11 for the caisson and centreboard breakwaters results from variations in water depth, breakwater draft and hydrodynamic characteristics between the breakwaters. The wide envelope spread for rubber tire breakwaters in Figure 12 was largely due to differences in tire arrangements.

Generally, model test results plotted near the top of the envelope in Figure 12 were determined with Goodyear module breakwaters whereas results at the bottom, more efficient part of the envelope were obtained in tests of the pole-tire breakwater arrangement.

The value of establishing breakwater designs on the basis of B/L values determined from earlier model studies was investigated by scaling up results from several model studies to develop breakwater designs suitable for reducing by 67% the height of 2, 3 and 4 second period incident waves. Figure 13 shows the variation in comparative breakwater widths calculated as suitable for 3 second period waves. The variation in sizes was even greater for 2 and 4 second wave periods.

The wide variation in dimensions illustrated in Figure 13 for similar types of breakwaters having similar efficiencies showed that the collected previous model study results produced no reliable agreement on the size of a structure required to achieve a given efficiency.

Results from A-frame, caisson and rubber tire model studies by one source were scaled to predict performance of other model breakwaters tested in other studies, Figure 14. In some instances, good agreement was found between comparative studies. However Figures 14A and 14B show that the results were inconsistent and that good agreement could not be relied upon.

Reasons for the unsatisfactory use of results from one model study to predict performance in other studies or to establish comparable design dimensions with other studies may be as follows:

i. The model tests were conducted at various model scales, some of which included considerable scale effects. It appears from a review of all existing data that scales of less than 1:10 may include excessive viscous or elastic scale effects. Dissipation of energy on a floating breakwater by either wave breaking or flow separation and damping effects of the mooring system may be overestimated at scales of less than 1:10.

ii. The dynamic response of individual floating breakwaters is unique to each structure and varies considerably for small changes in the breakwater's mass, geometry and distribution of mass. Dynamic response of the structure is of major importance in establishing wave transmissibility.

iii. Different mooring systems have been used for each set of model studies. Mooring restraints affect the dynamic response of the structure. Some parameters which were varied between tests were elasticity of the mooring system, point of attachment to the floating breakwater, scope and pretension in the mooring line and unit weight and diameter of the mooring line.

iv. Different values of wave steepness have been used in various model tests. The effect of wave steepness on breakwater performance is not clearly shown by an analysis of test results. However, it appears that rigid structures such as caissons are less efficient with steep waves than with less steep waves, at wave periods around the natural period of the structure.

All of the existing model results have been determined with regular waves approaching perpendicularly to the breakwater. Yamamoto and Yoshida (1980) found that floating breakwater performance was similar in regular and irregular waves whereas Ouellet and Morin (1975) found that the results were not the same. It is not clearly established that the dynamic response of a floating breakwater predicted from tests using regular waves is similar to the dynamic response of the same breakwater in irregular waves.

6.2.3 Comparative Transmissibility of Design Concepts

The widths of performance envelopes in Figures 10 to 12 shows the range of performance characteristics determined in various model studies for the three design concepts. These curves are based on the simplified assumption that breakwater hydrodynamic characteristics may be considered proportional to its breadth B. Performance of a well designed breakwater will be found towards the lower and right hand sides of the envelopes. Performances of less well designed breakwaters will be found closer to the top curve of the envelope.

Comparison of Figures 10 to 12 shows that although considerable overlap of the envelopes occurs, a generalized conclusion may be drawn that well designed caisson breakwaters can have lower transmission coefficients for longer period waves than are normally achieved with well-designed A-frames or rubber tire breakwaters of similar width.

A study for design of a breakwater at Nakusp marina (Western Canada Hydraulic Laboratories Ltd. (WCHL) 1979) showed that on the basis of comparative width, a hybrid centreboard breakwater supported by bolted caissons on either side produced lower transmissibility coefficients than the caissons without the centreboard. However, results from that study plotted on Figures 10 and 11 indicate that other more conventional caisson or A-frame centreboard designs of similar widths may be more efficient in reducing wave heights than the hybrid design.

6.2.4 Comparisons of Concepts at Similar Relative Depth

The transmission coefficient of a breakwater is known to vary with the relative depth, D/d, to which it extends, Figure 1. Results from model studies on approximately similar width caisson, A-frame and rubber tire breakwaters which had been tested at approximately the same relative water depths are compared in Figure 15. The caisson breakwater design tested by WCHL at D/d = 0.14 was found to be considerably more effective than a slightly wider pole-tire breakwater, Figure 15A. Caissons tested by Nece and Davidson were more effective than wider A-frame designs tested by WCHL and by Ofuya, Figure 15B. The pole-tire breakwater was more effective under most wave conditions than the A-frame design, Figure 15C, but the A-frame was found to be more effective than the Goodyear module design of approximately equal width.

The above comparisons support the generalized conclusion found in section 6.2.3, that in general, greater wave height reductions may be achieved by caisson design breakwaters than by either A-frame or rubber tire breakwaters of equal width.

6.2.5 Comparative Mooring Forces

Few model studies of mooring forces for each type of floating breakwater have been carried out. These have shown that mooring forces for a floating breakwater vary considerably with the dynamic response of the breakwater to waves, the elongation characteristics of the mooring system and the wave height and period of the incident waves.

Mooring forces have been reported in the literature as dimensionless ratios of mooring force and breakwater displacement, $\frac{F}{w\,B\,D}$, or of mooring force and incident wave energy, $\frac{F}{w\,H^2 L}$,

where:

F = mooring force

H = wave height

L = wave length

B = breakwater width

D = breakwater draft

w = unit weight of water

Results of three studies into caisson mooring forces (Beebe, 1966; Davidson, 1971; WCHL, 1966) were compared to determine if these dimensionless expressions for one breakwater were applicable to another breakwater of the same type. Comparison of the results of these studies showed wide scatter between data points for seemingly similar conditions. Forces determined in one study varied from those determined for comparable conditions in other studies by factors of up to 2 to 3. Mooring forces were found to be very sensitive to wave steepness.

Each of the existing studies were limited in one or more of the following ways:

i. The effect on mooring forces of variations in the breakwater's overall dimensions, mass or distribution of mass were not investigated.

ii. The elasticity of the mooring system was not modelled correctly. Usually a linear relationship between breakwater displacement and mooring force was assumed whereas actually the system was highly non-linear. A study at NCEL has shown that mooring forces vary considerably with mooring elasticity.

iii. In most cases the initial tension in the mooring system was not specified. Mooring forces vary considerably with the initial tension in the mooring line due to the non-linear elasticity of the system. Mooring line tension varies widely depending on the range of water level changes and the magnitude of current or wind loadings.

It was concluded that existing results of model studies of mooring forces could not be applied to other conditions of breakwater, mooring system and wave conditions than those tested.

6.3 Further Studies Required

Results of Section 6.2 indicate that previous hydraulic model tests have, in general, been carried out for site specific conditions under limited ranges of hydrodynamic variables. Therefore, further systematic laboratory testing of floating breakwater designs will have to be made in order to develop general relationships that can be used to predict the performance characteristics of prototype installations. Further theoretical work is also required to develop model scaling relations that will ensure dynamic similarity between model and prototype breakwaters. Ideally, development of new scaling criteria should relate the effects of wave heights, periods, steepness, angle of incidence and depth of water at the structure to the breakwater characteristics of width, depth, mass, radius of gyration, moment of inertia and vertical surface area to establish breakwater efficiencies. Finally, breakwater performance relationships derived from model studies should be verified by independant tests to confirm the validity of the scaling relations that were used. Such tests should include collecting field performance measurements at prototype breakwater installations.

6.4 Numerical Models

Numerical models have been developed for use in the design of floating breakwaters. The models use procedures developed to analyze ship motion problems which have been extended to include the restraining effects of the moorings.

The models may be used to determine floating body motions, coefficients of transmission and forces in the mooring system for given incident wave conditions. Numerical models which use the body motions as input have also been developed for the dynamic analysis of structural loads.

The development of numerical models dates back to the 1950's when the linear equations of motions of a floating body and the boundary conditions for their solution were first formulated. The earliest models were only valid for very simple shapes such as triangular or circular cross-sections and only for freely floating bodies. Recently, significant advances have been made both in numerical techniques for solving the governing equations of motion and in the ability to formulate complex body geometries and mooring systems. Procedures to describe the loss of energy by dissipation on the structure are still not available, and existing models are still only valid for simple structures, such as caisson or pontoon breakwaters. It is not clear that the performance of an A-frame breakwater could be represented in an existing numerical model due both to its complex geometry and the role that energy dissipation plays in its performance characteristics.

Three numerical models recently developed to describe the performance of floating breakwaters at the Universities of Washington, British Columbia and Oregon State are described in detail in the following reports: University of Washington, Adee, Richey and Christensen (1976), University of British Columbia, Fraser (1979) and Isaacson and Fraser (1979)., University of Oregon, Yamamoto and Yoshida (1978). Prepared software for these programs is not available. The numerical models are a developing analytical tool and a specialist in numerical modelling is required to properly calibrate, operate and interpret the results of these models.

All of the numerical models have limitations. In general, they are only valid for regular waves of low wave height. They do not model dissipation of energy by wave breaking, turbulence, flow separation or transformation of energy into work done by the structure. All the numerical models depend upon empirical coefficients to describe the added mass and the damping effects of the breakwater body and the mooring system.

The numerical model from the University of British Columbia differs in an important way from either of the other two numerical models. In this model, the non-linear restraining effect of the moorings on the breakwater motions are determined by an iterative solution technique. The non-linear restraining effect does allow for some effect of wave steepness. In the previous models, the motions of the breakwater are determined for a linear restraining effect of the moorings. A linear restraining effect is only valid for taut moored floating bodies which is seldom the case for a floating breakwater.

The motions of the floating bodies determined by the University of British Columbia model have been extensively compared with known solutions and the model represents an exact solution. However, the restraining effect of the moorings have not been compared with analytical solutions, model or field results. This model has been used in the design of a floating breakwater in British Columbia.

The numerical models discussed above are intended to aid in the two-dimensional problem of determining transmission coefficients and mooring forces for a unit length of the breakwater.

Numerical models to determine the dynamic structural loads due to wave forces along the breakwater have also been developed. Details of these models are not available, however, dynamic structural analysis of floating breakwaters in Washington and most recently of the Hood Canal floating bridge in the same state have been undertaken.

Numerical models can be cost-efficient when used correctly and a large number of variables, such as body dimensions, mass, distribution of mass, or mooring may be investigated very quickly. At the present time, the results of a numerical analysis would still have to be confirmed in a physical model to determine effect of wave steepness, irregular wave grouping and extreme wave heights on the performance of the model.

7.0 PREPARATION FOR DESIGN OF FLOATING BREAKWATERS

7.1 General

Following identification of a wave action problem at any existing site or in anticipation of such a problem at a proposed site, the following questions should be answered prior to considering any design for a floating breakwater.

i. Is the problem serious enough to warrant further investigation?

ii. What expenditure on the overall project and on the preliminary studies can be justified? The cost of preliminary studies and proper design should not be unduly restricted by limitations on the overall budget. If the project is to be economically unfeasible, this should be determined at this early study stage.

iii. Are there clear indications that a floating breakwater should be considered? For example:

- Subsurface soils will not support conventional gravity structures.
- Depth of water is such that a conventional gravity structure would be uneconomical.
- Only short term protection is required.

iv. Are there clear indications that a floating breakwater should not be considered? For example:

- The site may be exposed to a large open body of water with fetch distances suitable for frequent generation of waves with 4 to 6 second periods or longer. The exposed fetch limits for floating breakwaters depend on prevailing wind strengths and on the breakwater design concept chosen. Some breakwaters are generally ineffective against wave periods greater than 4 seconds. The width required for an effective floating breakwater becomes very large, and the structure may become uneconomic if waves with periods greater than 4 to 6 seconds occur frequently. Following review of the available information, the Project Manager should decide whether or not to proceed with the project and what resources should be committed to future work. Continual review of the project should take place during the design process, particularly when tentative schemes are developed and cost estimates are produced.

At this stage of the project, areas where insufficient information exists should be identified. Further studies to provide that information should be initiated if the project is to proceed. The following sequence of steps, discussed in the following sections, is recommended.

THE DESIGN PROCEDURE

i. Identification and definition of the problem
ii. Definition of wave criteria for the harbour
iii. Description of the site and the wave climate
iv. Definition of the performance standards for a floating breakwater
v. Preliminary design of a floating breakwater
vi. Comparison with other types of breakwaters
vii. Selection of most suitable type of breakwater
viii. Detailed design
ix. Construction and installation
x. Maintenance

7.2 Identification of the Problem

At this stage of the project, a tentative problem of unacceptable wave conditions in a harbour has been identified. Available information relevant to the problem should be summarized, an understanding of the problem developed and a plan of action defined based on initial information.

Contact should be made with users, operators, and owners (or representative) of the facility. A site visit should be made by the Project Engineer. All available reports describing the site and the local environment should be reviewed. The following points should be assessed to provide a clear picture of the problem.

A. Identification of the Problem.

A1. Is the breakwater required to protect an existing area that has been experiencing wave problems, an area to be developed adjacent to an existing facility, or an area that is to be developed? Describe.

A2. Provide details of the facility as follows:

A.2.1 Description of vessels with typical dimensions, displacements, etc.

A.2.2 Number of vessels using facilities

A.2.3 Type of mooring arrangements

A.2.4 Orientation of berths relative to wave direction

A.2.5 Season of operation in the harbour

A3. Provide details of problems experienced or expected as follows:

A.3.1 Are day to day operations in the harbour restricted?

A.3.2 Is wear and tear of moorings and fenders experienced?

A.3.3 Are vessels damaged in any way?

A.3.4 Are docks damaged in any way?

A.3.5 Have mooring lines failed?

A.3.6 If sailboats use the harbour, have masts of adjacent boats interlocked?

A.3.7 Have any serious accidents related to wave action occurred?

B. Descriptions of the Cause of the Problem

B1. Are problems discussed in A. caused or expected to be caused by wind generated waves?

B.1.1 How frequently do problems occur, or how frequently are complaints received?

B.1.2 What is the estimated significant wave height, or observed wave height, and the wave period that causes the problem?

B.1.3 Where, and from what directions do the waves come that cause the problem?

B.1.4 If wave heights, periods, and directions are unknown, is information on wind speeds, direction, and frequency available? Describe as above.

B.2 Are problems discussed in A. caused or expected to be caused by ship generated waves?

B.2.1 How frequently do problems occur, or how frequently are complaints received?

B.2.2 What is the estimated height and period of the waves?

B.2.3 What is the estimated length, beam, draft, and tonnage of the ship causing the waves?

B.2.4 What is the estimated speed of the ship?

B.2.5 Where, and from what directions, do the ship waves come?

C. Site Conditions

C.1 Provide a general description of the site including the following details:

C.1.1 Depth of water at the probable breakwater location

C.1.2 Soil conditions at the site of the breakwater

C.1.3 Water level variations or tidal range

C.1.4 Tidal currents in the vicinity of the probable breakwater location

C.1.5 Fetch, or distance over the water, where the wind can generate waves that will reach the breakwater

C.1.6 Wind velocity Tables describing the frequency of occurrence of wind velocity recorded at many meteoroligical stations are available from the Atmospheric Environment Service of Environment Canada.

C.1.7 Ice thickness, movement and periods of ice cover should be established

C.1.8 Waves - The existence of any recorded or hindcast wave data for a location close to the site should be established.

C.1.9 Water lot boundaries available for the breakwater location should be determined.

C.1.10 Land for construction and site access. Existing plans should be reviewed to determine the possible availability of land for construction, stockpiling of material, launching of the breakwater, and acces to the site.

C.1.11 Availability and relative cost of suitable local construction materials.

7.3 Wave Criteria for the Harbour

The primary objective of a breakwater is to limit the level of wave agitation experienced in a harbour. The limit to which wave agitation must be reduced depends on the use of the harbour, the types of vessels, their mooring arrangements, and the frequency of occurrence of the critical wave conditions.

A recent study of acceptable wave criteria in small harbours (Northwest Hydraulic Consultants Ltd, 1980) has recommended wave height criteria be given as a function of wave direction, wave period, quality of protection required, and frequency of the event, Table II. The criteria were developed for small craft harbours and were based on a study of pleasure craft.

The events with a return period of 1 and 50 years can be established by standard statistical analysis, providing that sufficient data is available. However, the analysis should be based only on the season during which the marina is being used. The once per week event indicates the maximum level that should normally be anticipated from passing boats or from frequently prevailing winds. The intent of the once per week event is to describe wave conditions that should not interrupt the day to day operations of a marina. It is proposed that the event may be better defined in future by wave conditions that are exceeded for no more than 1% of the time during the marina's operational season.

The criteria contained in Table II were proposed primarily for marinas containing sailing vessels. For fishing harbours where experienced fishermen handle sturdy boats, a significant wave height criterion of 0.3 m for limiting operations and a significant wave height of 0.6 m for the once a year event may be acceptable. At the present time there are no reports available to assist in defining criteria for Canadian fishing harbours. The most acceptable procedure for fishing craft harbours would be to relate the limiting wave heights to previous experience in the harbour, or to a similar harbour.

7.4 Physical Description of Site

Once it has been established that the harbour wave criteria are being exceeded and that some form of wave protection is necessary, the following further information should be collected in order to properly assess the optimum type of breakwater required.

i. Hydrographic Survey

A detailed hydrographic survey will be required of the area where the breakwater is to be located. Typically, distances between soundings may be in the order of 3 m in areas of rapidly changing bathymetry, and in the order of 10 m in areas of flat bathymetry.

The survey data are required for designing the anchor system of a floating breakwater and for designing the underwater details and estimating costs of a comparative conventional breakwater.

ii. Geotechnical Survey

A seabed material survey is required for the anchor design of floating breakwaters. A more complete geotechnical survey is required for conventional breakwaters to determine whether the seabed can support the structure and to assist in analysis of scour or seabed erosion in front of the breakwater.

iii. Ice Survey

In areas that ice forms to a significant thickness, information is required on the dates of freeze-up and thaw, thickness of ice, movement of ice and pressures that may be exerted by ice on a rigid structure.

An important consideration for the design of many coastal and harbour structures, including breakwaters, is the vertical forces that occur when ice has frozen and bonded to a structure and the ice is then moved vertically by changes in water levels.

iv. Access to the Site, Construction Area, Stockpiles, etc.

Suitable access to the site for trucks, heavy equipment, and possibly over-size trailers is required. Construction activities and the stockpiling of construction materials may need large areas of land.

v. Wind Climate

The wind velocity over the water is required to hindcast the wave climate and to calculate drag forces on a floating breakwater. Wind data may be obtained from the Atmospheric Environment Service of Environment Canda.

vi. Water Currents

The current velocities in the vicinity of a floating breakwater location must be determined because of the drag forces produced on the breakwater and its moorings. Currents should also be considered in establishing the breakwater plan layout to allow for vessel navigation and water quality.

vii. Water Level Variations

The elevation of the highest and lowest water levels which will be experienced at the site must be determined. Changes in water level are produced by

- tidal variation
- storm surge
- tsunamis
- harbour or bay resonance
- seiches
- wave set up
- variation in storage levels of lakes or reservoirs

The limiting effect of these factors may best be computed by a coastal engineer.

7.5 Definition of Site Wave Climate

7.5.1 Wind Waves

The site wave climate must be established and compared to the desired criteria to determine whether the problem results from lack of wave protection or from other causes.

The estimation of the wave climate, particularly in shallow water, is generally complex and should be undertaken by an experienced coastal engineer. The shore Protection Manual[2] published by the U.S. Corps of Engineers describes methods for establishing wave climate in non-complex aeas.

The wave climate determined for the breakwater site should include the effects of wave diffraction, refraction, reflection and shoaling as applicable. Wave climates determined from actual field measurement at the site will have had these last factors naturally incorporated into the measurements.

In general, the wave climate at a location should be expressed in terms of the average number of hours per year, or per season, during which wave heights from each direction at 45° spacings around the compass exceed specified values of significant wave height and period. The wave climate should be extended to predict the upper limit of wave heights anticipated to occur at the site during return periods of up to 50 years length.

An estimate of average annual wave conditions may be determined by installing a wave recorder for at least two years. Alternatively, the site wave climate may be calculated using a wind-wave hindcast procedure based on measured wind velocities. High wave conditions with low frequencies of occurrence must be calculated using wave hindcasting procedures unless many years of recorded wave data have been obtained. In some coastal locations the depth of water causes extreme waves to break before reaching a harbour or marina. In this case there is a limit to the maximum wave heights that can occur.

The accumulated site wave information, whether from site measurements or from wind hindcasting procedures with refraction analyses, and including any data on ship waves as discussed in section 7.5.2 should be summarized on a form similar to the following and should be plotted on a wave height recurrence graph similar to Figure 16.

SITE WAVE CLIMATE SUMMARY SHEET

Location of Observations ____________________

Source of Information ____________________

Dates of Observations ____________________

Direction of Wind ____________________

Direction of Waves ____________________

Direction of Swell ____________________

ANNUAL AVERAGE WAVE CLIMATE

Wave Conditions recorded for each of 8 major Compass Points

Direction	____		____		____		____	
% of time wave conditions are exceeded	Hs	Ts	Hs	Ts	Hs	Ts	Hs	Ts
50%	___	___	___	___	___	___	___	___
70%	___	___	___	___	___	___	___	___
90%	___	___	___	___	___	___	___	___

Average values of the maximum significant wave heights occurring during each week and the associated peak wave periods.

EXTREME WAVE CLIMATE

Direction	____	
Return Period	Wave Conditions	
Years	Hs	Ts
1	___ ± **	___ ±
10	___ ± **	___ ±
20	___ ± **	___ ±
50	___ ± **	___ ±

** an estimate of the reliability of the extreme values should be made.

7.5.2 Ship Waves

Waves generated by ships can produce wave agitation problems at adjacent harbours or marinas.

The magnitude of ship waves depends on the vessel size, hull shape and velocity. The most severe waves are not necessarily produced by the largest or fastest ships. Where ship waves are a major disturbing factor in the harbour, site data is best accumulated from regular wave recordings or observations. Records should be kept of wave heights, period, frequency of occurrence and direction of wave incidence. Should ship waves create a problem, this data should be incorporated on a frequency-direction basis with the wind wave data in the Site Wave Climate Summary Sheet and into the wave height recurrence graphs.

7.5.3 Wave Height Recurrence Graph

A typical wave height recurrence graph for an unnamed harbour is shown on Figure 16. This graph was developed by the following steps:

i. The effective fetch for wave generation by winds blowing from various compass points were calculated using procedures set out in the Shore Protection Manual of the U.S. Corps of Engineers.

ii. Hourly wind records over 10 years of observations were analysed to establish the frequency of occurrence of strong winds from each direction.

iii. The wind records were compared with the effective fetches to hindcast deepwater wave heights from each direction on a frequency basis.

iv. A refraction analysis was conducted to establish wave heights and directions in shallow water near shore.

The bottom scale of Figure 16 shows the mathematical percentage of time that waves of a significant height, shown on the left scale, have been hindcast to occur for the directions shown. The scale along the top of the figure indicates the average number of hours per year that corresponds to the frequency of occurrence scale along the bottom.

Figure 16 shows that for this harbour, waves of 2.8 m significant height may be expected to approach the harbour from the northwest for 1 hour in 10 years and of 1.1 m significant height for an average of 10 hrs/yr. Waves from the south would be 0.7 and 0.4 m for these return periods respectively.

7.5.4 Significant Wave Period Recurrence

The efficiency of floating breakwaters varies markedly with the length and period of incident waves. The width required in a floating breakwater to effectively reduce incident waves increases rapidly as the incident wave period increases beyond 3 seconds. Due to width limitations for cost, space or structural reasons, most feasible floating breakwater designs are not very effective against waves longer than 4 seconds period and are of very little use against waves of 6 seconds period. The wave periods associated with the wave height recurrence graph must next be determined in order to assess the width of breakwater required.

Floating breakwaters are usually used in relatively sheltered locations where fetches are too short to permit wind generation of waves with longer significant periods than 4 to 6 secondss. Under these conditions the SMB wave hindcasting technique, normally used in defining wave climates, gives a close relationship between generated significant wave height and period. This relationship is shown by the shaded areas in Figures 17 and 18 for waves generated in deep water and for waves generated in a comparatively shallow water depth of 4.6 m. A significant wave period recurrence graph similar to the one shown in Figure 19 should be prepared for the site based on comparison of the wave height return frequencies determined for the site, Figure 16, with the mean applicable wave height to wave period relationships shown on Figures 17 and 18.

Waves with height to period relationships outside the shaded areas shown in Figures 17 and 19 may occur in prototype as a result of factors such as wave refraction by irregular seabed topography, or through swell arriving from outside the normal fetch area. In such instances, engineering advice should be obtained prior to implementation of protective works.

8.0 DESIGN OF A FLOATING BREAKWATER

8.1 General

The results of investigations into prototype experience and model studies of floating breakwaters, section 6.0, showed that no reliable formulae can at present be determined for designing a floating breakwater of known, preplanned characteristics. However, guidelines can be drawn up to enable prospective floating breakwater developers to select the most suitable design concepts for their purposes and to establish dimensions with sufficient accuracy to enable preliminary cost estimates to be prepared.

Section 8.2 sets out the factors to be considered in selecting the most suitable floating breakwater design concept for the given situation. Section 8.3 describes how to determine the approximate required width and design for various concepts of breakwater. Physical model studies must then be undertaken to enable the designer to economically develop any suitable floating breakwater design. The wave transmission characteristics found for breakwater designs in model studies will be representative of situations with very long breakwaters where neither wave diffraction nor reflection occurs into the protected area behind the breakwater In reality some waves will diffract around the ends of the breakwater into the area to be protected. This may be accounted for either by reducing transmission coefficients allowed in the model studies or by altering the breakwater layout. The effects of reflected or refracted waves in the berthing area should be minimized by appropriate harbour design.

8.2 Factors Affecting Choice of Design

8.2.1 General

The most important factors in selecting the characteristic design principle for a given breakwater installation are discussed below. Local considerations will govern their order of relative importance at any specific site.

8.2.2 Use of Structure

The choice of breakwater concept is frequently governed by required secondary use of the structure. For example:

Flat-topped caisson breakwaters can also function as mooring floats or as access ways for pedestrian traffic;

A-frame breakwater centreboards may be extended vertically upwards to act as wind breaks creating sheltered areas or to prevent overtopping of large waves onto electrical or fuel conduits;

Tethered float or submerged slope breakwaters were developed to float in a submerged or semi-submerged state, leaving a visually clear and aesthetically pleasing outlook from the protected area.

Major and possible secondary uses for the breakwater should be thoroughly considered prior to selecting the design concept as secondary usage may justify added expense over that required for minimum wave protection.

8.2.3 Cost of Structure

A breakwater is normally expected to be an economically viable investment. The full assessment of breakwater costs must include planning, design, construction, transport, installation and mooring costs plus maintenance. These costs must be weighed against return on investment from moorings, shore protection and potential damage and liabilities for damages due to inadequate design or construction. In general, caisson breakwaters have provided the most effective and long lasting protection for an initially higher capital cost. This may be compensated for later by reduced maintenance and renewal costs. Rubber tire breakwaters have generally involved the lowest initial costs but have required frequent repairs. However, the complexity of construction techniques, locally available inexpensive materials or volunteer labour can make any of the common breakwater designs more economically atrractive than any of the others at a specific site.

8.2.4 Navigation and Physical Considerations

Limitations are often imposed on the physical size of the breakwater system by local topography or water lot dimensions. Such situations may arise where

i. structural width limitations preclude installation of very wide multi-module rubber tire breakwaters which would be more economical than equally efficient but narrower centreboard or caisson designs,

ii. anchoring forces in exposed deep water preclude use of centreboard designs having high windage or surface area;

iii. a need to maintain sufficient manoeuvering space for moving vessels requires a narrow deep breakwater be selected in place of a wider design. The ability to pass drifting garbage should also be considered in selecting between flow-through caisson, solid caisson or a less permeable centreboard design.

8.2.5 Exposure

Rubber tire breakwaters should not be left in exposed locations during periods of ice formation as fractured ice has been found to seriously slash both tire casings and flexible bindings. It is common practice to relocate rubber tire breakwaters from exposed to sheltered areas during winter months.

It is not possible to conclusively advise at this time on the design concept which would induce lowest anchoring forces in comparable storm conditions. However, the frequent history of shifting anchors and abraided connections in rubber tire breakwaters indicates that these designs may not be suitable for exposed locations unless significant improvements are made in design.

8.2.6 General Guidelines

The following general guidelines developed from Section 6 of this report are presented to assist in selecting the adopted breakwater concept for those situations where consideration of the above features has not indicated any one particular concept to be most suitable.

i. caisson breakwaters generally produce the lowest transmission coefficients for a given width of breakwater

ii. rubber tire breakwaters are generally cheaper to construct than caisson breakwaters but do not have as long an average life expectancy. They may be constructed by volunteer labour with freely available components

iii. centreboard A-frame breakwaters are generally more expensive to construct and are less efficient for a given width than caisson breakwaters

iv. pontoon breakwaters, have been found to be more expensive to construct and install than comparable caisson breakwaters. They may however be considerably more economical than caisson breakwaters for sites remote from the place of fabrication.

8.3 Preliminary Dimensions

8.3.1 Reduction of Wave Heights Required

Comparison of the once per week, once per year and once per 50 years allowable wave height criteria in the harbour with comparable wave heights shown on the site specific wave height recurrence graph, Figure 16, will give the transmission coefficients required.

The required transmission coefficients of a floating breakwater are determined for the site by dividing the allowable wave height criteria for the harbour, as discussed in Section 7.3 and indicated in Table II, by the incident wave height for the applicable return period. Examples of this calculation are given in Table III for two situations:

i. the wave climate shown in Figure 16 with Acceptable Criteria as given in Table III

ii. the wave climate from S and SW directions only as shown in Figure 16 with Acceptable Criteria as given in Table II.

The required transmission characteristics of the breakwater at given wave periods are then determined by plotting the lowest values of the required transmission coefficients in Table III against the wave periods for which they have been calculated to be required, Figure 20.

A curve drawn similar to Figure 20 for a specific site defines the wave height reduction characteristics of any required floating breakwater which would be suitable to protect that site.

8.3.2 Dimensions from Previous Model Studies

After consideration of the factors listed in 8.2, preliminary dimensions should be established for each seemingly suitable breakwater design concept to enable the relative costs of the design concepts to be determined. Transmission coefficients should have been developed for the full range of wave periods, particularly the higher values, indicated on the required performance curve. Floating breakwaters which are suitable for the location will have transmission coefficients that lie on or below the plotted curve.

Existing data describing the performance of specific floating breakwaters from model studies are contained in Appendix C. These data were obtained in model studies undertaken at a scale of 1:30 or greater. The available data were plotted on graphs using the same scale as that used in Figure 20 for the required performance curves. Each sheet in Appendix C describes the test conditions including breakwater dimensions, water depth, mooring system, and wave periods for which the data was obtained.

Placement of the required performance curve which has been developed for a given project directly over each graph in Appendix C will indicate whether or not the model breakwater studied had performance characteristics meeting the required standards. For the breakwater to be acceptable, the transmission coefficients determined in the model study should not have exceeded the required transmission coefficients throughout the range of wave periods under consideration. The comparison should be made for all model study results presented in Appendix C. All breakwaters that have acceptable performance characteristics should be selected for further consideration. If the performance characteristics of a breakwater are not entirely acceptable but are reasonably close to that required, then that breakwater may also be considered in anticipation that its design may be improved by subsequent model studies.

Following comparison of the site breakwate requirements with previous model study results, the test conditions during model studies which produced acceptable transmission coefficients should be compared to anticipated prototype conditions with respect to the following factors:

water depths
orientation of breakwater
wave steepness
mooring conditions

The comparison of model and prototype conditions will indicate the suitability of applying the model design to the site. Some general guidelines to assist in assessing suitability of model studies to a given site are:

i. breakwater efficiency will improve with increasing relative depth of the structure;

ii. transmission coefficients of breakwaters in shallow water will generally be lower than obtained from model studies in deep water;

iii. breakwaters lying at an oblique angle to the incident wave will likely have lower transmission coefficients than similar designs tested perpendicular to the waves;

iv. the effect of wave steepness is seen from the model study results to affect breakwater performance although no general conclusion is drawn at this time.

Approximate breakwater dimensions derived from comparison of required breakwater performance with Appendix C test results will enable a preliminary assessment of breakwater size and cost to be obtained. The preliminary dimensions must be confirmed or modified by further physical model studies prior to being adopted for detailed design.

Determination of the breakwater length requires consideration of incident wave direction, seabed and shoreline topography and of diffraction patterns behind the breakwater. The breakwater length may be limited by the available water lot or by the fact that a breakwater or its moorings should not be located too close to entrance or navigation channels. Determination of the most economical breakwater length should be carried out by an engineer.

8.3.3 Dimensions Where Previous Studies not Applicable

It is possible that none of the available model test results will have direct application to a new location. In those instances, very approximate breakwater dimensions may be obtained through judicious use of the transmission coefficient versus B/L curves shown for the three main breakwater concepts in Figures 10 to 12. Application of results from these curves will give first estimates of suitable breakwater dimensions that are subject to the cautions discussed in Section 6.2. Dimensions obtained from these curves may be subject to considerable error and must be confirmed by further physical model studies.

Figures 10 to 12 show the breakwater transmission coefficient as a function of width, B, to incident wave length L. The incident wave length in metres may be approximated for water depths greater than 10 m by $L = 1.56\ T^2$ where T is the wave period. In shallower water than 10 m or with wave periods longer than 6 secs the wave length should be calculated using methods set out in the Shore Protection Manual[2].

Test results from the most efficient breakwater designs tested are shown on Figures 10 to 12. It is recommended that preliminary dimensioning for the breakwater width at this stage be based on values closer to the mean than to the bottom of the envelopes. The relative depth to width ratios found in the more efficient model studies indicated in Figures 10 to 12 and detailed in Appendix C may then be used to determine an approximate breakwater depth.

8.3.4 Examples of Selecting Preliminary Dimensions

The methods for determining suitable preliminary breakwater dimensions for the two wave climate situations discussed in Section 7 are set out below. By this stage of the project, the required breakwater performance curves described in Section 8.3 and typically illustrated on Figure 20 should have been prepared.

a. Example 1

The harbour in example 1 was exposed to waves from S to NW having the recurrence frequencies shown on Figure 16. The calculated required breakwater performance was shown on Figure 20A. Water depth at the proposed breakwater location was approximately 15 m over a sandy bottom.

Comparison through overlaying transparent copies of Figure 20A with the model data plots in Appendix C showed that, due to the required low transmission coefficients at wave periods above 4 sec, none of the previously tested model studies was directly applicable to the site. Although several designs reduced waves of less than 3 sec period to below the desired criteria, none of the designs satisfied the performance requirements of a transmission coefficient between 0.15 and 0.20 for waves with 5 or 6 sec periods.

A 6 sec period wave in 15 m depth of water has a wave length L, of 53 m.

Figure 10 shows that the best caisson design breakwaters could only achieve the required transmission coefficient of 0.15 with a B/L ratio of about 1.1 and with a centreboard added. This transmissibility was at the marginal limit of model caisson performance.

Figure 11 shows that Ofuya's 1968 tests on a 3.69 m wide A-frame and WCHL's 1979 tests on an 8.2 m wide caisson centreboard were the only centreboard design breakwaters to achieve a transmission coefficient of 0.15. Figure 11 also shows that Ofuya's wider A-frames did not achieve the same efficiencies at low values of B/L as his narrower one, indicating that scaling up the narrow breakwater tests results with relatively short waves to the required prototype 6 sec wave period dimension would produce an unreliable answer. The caisson centreboard breakwater required a B/L ratio of 1.1 to achieve a transmission coefficient of 0.15.

A single depth of pole tire breakwater using 1.0 m diameter tires would have a draft to depth ratio, D/d of 0.17. Figure 12 shows that such a pole-tire breakwater with a B/L ratio of 1.5 may achieve the required transmission coefficient of 0.15.

Consideration of Figures 10 to 12 indicates the following:

i. a caisson breakwater would probably require a width of well over 60 m, i.e., greater than 1.2 times the wave length of 53m, to reduce 6 sec waves to 15% of their incident height.

ii. a caisson supported centreboard breakwater may achieve the required performance but would have to be completely designed through model studies. The dimensions for the first model to be tested would be a matter of engineering judgement. Based on the relative dimensions of the caisson centreboard design shown on Figure C-27 and on Figure 11, starting dimensions for the model study may be a width of 20 m and a draft of 10 m.

iii. a pole-tire breakwater design of 84 m width, i.e., 1.5 times the wave length of 53 m, should be effective in reducing 6 sec waves to 15% of their incident height

iv. serious consideration should be given to increasing the allowable wave height criteria in the harbour, especially with respect to the 5 and 6 sec wave periods, to permit narrower floating breakwaters to be used

v. economic consideration should be given to constructing a solid breakwater to provide protection against the long period NW and W waves, Table III, and in protecting the harbour against S and SW waves by use of a narrower floating breakwater.

b. Example 2

The required performance curves in Figure 20B were derived for a similar harbour as discussed above but with a headland protecting the harbour against W and NW waves. This condition would be similar to example 1 if protection against NW and W waves were provided through construction of a solid breakwater. In this harbour, the mooring arrangement allowed vessels to be moored head to sea against both S and SW waves.

Comparison of Figure 20B to Appendix C showed that the following caisson and cetreboard caisson designs may be suitable:

i. 9.15 m caisson, Figure C14, 15
ii. 9.15 m chambered caisson, Figure C16, 17
iii. 18.29 m chambered caisson, Figure C25
iv. 8.1 m centreboard caisson, Figure C27

These structures were all tested in water depths comparable to those on site.

Pole-tire breakwaters Figures C33, 34, 35 and 36 also showed marginally acceptable performance characteristics but these had been tested in water depths of 4.6 m or less. It was anticipated that these designs would prove less satisfactory when tested in a deeper 15 m water depth.

Dimensions of the 9.15 m chambered and unchambered caissons and the 8.1 m centreboard caisson designs shown in Figures C16, C14, and C27 should be used to estimate preliminary costs and to establish the first design for model testing.

8.4 Mooring Systems

8.4.1 Anchoring

Design of anchoring systems for either permanent or semi-permanent structures requires detailed studies of site seabed conditions. The seabed survey should determine the sea floor material, depth of sediments, areal variability, estimate of soil cohesion, sensitivity, grain size, friction angle, density and origin of sediments. In addition, if piles are to be used the soil modulus of subgrade reaction should be determined. Seabed surveys should only be conducted by geotechnical engineers. Based on the results of such studies plus consideration of the economics and purpose of the moored structure, the optimum selection of the many numerous and sometimes complex types of anchoring system can be made.

Information on design of anchoring systems can be found in reports by Myers (1969); Taylor et al. (1975) and Taylor (1981). Information and results of anchoring systems of existing breakwaters has been discussed in section 6.1.10 of this report.

The selection of an optimum anchoring system for a floating breakwater is simplified by the fact that the anchor forces will generally be uniplanar. Anchor forces will be exerted in one horizontal direction with a vertical component. The selection may be complicated by possible water level changes and the permissible excursion limits of the breakwater. The following anchoring systems are recommended for consideration;

i. piles or dolphins to above water line
ii. battered stub piles protruding just above the seabed
iii. embedment anchors
iv. unidirectional deadweight anchors
v. combination deadweight anchors constructed around stub piles.

Of these, piles or dolphins would provide the most certain holding in a lateral direction. It is probable that they would also be the most expensive anchoring method. Battered stub piles would probably require the least scope of anchor chain of the seabed anchoring systems. The above two systems could only be used if the seabed material was suitable.

Selection of the most suitable embedment anchor or deadweight anchor system would depend on seabed soil and cost considerations. Reinforced and unreinforced concrete blocks have been successfully used as anchors in many situations. Anchor forces should be calculated considering wave reflectivity and drag forces, current drag, wind loads and ice forces plus some allowance for pretensioning and collisions. Wave forces may be determined from physical model studies. The size of anchor required to resist the above forces can be calculated using the seabed characteristics determined by site investigations. Field studies at Tenakee have indicated that such calculations produce conservative mooring forces.

Anchors should best be connected to the floating breakwater by chain to combine longevity with simplicity of connection and to provide weight to assist the holding power of the anchor. Sizing of anchor chain and shackle connections should be designed on the basis of wave force calculations considering both wave reflectivity and wave drag forces. It would be better practice to oversize the anchor chain and allow for loss of strength through corrosion than to install a thin chain which required frequent maintenance or replacement.

8.4.2 Mooring Forces

Mooring loads from winds and currents may be calculated for a breakwater using an equation of the form $F=f(C_D \cdot A \cdot V^2)$.The information required includes:

- Current speeds and direction (V_c)
- Drag coefficients for the breakwater (C_{Dc})
- Wind speeds (V_w)
- Drag coefficients for the breakwater superstructure (C_{Dw}) and vessels if used for moorings.
- Exposed surface areas (A)

Calculation of wave induced mooring forces is a varied and complex problem. Wave forces are best determined from carefully controlled model studies which reproduce the breakwater design, mooring system and entire spectrum of incident waves. No simple generalities can be determined for reliably calculating wave induced mooring forces.

Ice loadings can only be reliably estimated from experience with similar structures. Rough estimates can be made by assuming an ice thickness and a value of the crushing strength of ice.

The pretension of the mooring lines determines both the excursion of the breakwater and its dynamic response. High pretension forces allow less excursion and produce an improved breakwater performance compared to low pretension forces. At the same time, both the steady and dynamic mooring loads increase with increasing pretension.

8.4.3 Float Connections

Connections between floats have been a major source of weakness in floating breakwater design. Satisfactory results have been achieved however at Tenakee harbour in Alaska with 30 cm thick hard rubber rings, Figure 21. These rings are spaced vertically at each corner of connecting breakwater sections to absorb compression, tension, and torsion forces. Steel chains between the sections limit the maximum tension to which the rubber rings can be exposed

The rubber rings both cushion tension shocks on the chains and act as compressive buffers between the floats. This technique has not been long in practice but has overcome wear problems previously experienced by buffer systems wherein tension was taken only by chains attached to fixed connection points.

It is important for any design that the connections be exposed for continual inspection and that components subject to continuous movement be easily replaced.

8.5 Detailed Design Studies

8.5.1 Breakwater Performance

At this stage of the project, a preliminary design concept for a floating breakwater has been adopted that may meet the requirements of the harbour. The subsequent detailed design process should refine and finalize the dimensions and other physical characteristics of the breakwater, including moorings, on the basis of the required performance characteristics. The overall layout of the breakwater relative to the harbour should also be established. Plans and specifications should be produced. The steps in this stage of the design process are illustrated in Figure 22, and are discussed in this section.

The most important part of the detailed design is a model study required to confirm and refine the required breakwater performance and to economically optimize its dimensions. There appears to be no satisfactory alternative to undertaking this model study. Section 6.2 demonstrated that existing data cannot be reliably extrapolated and that numerical model studies require physical model studies for calibration and verification. The model studies would form a small part of the overall project budget and would be offset by savings in achieving an optimal breakwater design. The model studies should be conducted at a scale of 1:10 or greater and should reproduce the following features as accurately as feasible in order of importance:

- the breakwater geometry
- the weight and weight distribution of the breakwater
- the radii of gyration of the breakwater about the principal axes
- wave climate representative of site conditions
- depths of water representative of site conditions
- direction of prevailing waves relative to the breakwater
- unit weight, diameter, elasticity, length and catenary of mooring lines
- number of connected breakwater modules, side by side, with their connectors
- breakwater elasticity for rubber tire designs
- loadings on the structure resulting from secondary use

- snow, ice, marine growth and sediment loadings if applicable

In general, the performance of a breakwater will be improved if its mass is increased, if the natural periods of motion are increased or if the pre-tension in the moorings is increased. The performance of a floating breakwater may also be improved if the waves arrive at an oblique angle to the breakwater.

8.5.2 Breakwater Layout

In many situations, three dimensional model studies of the layout of the breakwater and harbour are required. The objective of these studies would be to determine the length of the breakwater and the orientation that provides the required protection to the harbour area for all wave directions. Three dimensional model studies, if properly designed, can accurately simulate wave refraction, diffraction, shoaling and reflection.

As an alternative to physical modelling of the diffraction process, numerical models of wave diffraction past the ends of the breakwater may be undertaken to establish the breakwater's required length and orientation.

These studies should be undertaken at the same time as the model studies to refine the design dimensions of the breakwater and mooring system and an active exchange of information should be organized between the studies.

8.5.3 Possible Problems in Construction

Construction problems can be minimized by thorough project planning, management and supervision. The preparation of good specifications and attentive inspection of all aspects of the construction is essential to the success of a floating breakwater project.

Some problems which have often been encountered during construction include:

- considerable variation in the unit mass or draft of the breakwater or components of the breakwater. This has caused difficulty in assembling pontoon breakwaters and has caused listing of caisson breakwaters;

- lack of inspection during construction. Problems which develop in the breakwater during construction have often become magnified when a lack of constant and attentive inspection makes it impossible to isolate the problem to one area of concern;

- facilities for the construction, launching or asembly of a floating breakwater have limited the size or weight of a structure. The capability of existing equipment for breakwater construction may limit the design of a breakwater or subject it to hazardous conditions during its transportation and handling which must be considered in the design.

8.6 Maintenance

The maintenance of a floating beakwater is essential to its continuing success. Regular programs of inspection should be established and should include both structural and underwater inspections.

Connections between breakwater sections and between the structure and its anchoring system should be given special attention. The performance of a floating breakwater should be monitored in the field with consideration given to:

- the recording of both incident and transmitted wave climates at the breakwater;
- measurement of both mooring and connection forces.

These will ensure that performance requirements for the harbour have been met by the breakwater design and will provide invaluable information if required for a breakwater extension or replacement as well as assisting in future physical and numerical model studies.

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TABLES

TABLE I

SUMMARY OF EXPERIENCE FROM SITES VISITED

Type	Place	Year	Cost/m ($ 1980 est)	Mooring	Wave Protection	Major Problems	Acceptance
RUBBER TIRE BREAKWATERS							
Goodyear 2 Modules	Diversey Harbour	1978	1000	Concrete blocks weight unknown	Not wide enough for adequate protection.	Lost flotation due to foam absorbing water. Anchors dragged in first storm.	Unsatisfactory
Goodyear 3 Modules	Cataumet	1975	220	2000 kg concrete blocks 300 kg/m length	Satisfactory with waves to 1.8 m high	Cable and chain replaced due to corrosion. Tires filled with foam after sinking.	Satisfactory
Goodyear 3 Modules	Newington	1976	N/A	1750 kg stone blocks	Satisfactory	Anchors dragged in 3 m/s current	Satisfactory
Goodyear 3 Modules	Riviere au Renaud	1977	480	68 kg Danforth with 630 kg concrete blocks 50 kg/m length	Satisfactory	Unpopular with fishermen in poor visibility	Satisfactory
Goodyear 3 Modules	North Vancouver	1978	N/A	230 kg Danforths replaced by 12000 kg blocks	Satisfactory	Constantly broke moorings and dragged anchors in currents to 1 m/s	Unsatisfactory
Goodyear 3 Module 6 Module	Pickering Beach	1978	650 1085	2900 kg concrete blocks 350 kg/m length Replaced with piles	N/A	Connections failed; loss of foam buoyancy; anchors dragged	Satisfactory
Wave Maze 7 Module	Pickering Beach	1978	850 1500	2900 kg conc. blocks 350 kg/m length	N/A	Breakwater failed structurally and anchors dragged	Unsatisfactory
Pole Tire	Mamaroneck	1980	575	2700 kg anchor 2300 kg conc. blocks. 700 kg/m length	N/A	None	Recent installation

TABLE I

SUMMARY OF EXPERIENCE FROM SITES VISITED

Type	Place	Year	Cost/m ($ 1980 est)	Mooring	Wave Protection	Major Problems	Acceptance
CAISSON BREAKWATERS							
Caisson	Richmond	1969	1020	N/A	N/A	Caissons damaged by end collissions; Filled with foam after sinking	Satisfactory
Caisson	Nanaimo	1974	N/A	perimeter dolphins	Satisfactory	None	Satisfactory
Caisson	Pt. Orchard	1974	1020	stub piles	Satisfactory	Corrosion of anchor chains. Failure of caisson connection to anchor	Satisfactory
Caisson	N. Vancouver	1977	2750	11000 kg concrete blocks. 1100 kg/m length	Satisfactory	Corners damaged prior to cross-connecting caissons	Satisfactory
Caisson	Maple Bay	1977	2050	stub piles	Satisfactory	None	Satisfactory
Caisson	Friday Harbour	1978	N/A	15000 kg conc. blocks. 40000 kg clump weights, 1000 kg/m length	Satisfactory	Poor construction techniques	Satisfactory
PONTOON BREAKWATERS							
Pontoon	Friday Harbour	1972	2300	N/A	Satisfactory	Fatigue failure in plastic pontoons	Satisfactory
Pontoon	Sitka	1973	3040	Concrete blocks with piles	Satisfactory	Wear on chain connections	Satisfactory
Pontoon	Ketchican	1980	4600	100000 kg conc. 5400 kg/m length	Satisfactory	None	Recent Installation

TABLE I

SUMMARY OF EXPERIENCE FROM SITES VISITED

Type	Place	Year	Cost/m ($ 1980 est)	Mooring	Wave Protection	Major Problems	Acceptance
A-FRAME BREAKWATERS							
A-frame	Lund	1963	2500	11000 kg conc blocks 600 kg/m length	Satisfactory	Corrosion of steel floats, frame and chains. Damage to float ends from collision	Satisfactory
A-frame	Queen Charlotte City	1967	N/A	11000 kg conc. blocks 600 kg/m length	Satisfactory	None	Satisfactory
A-frame	Gananoque	1968	2800	11000 kg conc. blocks 700 kg/m length	Satisfactory	None	Satisfactory
A-frame	Thunder Bay	1968	2900	11000 kg conc. blocks 600 kg/m length	Satisfactory	None	Satisfactory

TABLE II

PROVISIONALLY RECOMMENDED CRITERIA

FOR THE SIGNIFICANT WAVE HEIGHT, Hs,

IN A SMALL CRAFT MARINA

WAVE DIRECTION RELATIVE TO VESSEL	PEAK WAVE PERIOD, T, SECS	RETURN PERIOD OF THE EVENT 1 IN 50 YR	1 IN 1 YEAR	1 IN EACH WEEK
HEAD SEAS	T < 2	/	Hs 0.30 m	HS 0.30 m
HEAD SEAS	2 < T < 6	Hs 0.61 m	Hs 0.30 m	HS 0.15 m
HEAD SEAS	6 < T	HS 0.61 m	HS 0.30 m	HS 0.15 m
BEAM SEAS	T < 2	/	HS 0.30 m	HS 0.30 m
BEAM SEAS	2 < T < 6	HS 0.23 m	HS 0.15 m	HS 0.08 m
BEAM SEAS	6 < T	HS 0.23 m	HS 0.15 m	HS 0.08 m

For Excellent wave climate multiply criteria by 0.75

For Moderate wave climate multiply criteria by 1.25

TABLE III

TRANSMISSION COEFFICIENTS AT VARYING WAVE PERIODS

FOR EXAMPLE GIVEN IN TEXT

Wave Direction		Wave	Wave Height Criteria, m			Incident Wave Height			Required Breakwater Transmission Coefficient		
Bearing	to Vessel	Period, sec	1 hr/wk	1 hr/yr	1 hr/50 yr	1 hr/wk	1 hr/yr	1 hr/50 yr	1 hr/wk	1 hr/yr	1 hr/50 yr
Example 1- Wave Climate as shown on Figures 16 and 19. Vessels moored with bows facing West.											
NW	Head	2	0.3	0.3	-	-	0.2	-	-	1.0	-
	Head	3	0.2	0.3	0.6	-	-	-	-	-	-
	Head	4	0.2	0.3	0.6	-	-	-	-	-	-
	Head	5	0.2	0.3	0.6	-	-	-	-	-	-
	Head	6	0.2	0.3	0.6	-	2.0	3.2	-	0.15	0.19
W	Head	2	0.3	0.3	-	-	-	-	-	-	-
	Head	3	0.2	0.3	0.6	0.6	-	-	0.33	-	-
	Head	4	0.2	0.3	0.6	-	-	-	-	-	-
	Head	5	0.2	0.3	0.6	-	1.2	-	-	0.25	-
	Head	6	0.2	0.3	0.6	-	-	1.9	-	-	0.31
SW	Head	2	0.3	0.3	-	0.2	-	-	1.0	-	-
	Head	3	0.2	0.3	0.6	-	0.7	-	-	0.43	-
	Head	4	0.2	0.3	0.6	-	-	1.1	-	-	0.55
	Head	5	0.2	0.3	0.6	-	-	-	-	-	-
	Head	6	0.2	0.3	0.6	-	-	-	-	-	-
S	Beam	2	0.3	0.3	-	0.2	-	-	1.0	-	-
	Beam	3	0.1	0.2	0.2	-	0.5	-	-	0.40	-
	Beam	4	0.1	0.2	0.2	-	-	0.9	-	-	0.22
	Beam	5	0.1	0.2	0.2	-	-	-	-	-	-
	Beam	6	0.1	0.2	0.2	-	-	-	-	-	-
Example 2- Wave climate similar to that shown on Figures 16 and 19 but protected from NW and W waves by headland. Vessels moored with bows facing south.											
S	Head	2	0.3	0.3	-	0.2	-	-	1.0	-	-
	Head	3	0.2	0.3	0.6	-	0.5	-	-	0.6	-
	Head	4	0.2	0.3	0.6	-	-	0.9	-	-	0.7

TABLE III

TRANSMISSION COEFFICIENTS AT VARYING WAVE PERIODS

FOR EXAMPLE GIVEN IN TEXT

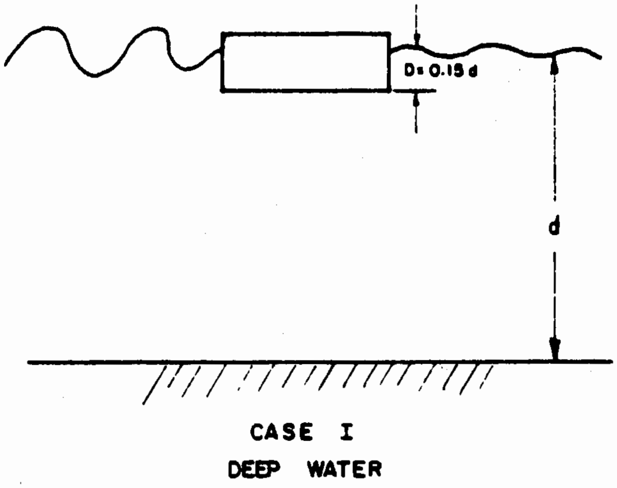
Wave Direction		Wave	Wave Height Criteria, m			Incident Wave Height			Required Breakwater Transmission Coefficient		
Bearing	to Vessel	Period, sec	1 hr/wk	1 hr/yr	1 hr/50 yr	1 hr/wk	1 hr/yr	1 hr/50 yr	1 hr/wk	1 hr/yr	1 hr/50 yr
S	Head	5	0.2	0.3	0.6	-	-	-	-	-	-
	Head	6	0.2	0.3	0.6	-	-	-	-	-	-
SW	Head	2	0.3	0.3	-	0.2	-	-	1.0	-	-
	Head	3	0.2	0.3	0.6	-	0.7	-	-	$\underline{0.43}$	-
	Head	4	0.2	0.3	0.6	-	-	1.1	-	-	$\underline{0.55}$
	Head	5	0.2	0.3	0.6	-	-	-	-	-	-
	Head	6	0.2	0.3	0.6	-	-	-	-	-	-

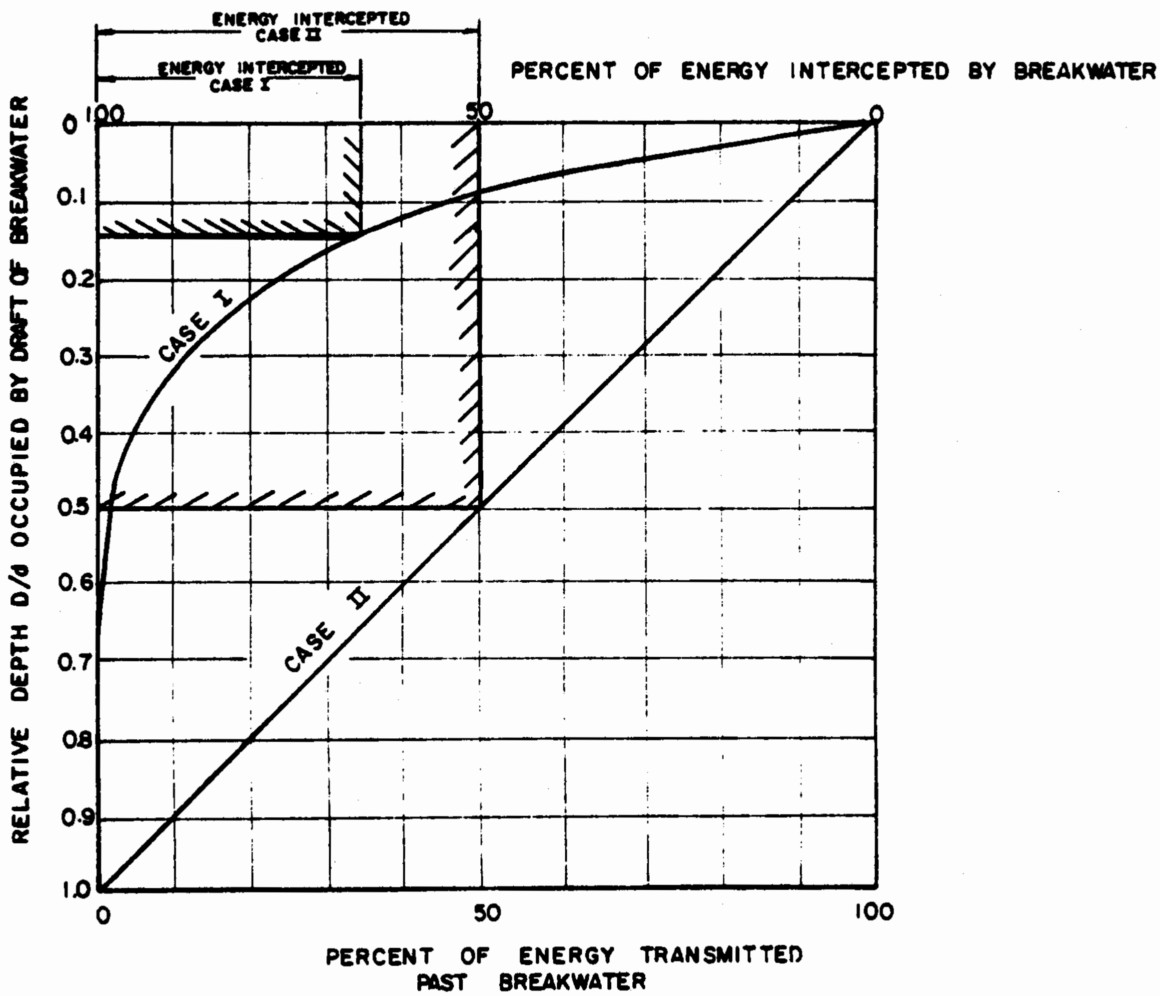
Notes:

1. Wave height criteria for harbour as given in Table II
2. Required Breakwater transmission coefficient determined by dividing the Wave Height Criteria by the respective Incident Wave Height.
3. Underlined numbers are minimum values for each wave period and are plotted in Figure 20.

FIGURES

FIGURE 1



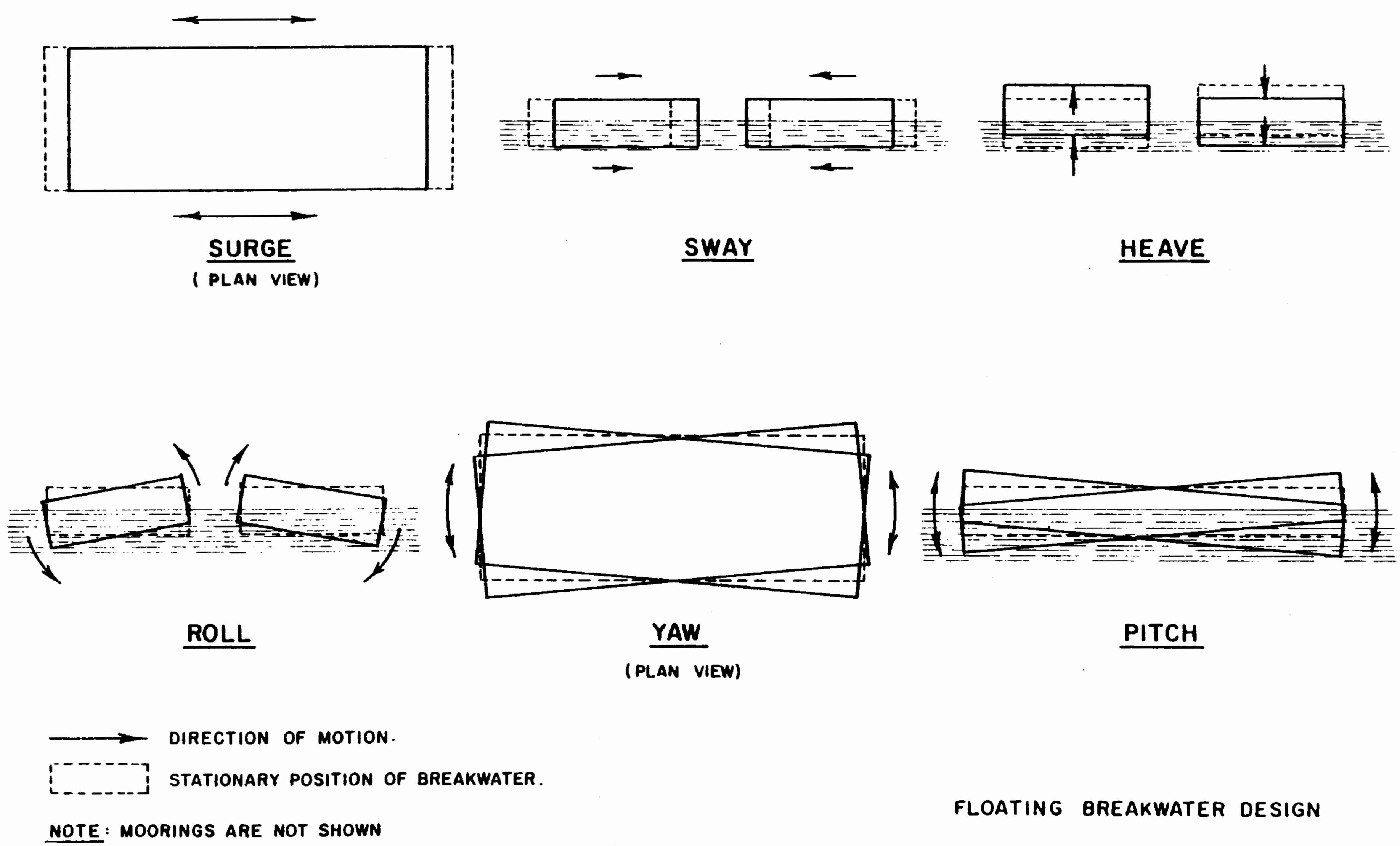


FLOATING BREAKWATER DESIGN

EFFECT OF WATER DEPTH

WESTERN CANADA HYDRAULIC LABORATORIES LTD.

bcil 6639A-WCH



FLOATING BREAKWATER DESIGN

DEFINITION OF MOTIONS

FIGURE 2

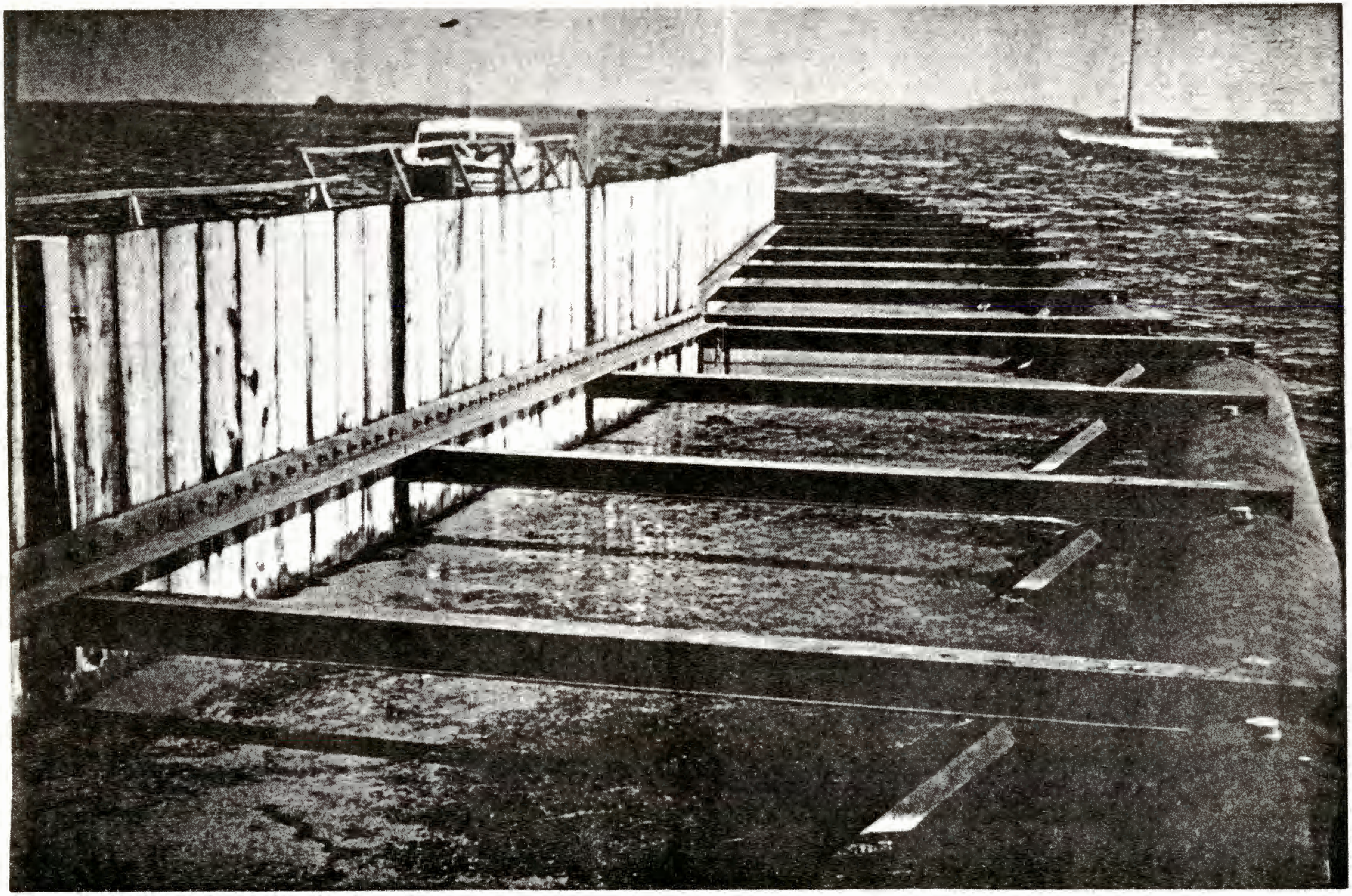
WESTERN CANADA HYDRAULIC LABORATORIES LTD.

bcil 6638 A-WCH

A. Breakwater at Lund, B.C., with centreboard support frames above water level

B. Lund A-Frame breakwater construction

A-FRAME BREAKWATERS - SUPPORT FRAME ABOVE WATER



A-FRAME BREAKWATER SUITABLE FOR DECKING - in use on Wharf at Gananogue, Ont.

FIGURE 4

A. Floating Caisson breakwater of 4.6 m width and 1.1 m draft at Friday Harbour

B. Friday Harbour breakwater in 2.8 sec. waves

CAISSON BREAKWATERS

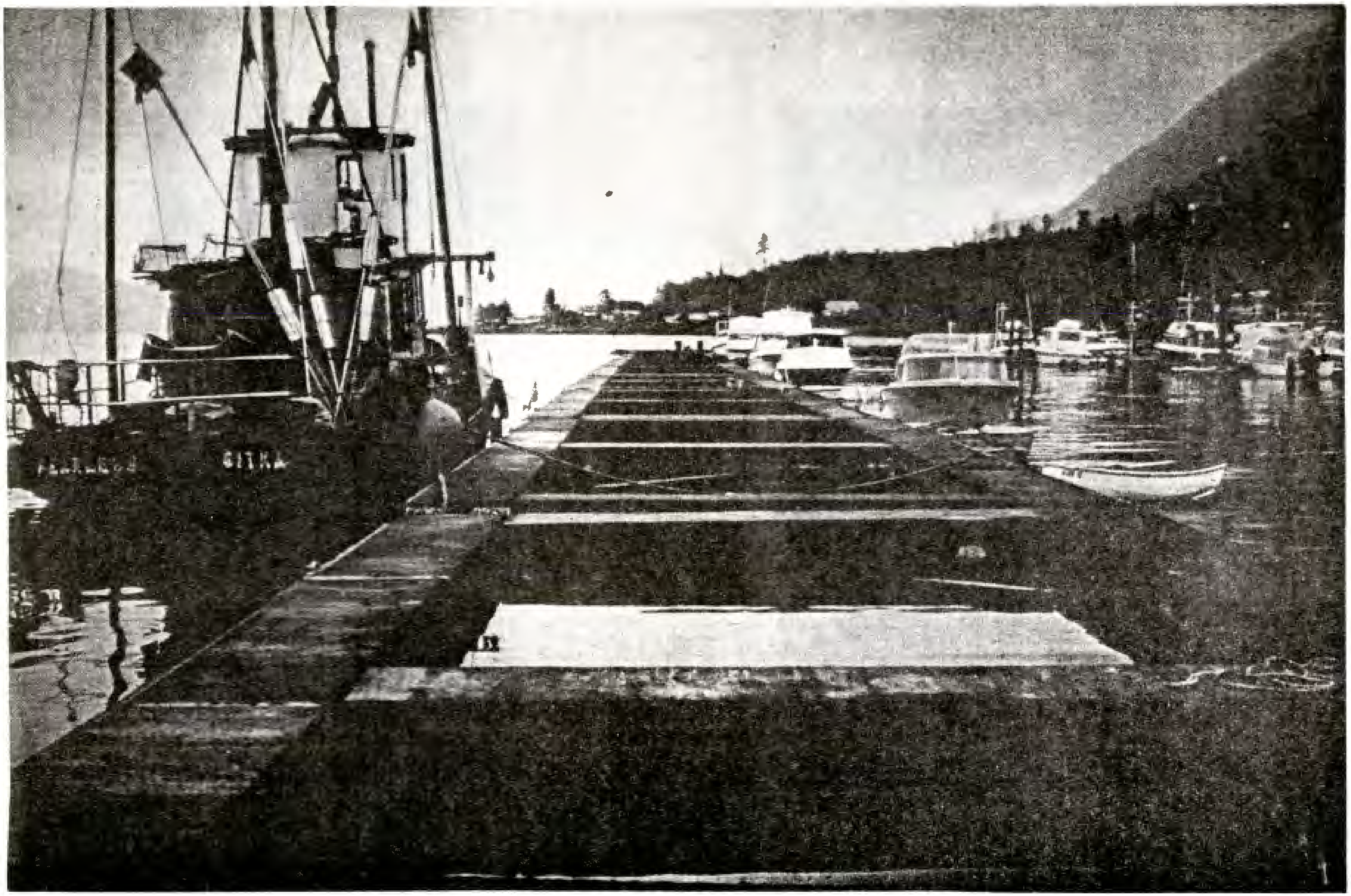
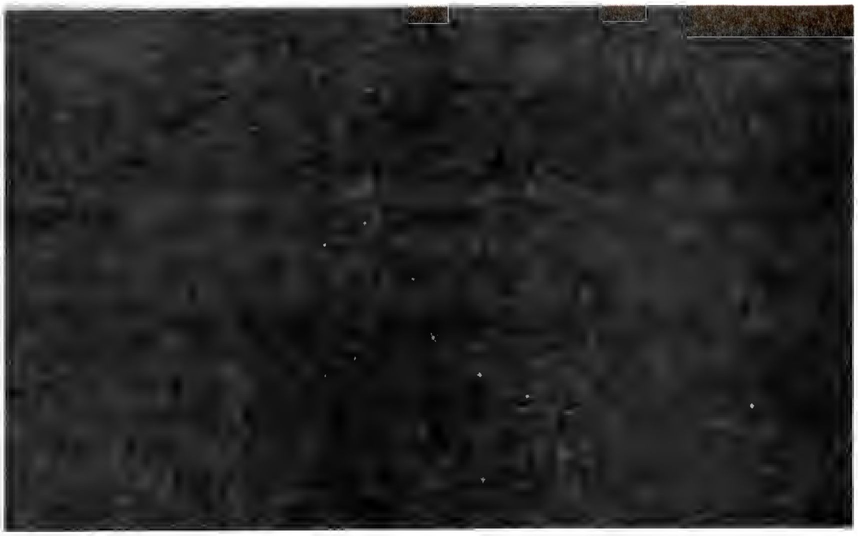


FIGURE 6

PONTOON BREAKWATER AT SITKA, ALASKA

FIGURE 7



A. 3 module Goodyear design breakwater at Pickering Beach - photo by U.S. Army Corps of Engineers

B. Goodyear module design breakwater at North Vancouver, B.C.

C. Wave Breaking action induced by Wave Maze Breakwater

FLOATING TIRE BREAKWATER

FIGURE 8

A. Floating log bundle breakwater

B. Breakwater at Powell River B.C. of Scrapped Ship Hulls

BREAKWATERS FROM AVAILABLE MATERIALS

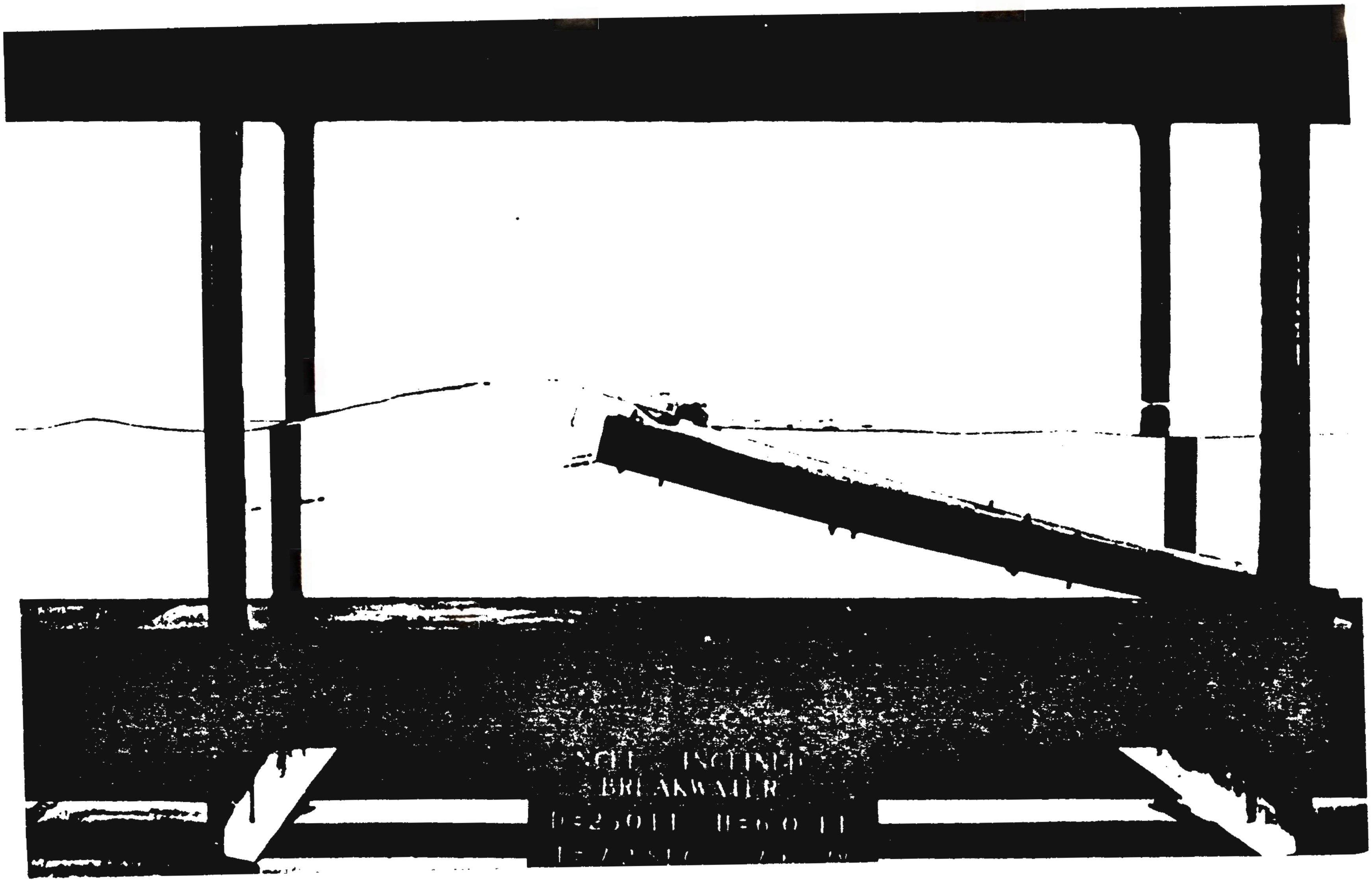
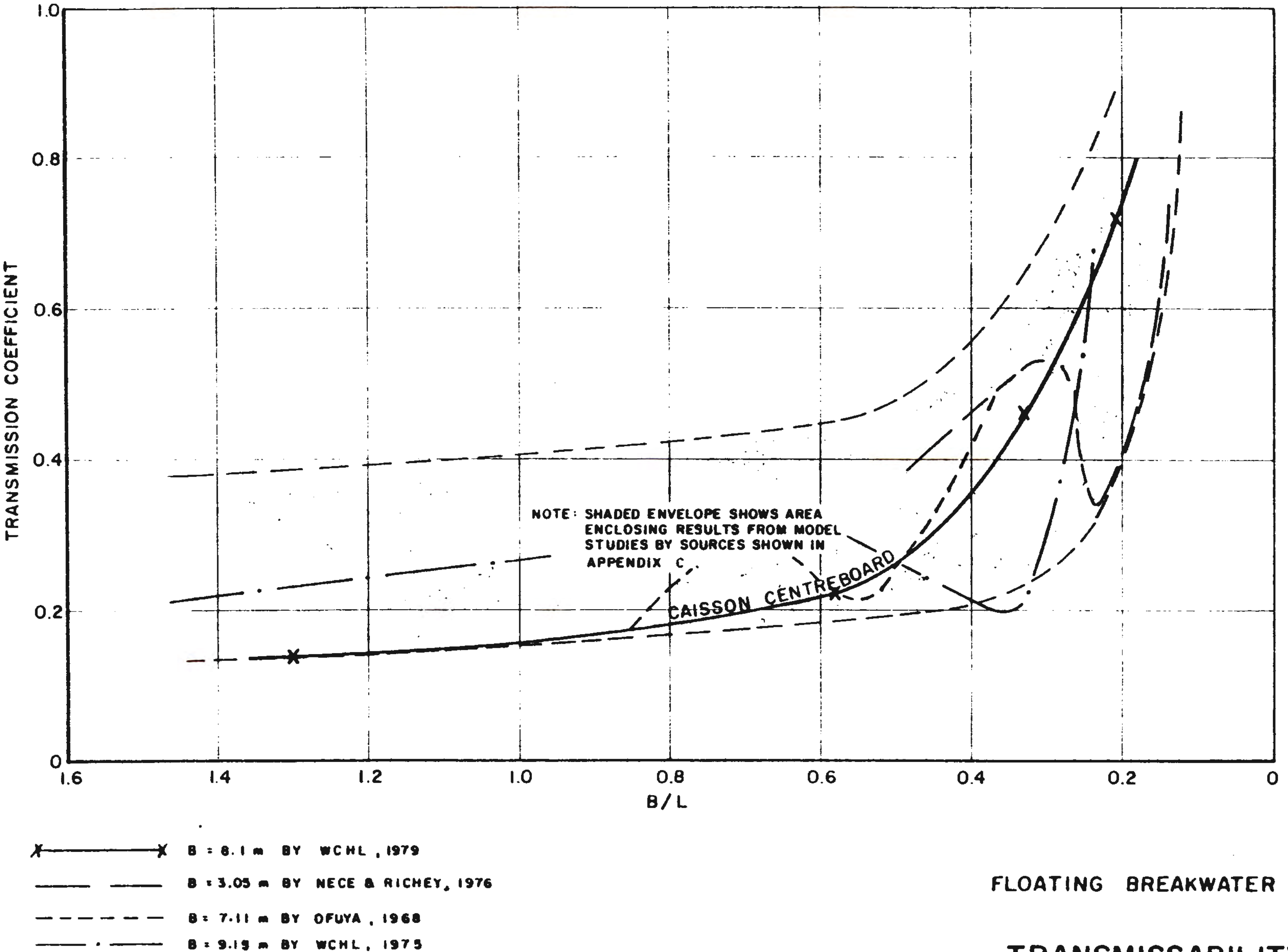


FIGURE 9

INCLINED FLOAT BREAKWATER. Photo by U.S. Corps of Engineers



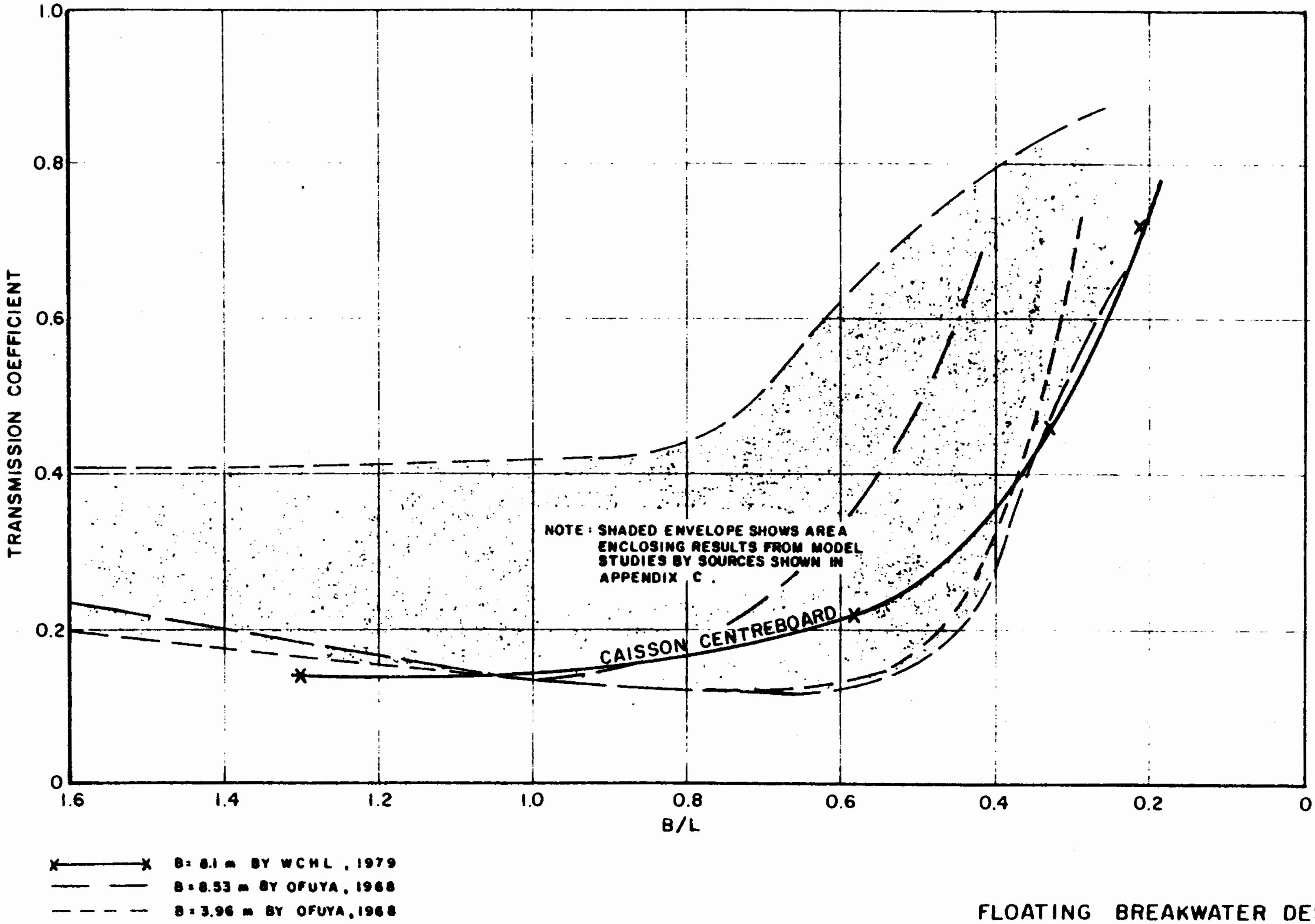
FLOATING BREAKWATER DESIGN

FIGURE 10

TRANSMISSABILITY OF MODEL CAISSON BREAKWATERS

WESTERN CANADA HYDRAULIC LABORATORIES LTD.

bcil 6638 A-WCH



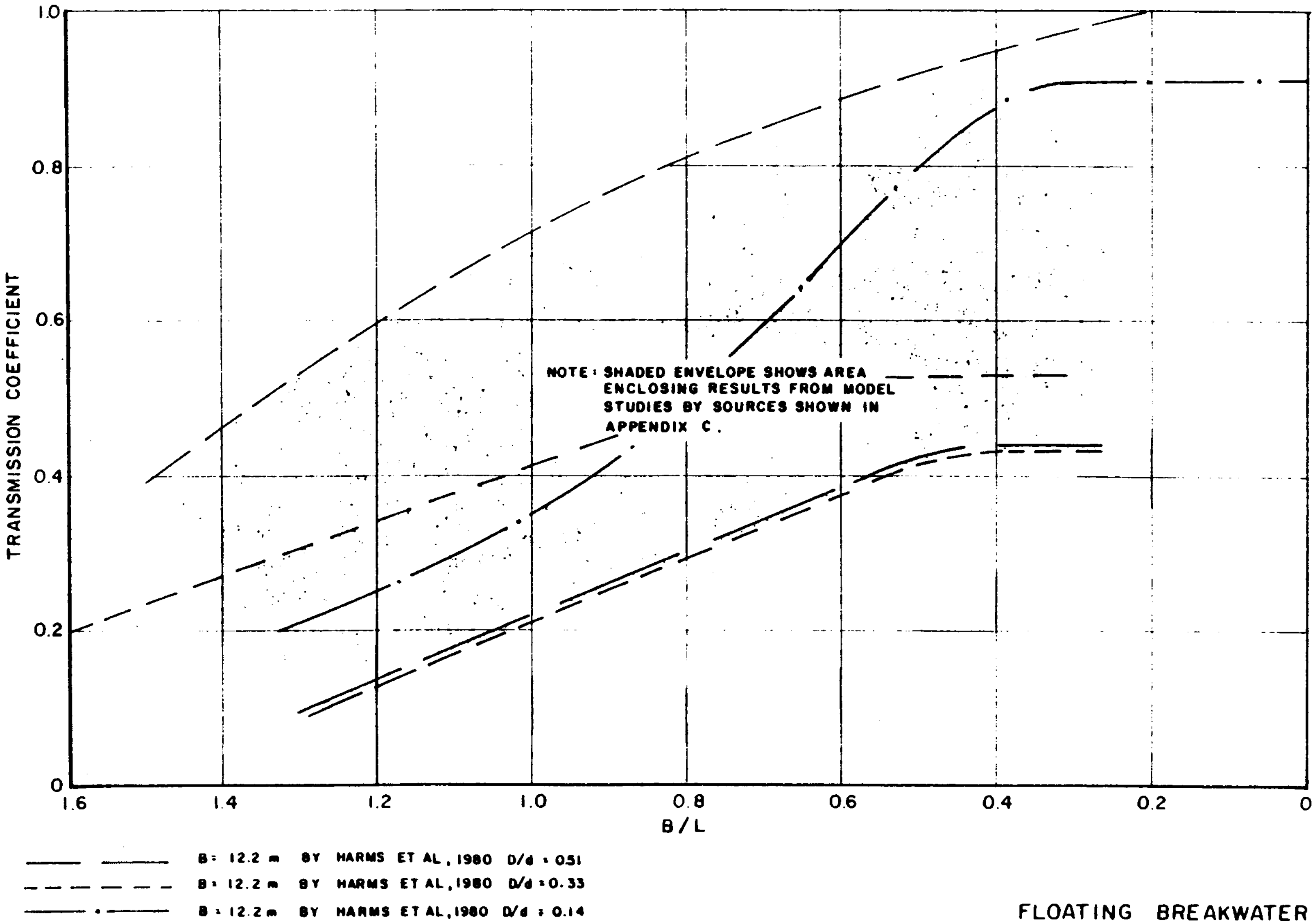
FLOATING BREAKWATER DESIGN

TRANSMISSABILITY OF MODEL CENTREBOARD BREAKWATERS

FIGURE 11

WESTERN CANADA HYDRAULIC LABORATORIES LTD.

bcl 6638 A-WCH



FLOATING BREAKWATER DESIGN

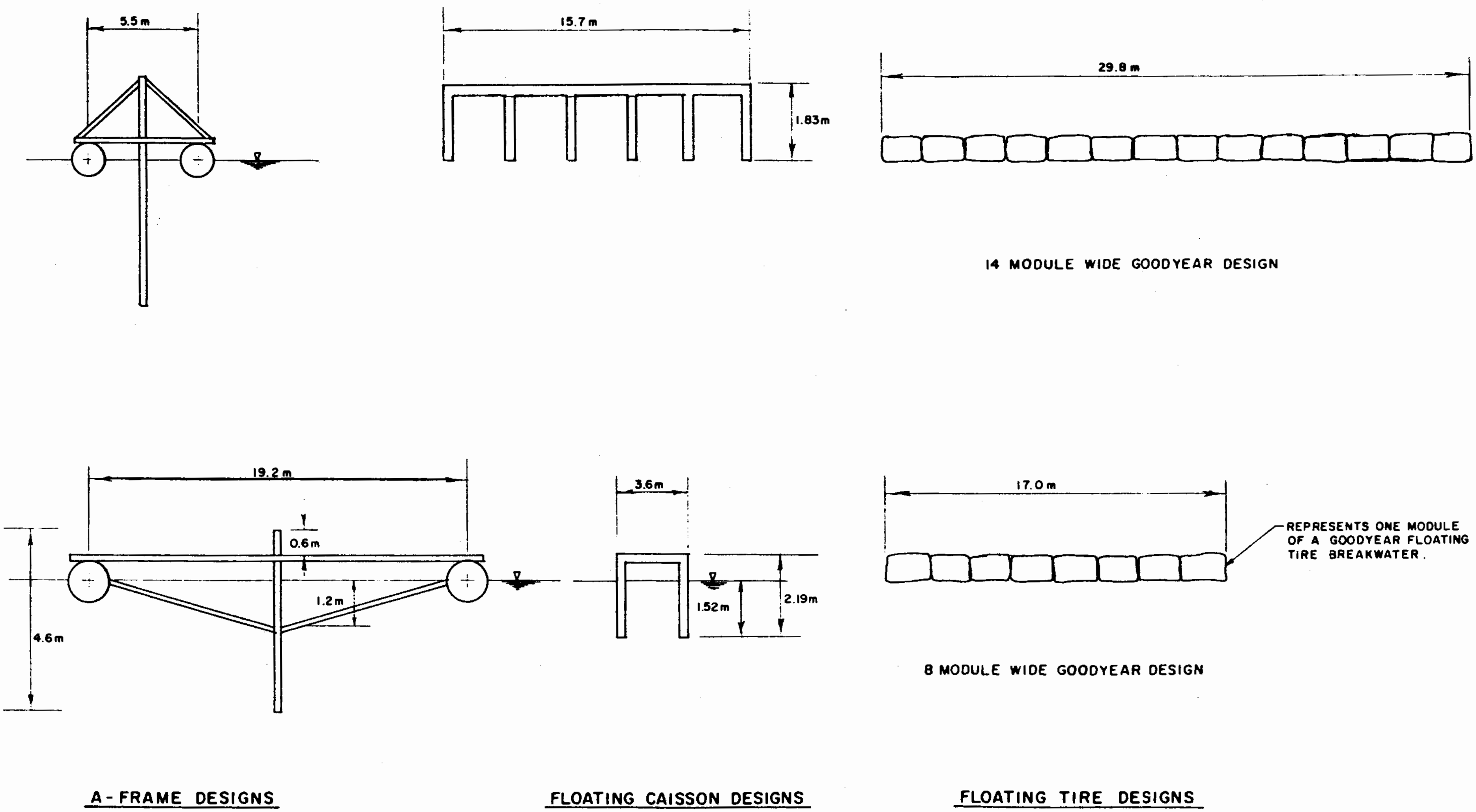
TRANSMISSABILITY OF RUBBER TIRE BREAKWATERS

FIGURE 12

WESTERN CANADA HYDRAULIC LABORATORIES LTD.

bci16638 A-WCH

FIGURE 13



NOTE: ALL DESIGNS SHOWN HAVE BEEN EXTRAPOLATED FROM RESULTS OF DIFFERENT MODEL STUDIES TO PRODUCE A TRANSMISSION COEFFICIENT OF 0.33 WITH WAVES UP TO 0.9 m SIGNIFICANT HEIGHT AND 3 sec PEAK PERIOD.

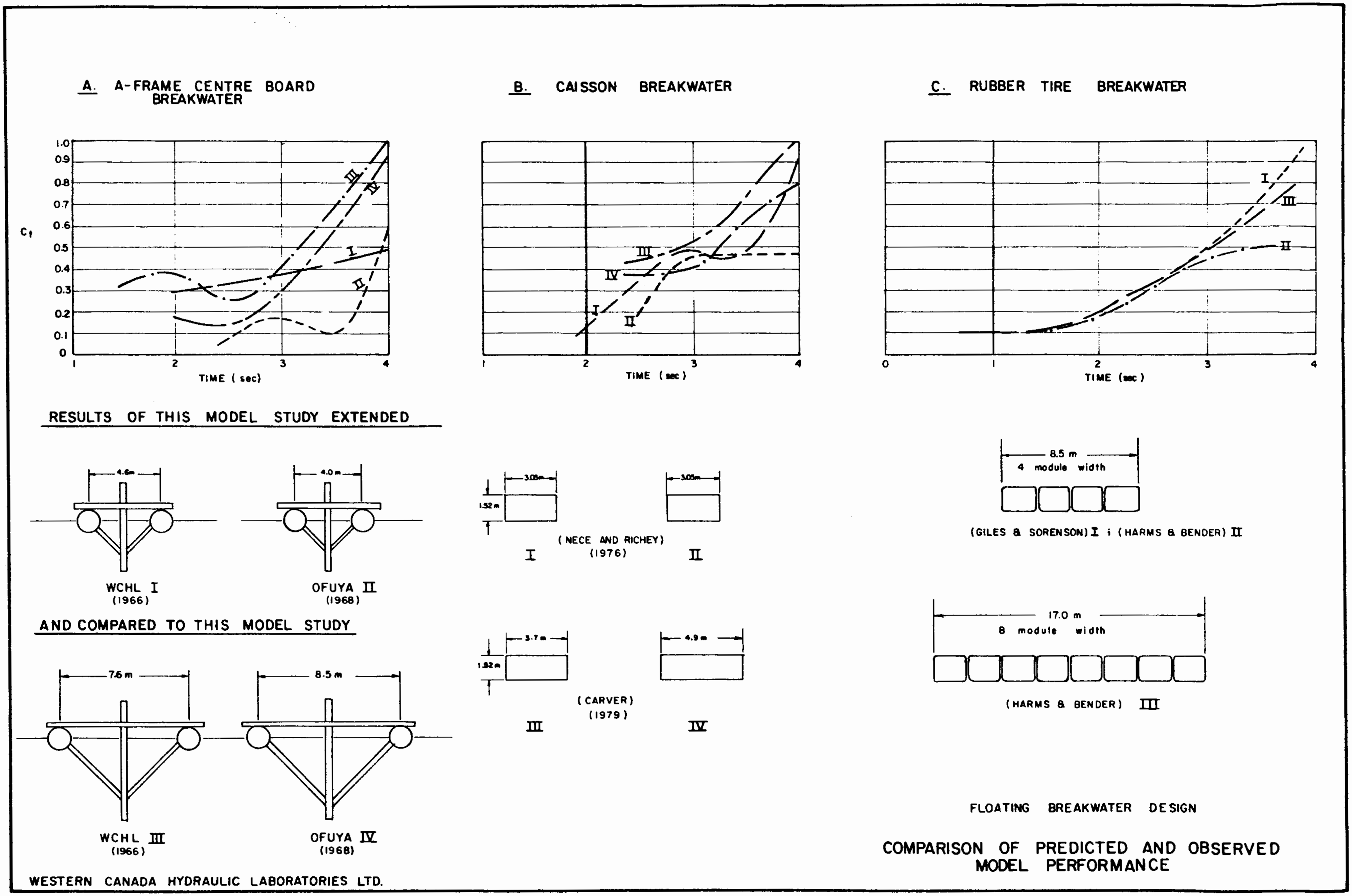
FLOATING BREAKWATER DESIGN

VARIATION IN DESIGNS FROM EXISTING MODEL STUDIES

WESTERN CANADA HYDRAULIC LABORATORIES LTD.

bcil 6640b-WCH

FIGURE 14

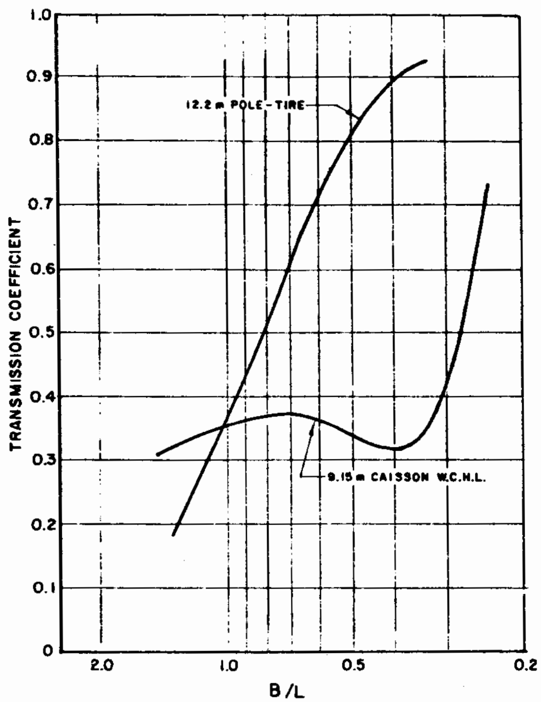


FLOATING BREAKWATER DESIGN

COMPARISON OF PREDICTED AND OBSERVED MODEL PERFORMANCE

bcil 6640b-WCH

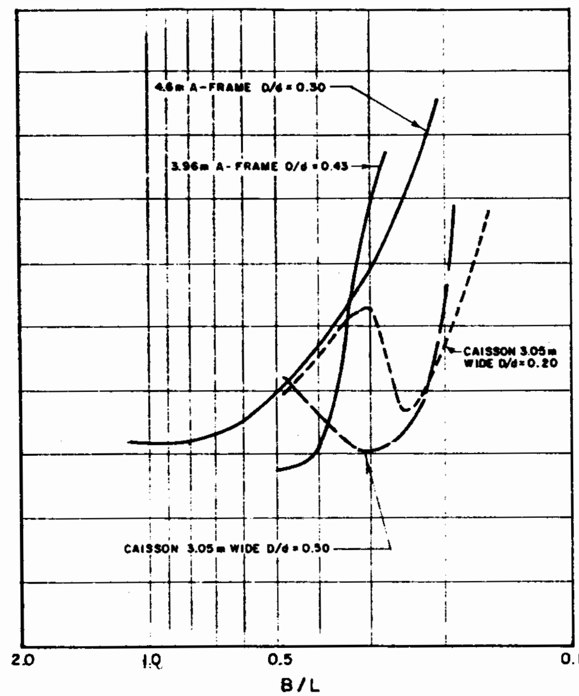
FIGURE 15



A - COMPARISON OF CAISSON WITH RUBBER TIRE BREAKWATER.

RELATIVE DEPTH D/d = 0.14

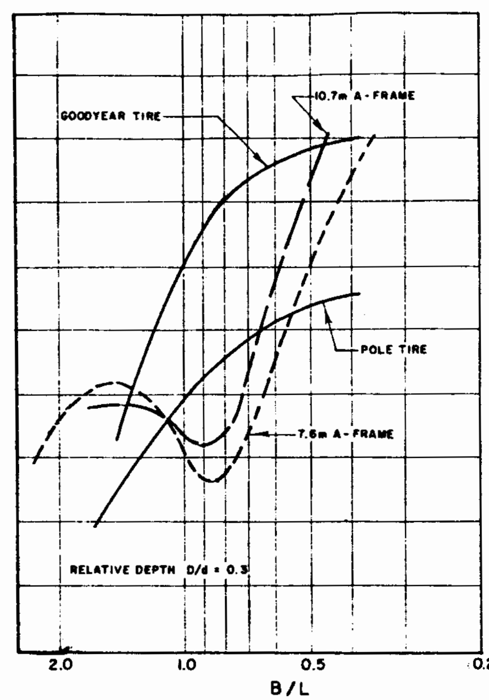
POLE-TIRE 12.2 m WIDE BY HARMS, ET AL 1980 d = 2.0 m
CAISSON 9.15 m WIDE BY WCHL, 1975 d = 13.7 m



B - COMPARISON OF CAISSON WITH A-FRAME BREAKWATER.

RELATIVE DEPTH D/d = 0.30

A-FRAME D/d OF 0.43 BY OFUYA 1968 d = 9.1 m
" " D/d OF 0.30 BY WCHL 1966 d = 12.2 m
CAISSON D/d OF 0.50 BY DAVID SON 1971 d = 3.1
" D/d OF 0.20 BY NECE & RICKEY 1976 d = 3.1



B - COMPARISON OF A-FRAME WITH RUBBER TIRE BREAKWATER.

RELATIVE DEPTH D/d = 0.30

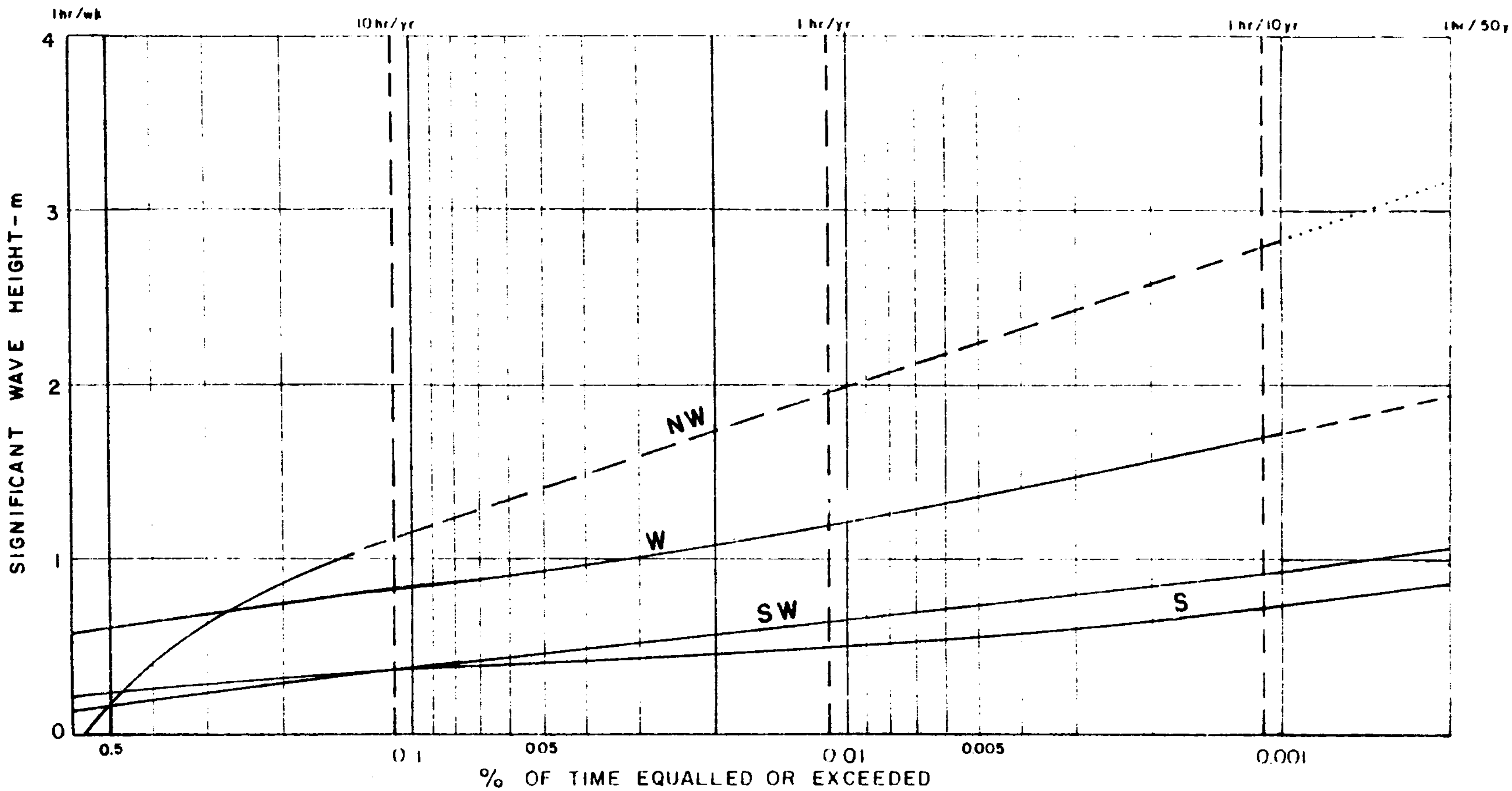
A-FRAME 7.6 m WIDE WCHL 1966 d = 12.2 m
POLE TIRE 12.2 m WIDE HARMS ET AL 1980 d = 2.0 m
GOODYEAR TIRE 8.5 m WIDE GILES & SORENSON 1978 d = 2.0 m

FLOATING BREAKWATER DESIGN

COMPARISON OF BREAKWATER CONCEPTS

WESTERN CANADA HYDRAULIC LABORATORIES LTD.

bcil6640b-WCH



NOTE WAVE CLIMATE ESTIMATED FROM 10 YRS OF WIND RECORDS.

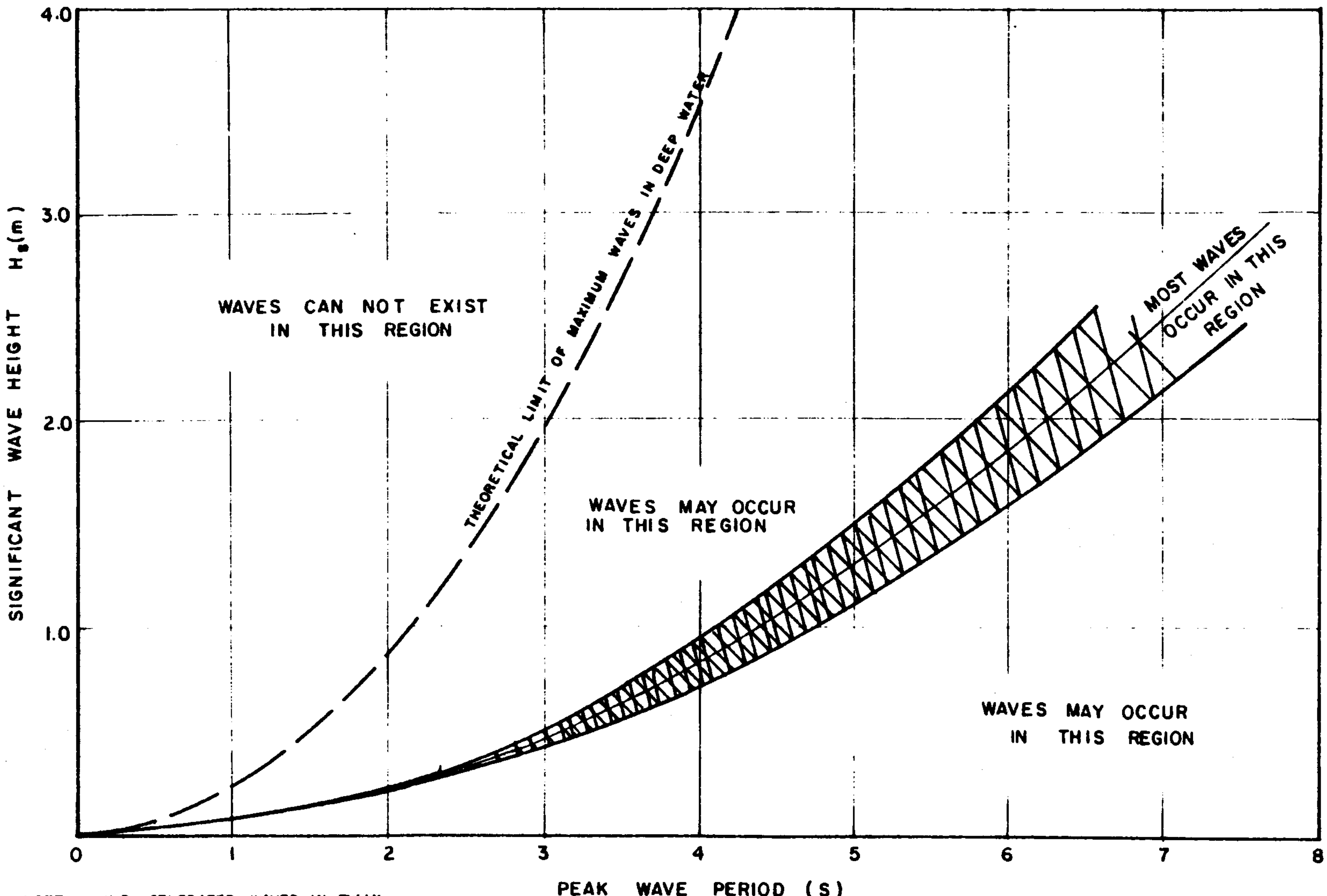
FLOATING BREAKWATER DESIGN

TYPICAL WAVE HEIGHT RECURRANCE

FIGURE 16

WESTERN CANADA HYDRAULIC LABORATORIES LTD.

bc-1 6638 A-WCH



NOTE: WIND GENERATED WAVES IN FULLY DEVELOPED SEAS GENERATED BY WINDSPEEDS OF 0–160 kph BLOWING OVER DEEP WATER FETCHES OF 2–95 km AS RELATED BY THE SVERDRUP-MUNK-BRETSCHNEIDER HINDCASTING TECHNIQUE.

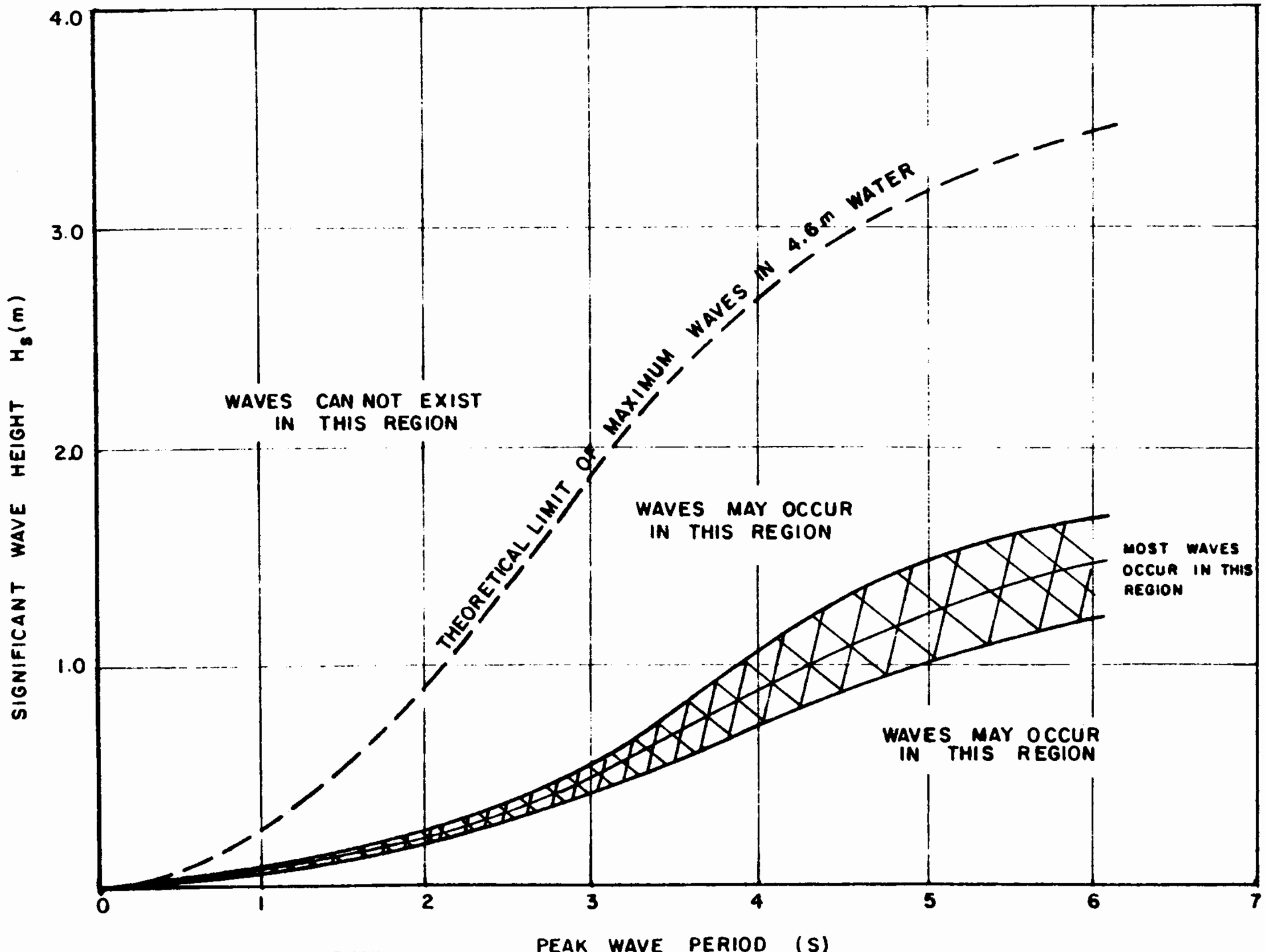
FLOATING BREAKWATER DESIGN

HINDCAST WAVE HEIGHT VS PERIOD
DEEPWATER

FIGURE 17

WESTERN CANADA HYDRAULIC LABORATORIES LTD.

bcil 6638 A-WCH



NOTE: WIND GENERATED WAVES IN FULLY DEVELOPED SEAS GENERATED BY WINDSPEEDS OF 0-160 kph BLOWING OVER DEEP WATER FETCHES OF 2-95 km AS RELATED BY THE SVERDRUP-MUNK-BRETSCHNEIDER HINDCASTING TECHNIQUE.

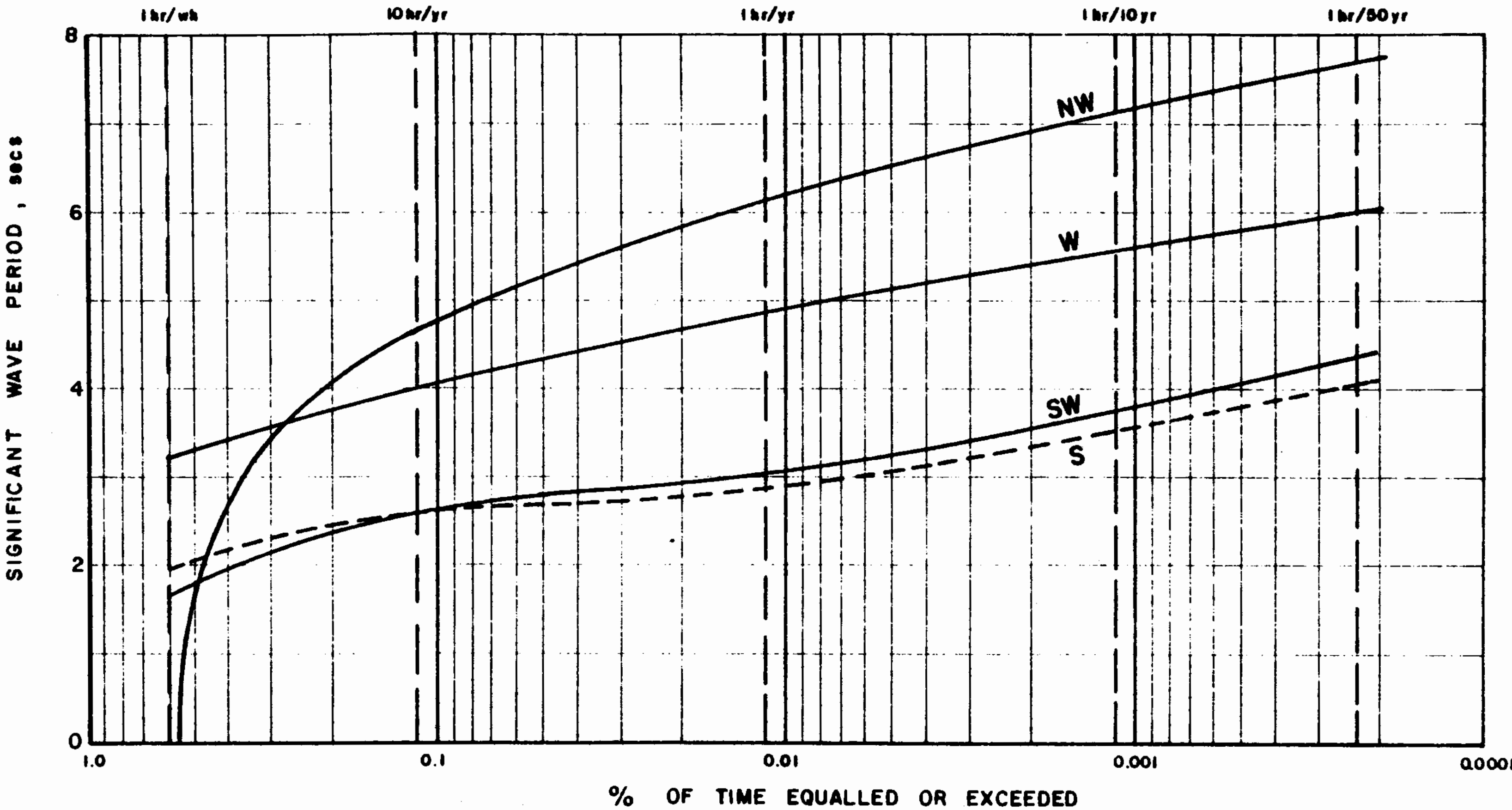
FLOATING BREAKWATER DESIGN

HINDCAST WAVE HEIGHT VS PERIOD
4.6 m DEPTH

WESTERN CANADA HYDRAULIC LABORATORIES LTD.

FIGURE 18

bcil 6638 A-WCH



<u>NOTE</u>: WAVE PERIODS DETERMINED FROM WAVE HEIGHT CURVES ON FIGURE 16 USING MEAN CURVE RELATIONSHIP SHOWN ON FIGURE 17.

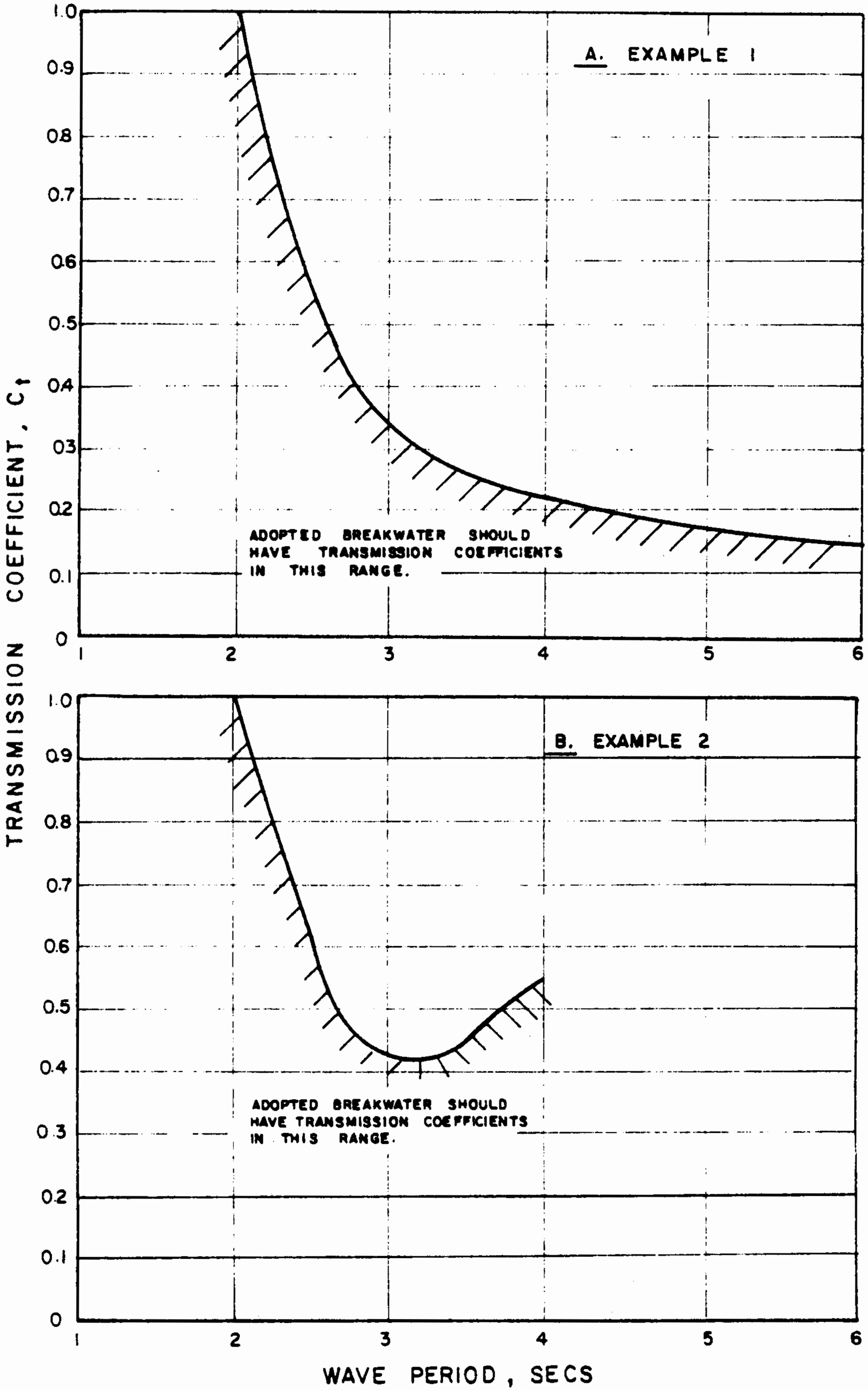
FLOATING BREAKWATER DESIGN

RECURRANCE FREQUENCY OF SIGNIFICANT WAVE PERIODS

FIGURE 19

WESTERN CANADA HYDRAULIC LABORATORIES LTD.

bcil 6638 A-WCH



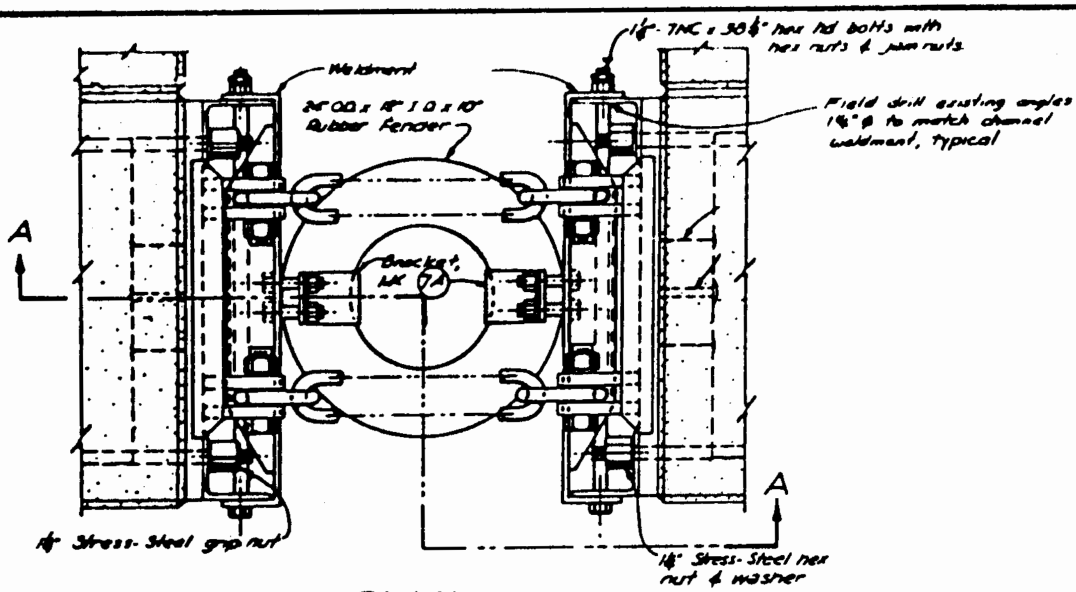
FLOATING BREAKWATER DESIGN

REQUIRED BREAKWATER PERFORMANCE

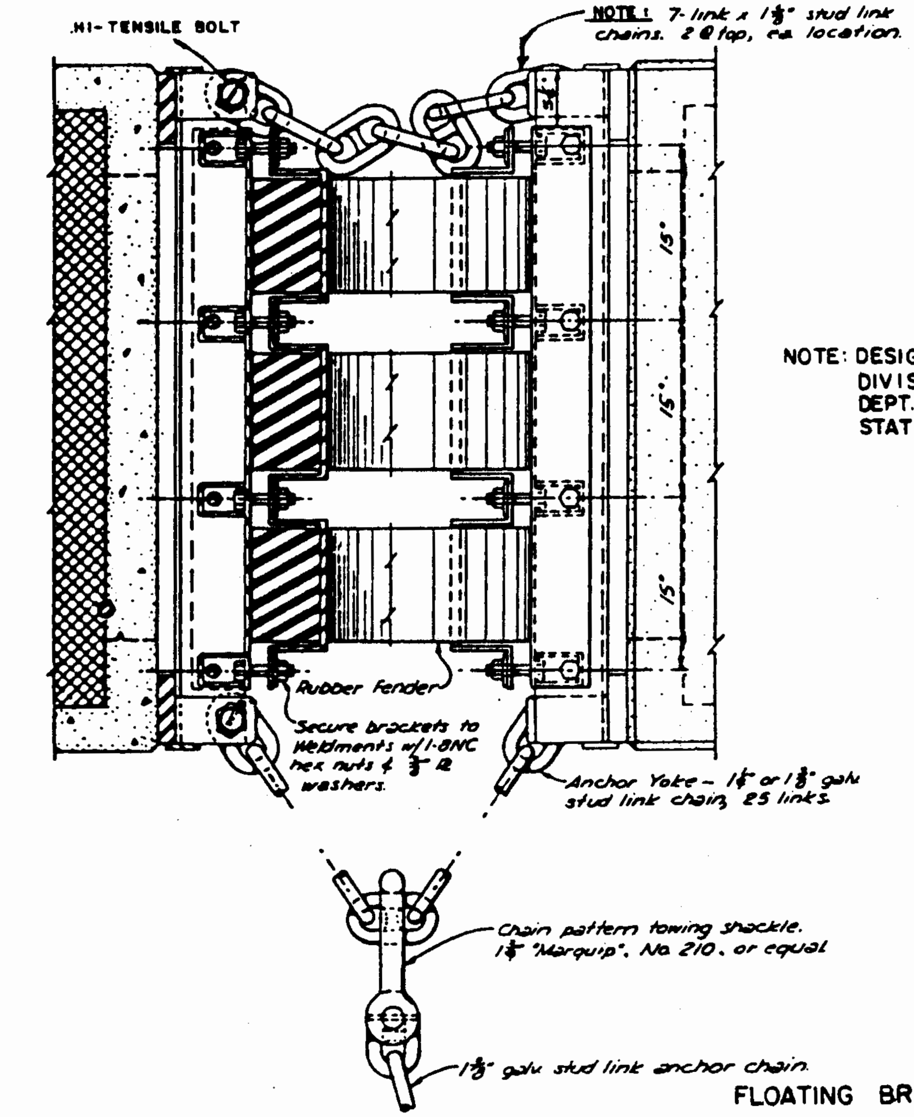
WESTERN CANADA HYDRAULIC LABORATORIES LTD.

bcil 6639 A-WCH

FIGURE 21



PLAN



NOTE: DESIGN BY
DIVISION OF WATER & HARBORS
DEPT. OF PUBLIC WORKS
STATE OF ALASKA.

SECTIONAL ELEVATION A-A

CONNECTION DETAIL

TYPICAL FOR EXISTING BREAKWATER

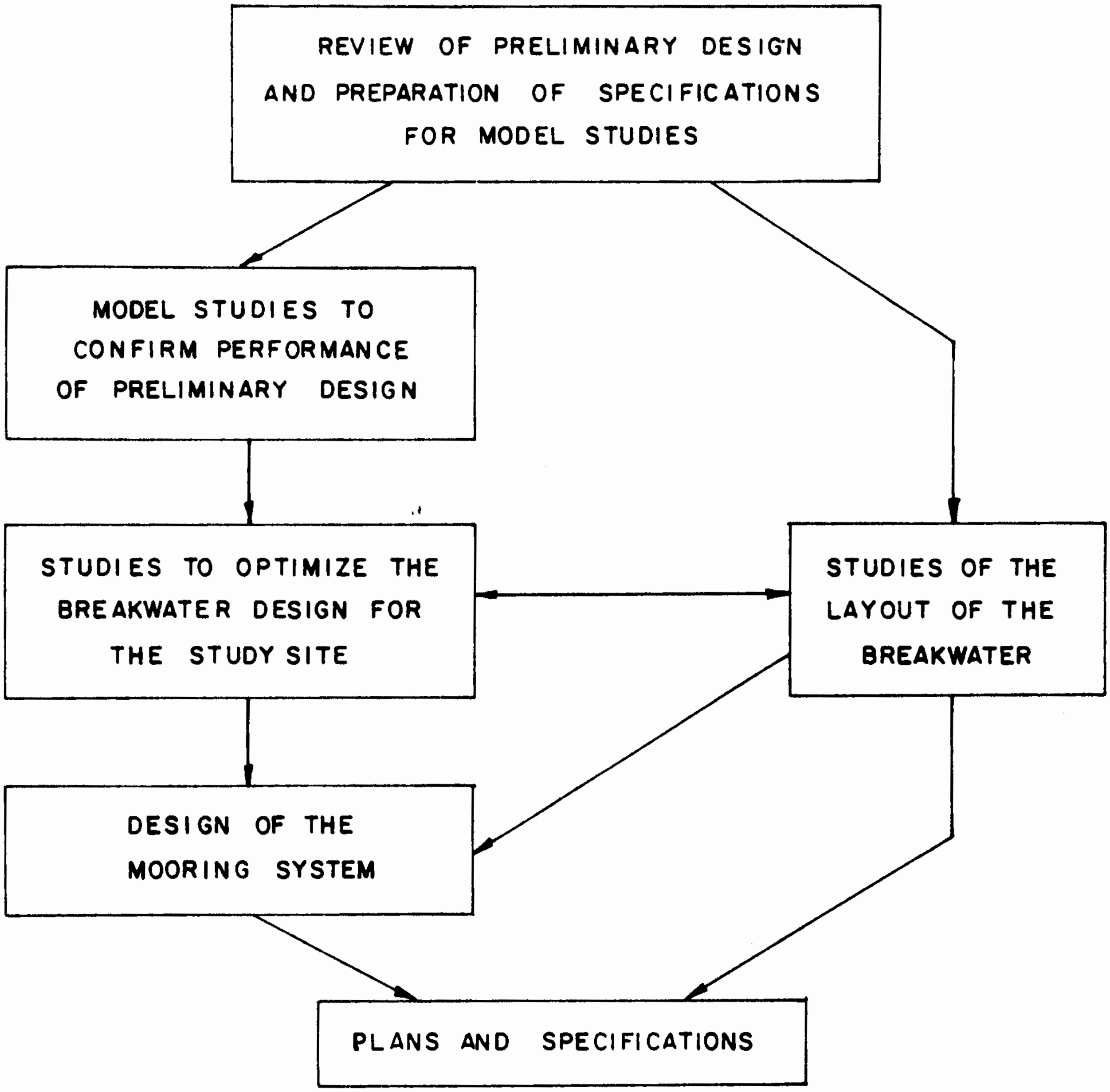
$1\frac{1}{2}" = 1'-0"$

FLOATING BREAKWATER DESIGN

BREAKWATER CONNECTION AT TENAKEE ALASKA

WESTERN CANADA HYDRAULIC LABORATORIES LTD.

FIGURE 22



FLOATING BREAKWATER DESIGN

DETAIL DESIGN PROCESS

WESTERN CANADA HYDRAULIC LABORATORIES LTD.

bcil 6639A-WCH

APPENDICES

APPENDIX A

SITE OBSERVATIONS AND NOTES ON EXISTING BREAKWATERS

A.0 FLOATING BREAKWATER DATA

All information collected on existing floating breakwaters is summarized. The information is divided into two categories; Floating Breakwaters in North America, and Floating Breakwaters outside North America.

Floating breakwaters in North America are arranged by the type of breakwater. Floating breakwaters outside North America are arranged by country.

Floating breakwaters in North America, where insufficient information to summarize the facility was available, are summarized by location at the end of Section

A.1 Floating Breakwaters in North America

The following comments apply to the headings on each breakwater description.

Winds: In general, accurate wind data was not available. Local observations of winds are summarized where provided.

Waves: In general, accurate wave data was not available. Observed wave heights should be regarded as maximum observed significant wave heights. Where design criteria was available, it is summarized.

Dimensions: Dimensions are provided in the following order; length, width, and depth. The depth is the sum of freeboard and draft. Where draft was available, mass has been calculated.

Costs: Costs are in currency of the country.

A - FRAME AND INVERTED A - FRAME BREAKWATERS

Location: LUND, BRITISH COLUMBIA

Breakwater Type: A-Frame

Reference Location: Canadian Hydrographic Services' Chart No. 3591

Type of Harbour: Small craft harbour

Contact: Public Works Canada, Pacific Region, Vancouver, British Columbia

Site Description: The breakwater is located inside the entrance to a small bay which is exposed to the west and northwest. Protection is provided from the southwest by a small rubble mound structure protruding from the headland on the southside of the bay.

Depth of Water: 15 m to 21 m below chart datum

Tide Range: 3.4 m (average tide)

Bottom Soil Conditions: Not available

Wind Climate: Subjected to two or three strong northwest storms per year. Local recorded wind not available. Afternoon northwest wind (8 - 10 m/s approximately) frequent during summer. Breakwater is protected by adjacent land from most frequent southeast storms.

Fetches: Longest fetch is northwest 12 km.

Waves: No recorded wave data for design. Offshore wave data recorded in 1977 (MEDS Station 117). Maximum observed height 1.4 m. Maximum height = 1.4 m, Maximum period = 2.8 s for design.

Currents: No strong currents in area.

Shipwaves: Only from vessels using the facility.

Ice: No ice during winter.

Breakwater Description: A-frame breakwater constructed of cylindrical steel pontoons separated laterally by steel I-beam space frame. Heavy timber centreboard. Pontoons sub-divided into 3 m long watertight compartments each with two pump-out ports.

Dimensions: Six modules, four modules, each = each 18.3 m x 7.6 m
two modules, each = 15.2 m x 7.6 m
Diameter of pontoons = .76 m
Draft (including centreboard) = 3.7 m Draft of pontoons variable (1980)
Freeboard (including centreboard) = 2 m
Mass of breakwater = Not calculated

Construction Site: Fabricated in Vancouver, B.C., towed to Lund and moored.

Connections: Modules cross-connected with 15 kg/m chain. Tires threaded on chain placed between modules at pontoons.

Reserve Buoyancy: Original design had watertight compartments. Later modified by filling pontoons with styrofoam rounds.

Provision for Corrosion: None

Inspection Program: Annual Inspection, wharfinger on location.

Notes: Concept model tested. No field measurements conducted. Transmission coefficient and mooring forces obtained.

Mooring Details:

Anchor System: 1.8 m x 1.8 m x 1.5 m unreinforced concrete blocks with 50 mm diameter steel rod embedded for chain connection. Windward anchor chains, 25 mm chain. Leeward anchor chains 19 mm. Chain was 14.9 kg/m, Prooftest = 249 kN. Breaking load 355 kN.

Mooring Layout: Four anchors per module on windward side. Two anchors per module on leeward side.

Maintenance Requirements:

Structural: Structural steel corroded. Worst where water was both sides of steel. End of pontoons damaged from inter module collisions.

1973-1974, end compartments filled with round styrofoam beads. Further damage to ends allowed beads to escape. Replaced with discs of styrofoam forced into pontoons.

1979-1980, additional 0.76 m diameter pontoons added full length, each side to 15.2 m modules. Pontoons foam filled. Additional pontoons to be added to 18.5 modules in near future.

Connections: Connecting chains break or pull out. Tires replaced. Chain lengths adjusted.

Mooring: Rechained 1977-1978.

Performance: Adequate. During northwest 18 m/s, some users find it necessary to vacate harbour for more secure mooring. Considerable motion in harbour. Field transmission coefficients unknown.

Costs: \$754/m (Total cost on site, 1963-1966)

Maintenance costs todate, \$52,000. Breakdown between materials and labour not available.

Notes: Depth of water made floating breakwater more economical than conventional breakwater.

Location: QUEEN CHARLOTTE CITY, BRITISH COLUMBIA

Breakwater Type: A-Frame

Location Reference: Canadian Hydrographic Services' Chart No. 3806

Type of Harbour: Small craft harbour

Year Constructed: 1967

Contact: Public Works Canada, Pacific Region, Vancouver, B.C.

Site Description: Harbour is exposed to south quadrant. Breakwater protects southern exposure of harbour facility. Protection to the southeast is provided by a rockfill causeway and a pile A-frame breakwater. Southwest exposure protected by log bundle breakwaters.

Depth of Water: 3.5 m below chart datum

Tide Range: 7.8 m, large tide

Bottom Soil Conditions: Not available

Wind Climate: Not available

Fetches: Fetch normal to breakwater, 2.4 km

Waves: No recorded wave data. Predominant direction is southwest. Maximum observed wave heights of 0.8 to 0.9 m. No wave period data available.

Currents: No strong currents in area.

Shipwaves: Not a problem

Ice: No ice during winter

Breakwater Description: A-frame breakwater, constructed of cylindrical steel pontoons separated laterally by steel I-beam space frame. Heavy timber centreboard. Pontoons sub-divided into 3.0 m long water tight compartments each with two pump-outs ports. Construction is same as breakwater at Lund, B.C.

Dimensions: Three modules, each = 18.3 m x 7.6 m

Diameter of pontoons = .76 m

Draft (including centreboard) = 3.7 m

Freeboard (including centreboard) = 2 m

Mass of breakwater = not calculated

Construction Site: Not known

Connections: Not known

Reserve Buoyancy: Water tight compartments (pumping has not been required)

Provision for Corrosion: Sacrificial anodes

Inspection Program: Annual inspection

Notes: Original concept model tested. No field measurements conducted

Mooring Details:

Anchor System: 1.8 m x 1.8 m x 1.5 m unreinforced concrete blocks with 51 mm diameter steel rod embedded for chain connection. Windward anchor chains 25 mm chain. Leeward chains 19 mm chain.

Mooring Layout: Two anchors per module on windward side. Two anchors per module on leeward side.

Maintenance Requirements: Only minor maintenance has been required.

Performance: Not possible to separate breakwater performance from influence of other structures in harbour.

Costs: Not available

Notes: Depth of water made floating breakwater more economical than conventional breakwater

Location: **GANANOQUE ONTARIO**

Breakwater Type: Inverted A-Frame

Location Reference: Canadian Hydrographic Services' Chart No. 1420

Type of Harbour: Private Marina

Year Constructed: 1968

Contact: Public Works Canada, Ontario Region, Toronto, Ontario

Site Description: Breakwater is located in the St. Lawrence River in Thousand Islands area. The breakwater protects the southwest and west exposure of the marina. No protection is provided to the east and southeast.

Depth of Water: 3.9 m to 4.6 m

Water Level Fluctuation: Water levels may vary by 1.5 m depending on season.

Bottom Soil Conditions: Soft organic.

Wind Climate: Storm direction is southeast, however, the predominant wind direction is southwest.

Fetches: Southeast fetch 5-6 km. Southwest fetch 1.9 km.

Waves: No recorded waves available. Most severe waves come from southeast during which time the breakwater reflects waves into marina. Highest observed southwest waves 0.9 m. No period data available.

Shipwaves: Wake of vessels using Granonoque Harbour is a problem.

Currents: Steady 0.4 to 0.3 m/s current from west perpendicular to breakwater.

Ice: Ice forms in winter. Breakwater is left in during winter. Ice tends to buildup and submerge breakwater. Refloats in the spring.

Breakwater Description: The breakwater is an inverted A-frame breakwater. The construction is similar to the Lund and Queen Charlotte City breakwaters, however, the steel framework bracing the centreboard to the outer pontoons is inverted and now below the water surface. This leaves an uncluttered surface above water. Decking has been placed on the leeward side for decking.

Dimensions: Four modules, each = 15.3 m x 8.5 m
Diameter of pontoons = 0.92 m
Draft (including centreboard) = 3.2 m
Freeboard (including centreboard) = 1.7 m
Mass of breakwater = Approximately 3,250 kg/m (including buoyancy of the centreboard)

Construction Site: Not available

Connections: Modules cross-connected with 15 kg/m chain. Tire fenders placed between modules at ends of pontoons.

Reserve Buoyancy: Watertight compartments

Provision for Corrosion: Steel painted with a bituminous epoxy coating.

Inspection Program: Inspected annually. Marina operator maintains decking.

Notes: Original concept and inverted configuration model tested. No field measurements conducted.

Mooring Details:

Anchor System: 1.8 m x 1.8 m x 1.5 m unreinforced concrete blocks with 51 mm steel rod embedded for chain connection. Anchor chains 15 kg/m. Anchor chains connected to apex of A-framework. Anchors buried in organic silt.

Mooring Layout: Four anchors per module on windward side. Two anchors per module on leeward side. One anchor from each end of a pair of modules.

Maintenance Requirements:

Structural: Occasional pumping. Never needed major repairs.

Connections: Minor.

Moorings: Possibility that breakwater has dragged moorings in direction of river current.

Performance: The breakwater performs very well for southwest waves. Due to the lack of protection to the southeast, the breakwater increases wave agitation in the harbour by reflection. Upgrading of marina facilities has been undertaken partially as a result of the additional protection provided by the breakwater.

Costs: $1115/m (Total cost, 1968)

Maintenance costs are unavailable but minor.

Notes: Soil would not support conventional breakwater at this location.

Location: THUNDER BAY, ONTARIO

Breakwater Type: Inverted A-Frame

Location Reference: Canadian Hydrographic Services' Chart No. 2301, 2314

Year Constructed: 1968

Contact: Public Works Canada, Ontario Region, Toronto, Ontario

Site Description: The breakwater is located inside the main harbour of Thunder Bay and is protected from Lake Superior by concrete and rubble mound breakwaters. The site is exposed to waves within the harbour from the east and north-east.

Depth of Water: 3.1 m to 6.2 m below chart datum.

Water Level Fluctuation: Lake Superior water levels vary up to 1 m depending on the season.

Bottom Soil Conditions: Not available

Wind Climate: Details not available. Predominant wind directions are southwest and northwest.

Fetches: Maximum fetch to the northeast of 3.2 km.

Waves: Maximum observed wave height estimated to be 0.5 m.

Shipwaves: Not considered in design, not a problem.

Currents: Not a problem.

Ice: Breakwater is locked in by ice during the winter. Has not been a problem.

Breakwater Description: The breakwater is an inverted A-frame breakwater similar to the floating breakwater at Gananoque.

Dimensions: Six modules, each = 18.3 m x 8.5 m

Diameter of pontoons = 0.92 m

Draft (including centreboard) = 3.2 m (3 modules)

Freeboard (including centreboard) = 1.7 m

Mass of breakwater = Approximately 3,250 kg/m (incldiung buoyancy of centreboard)

Construction Site: Thunder Bay

Connections: Modules are cross-connected with 15 kg/m chain. Tire fenders placed between modules at ends of breakwater.

Reserve Buoyancy: Watertight compartments.

Provision for Corrosion: Steel painted with a bituminous epoxy coating.

Notes: Original concept and inverted configuration model tested. No field measurements conducted.

Mooring Details:

Anchor System: 1.8 m x 1.8 m x 1.5 m unreinforced concrete blocks with 51 mm steel rod embedded for chain connections. Anchor chains 15 kg/m. Anchor chains connected to apex of A-framework.

Mooring Layout: Four anchors per module on windward side. Two anchors per module on leeward side. One longitudinal anchor for group of three modules.

Maintenance Requirements:

Structural: None, Pontoons will require re-painting in near future.

Connections: None reported.

Moorings: Minor repairs

Performance: The users of the marina are satisfied with the protection provided by the breakwater. The leeward pontoon is often used as a berth for boats.

Costs: $1122/m (total cost, 1968)

Note: Design notes indicated a lack of information available at the time to determine:

a) Scope of anchor chains

b) Stresses on members and welds due to ice and temperature variations.

Depth of water made floating breakwater more economical than conventional breakwater.

FLOATING LOG BUNDLES

FLOATING LOG BUNDLES

Note: Floating log bundle breakwaters are extensively used in British Columbia. This section summarizes the locations of the major structures and known experience with log bundles.

Information From: Public Works Canada, Pacific Region, Vancouver, B.C.

Locations:

Queen Charlotte City: Fetch = 4.4 km to southwest. 98 m of three-five marine piling bundles chained together. Styrofoam billets included in bundles. Pilings creosoted.

Kelowna: Constructed in 1978. 238 m of three-seven log bundles. Core of each bundle contains polyurethane billet. Bundles tied together with wraps of number 25 steel chain secured with turnbuckles.

Anchored with two, 1.25 m x 1.25 m x 1.25 m reinforced concrete anchors per 34 m of bundles, each side.

Polyurethane core breaks up with flexing of bundles.

Bedwell Harbour: 128 m of floating log bundles. Details not available.

Fairview Bay, Prince Rupert: 585 m of floating log raft consisting of five 0.92 to 1.22 m diameter logs. Breakwater protects coop fishing terminal.

Site is exposed. Fetch not available.

Storm damage experienced on floats.

Rushbrook, Prince Rupert: 390 m of five log bundles.

Cost $650/m, data not available.

Browning Harbour: 106 m of five log bundles. Details not available. Cost $405/m.

Reed Point Marina, Port Moody: Breakwater located east end of Burrard Inlet.

Exposed to west and north. Local chop only and tug and large ship traffic.

990 m of three-seven log bundles. Core of two bundles alternates log and polyurethane core. Four external billets of polyurethane strapped to each section of bundles. Section 48.8 m and 34.2 m long.

Cost, 1976, $488/m.

Anchored with three, each side 1.2 m x 1.2 m x 1.2 m reinforced concrete anchor blocks per section.

Breakwater performs well with exception of shipwaves.

Maintenance - untreated logs lose buoyancy.

FLOATING TIRE BREAKWATERS INCLUDING

GOODYEAR FLOATING TIRE BREAKWATERS, "WAVE MAZE" FLOATING TIRE BREAKWATERS, AND POLE-TIRE FLOATING TIRE BREAKWATERS

Location: CATAUMET, MASSACHUSETTS

Breakwater Type: Floating Tire

Location Reference: Not available

Type of Harbour: Private Marina

Year Constructed: 1975

Contact: Kingman Marine Incorporated, Cataumet, Massachusetts

Site Description: The breakwater is located on the east side of Red Brook Harbour and is exposed to the southwest and west. The breakwater protects a single row of docks from the prevailing southwest winds. A small creek flows into the basin on the lee of the breakwater.

Depth of water: 2.4 m below low water

Tide Range: 1.2 m (mean tide)

Bottom Soil Conditions: Mud and sand

Winds: Southwest prevailing winds and storms. Winds 56 km/hr to 81 km/hr. Tail end of hurricanes produce less frequent but more severe storms.

Fetches:

Waves: Maximum observed, west 0.62 m, period 4 sec. maximum. Storm surge associated with hurricane in 1976 overtopped islands limiting fetch. Seas of height 1.83 m encountered for 1/2 hour. Switched to swells 7 secs or longer. Breakwater prevented extensive damage.

Currents: 0.13 m/s to 0.26 m/s

Ice: Ice in winter. Breakwater is left in the water. No damage.

Breakwater Description: The breakwater is a three module wide Goodyear floating tire breakwater.

Dimensions: 21.4 m x 6.4 m

Construction Site: Cataumet Marina

Connections: Present connections rubber conveyor belting and nylon bolts. Originally stainless steel cable and chain used, but both materials corroded and failed.

Reserve Buoyancy: Originally none. Now tires filled with urethane foam.

Provision for Corrosion: Details not available.

Inspection Program: Not available.

Mooring Details:

Anchor System: Seaward: Concrete blocks 1,816 kg
Landward: Concrete blocks 454 kg

Mooring Layout: Seaward: 3 blocks
Landward: 2 blocks
Spacing not available.

Maintenance Requirements:

Structural: Both connector and buoyancy were insufficient in original design. Breakwater sank from accumulation of sediment and marine growth. Tires foamed. Connectors replaced with conveyor belting and nylon bolts.

Moorings: None

Performance: Performance has been satisfactory. Application for 275 m extension pending in 1980.

Costs: Initial cost $64/m (1975). Anchors obtained free.
Repairs (Connections) $88/m (1977)
Manhours to build, 86.5 hours

Location: DIVERSEY HARBOUR, MICHIGAN

Breakwater Type: Floating Tire

Location Reference:

Type of Harbour: Small craft marina

Year Constructed: 1978

Contact: Chicago Park District

Site Description: Breakwater protects entrance to Diversey Harbour and is fully exposed to waves on Lake Michigan.

Depth of Water: Unknown

Water Level Fluctuation: Unknown

Bottom Soil Conditions: Unknown

Winds: Unknown

Fetches: Unknown

Waves: Unknown

Shipwaves: Unknown

Currents: Unknown

Ice: Unknown

Breakwater Description: Goodyear floating tire breakwater.

Dimensions: 91.4 m x 8.5 m

Mass of breakwater = unknown

Construction Site: Unknown

Connections: Unknown

Reserve Buoyancy: Tire partially filled with polyurethane foam.

Inspection Program: Unknown

Notes: Bridle around module of galvanized steel chain, maximum working load 20,017 N, weight 4 kg/m.

Mooring Details: Twenty concrete anchors, 7.6 m apart on east (exposed) side and 15.2 m apart on west (leeward) side. Load distributed over two modules.

Performance: Breakwater was moved in the first storm and it was concluded that mooring was insufficient. The breakwater was not wide enough to provide the intended protection. Polyurethane flotation lost buoyancy due to water absorption. Breakwater did not provide protection against long period waves which were damaging boat slips in harbour.

Costs: $833/m (US$, 1978)

Notes: This experimental breakwater was found to be unsatisfactory for this location. Design data for breakwater was supplied by Engineering Research Department, Research Division, Goodyear Tire and Rubber Company, Akron, Ohio.

Location: NEWINGTON, NEW HAMPSHIRE

Breakwater Type: Floating Tire

Location Reference: NOAA Chart No. 13285

Type of Harbour: Private Marina

Year Constructed: 1976

Contact: Great Bay Marina, Newington, New Hampshire

Site Description: The breakwater is located on the south side of Little Bay, part of a tidal estuary, having strong tidal currents. The breakwater protects the marina from a northwest exposure and is located off the end of a rubble breakwater serving a similar purpose.

Depth of Water: 2.7 m to 9.2 m at low water

Tide Range: 2.9 m (mean tide)

Bottom Soil Conditions: Mud and clay

Winds: Storms blow regularly from the northwest, during the winter two to four days per week and during the boating season for a total of two to three weeks. Storm duration is typically one day with speeds of 48 km/hr.

Fetches:

Waves: Maximum observed 0.92 m. No period information available.

Shipwaves: Not a problem

Currents: 3.1 m/s flood and ebb.

Ice: Large flows at spring break-up regularly snap pilings on marina. Breakwater stored in lee of rubble breakwater.

Breakwater Description: Breakwater is a three module Goodyear type floating tire breakwater.

Dimensions: 45.8 m x 6.4 m

Construction Site: Great Bay Marina

Connections: Tires and modules connected with 50 mm wide 0.75 mm conveyor belt edging. Nylon (12.5 mm diameter) bolts, nuts, and washers used. Details of number per connection not available.

Reserve Buoyancy: None

Provision for Corrosion: Nylon bolts dyed black. Dye has worn off all bolts. No problem encountered to date.

Inspection Program: Operator on site.

Mooring Details:

Anchor System: Granite blocks, 1,589 kg. Details of chain not available.

Mooring Layout: Details not available.

Maintenance Requirements:

Structural: None reported. Debris accumulates and must be cleaned off.

Connections: None reported.

Moorings: Anchors dragged om currents. Breakwater has a large curve reverses with current. Only half the designed protection available. Insufficient time available at slack water to work. No information available to size anchors.

Performance: Breakwater has provided good protection. Breakwater regarded as temporary strucutre.

Costs: Not available.

Location: NORTH VANCOUVER, BRITISH COLUMBIA

Breakwater Type: Floating Tire Breakwater

Location Reference: Canadian Hydrographic Services' Chart No. 3482

Type of Harbour: Private yacht club

Year Constructed: 1978, deployed in 1979, destroyed in 1980.

Contact: Burrard Yacht Club, North Vancouver, British Columbia

Site Description: The yacht club is located on the north shore of Vancouver Harbour. The breakwater protected docks and boat houses.

Depth of Water: 21 m

Tide Range:

Bottom Soil Conditions: Light consolidated silt.

Winds: Predominant storm direction is east southeast and southwest. Winds are strong enough to cause problems in the marina approximately four to five times per year.

Fetches: Southeast 3.1 km, South 2.4 km, Southwest 3.0 km.

Waves: Not available

Shipwaves: Constant ship traffic. Maximum observed shipwave 0.76 m, period 6 sec.

Currents: Maximum 1 m/s. Parallel to breakwater.

Ice: None

Breakwater Description: The breakwater is three module wide Goodyear floating tire breakwater. Tires are bound together using scrap conveyor belting bolted with nylon bolts and washers.

Dimensions: Not available

Construction Site: Built by volunteer labour.

Reserve Buoyancy: Crest of tires foam filled.

Connections: Two nylon bolts per connection.

Inspection Program: Details not available.

Mooring Details:

Anchor System: Eight 227 kg Danforth anchors. Nylon belting anchor rode. Anchor rode connected through three tire modules with double loop belting.

Mooring Layout:

Maintenance Requirements:

Structural: None reported

Connections: Nylon bolts for inter module connection failed.

Moorings: Large drag in tidal current. Dragged 0.5 tonne anchors with 15 m of 32 mm chain and 180 kg sinkers. Replaced 12 tonne blocks which held.

Performance: Breakwater adequate when on location, however, moorings constantly broke. Transmitted shipwaves still cause some maintenance problems to the dock and boat houses. Yacht club has purchased two wooden barges which will be sunk to a draft of 4.58 m to act as a new breakwater.

Costs: Not available.

Location: MAMARONECK, NEW YORK

Breakwater Type: Pole-Tire Floating

Location Reference: Not available

Type of Harbour: Private Yacht Club

Year Constructed: 1980

Contact: Mamaroneck Beach and Yacht Club, Mamaroneck, New York.

Site Description: Breakwater is located at entrance to a small bay exposed to the south and southeast to Long Island Sound.

Depth of Water: 1.37 m (mean low water)

Tide Range: 2.23 m (mean tide)

Bottom Soil Conditions: Sand

Winds: Predominant southeast and southwest winds during summer. Remnants of hurricanes in September produce major storms from southeast. Wind data available from local observation and La Gaurdia Airport, N.Y.C.

Fetches: Southeast 8.5 m; South 11.0 km

Waves: No recorded data. Design based on hindcast procedure, 20 year return period. Significant wave height of 1.1 m to 1.8 m depending on tide. Peak period 4.9 secs. Wave height limited by water depth at low water.

Shipwaves: Not a problem

Ice: Some ice.

Note: Application pending (September 1980) to use breakwater year round.

Breakwater Description: The breakwater is a Pole-Tire breakwater developed by Dr. V.W. Harms, details available elsewhere. This installation is the first field prototype installation of this type of floating breakwater. The breakwater is constructed from nineteen modules; each module consisting of ten rows of conveyor belt edging (153 mm wide, 12.5 mm thick) on to which truck tires are threaded. The belting is connected at right angles to steel pipes (12.2 m long, 0.46 m outside diameter) spaced at 3.66 m, centre to centre. The steel pipes are threaded with truck tires and are foam filled.

Dimensions: 69.5 m x 12.2 m

Draft: Varies depending extent of marine growth, diameter of tires and pipe used. Details of draft at MBYC not available.

Mass of breakwater = Not available.

Construction Site: Modules assembled on land at Mamaroneck Beach and Yacht Club, modules connected in the water and towed to site.

Connections: Conveyor belting connected with 12.5 mm diameter nylon bolts and nuts and washers. Number of bolts per connection not available.

Reserve Buoyancy: Steel pipes foam filled. Reported that only one half of pipes were filled with foam. No foam in tires. Air trapped in tires.

Provision for Corrosion: Interior of pipes coated with diesel oil.

Inspection Program: Details not available.

Mooring Details:

Anchor System: Four 2,724 Kg Navy Anchors and sixteen 2, 270 kg concrete blocks on seaward side. Connected to pipes by tire shock absorbers. Approximate six automobile tires per absorber. Tires connected with 153 mm wide by 12.5 mm thick conveyor belting, five 12.5 mm diameter nylon bolts per connection. Details of anchor rode not available.

Mooring Layout: Seaward: One anchor per steel pipe

Landward: No details available.

Maintenance Requirements: None to date.

Performance: Breakwater has not been subjected to any storms yet. Breakwater performed very well in squall with 0.9 m observed waves. Tires and poles reported to rotate sufficiently during operation that tires remain free of marina growth. Insufficent compression applied when tires strung on pipe. One meter of pipe now exposed.

Costs: $575/m (Approximate total, 1980), including materials, labour, construction, and installation.

Location: PICKERING BEACH, DELAWARE

Breakwater Type: Floating Tire

Location Reference: Not available.

Type of Harbour: Part of US Army Corps Shoreline Erosion Demonstration Project.

Year Constructed: 1978

Contact: US Army Corps of Engineers, Philadelphia District, Philodelphia, Pennsylvania.

Site Description: The breakwater is located on the west side of Delaware Bay, 214 m seaward of a beach nourishment project.

Depth of Water: 0.61 m at low water

Tide Range: 1.5 m mean tide

Bottom Soil Conditions: Silt and mud

Winds: Not available

Fetches: East: 24.0 km, Southeast: 48 km, Other fetches not available.

Waves: Maximum design wave height 1.83 m, period 4 sec.

Shipwaves: Not a problem

Currents: 0.52 m/s parrallel to breakwater

Ice: In winter. Usually thin ice. Recent years thick ice.

Breakwater Description: Two type of floating tire breakwaters were built at Pickering Beach, a Wave Maze breakwater and a Goodyear tire breakwater. The Wave Maze breakwater was damaged and discarded.

Goodyear breakwater: Two sizes of Goodyear breakwaters were built. Sixty-two m of a three module breakwater and 62 m of a six module breakwater.

Dimensions: Three module: 62 m x 6.4 m

Six module: 62 m x 12.8 m

Construction Site: Constructed onshore of Pickering Beach.

Connections: Tires and modules connected with 50 mm wide, 9.5 mm thick rubber conveyor belt edging. Three 12.5 mm diameter nylon nuts, bolts, and washers per connection. Continuous 12.5 mm diameter polypropylene perimeter line.

Reserve Buoyancy: All tires filled with 0.23 kg of polyurethane foam.

Provision for Corrosion: Nylon belts dyed black for ultraviolet resistance.

Inspection Program: Inspected monthly.

Mooring Details:

Anchor System: Breakwater originally moored with 1.53 m c 1.22 m x 0.69 m reinforced concrete blocks, with 12.5 mm welded steel anchor chain. Concrete blocks replaced with timber stake piles, 0.31 m diameter, 7.63 m long. Anchor chain connected to breakwater by looping through the five tires which form the intermodule connections on the Goodyear design.

Mooring Layout: Both breakwater: Seaward: 7 anchors spaced at 8.34 m. First anchor in line 5.80 m from end of breakwater.

Landward: 4 anchors spaced 14.6 m, First anchor in line 8.85 m from end of breakwater.

Two chains per anchor block. Connection to breakwater spaced 8.34 m apart.

Maintenance Requirements:

Structural: In July 1980, one intermodule connection had failed. Not possible to determine if a weak slashed tire or the connection had failed. A random check of 20 tires around the windward and leeward perimeter of the breakwater showed that 60% no longer had foam in the crest. A further 10-20% of the tires had only loose pieces of foam wedged in the crest. Examination showed that the foam was being worn by abrasion on the tire casing. The foam had originally been sprayed into the tires with the intent it would bond to the tire. This does not appear to be happening. A slurry of fine sediment was trapped in the base of each tire. Weight of sediment slurry was not determined. Marine growth on the tires amounts to approximately a 12.5 mm thick coating of shells and seaweed. Not possible to determine if breakwater was sinking.

Connections: One failure as noted above. Details not available.

Moorings: During a storm with the following observed conditions: (LEO) wind east 48 km/hr, breaking wave height 0.76 m, tide range 1.65 m to -0.09 m during storm, the Goodyear breakwater dragged its seaward anchors. Concrete anchors replaced with stake piles.

Performance: Breakwater is monitored for its performance as a Shore Erosion Protection Device. No information on transmitted wave heights or performance with this regard is available. A small mud tombola is forming behind the breakwater, however, breakwater appears too far from the beach to have any significant impact on shore erosion.

Costs: Estimated total cost, 6 module wide breakwater \$895/m

Estimated total cost, 3 module wide breakwater \$540/m

Notes: Information based on data supplied by US Army Corps of Engineers and a field visit by Hydrotechnology. Conclusions drawn are Hydrotechnology's.

Location: PICKERING BEACH, DELAWARE

Breakwater Type: Floating Tire

Note: Location and Site Description details same as previous breakwater.

Breakwater Description: Wave Maze Breakwater: Sixty-two meters of a 12.6 m wide and 62 m of a 5.87 m wide Wave Maze breakwater were constructed at Pickering Beach in conjunction with the Goodyear breakwater described earlier. The breakwaters were constructed from .36 m to .38 m automobile tires.

Dimensions: Wide breakwater = 15 modules = 62 m x 12.6 m
Narrow breakwater = 7 modules = 62 m x 5.87 m

Construction Site: Assembled at Pickering Beach

Connections: 15.3 m of narrow breakwater connected using black nylon bolts (12.5 mm diameter), nuts, and washers, backed by 100 mm x 100 mm x 9.5 mm (min) rubber patches.

Remainder of Wave Maze connected using galvanized steel bolts (12.5 mm diameter), nuts, and two 50 mm outer diameter washers.

Reserve Buoyancy: All vertical tires in module filled with .45 kg of polyurethane foam.

Inspection Program: Inspected monthly.

Mooring Details:

Anchor System: Reinforced concrete anchors, 1.53 m x 1.22 m x 0.69 m
Anchor chain 12.5 mm welded steel chain.

Mooring Layout: Seaward: 7 anchors, spaced 8.79 m, First anchor 4.56 m from breakwater end.

Landward: 4 anchors, spaced 15.4 m, First anchor 7.63 m from breakwater end.

Two chains per anchor block. Connections to breakwater spaced 8,34 m apart.

Maintenance Requirements:

Structural: Sections connected by galvanized steel bolts broke up during winter. Failure attributed to ice and waves overstressing bolts. Bolts pulled through tires. Sections constructed with nylon and rubber patches were not damaged.

Moorings: Anchors dragged in same storm as described for Goodyear breakwater.

Performance: Wave Maze breakwaters were discarded after damage occurred.

Costs: Estimated total cost 12.6 m wide breakwater $1,236/m

Estimated total cost 5.87 m wide breakwater $703/m

Notes: Information based on data supplied by US Army Corps of Engineers.

Conclusions drawn are Hydrotechnology's.

Location: RIVIERE AU RENARD, QUEBEC

Breakwater Type: Floating Tire

Location Reference:

Type of Harbour: Fishing

Year Constructed: 1977

Contact: Public Works Canada, Headquarters, Ottawa, Ontario

Site Description: The breakwater is located inside the harbour and protects a marginal wharf from waves generated in the harbour. Longer period waves diffract into harbour through entrance.

Depth of Water: 4.6 m below chart datum

Tide Range: 1.8 m

Bottom Soil Conditions: Sand

Winds: Design wind 102 km/hr, National Building Code

Fetches: 1.22 km across harbour

Waves: Hindcast from SMB charts. Significant wave height 0.6 m, peak period 2.4 sec. No recorded wave data.

Shipwaves: Not a problem

Currents:

Ice: Ice in winter. Removed in winter.

Breakwater Description: Goodyear floating tire breakwater. Three modules wide. Originally two breakwaters constructed 186 m x 6.1 m and 135 mm x 6.1 m. In 1980, breakwaters tied together to form breakwater 147.2 m x 12.2 m (six modules wide).

Construction Site: Assembled in Quebec City, transported to Riviere au Renard by boat.

Connections: Modules assembled with 19 mm nylon rope. Nylon tied with double sheet bend and bow line and whipped. Perimeter chaim 9.5 mm trade chain with shackles.

Reserve Buoyancy: Four tires per module fitted with polyethylene foam (foam wedged into tire) cylinder (0.92 m x 0.15 m diameter)

Inspection Program: Annual

Note: Design based on University of Rhode Island, Marine Bulletin 21. Insufficient detail for design. Connections designed to lift breakwater from water.

Mooring Details:

Anchor System: 68 kg Danforth anchors with 0.94 m x 0.94 m x 0.31 m reinforced concrete clump weights connected to breakwater with 9.5 mm trade chain.

Mooring Layout: Anchor chains crossed under breakwater. Anchors spaced 12.8 m. No longitudinal anchors.

Maintenance Requirements:

Structural: Shackles on perimeter chain being stolen. Modules separating. Fishing boats get stuck on breakwater at night. Cut ropes to get off. Modules separating.

Connections: None reported other than noted above.

Moorings: None reported. Difficult to locate anchors in spring.

Performance: Local fisherman did not like breakwater. It was hard to see at night and was a hinderance to vessel movement. It is likely that waves diffracting into harbour are greater problem than waves generated across harbour.

Costs: \$361/m (Contract cost, 1977)

Construction took two to three weeks

Installation took one week

Notes: Floating tire breakwater was selected for ability to relocate if necessary. Breakwater was also to be learning experience and low cost was attractive for this reason.

REINFORCED CONCRETE CAISSON BREAKWATERS

Location: FRIDAY HARBOUR, WASHINGTON

Breakwater Type: Reinforced Concrete Caisson

Location Reference: Not available

Type of Harbour: Oceanographic Research Facility

Year Constructed: 1978

Contact: University of Washington, Seattle, Washington

Site Description: Breakwater is located on north side of Friday Harbour and is exposed to the east and south. Friday Harbour is used by recreational and fishing vessels and is served by large ferrys every 2 hours.

Depth of Water: 9.2 m to 2.4 m

Tide Range: 2.3 (mean Higher high water)

Bottom Soil Conditions:

Winds: Storms from southeast and northeast. Designed for 40 minutes duration of 174 km/hr northeast. Most frequent storms from southeast.

Fetches: Northeast 2.2 km, East 6.5 km, Southeast 1.9 km, South 1.3 km.

Wave: No recorded data. Design wave H = .9 m, T = 3.5 sec.

Shipwaves: Frequent shipwaves 0.3 to 0.6 m.

Currents: 1.5 knots parallel to breakwater

Ice: None

Breakwater Description: Breakwater has L-shaped layout with both legs aligned at 45° to longest fetch. Regular weight reinforced concrete caissons. Three caissons per module post-tensioned together. Typical wall thickness 121 mm. Welded wire fabric reinforcing with #8 and #5 bar. Cover not available.

Dimensions: Three modules, each = 40 m x 4.6 m x 1.4 m

Draft = 1.1 m

Individual caissons = 13.2 m x 4.6 m x 1.4 m

Mass of breakwater = 1,128 kg/m^2

Construction Site: Not available. Towed to site.

Connections: None. Each caisson individually moored. Linked by gangway.

Reserve Buoyancy: Concrete formed over styrofoam billets.

Provision for Corrosion: Details not available.

Inspection Program: Wharfinger on location.

Mooring Details:

Anchor System: Reinforced concrete anchor blocks, 1.6 m x 2.0 m x 2.1 m.

Anchor chain 25 mm stud link chain with 1.4 m x 1.4 m x 0.9 m reinforced concrete clump weight on chain.

Mooring System: Four anchors per caisson, two each side, splayed at 45° to caisson. Attached at end to caisson. Additional anchors perpendicular to caissons at end of each caisson. Two per caisson.

Maintenance Requirements:

Structural: Forms sprang during construction. Wall thickness uneven and caissons floated unevenly. Styrofoam billets strapped to underside. No problem since.

Connections: Original gangways fixed for rotational degrees of freedom. Breaking from motion. Now modified no problems.

Anchor: A lack of surface soils information resulted in expensive modifications to anchors during construction.

Performance: Breakwater has performed very satisfactorily.

Costs: Not available

Notes: Due to the depth of water a floating breakwater was more economical alternative to a rubble mound structure.

Location: **MAPLE BAY, BRITISH COLUMBIA**

Breakwater Type: Reinforced Concrete Caisson

Location Reference: Canadian Hydrographic Services' Chart No. 3452

Type of Harbour: Private Yacht Club.

Year Constructed: 1977

Contact: Maple Bay Yacht Club, Duncan, British Columbia

Site Description: The breakwater is located at the southern end of Maple Bay and is exposed to winds blowing down Sansum narrows and across Maple Bay. The breakwater is aligned parallel to the shoreline contours and protects approximately 180 vessels. Thirty-six vessels moor to lee side of breakwater alongside finger piers that are integral part of caissons.

Depth of Water: 13.7 m below chart datum

Tide Range: 2.7 m (mean tide); 4.2 m (large tide)

Bottom Soil Conditions: Silt and Rock

Wind Climate: Designed for five year storm from north to north northeast.

Fetch: North 1.85 km; Northeast 4.3 km; East 0.5 km.

Waves: No recorded wave data. Five year design storm criteria: significant wave height 0.6 m, peak period 3 seconds. Design criteria: to provide 0.3 m to 0.4 m significant wave height at lee of breakwater for these conditions.

Shipwaves: Minor problem.

Currents: .78 m/s parallel to breakwater, either direction.

Ice: None

Breakwater Description: The breakwater consists of eight pre-stressed reinforced (wire mesh) concrete caissons, compartmentalized and filled with styrofoam. Wall Thickness = 50 mm. The caissons are connected to form one continous breakwater. The breakwater is ballasted with seawater. Mooring fingers on the leeward side at spacing of 8 m.

Dimensions: Seven caissons, each 22. 3 m x 4.6 m x 1.2 m, with 7.6 m 1.1 m x 1.2 m fingers on leeward side. The caissons are bolted together to form a continous structure 156 m in lenght. Mass of breakwater = Not available.

Construction Site: Richmond, B.C., towed to and assembled at Maple Bay.

Connections: Four 50 mm bolts per caisson. Bolts are set in a neoprene rubber sleeve, accessible from deck for tightening.

Reserve Buoyancy: Caissons filled with styrofoam to provide 1.0 m freeboard.

Provision for Corrosion: Prestressing tendons set in polyethene sheath prestressed to prevent concrete cracking.

Inspection Program: Inspected annually, wharfinger in attendance.

Mooring Details:

Anchor System: 4.8 m stud piles to seaward. Concrete anchor blocks and stud piles towards shore depending on depth of overburden. Design anchor load = 4,086 N/m.

Seaward anchors field tested to 80 k. Landward anchors field tested to 44.5 kN.

Mooring Layout: One anchor each side per caisson. Scope of windward side 6:1. Scope of leeward side 3:7:1.

Maintenance Requirements:

Structural: Minor repairs

Connections: Occasional tightening of bolts required.

Moorings: None

Performance: The breakwater has met the expectations of the owners and has provided satisfactory protection for vessels mooring behind the breakwater.

Costs: $1,540/m (Total cost, 1977)

Notes: Depth of water made the floating breakwater more economical than conventional breakwater.

Insufficient information exists to define concrete cover for prestressing that is required in this marine environment.

Location: **NANAIMO, BRITISH COLUMBIA**

Breakwater Type: Reinforced Concrete Caisson

Location Reference: Canadian Hydrographic Services' Chart No. 3457

Type of Harbour: Private, Yacht Club

Conatct: Nanaimo Yacht Club, Nanaimo, British Columbia

Site Description: The breakwater is located within Nanaimo Harbour and is exposed to the east and south.

Depth of Water: 0.8 m to 4.4 m

Tide Range: 3.1 m (mean tides), 4.9 m (large tides)

Bottom Soil Conditions: Soft, silty bottom

Winds: Frequent storms from southeast

Fetches: Fetch varies depending on stage of tide. At high water, East 1 km, Southeast 5.3 km.

Waves: Details not available.

Shipwaves: Constant ship traffic.

Currents: 0.6 m/s current either direction

Ice: None

Breakwater Description: The breakwater consists of three reinforced concrete (wire mesh) caissons, Wall thickness = 50 mm.

Dimensions: Mass of breakwater = 390 kg/m^2

Construction Site: Richmond, B.C., and towed to site.

Connections: Bolts with rubber pads in between.

Reserve Buoyancy: Hollow

Provision for Corrosion:

Mooring Details:

Anchor System: Piles on perimeter of breakwater.

Mooring Layout: Two, Six pile dolphins per caisson on seaward side only.

Maintenance Requirements:

Structural: Pumping required in 1980 for first time.

Connections: None

Moorings: None

Performance: Users are pleased with breakwater. Intend to expand the breakwater in future.

Costs: Not available.

Notes: Floating breakwater required because of concern for possible siltation from fixed breakwater.

Location: NORTH VANCOUVER, BRITISH COLUMBIA

Breakwater Type: Reinforced Concrete Caisson

Location Reference: Canadian Hydrographic Services' Chart No. 3482

Type of Harbour: Private Marina

Year Constructed: Installed at North Vancouver in 1977, Previous History of caissons - unknown.

Contact: Public Works Canada, Pacific Region, Vancouver, British Columbia

Site Description: The breakwater is located in Vancouver Harbour on the north side of the harbour and protects approximately 500 boats.

Depth of Water: 3.1 m to 12.2 m

Tide Range: 4.9 m, large tide

Bottom Soil Conditions: Mud and silt.

Winds: Storms blow from the east and southeast across the harbour. Winds are strong enough to cause problems in the marina approximately four or five times per year.

Fetches: Southeast 3.1 km, South 2.4 km, Southeast 3.0 km.

Waves: Details not available.

Shipwaves: Constant ship traffic past the marina produce up to 0.6 m waves.

Currents: Not a problem

Ice: None

Breakwater Description: Nine compartmentalized reinforced concrete caissons.

Dimensions: 20.9 m x 7.2 m x 2.3 m

Mass of breakwater = 1,876 kg/m^2

Construction Site: Details not available

Connections: Cross-connected with 32 mm diameter chains and with truck tire fenders between caissons.

Reserve Buoyancy: Compartments filled with styrofoam

Provision for Corrosion: No details available

Inspection Program: Annual inspection, wharfinger at marina.

Mooring Details:

Anchor System: Reinforced concrete blocks

Seaward side: 1.8 m x 1.8 m x 1.5 m

Landward side: 1.8 m x 1.8 m x 1.5 m

Anchor chain: 38 mm diameter

Mooring Layout: Two anchors per side each module. Anchors buried in mud and silt.

Maintenance Requirements:

Structural: Corners of caissons heavily damaged in previous location. Repaired and cross-connected.

Connections: None reported.

Moorings: None reported.

Performance: Breakwater does not attenuate shipwaves as much as is desired. Otherwise, breakwater is satisfactory. Breakwater prevent debris entering the harbour.

Costs: $2,066/m (Total cost, 1976)

Note: Depth of water made floating breakwater more economical than conventional breakwater.

Location: NORTHWEST DISTRICT, US ARMY CORPS OF ENGINEERS

Breakwater Type: Reinforced Concrete Caisson

Location Reference: Breakwater is design stage for Brownsville, East Bay, and Friday Harbour. Design for Elliot Bay, Seattle to begin 1980.

Type of Harbour: Small craft marinas, public facilities

Year Constructed: Designed 1980

Site Description: Breakwaters are being designed for Brownsville, East Bay, and Friday Harbour which are all fetch limited locations. Breakwaters are all integral part of extensive marina facilities. Comments below describe the design process.

Note: A field program is planned for Elliot Bay, Seattle. The breakwater is located in the entrance to the main harbour for Seattle, Washington and shipwaves will be a major problem.

Depth of Water: East Bay = 15.3 m below MLLW (typical)
Friday Harbour = 3.0 m to 8.5 below MLLW
Brownsville = 3.0 m (typical) below MLLW
Elliot Bay = 38 m (typical) below MLLW

Tide Range: Typical 2 to 4.5 m (MHHW) all sites

Bottom Soil Conditions: East Bay = soft to very soft silt
Friday Harbour = Not available
Brownsville = not available
Elliot Bay = not available

Winds: Wind velocity versus duration curves prepared from nearest airport recorded data. Design wind 1/2 hour duration to fetch limiting duration.

Fetches: In general, sites fetch limited
East Bay = fetch varies from 3.5 km to 12 km depending on method used (direct vs. effective fetch)
Friday Harbour = details not available
Brownsville = details not available
Elliot Bay = 10.3 km (based on 24 degree effective fetch fan)

Waves: Waves hindcast for design wind conditions. In general significant wave height less than 0.9 m, peak period less than 3.5 secs.

Notes: Structural design based on 10 percent exceedance wave for same wind conditions. Width selected to permit 0.3 m transmitted wave under the same wind conditions.

Shipwaves: Shipwaves will be measured at Elliot Bay. Width of breakwaters at Friday Harbour and Elliot Bay based on shipwaves. Not a problem at other sites.

Currents: Not a problem

Ice: None

Breakwater Description: The proposed breakwaters are all post-tensioned (6 tendon per caisson) reinforced concrete caissons. Caisson walls are 120 mm thick with welded wire fabric reinforcing. Cover not available. Two widths of caissons are proposed 4.9 m and 6.4 m.

Dimensions: Caissons = 30.5 m x 4.9 m or 6.4 m x 1.5 m
Draft = 1.07 m
East Bay = 215 m of breakwater
Friday Harbour = 488 m of breakwater
Elliot Bay = 625 m approximately of breakwater
Brownsville = 460 m of breakwater
Mass of breakwater = 1,097 kg/m^2 (based on draft of 1.07 m)

Construction Site: Not available. In general towed site.

Connections: Not final. Field program requested to determine connection loads. Proposed connection include: Steel weldments with rubber doughnuts (port Orchard) and 16 threaded bar tendons per caisson compressing 37.5 mm thick rubber gaskets. Tendons accessible through manhole covers.

Reserve Buoyancy: Positive floatation by polystyrofoam inside caissons.

Provision for Corrosion: Moorings, impressed current cathodic protection.
Fitting, galvanized steel or aluminum plate.

Inspection Program: Details not available. Regularly scheduled inspections planned.

Notes: Design predicated on requirement for access and use of floating breakwater by public.

Mooring Details:

Anchor System: Stake Piles. Details not available. Mooring forces based on Miche-Rundgren analysis and drift force measured in previous field tests. Anchor cable galvanized bridge strand cable with clump weights.

Mooring Layout: Two anchors per caisson each side.

Maintenance Requirements: Not available

Performance: Model tested. Results available.

Costs: Estimated (East Bay) $3,280/m (US$)

Location: RICHMOND, BRITISH COLUMBIA

Breakwater Type: Reinforced Concrete Caisson

Location Reference: Canadian Hydrographic Services' Chart No. 3489

Type of Harbour: Seaplane Base

Year Constructed: 1969

Contact: Public Works Canada, Pacific Region, Vancouver, British Columbia

Site Description: Seaplane base located on middle arm of Fraser River, 2 km south of Vancouver airport. The middle arm is exposed to Strait of Georgia only at high tide. Breakwater protects the mooring floats of several commercial float plane services.

Depth of Water: 0.3 m to 3.5 m

Tide Range: 4.8 m, large tide

Bottom Soil Conditions: Sandy

Winds: Predominant wind direction is from west.

Fetches: West 1.6 km

Waves: Not available

Shipwaves: Not a problem

Currents: Currents in river vary with tide and river discharge. Large debris carried by currents.

Ice: None

Breakwater Description: The breakwater consists of five reinforced concrete (wire mesh) caissons, chained together in a string across the middle arm of the Fraser River. The individual caissons are post-tensioned with 8-12.5 mm diameter cables. The caissons are compartmentalized, exterior wall thickness = 50 mm, interior wall thickness = 25 mm.

Dimensions: Five caissons, each 24.4 m 5.5 m x 1.2 m

Mass of breakwater = not available

Construction site: Richmond, British Columbia, and towed to site

Connections: Chain yoke through veritcal pipe sleeves.

Reserve Buoyancy: Originally no positive floatation. Later injected with urethane foam.

Provision for Corrosion: Caissons post-tensioned to control concrete cracking.

Inspection Program: Not available

Mooring Details: Not available

Notes: Floating breakwater required in order not to produce deposition of river sediment. A low profile for approaching aircraft required.

Maintenance Requirements:

Structural: Extensive damage to the ends of the caissons (smashed bulkheads) was sustained in a number of storms. Damage caused by caissons colliding together. Caissons sank, refloated, and filled with foam. Prior to completion of foaming, one caisson was holed and sank. Refloated. Caissons now awash and need maintenance.

Connections: Connections have required constant maintenance. Heavily damaged when caissons collide.

Moorings: No details available.

Costs: $410/m (Total cost, 1969)

Location: PORT ORCHARD, WASHINGTON

Breakwater Type: Reinforced Concrete Caisson

Location Reference: Not available

Type of Harbour: Public marina

Year Constructed: 1974

Contact: Port of Bremerton, Bremerton, Washington

Site Description: Two breakwaters protect a large public marina on Sinclair Inlet just off Puget Sound. The breakwaters are exposed to the southwest and northeast.

Depth of Water: 10.7 m below low water

Tide Range: Not available.

Bottom Soil Conditions: Not available

Winds: Prevailing wind direction, southerly. Eight to ten storms per year from southeast to southwest. Wind speed from 40 km/hr to 120 km/hr (maximum, 1 hour duration). Occasional northerly storms.

Fetches: Southeast = 3.0 km
North = 0.8 km
Northeast = 6.7 km

Waves: No recorded wave data. Reported most severe waves come from southeast and southwest.

Shipwaves: Minor problem.

Currents: Not a problem

Ice: None

Breakwater Description: Two breakwaters, one L-shaped protecting north and east exposure. Second breakwater protecting west exposure.

L-shaped breakwater consists of a series of lightweight reinforced concrete caissons constructed from 6.4 m long reinforced concrete floats, post-tensioned together. Caissons joined to form a continous breakwater.

Dimensions: L-shaped breakwater = 457 m x 3.7 m, total length
Caissons = 19 m x 3.7 m x .91 m
Floats = 6.4 m x 3.7 m x 91 m
Mass of breakwater = 626 kg/m^3

Construction Site: Not available.

Connections: L-shaped breakwater, Steel weldments bolted each side to caissons and threaded through neoprene fender mounts. Two connections per caisson.

West breakwater, connected with three 75 m x 250 mm timbers.

Reserve Buoyancy: Concrete poured over styrofoam billets.

Provision for Corrosion: None reported

Inspection Program: Harbour Master on location.

Note: Connections designed by considering maximum possible differential displacement of adjacent connections.

Mooring Details:

Anchor System: L-shaped breakwater; stub piles, anchor chain seward side (typical) 7.3 m of chain, diameter = 12.5 mm, 120 m double braided nylon rope, diameter = 22 mm, 9 m chain connected to stub pile.

West breakwater = pilings, details not available

Mooring Layout: L-shaped breakwater: stub piles spaced 13.7 m each side.

West breakwater: not available

Maintenance Requirements;

Structure: None reported.

Connections: One failure of rubber connections linked with anchor failure. Timbers on west breakwater frequently break.

Moorings: One anchor failure. Details not available. Chains needed replacing due to corrosion.

Performance: Satisfactory. Reported shipwaves from larger harbour craft pass through breakwater unobstructed.

Costs: $574/m, (Total cost, 1974)

Note: Depth of water made floating breakwater more economical than conventional breakwater.

REINFORCED CONCRETE PONTOON BREAKWATERS

Location: KETCHICAN, ALASKA

Breakwater Type: Reinforced Concrete Pontoon

Location Reference: Not available

Type of Harbour: Public Marina

Year Constructed: 1980

Contact: Alaska District, U.S. Army Corps of Engineers

Site Description: The breakwater is located on the south side and parallel to a long narrow channel used extensively by fishing boats during the fishing season. A short breakwater is located at the west end of the marina site perpendicular to the shoreline. Service and mooring façilities for the fishing fleet are located on either side of the marina site. Heavily loaded vessels operating at speeds up to 22 km/ hr use the channel. One hundred vessels/day in fishing season past breakwater.

Depth of Water: 18.3 m

Tide Range: 6.1 m

Winds: Predominant storm direction from west. Frequency; 5 to 6 per month. Duration; 12 hours to 3 days. Peak velocity; 80 to 128 km/hr. Occasional storms from east. Peak velocity; 48 km/hr

Fetches: Southeast; 13 km. Southwest; 0.7 km (perpendicular).
Northwest; greater than 17 km. (details not available).

Waves: No recorded waves. Waves propogate parallel to breakwater.

Shipwaves: Constant ship traffic. Vessels 15 m to 61 m in length. Often heavily loaded.

Currents: Tidal currents up to 11.1 km/hr.

Ice: Not aproblem

Note: Shipwaves diffract through entrance into marina site creating some problems.

Breakwater Description: Breakwater fabricated from lightweight reinforced concrete floats, post-tensioned into modules in a ladder type of pattern. Modules are connected together for total length of breakwater.

Dimensions: Floats = 4.58 m 1.22 m x 1.53 m
Draft = 1.22 m, Draft is 0.15 m greater than designed.
Modules, 11 floats = 18.3 m x 6.41 m x 1.53 m
Overall Breakwater = 366 m x 6.41 m x 1.53 m
Mass of Breakwater = 1,250 kg/m^2 (based on horizontal float area)

Construction Site: Floats fabricated in Washington State. Shipped by barge, assembled on site.

Connections: Similar to Sitka, details not available

Reserve Buoyancy: Concrete cast over polystyrofoam core

Provision for Corrosion: External steel galvanized

Inspection Program; Not available

Note: Breakwater completed spring of 1980. Not tested to date.

Mooring Details:

Anchor System: Concrete anchor blocks. 60 tonne and tonne. Details not available.

Mooring System: Anchor blocks both sides spaced 18.6 m. Anchor chain 32 mm galvanized chain.

Maintenance Requirements:

Structural: None reported

Connections: None reported

Anchors: There was some difficulty placing anchors due to a lack of information on bottom topography and sub surface soil conditions.

Performance: Not tested by storm. Breakwater works well for shipwaves. Layout permits extensive diffraction of wakes into basin.

Cost: $4,592/m (Total cost, 1980)

Note: Due to depth of water a floating breakwater was more economical than a rubble mound structure.

Problems were experienced during installation with lining up the post tensioning ducts. Each module floated with a slightly different draft. Difficulty was also experienced with tightening the post-tensioning cables. As the nuts were tensioned the cable would rotate with the nuts.

Location: **SITKA, ALASKA**

Breakwater Type: Reinforced Concrete Pontoon

Location Reference: Not available

Type of Harbour: Public Marina

Year Constructed: 1973

Contact: Division of Water and Harbours, State of Alaska

Site Description: Breakwater is located on Sitka Sound which is protected from the Gulf of Alaska. The breakwater consists of two legs. One leg is exposed to the southwest, the other to the northwest. Marina facilities used by fishing and recreational vessels.

Depth of Water: 11 m south breakwater. 11 m to 3 m north breakwater

Tide Range: 3.67 m

Bottom Soil Conditions:

Winds: Storms in the Gulf of Alaska produce winds up to 105 km/hr, northwest; andf 145 km/hr, southeast-southwest. Maximum duration two to three days.

Fetches: Northwest greater than 3.7 km (details not available),
West 1.8 km, southwest 0.4 km.

Waves: Recorded data not available. Maximum wave height over direct fetch observed 1.22 m. Swell from Gulf of Alaska enters Sitka Sound and hits breakwater.

Shipwaves: Fishing vessels use channel parallel to breakwater.

Currents; Not a problem. Tidal currents run parallel to breakwater.

Ice: None

Breakwater Description: The breakwater is fabricated from lightweight reinforced concrete floats, post-tensioned into modules in a ladder type pattern. Modules are connected to form total length of breakwater. Reinforcing #3 and #4 bars and stirrups.

Diomensions: Wall thickness = 100 mm
Floats = 4.58 m x 0.92 m x 1.53 m
Draft = 1.22 m
Modules = 11 floats, 18.3 m x 6.41 m x 1.53 m
Overall breakwater = south legs 209 m x 6.41 m
north legs 86 m x 6.41 m
Mass of breakwater = 1,250 kg/m^2 (based on horizontal float area)

Construction Site: Floats fabricated in Bellingham, Washington. Shipped by barge and assembled on site.

Connections: Floats are connected with 25 mm post-tension rod (transverse) and 32 mm rods (longitudinal). Transverse floats cushioned at ends with 6 mm neoprene pads. Tensioned to 160 kN.

Modules connected with three links, 32 mm Stud-link, galvanized chain bolted to galvanized plate steel weldment cast in floats. Weldment also terminal for post-tension rods and anchor chains. Neoprene rubber fender blocks with timber bearing plates wedged between modules. Compressed by chain links.

Reserve Buoyancy: Concrete cast over polystyrofoam core.

Provision for Corrosion: External steel galvanized.
Mooring chain galvanized.
32 mm concrete cover.

Inspection Program: Harbour Master

Notes: Breakwater designed for ease of transportation and assembly in remote locations.

Mooring Details:

Anchor System: Reinforced concrete blocks 3.0 m x 2.75 m x 1.22 m and 1.83 m x 1.83 m x 1.22 m. All blocks have .46 x .46 m hole for H-pile. H-section stake piles. Anchor chain 38 mm galvanized stud-link.

Mooring Layout: North breakwater. Windward side. Large blocks and large blocks with stake piles. Leeward: Stake piles. South breakwater: Windward side. Small blocks and stake piles. Leeward: stake piles.

All anchors spaced 18.9 m

Nominal scope: Windward side 1:5, Leeward side 1:25 (Based on 11.0 m depth).

Maintenance Requirements:

Structural: None reported

Connections: Chain links wearing due to breakwater motion. Rubber bumpers missing on several connections. Continual replacement.

Anchors: None reported.

Note: The north leg is most exposed and most problems occur on this breakwater.

Performance: Users pleased with breakwater. During storms, motion still rough. Details not available. Boats all require slack moorings and several springs.

Costs: $1,560/m (Total cost including transportation, 1973).

Notes: Due to the depth of water a floating breakwater was a more economical alternative than a rubble mound structure.

TETHERED FLOAT BREAKWATERS

Location: SEABECK, WASHINGTON

Breakwater Type: Tethered Float

Location Reference: NOAA Chart No. 18458

Type of Harbour: Private Marina

Year Constructed: 1979

Contact: Seabeck Marina, Seabeck, Washington

Site Description: The marina is located on the Hood Canal and is exposed to winds from the northeast and north.

Depth of Water: 13.7 m

Tide Range: Unknown

Bottom Soil Conditions: Sandy with some silt and mud.

Winds: Predominant wind direction is south. Storms from the north occur on average three or four times per year. Mainly in January and December.

Fetches: Northeast 9 km, North 4.5 km

Waves: Design significant wave height 2.3 m, period 7-8 secs.

Shipwaves: Not a problem

Currents: Not a problem

Ice: None

Breakwater Description: The breakwater consists of 1.5 m diameter steel spherical balls tethered by cable and chain to a chain matrix anchored to the sea bed.

Dimensions: 326 spherical steel balls = 1.5 m diameter, spaced 10 feet apart. Five rows.

Construction Site: On-site by divers and crane.

Connections: Spheres schackled with 4.5 kg galvanized shackles to tether. No other connections.

Reserve Buoyancy: None. Relies on integrity of sphere.

Provision for Corrosion: Spheres sand blasted and sprayed with bituminous paint. Coating expected to last 5 years. Mooring chains coated with antifouling paint and grease.

Inspection Program: Marina operators on site.

Mooring Details:

Anchor System: Tether 3.1 m of cable (diameter unknown) connected to 10.7 m of chain (4.1 kg/link). Chain shackled to matrix of 5 longitudinal chains (4.1 kg/link), 183 m length, and 5 transverse chains. Chains anchored with 1,362 kg Navy anchors, one each end.

Maintenance Requirements:

Note: Length of service too short to define. Expect spheres must be retarred every five years.

Performance: Breakwater has been through two storms when conditions behind breakwater were severe (not defined). Otherwise, breakwater works well. Damage to docks less with breakwater installed.

Costs: Not available. Built with volunteer labour.

Notes: Depth of water made floating breakwater more economical than conventional breakwater.

Submergence of individual floats considerably less than called for in design. Unlikely performance will be as predicted during design.

TIMBER AND PLASTIC PONTOON BREAKWATERS

Location: FRIDAY HARBOUR, WASHINGTON

Breakwater Type: Plastic Pontoon and Timber

Location Reference:

Type of Harbour: Public Marina

Year Constructed: 1972

Contact: Port of Friday Harbour, Friday Harbour, Washington

Site Description: Breakwater located west side of Friday Harbour and exposed to northeast and south. Washington State Ferry slip adjacent to water lot. Frequency of service, every two hours.

Depth of Water: 134. m to 6.1 m

Tide Range: 2.3 m (higher high water)

Bottom Soil Conditions: Soft mud

Winds: Prevailing winds from south. Storms from southeast and northeast. Details of design winds not available.

Fetches: Northeast 3.3 km, East 6.5 km, Southeast 1.6 km

Waves: Observed maximum wave height 1.1 m, period 3 secs. Associated with northeast storm. Transmitted wave height 0.3 m.

Shipwaves: Frequent shipwaves from washington State Ferry

Currents: None reported.

Ice: None

Breakwater Description: The breakwater han an L-shaped configuration with the long leg perpendicular to the northeast fetch. The breakwater is constructed of heavy timbers and decking, floating on polyolefin pontoons. The pontoons have "milk bottle" outline and are connected by stringers which fit in the "neck" of the pontoon. Stringers run along the breakwater. There are four pontoons across the beam with a gap in the center equal to the width of one pontoon. Pontoons have a 12.5 mm wall thickness.

Dimensions: Pontoons = 3 x 1.5 m x 1.5 m, overall dimension (Pontoons have irregular outline)

Breakwater = 275 m x 7.6 m, overall dimensions

Draft = 0.46 m, freeboard not available

Mass of breakwater = not available. Breakwater ballasted with 0.15 m of water in pontoons.

Construction Site: Not available.

Connections: Timber bolted with galvanized fastenings. Pontoons supported by heavy timber stringers wedged into "neck" of pontoon. Held by nylon webbing.

Reserve Buoyancy: Details not available.

Provision for Corrosion: All metal hot-dipped galvanized. Carbon black added to polymer to prevent ultra violet degradation. Timbers pressure treated with coal tar creosote. Decking treated with Ammonial Copper Arsenite.

Inspection Program: Harbour Master on location.

Notes: Pontoons subject to fatigue failure in neck. Ballast reduced from design of 0.48 m to 0.15 m.

Mooring Details:

Anchor System: Stake piles minimum length of 3.1 m. Typical anchor line (seaward) 58 m of 38 mm double braided nylon rope with 27 m of chain on lower end. Seaward scope typically 1:7. Landward scope typically 1:2.

Mooring Layout: Anchor lines spaced at 15 m along breakwater

Note: Anchors field tested to 120 kN on leeward anchors and 165 kN on windward anchors.

Maintenance Requirements:

Structural: Pontoons have undergone extensive fatigue failure and crack at neck where supported by timbers. Polymer has been changed and pontoons appear to be standing up. Structure in general is not standing up. Extensive maintenance required.

Connections: None reported other than role of timbers in neck of pontoon.

Moorings: None reported

Performance: Other than fatigue problem with pontoons, breakwater reasonably effective. Plan to replace with concrete caisson in association with expansion plans.

Costs: $1,076/m (Total cost, 1972)

Replacement pontoons $1,000/pontoon

Maintenance costs not available

Notes: Due to depth of water, floating breakwater was a more economical alternative to a rubble mound structure.

FLOATING BREAKWATERS
WHERE INSUFFICIENT DATA IS AVAILABLE

1. EAGLE HARBOUR YACHT CLUB, W. VANCOUVER, B.C.

Floating pontoon breakwater with car tires suspended between pontoons. Anti-submarine net suspended from pontoons.
Dimensions: 61 m x 6.1 m; Pontoon diameter 1.07 m; Wall thickness 6.75 mm.
Concrete anchor blocks.
Exposed to southwest, Strait of Georgia
Operational for 4 1/2 years, works reasonably well.

2. MENTOR HARBOUR YACHT CLUB, MENTOR, OHIO

Floating type breakwater constructed in 1978. Chains holding tires and floatation did not perform well and project abandoned.
Likely Goodyear floating tire breakwater.

3. CLEVELAND YACHT CLUB, ROCKY RIVER, OHIO

Trial installation planned July-August 1980 of a new pole and tire type breakwater.

Design consists of three telephone poles, 10 m long, threaded with tires. Poles are connected with 100 mm channel and threaded rod.

4. MIDDLE CARAQUET, NEW BRUNSWICK

Goodyear floating tire breakwater, three modules wide, 100 m x 6.4 m, connected with 19 mm nylon rope, spliced connections, three tires per module with unspecified floatation added. Moored with 0.62 x 0.62 m x 0.62 m mass concrete anchor blocks, 25 mm nylon anchor rode.
Built 1976, based on University of Rhode Island, Marine Bulletin 21.
Performed well in waves up to 1 m. Failed during storm with 4 m waves. Ropes broke.

5. LITTLE HARBOUR LABORATORY, GUILDFORD, CONNECTICUT

Report received describing operational experience with Goodyear floating tire breakwater.
Breakwater failed and project abandoned.

6. PIER 39, SAN FRANCISCO, CALIFORNIA

Report received describing operational experience with Wave Maze floating tire breakwater.

Was not a success and breakwater is being replaced with a fixed sheet pile structure.

7. PORT OF LANGLEY, WASHINGTON

Eight module wide floating tire breakwater.

Performance unsatisfactory.

8. OREGON INLET, NORTH CAROLINA

Application of floating slope breakwater planned to extend the dredging season at an inlet bypassing site.

Breakwater wave climate - significant wave height 0.31 m to 1.53 m, peak period 5 to 12 secs.

Depth of water 4.6 m to 6.1 m

Possibility of encountering tail of hurricane. System must survive 4.6 m wave height, period 15 secs.

Description of Breakwater: Consists of modules of 6 to 8 Navy lighter barges connected with non-rigid connectors. Barges made up to three 27.5 m x 2.14 m x 1.5 m pontoons bolted together. Valves can be opened to flood compartments. Barges sunk on an incline into incident waves. Bottom sits on sea bed. Anchored from top into waves. Positive floatation provided by styrofoam.

Design Procedure: Concept has been tested in general in model tests. Attenuation of 2.1 m waves, period unspecified by 50 percent. Following tests planned.

Field Tests: 1979 - Two barges field tested in California

1980 - Two barges to be installed, Port Hueneme, California instrumented with wave guages, load cells. Connections evaluated. A high speed movie will be made to reconstruct motion of barges for a numerical simulation of breakwater.

Model Tests: 1980 - Further two dimensional tests to determine effects of mooring lines and structural changes to improve performance. Regular waves representative of Oregon Inlet.

1980 - Three dimensional tests and two dimensional spectral wave model tests.

Note: Breakwater is intended to extend dredging season by one half. Currently 0.4 million cubic meters dredged per year over three month period. 0.75 to 1 million cubic meter/year net transport at Oregon Inlet.

Cost: Estimated each pontoon - $1000, 000 ($US)
Per meter installed - $21,000 ($US)

9. NEW ORLEANS LEVEE BOARD, NEW ORLEANS

Recently completed 244 m of Goodyear floating tire breakwater as experiment to determine effectiveness for preventing erosion behind a seawall.

Site exposed to regular storms from northwest on Lake Ponchatrain.
Breakwater has L-shape: 122 m x 18.4 m and 122 m x 10.7 m. Anchored by .31 m diameter helix screww anchors, 6.4 m long in sandy bottom. Anchors do not hold. (Spacing not available).
Breakwater has worked well in up to 0.92 m waves, however, will stay on location. Sold to neighbouring parish for another use.
Cost $820/m ($US)
Manpower requirements 100 modules/5 mandays.

10. PLATTSBURG, NEW YORK

A 123 m c 8.54 Goodyear floating tire breakwater has been built in lee of rubble mound breakwater to provide protection during southwest storms.
Breakwater was built on ice in January 1976.
Details of design supplied by Goodyear Tire Company.
There is an ongoing problem with floatation. Originally the breakwater was built using styrofoam, however, this continually broke up. Foam replaced with 1/2 gallon milk bottles. These continue to break. Portions of the breakwater continue to sink.
Originally steel chain used to connections. Replaced with 6.4 mm diameter steel aircraft cable. Chain was too heavy and caused breakwater to sink.
Sediment also gathered in bottom tires causing breakwater to sink.
No problems with mooring system.

Ongoing maintenance - 6 to 12 mandays plus small barge and crane.

11. LAKE CHARLEVOIX, MICHIGAN

Goodyear floating tire breakwater installed winter of 1979. Breakwater constructed on ice.

Breakwater to support docks, connected to breakwater by rigid steel frames connected to tires by wire mesh.

Floatation provided by urethane foam in crest of tires.

Tires connected by conveyor belting 100 mm wide. Details not available.

12. DUNKIRK HARBOUR, NEW YORK

Goodyear floating tire breakwater constructed in 1975. 183 m x 18.3 m to protect marina from 1.1 m waves.

Found transmission coefficient to be approximately 60 percent. Diffraction around breakwater reduced effective length to 92 m.

Anchored with 227 kg concrete blocks with 24.4 m of chain, spaced every 30.5 m landward side. Anchored behind submerged reef. Anchors dragged during storms.

Connected with 12.5 mm steel chain and 6.4 mm diameter stainless steel wire. Chain worked well. Wire broke after three months.

Breakwater replaced with rubblemound structure.

13. STUART JENSEN CAUSEWAY, FLORIDA

Goodyear floating tire breakwater constructed as part of Shoreline Erosion Demonstration Project.

Breakwater washed ashore in hurricane. Breakwater reconstructed.

Connections: 12.5 mm polypropylene rope with nylon sheath failed. Replaced with 9.5 mm chain.

Buoyancy: Styrofoam blocks forced into tires unsatisfactory. Replaced with injected foam.

Length of service too short to evaluate performance.

14. NEWPORT, RHODE ISLAND

Newport International Sailboat Show.

Originally used Goodyear floating tire breakwater. Original breakwater part of development of concept.

Breakwater sank.

Location of show changed, planning caisson breakwater.

15. **DEEP COVE YACHT CLUB, DEEP COVE, BRITISH COLUMBIA**

64 m x 5.3 m x 1.5 m reinforced concrete caisson

Draft = 1.22 m

Design wave height 1.22 m, period 3.4 secs, Mooring forces from numerical analysis.

Estimated cost $1,953/m (Total)

16. **MIDLAND, ONTARIO, SEVEN SOUND MARINA AND YACHT SERVICE**

43 m Goodyear floating tire breakwater.

17. **BARCELONA HARBOUR COMMISSION, WESTFIELD, NEW YORK**

Goodyear floating tire breakwater. No floatation, sank. Severe wave reflection in harbour.

18. **NAVAL CIVIL ENGINEERING LABORATORY, PORT HUENEME, CALIFORNIA**

Planning first field test of sloping float breakwater in 1980.

19. **MISSION BAY, SAN DIEGO**

83 m of reinforced concrete caisson breakwater.

Details not available.

20. **TENAKEE SPRINGS, ALASKA**

Similar to Sitka and Ketchican Breakwaters.

Breakwater being tested in field.

A.2 Floating Breakwater Outside North America

A.2.1 Australia

a) Three floating breakwaters have been identified.

Spit Bridge Marina , Sydney, NSW - caisson design.

Royal Sydney Yacht Squadron, Sydney, NSW.

Renmark, Murray River

However, no details have been received in time for inclusion in this report.

b) Engineering research on submerged platform breakwaters, fixed and floating, is conducted at University of Western Australia.

A.2.2 Great Britain

a) Rhu Marina, Dunbartonshire, Scotland

Breakwater constructed in 1977. Consists of four 140 m sections and one 100 m section of "Harris" type breakwater units.

The sections are assembled from 35 m long prestressed "zig-zag" units, 10 m wide and 0.5 m deep. The units consist of ten 10 m long by 1.8 m wide beams arranged in a zig-zag pattern. Further details not available.

The 35 m units are post-tensioned together.

In October 1977, a storm exceeding the design conditions (details not available, period in excess of 4 seconds) caused fractures to occur in every fifth beam.

Breakwater was removed from service until 1980.

Reinstalled in five beam sections with tire fenders between sections.

Site Conditions: Depth 8 m

Fetch 12 km, Southeast

1 km, south

3 km, Southwest

Incomplete details received from marina authority.

b) Ardyne Point, Scotland

In 1976, a 55 m x 18 m wide breakwater protected a temporary bridge at the construction site for the Brent "C" platform for the North Sea oil fields.

Breakwater consisted of 18 m x .9 m x .3 m timbers braced by .3 m x .3 m timbers. Further details not available.
Breakwater is not longer in use.

c) Stokes Bay, England
Details of a 1:10 scale test of breakwater intended for use in open ocean available in a report.
Breakwater is similar to Rhu Marina.

d) Port Edgar, Scotland
Truck tire floating breakwater located inside two rubble mound breakwaters. Breakwaters protects exposure through harbour entrance to the west and northwest and is intended as a temporary structure.
Waves: Hindcast wave height = 0.3 m to 0.7 m
Period = 4 sec to 3 sec respectively
Depth of Water: 2 m to 5 m below low tide
Tide Range: 6.4 m
Bottom Soil Conditions: very soft silt
Connections: Conveyor belting
Reserve Buoyancy: None
Maintenance Requirements: After five months conveyor belting abraded. Replaced with chain. Chain will require replacement after three years due to corrosion. Polyethelene foam placed in crown of tires.
Performance: Satisfactory.

e) Plymouth, England, Mayflower Marina
No details available.

Italy

Caisson type FBW has been tested numerically and experimentally at Istituto Di Idraulica, Pavia, Italy. BW proposed for marinas in Northern Italy.
Oirelli Industries have tested a short section of a filled bag type of floating breakwater at Livovro. No details available.

A.2.3 Japan

a) Floating breakwaters have been installed or field tested at the following locations:

1) Ondo fishing harbour, Hiroshima prefecture. Caisson breakwater width 7 m, depth 4.5 m, draft 2.85 m, length of breakwater is unknown.
Built by Yawata Iron and Steel Co. Ltd. in 1964.

2) Hataura, Saga Prefecture, A steel caisson type of breakwater consisting of a steel caisson with two barriers, supported by steel truss work, on both sides of the caisson dimensions unknown.
Built by Ishikawajima - Harima Heavy Industries Co. Ltd. in 1977.

3) Fukura Bay, Awajishima Island, Hyogo Prefecture. Breakwater consists of 12 ports of three fibreglass interconnected shells ballasted with seawater. Positive floatation is provided in each shell. The shells are rectangular in plan and cross-section with dimensions 7 m x 2.35 m x 2 m, draft when ballasted is 1.4 m. Shells are interconnected with three 0.8 m diameter fibreglass pipes evenly spaced, to form a pod 10 m x 7 m x 2.3 m. The pod is oriented with the interconnecting tubes parallel to the wave direction. Each pod weighs 43 tonnes after ballasting.

4) Ohoma, Tsushima-cho, Ehime prefecture. A similar breakwater to the one in Fukura Bay has also been installed. Details are unavailable.
Both of these breakwaters were built by the Bridgestone Tire Co. Ltd. Tokyo.

5) Kataura Bay, Kagoshima Prefecture. A modified version of the breakwater at Hataura has been field tested between 1977 and 1979 by Ishikawajima - Harima Heavy Industries Co. Ltd. The breakwater was modified from that at Hataura, by adding a steel trusswork stabilizer below the caisson. Details of the performance are not available.

6) Miyako Bay, Iwafe Prefecture, a third type of breakwater manufactured by the Ishikawajima - Harima Heavy Industries Co. Ltd. have undergone 2 sets of field trials. The breakwater consists of small sections of the Hatavra breakwater, with the length of breakwater module made equal to its beam. Each module is individually moored. Details of any fenders, connections or moorings are not available. Maximum wave heights during the test period were 11.4 m.

Breakwater have also been built and field tested by the following companies in Japan:

1) Mitsubishi Heavy Industries Ltd. This breakwater is a steel caisson breakwater open at the top and bottom to allow water to enter the caisson. Two types of breakwater have been designed. One incorporates an anti-heaving tank, the other, both an anti-heaving tank and an anti-swaying tank. The principle is based on anti--pitching tanks developed for use in ships.

   The second breakwater has been field tested. Details are not available.

2) Tokyo Rope Mfg. Co. Ltd.

   This company has developed and field tested a beach type of floating breakwater. Steel trusswork supports a sloping platform constructed from steel sections and a second horizontal platform of similar construction suspended below. The two platforms are suspended from seven steel pontoons which provide the necessary floatation. The upper platform slopes seaward to form a beach. The lower platform is horizontal and acts as a damper to the structures motion.

The structure has been field tested. Details are not available.

a) The four companies above are all members of the Japan Floating Breakwater Association.

b) The Port and Harbour institute of Japan is conducting a survey of floating harbour facilities. No details are provided.

A.2.4 New Zealand

Christchurch, New Zealand. Floating Tire Breakwater.

Location Reference: Lyttleton Harbour

Type of Harbour: Small craft marina. Present breakwater is test section.

Year constructed: 1979 (May)

Contact: Lyttleton Harbour Board, P.O. Box 2108, Christchurch 1, New Zealand.

Site Description: The breakwater is a test installation for a marina to be located in Magazine Bay. Test section is exposed to short waves from the southwest.

Depth of Water: 2 m. Datum not available.

Tide Range: Not available

Bottom Soil Conditions: Not available.

Winds: Details of design winds not available. Extensive records of winds during trial period available. Eleven southwest storms between May 3, 1979, and January 17, 1980, recorded. Average velocity between 37 km/hr and 74 km/hr.

Fetch: Effective fetch 2.5 km

Waves: Expected maximum conditions, wave height 1.22 m, period 3.5 secs, wave length 13.7 m. Observed and recorded incident and transmitted wave data available from Lyttleton Harbour Board. Not available for this report.

Shipwaves: Not reported.

Currents: Not available

Ice: Not reported

Breakwater Description: Breakwater is a Goodyear floating tire breakwater. First constructed 7 modules wide. One month later made 10 modules wide.

Dimensions: Final dimensions 25 modules x 10 modules, 45.7 m x 5.2 m

Construction Site: Drydock, Christchurch, towed to site.

Connections: Rubber conveyor belting 75 mm x 11 mm and nylon bolts (15.9 mm diameter), two per connection.

Reserve Buoyancy: Two litre plastic containers placed in same modules. An unknown number of bottles were filled with foam. None in other modules. Performance monitored. Modules with no additional floatation had sunk after six months. Modules with plastic containers in general remained afloat. Approximately fifteen percent of bottles had failed, developing small holes to splits of 75 mm long after six months. Bottles filled with foam had remained intact after 10 months.

Provision for Corrosion: Black nylon bolts used to prevent breakdown by ultra violet radiation.

Inspection Program: Inspected monthly and during southwestern storms. Breakwater removed to drydock in February 1980. Testing program shall continue with several alternatives for floatation being examined.

Mooring Details:

Anchor System: Seaward: 6,000 kg and 4,000 kg concrete blocks.
Landward: 4,000 kg concrete blocks
Mooring Lines 40 mm polyester.

Mooring Layout: Seaward: 6,000 kg blocks each corner
4,000 kg block at third points. Spacing 15.3 m.
Landward: One 4,000 kg block at center.

Maintenance Requirements:

Structural: Chafing of conveyor belting connecting modules in direction of waves reported. See above floatation. No wear on bolts.

Moorings: Blocks turned over preventing inspection of connection. Base area being increased.

Performance: Performance has been acceptable. Test program is continuing (September 1980).

Cost: Cost available in New Zealand dollars. Manpower requirements for 3 x 25 module section - 12 mandays.

A.2.5 Norway

a) Translation of a Norweigan report provided by US Army Corps of Engineers identifying one harbour in Sweden and twelve in Norway.

Breakwaters are all concrete pontoons or caissons. Two have been replaced with rubble mound structures and a number of failures have occurred with others.
Not all details available in the report.

b) Det Norske Veritas, a Norweigan research institute has undertaken model studies for an additional harbour not included in the above report. Details not available.

c) Norweigan Hydrodynamic Laboratories conducted extensive model testing on floating breakwaters. Details not available. Reports in Norweigan.

A.2.6 <u>Switzerland</u>

a) A caisson type of floating breakwater has been used at a small private marina since 1977 on Lake of Luzen in Switzerland.

APPENDIX B

SOURCES OF INFORMATION CURRENTLY ENGAGED WITH FLOATING BREAKWATERS

IN NORTH AMERICA

INSTITUTIONS OR INDIVIDUALS CURRENTLY ENGAGED IN FLOATING BREAKWATER ACTIVITIES

a) The Goodyear Tire and Rubber Company.

Not actively engaged in research now. Their function is to direct people to others active in the field.

b) The University of Washington, Seattle.

Not currently active in floating breakwater research due to lack of funds. Professors Richey, Nece, Adee, and Hartz consult to industry to review proposed floating breakwaters.

Professor B. Adee has developed a numerical model for analysis of floating breakwater performances. The model is reported in the literature and is extensively used for evaluating proposed designs.

Professors Richey and Nece are conducting a survey of field installations of floating breakwaters in the western United States for the U.S. Army Corps of Engineers.

c) The University of Rhode Island, Sea Grant Marine Advisory Service, Mr. Neil Ross.

Not currently active other than to act as source for people interested in floating tire breakwaters.

d) The National Water Research Institute, Burlington, Ontario.

Conducting model tests on pole-tire breakwater design by Doctor V.W. Harms.

Published a "Design Manual for Floating Tire Breakwaters", by C.T. Bishop.

e) Memorial University of Newfoundland, Faculty of Engineering and Applied Science.

Conducting model tests on tethered float breakwaters in irregular waves.

f) Queen's University, Kingston, Ontario.

Conducting model tests of floating breakwater for Jamaica.

g) Doctor V.W. Harms, University of California, Berkeley.

Conducting further studies on pole-tire breakwaters at the National Water Research Institute, Burlington.

h) Waterways Experiment Station, U.S. Army Corps of Engineers.

Undertakes model studies for specific projects.

Conducting a literature search on floating breakwaters.

i.) U.S. Army Corps of Engineers, Coastal Engineer Research Center.

Initiated field assessments of floating breakwaters in east and west United States and extensive literature survey.

Work to lead to a long-term research program for floating breakwaters.

j) Western Canada Hydraulic Laboratories Ltd., Port Coquitlam, B.C.

Undertaking floating breakwater model studies for clients.

k) Small Craft Harbour Branch, Fisheries and Oceans Canada.

Develop and operate small craft harbours using floating breakwaters.

l) Public Works Canada, Ottawa.

Construct, install and maintain floating breakwaters for public harbours.

MANUFACTURERS OF FLOATING BREAKWATERS

a) Builders Concrete, Inc., Bellingham, Washington.

Reinforced concrete caissons.

b) Cefer Designs Ltd., Richmond, British Columbia.

Reinforced concrete caissons.

c) Thompson Floatation Company, Newport Beach, California.

Timber and polyethylene floats damped with sea anchors suspended from floats.

Model tests planned at University of Washington.

d) Lane Instrument Company.

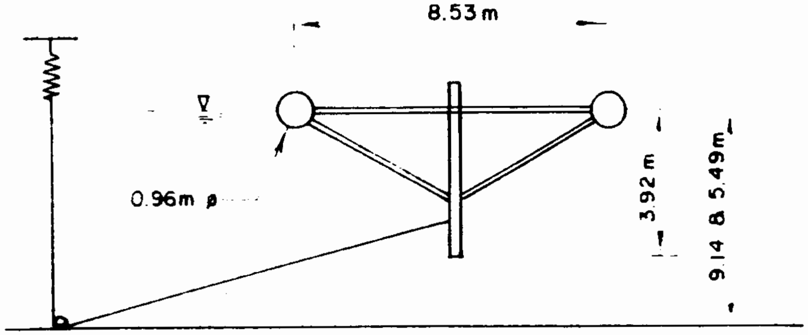
Tethered float breakwaters. Details not available.

e) Aqua Terra Floatations Ltd. - Cassidy, B.C.

Fabrication and installation of Caisson Breakwaters.

APPENDIX C

RESULTS OF MODEL STUDIES



SCALE 1:12
REGULAR WAVES

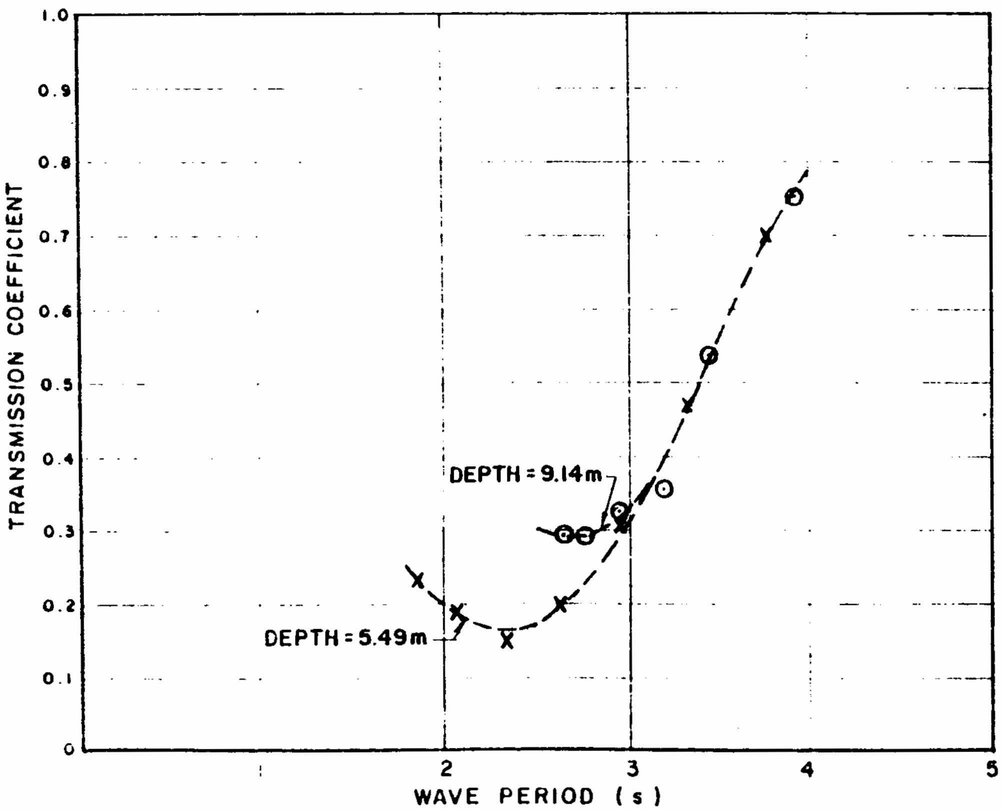
TEST CONDITIONS

WAVE PERIOD	1.86 to 3.94 s
WAVE HEIGHT	Not available
DEPTH OF WATER	5.49 and 9.14 m
WAVE STEEPNESS	Maintained between 0.06 and 0.08
RELATIVE DEPTH	0.27 to 1.30

MOORING DETAILS

Moored with 1.5 mm (model) stainless steel cable. A spring with a linear spring constant connected the cable to a force meter. Stiffness not available.

Initial tension = 0. Scope not available.



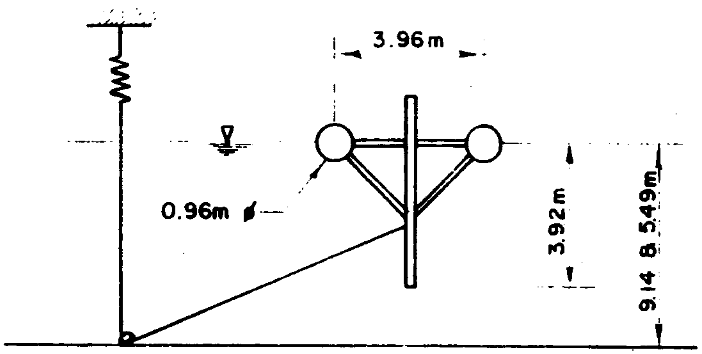
FLOATING BREAKWATER DESIGN

OFUYA, 1968, A- FRAME

FIGURE C1

WESTERN CANADA HYDRAULIC LABORATORIES LTD.

bcl 6638 A-WCH



SCALE 1:12
REGULAR WAVES

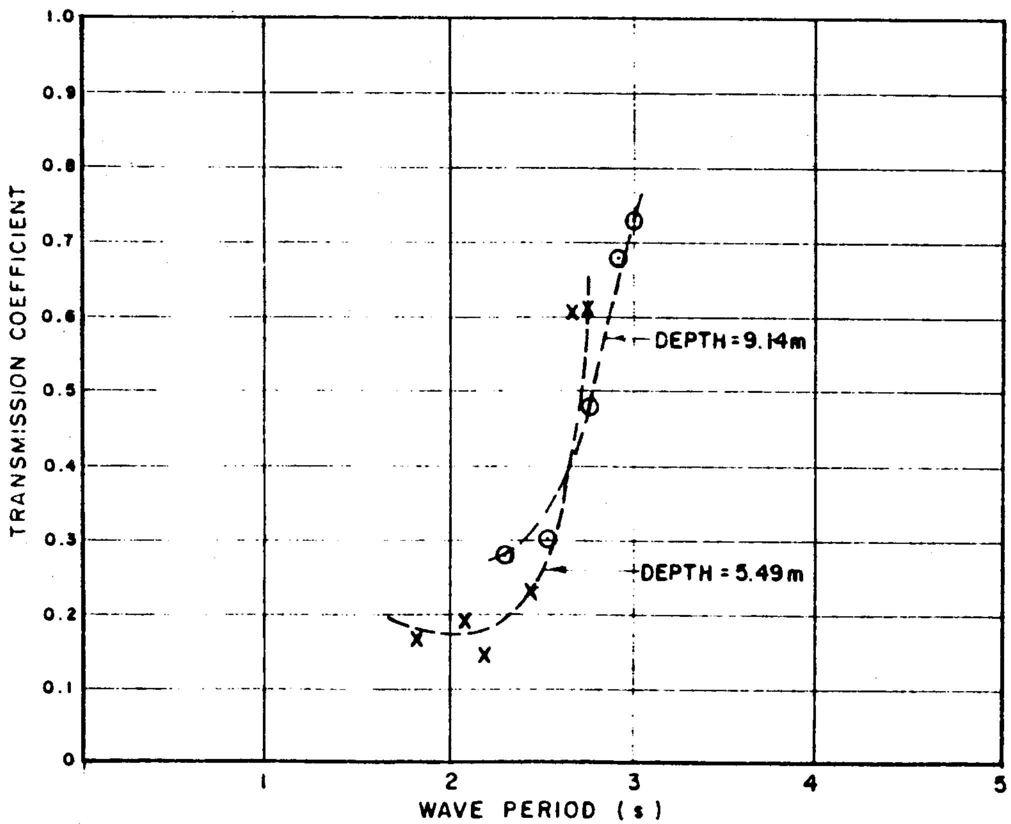
TEST CONDITIONS

WAVE PERIOD	1.86 to 3.94 s
WAVE HEIGHT	Not available
DEPTH OF WATER	5.49 and 9.14 m
WAVE STEEPNESS	Maintained between 0.06 and 0.08
RELATIVE DEPTH	0.27 to 1.36

MOORING DETAILS

Moored with 1.5 mm (model) stainless steel cable. A spring with a linear spring constant connected the cable to a force meter. Stiffness not available.

Initial tension = 0. Scope not available.



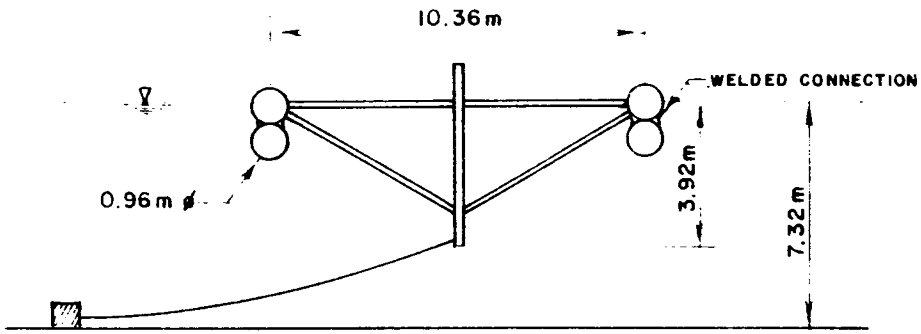
FLOATING BREAKWATER DESIGN

OFUYA, 1968, A-FRAME

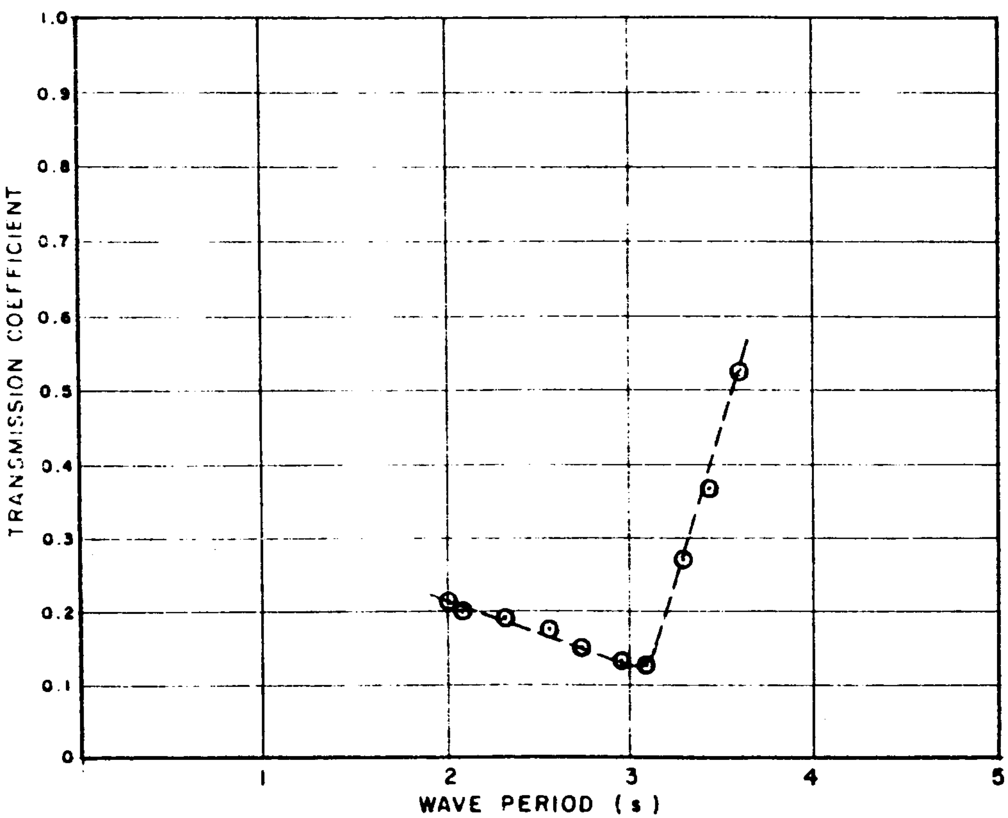
FIGURE C2

WESTERN CANADA HYDRAULIC LABORATORIES LTD.

bc-16638 A-WCH



NOTE: LOWER CYLINDER PERFORATED



SCALE 1:12
REGULAR WAVES

TEST CONDITIONS

WAVE PERIOD	2.0 to 3.6 s
WAVE HEIGHT	Not available
DEPTH OF WATER	7.32 m
WAVE STEEPNESS	0.07 to 0.08
RELATIVE DEPTH	.38 to 1.0

MOORING DETAILS

Moored with 1.5 mm (model) stainless steel cable. A Spring with a linear spring constant connected the cable to a force meter. Stiffness not available. Initial tension = 0. Scope not available.

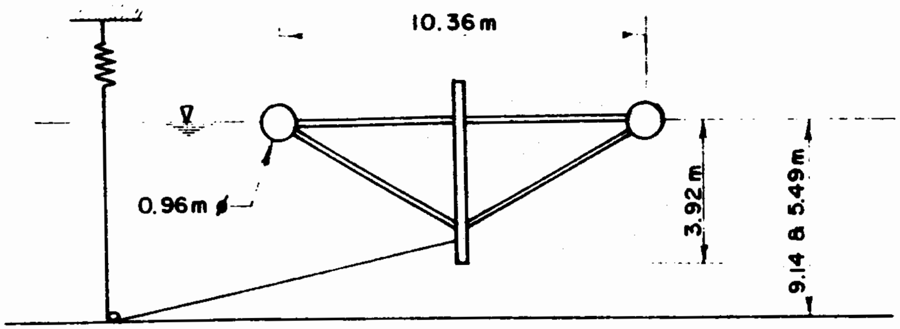
FLOATING BREAKWATER DESIGN

OFUYA, 1968, 4-CYLINDER A-FRAME

FIGURE C3

WESTERN CANADA HYDRAULIC LABORATORIES LTD.

W-1 6638 A-WCH



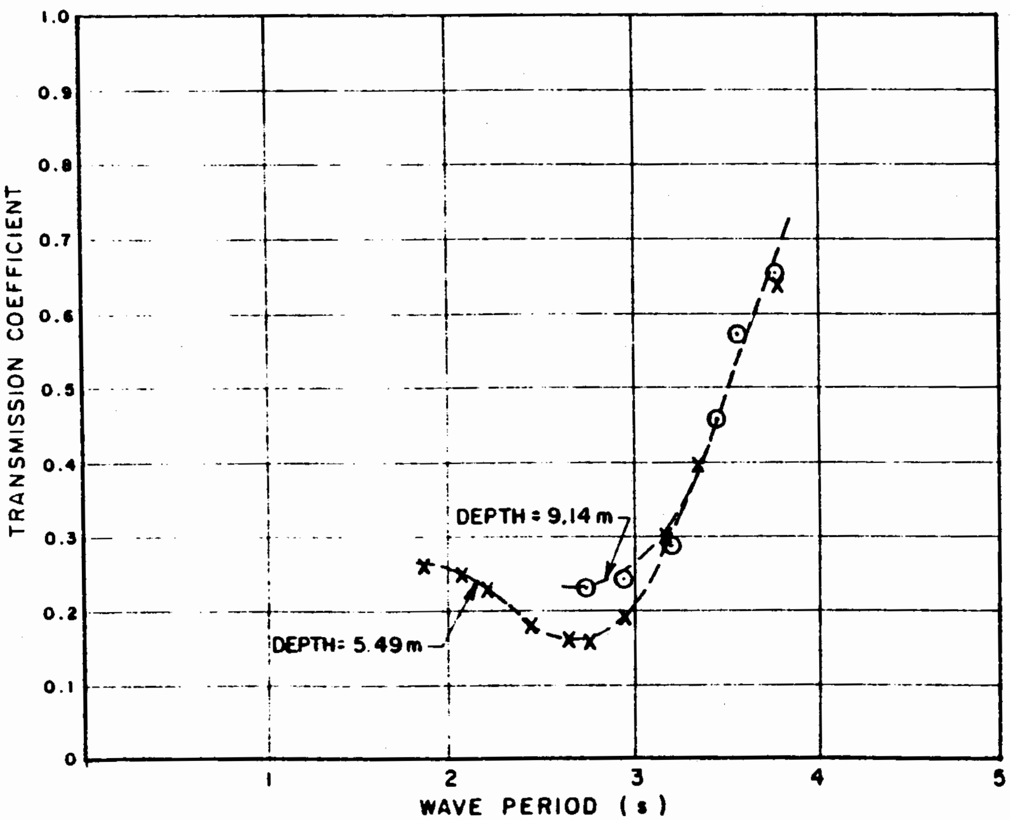
SCALE 1:12
REGULAR WAVES

TEST CONDITIONS

WAVE PERIOD	1.86 to 3.94 s
WAVE HEIGHT	Not available
DEPTH OF WATER	5.49 and 9.14 m
WAVE STEEPNESS	Maintained between 0.06 and 0.08
RELATIVE DEPTH	0.27 and 1.36

MOORING DETAILS

Moored with 1.5 mm (model) stainless steel cable. A spring with a linear spring constant connected the cable to a force meter. Stiffness not available. Initial tension = 0. Scope not available.



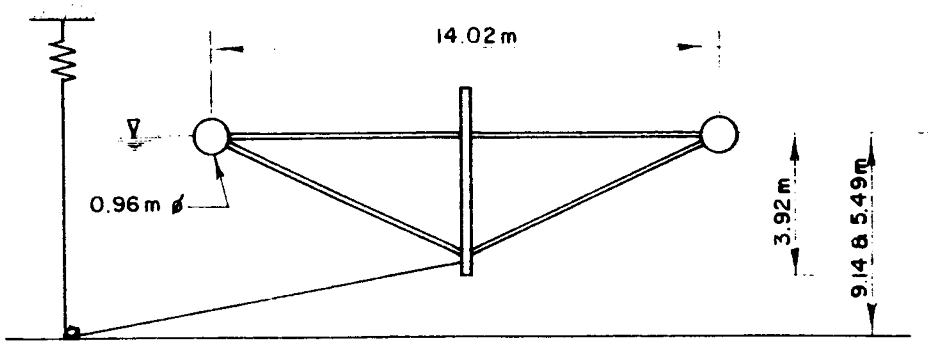
FLOATING BREAKWATER DESIGN

OFUYA, 1968, A- FRAME

FIGURE C4

WESTERN CANADA HYDRAULIC LABORATORIES LTD.

bc-16638 A-WCH



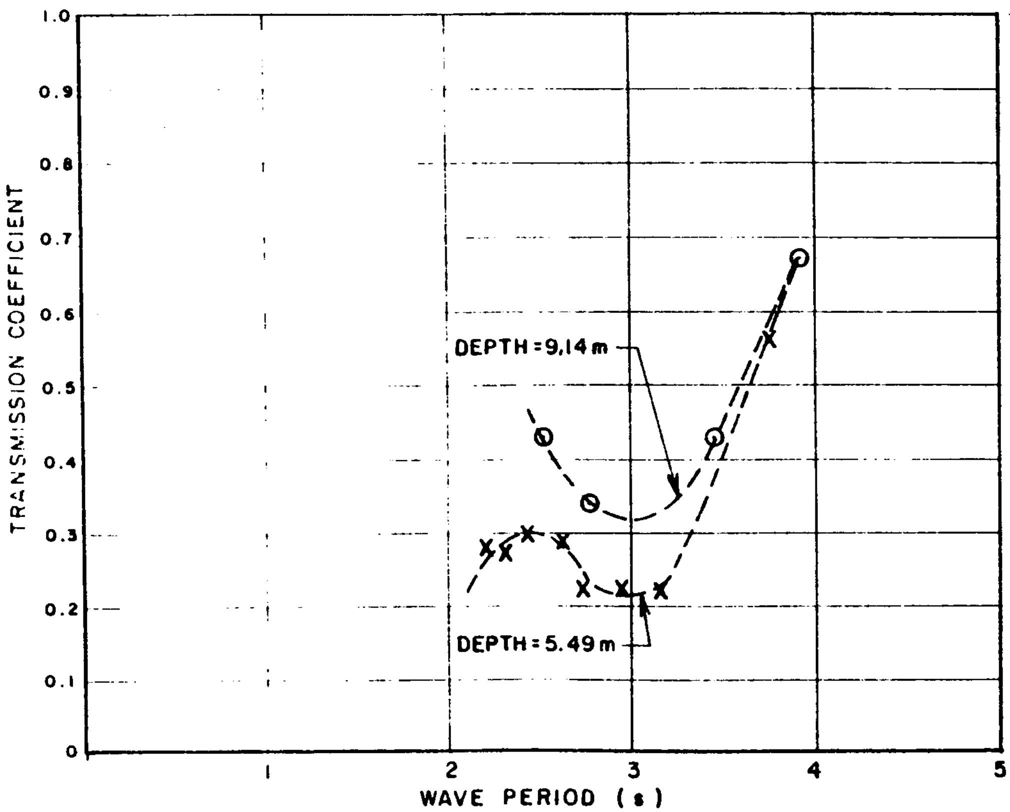
SCALE 1:12
REGULAR WAVES

TEST CONDITIONS

WAVE PERIOD	1:86 to 3.94 s
WAVE HEIGHT	Not available
DEPTH OF WATER	5.49 and 9.14 m
WAVE STEEPNESS	Maintained between 0.06 and 0.08
RELATIVE DEPTH	0.27 to 1.36

MOORING DETAILS

Moored with 1.5 mm (model) stainless steel cable. A spring with a linear spring constant connected the cable to a force meter. Stiffness not available. Initial tension = 0. Scope not available.



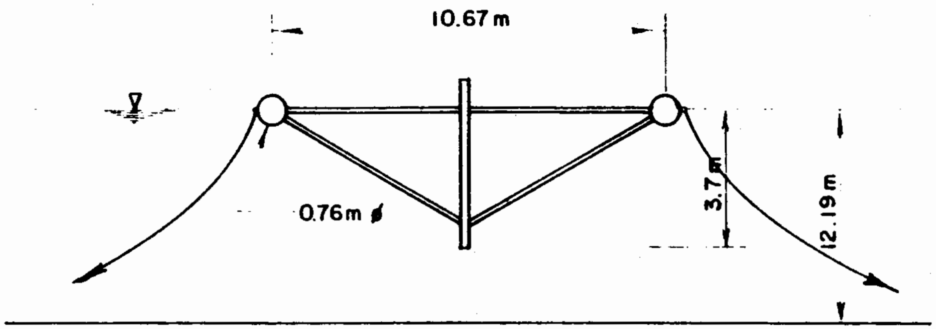
FLOATING BREAKWATER DESIGN

OFUYA, 1968, A-FRAME

FIGURE C5

WESTERN CANADA HYDRAULIC LABORATORIES LTD.

bcil 6638 A-WCH



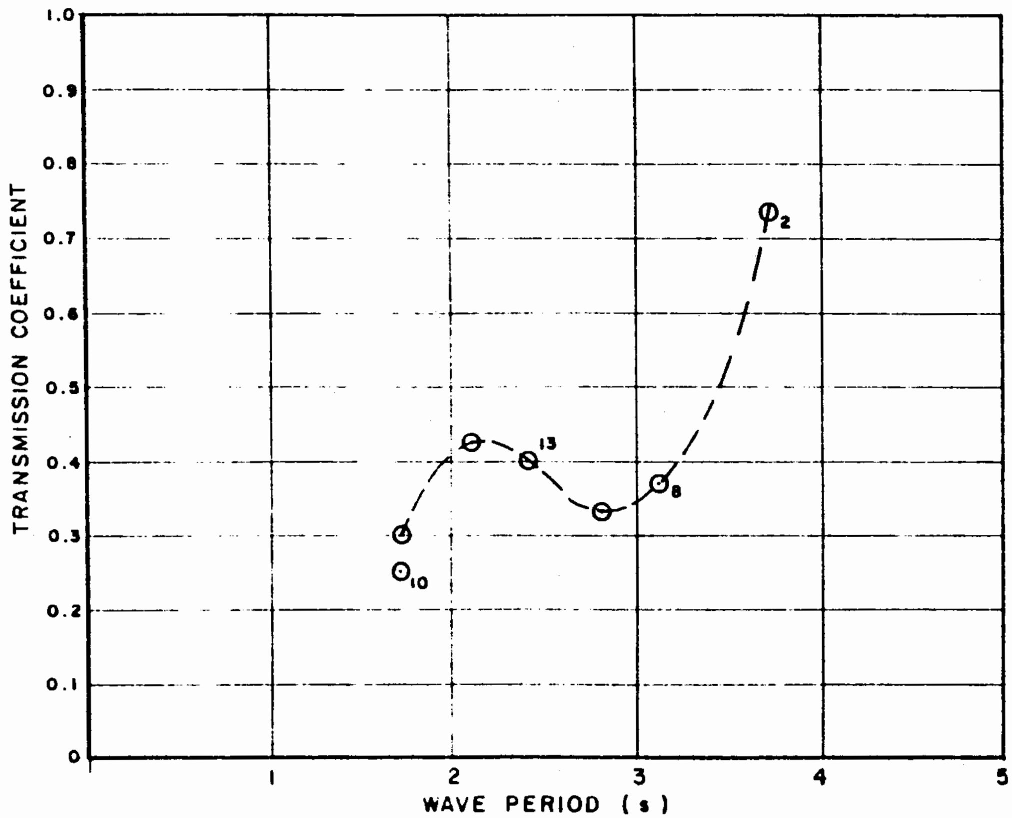
SCALE 1:24
REGULAR WAVES

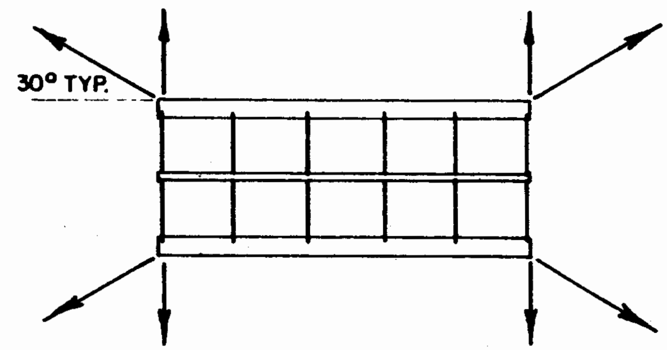
TEST CONDITIONS

WAVE PERIOD	1.7 to 3.7 s
WAVE HEIGHT	.46 to 1.8 m
(Not available for all tests)	
DEPTH OF WATER	12.2 m
WAVE STEEPNESS	.08 to .13
RELATIVE DEPTH	.57

MOORING DETAILS

Moored with chain, unit mass = 16.4 gk/m. Diameter and initial tension not available. Scope of mooring line = 2.5. Eight mooring lines, one at each corner and 2 each side used to moor breakwater.





PLAN

NOTE

Numbers on graph show —— percent wave steepness

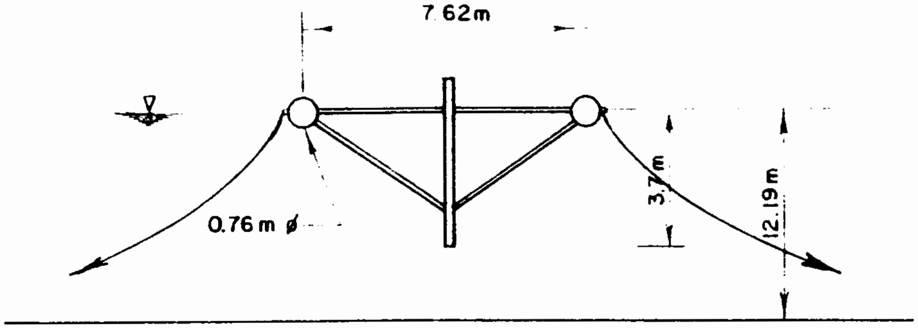
FLOATING BREAKWATER DESIGN

WCHL, 1966, INVERTED A-FRAME

WESTERN CANADA HYDRAULIC LABORATORIES LTD.

FIGURE C6

bcl 6638 A-WCH



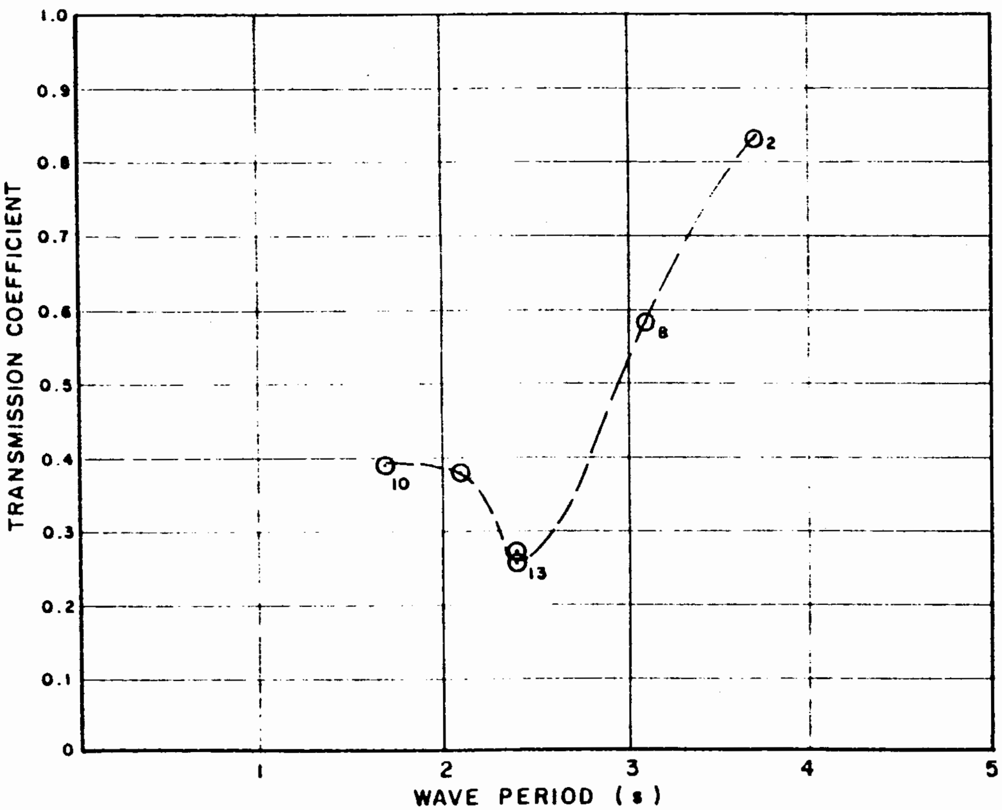
SCALE 1:24
REGULAR WAVES

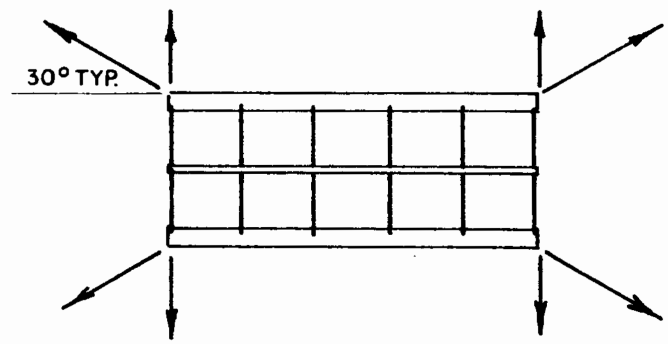
TEST CONDITIONS

WAVE PERIOD	1.7 to 3.7 s
WAVE HEIGHT	.46 to 1.8 m
(Not available for all tests)	
DEPTH OF WATER	12.2 m
WAVE STEEPNESS	0.08 to 0.13
RELATIVE DEPTH	.57

MOORING DETAILS

Moored with chain, unit mass = 16.4 kg/m. Diameter and initial tension not available. Scope of mooring line = 2.5. Eight mooring lines, one at each corner and 2 each side used to moor breakwater.





PLAN

NOTE

Numbers on graph show — percent wave steepness

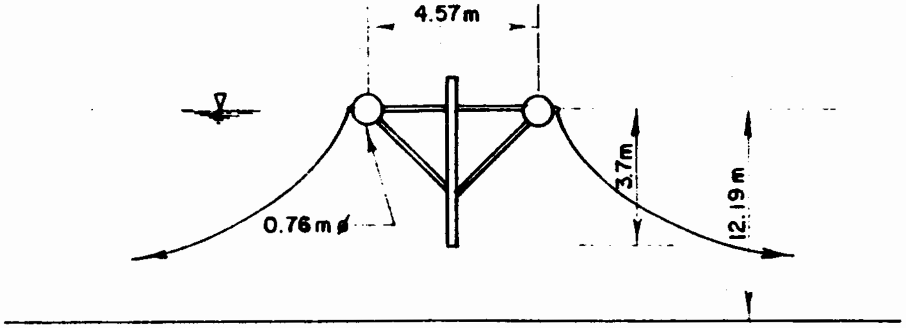
FLOATING BREAKWATER DESIGN

WCHL, 1966, INVERTED A-FRAME

WESTERN CANADA HYDRAULIC LABORATORIES LTD.

FIGURE C7

bcil 6638 A-WCH



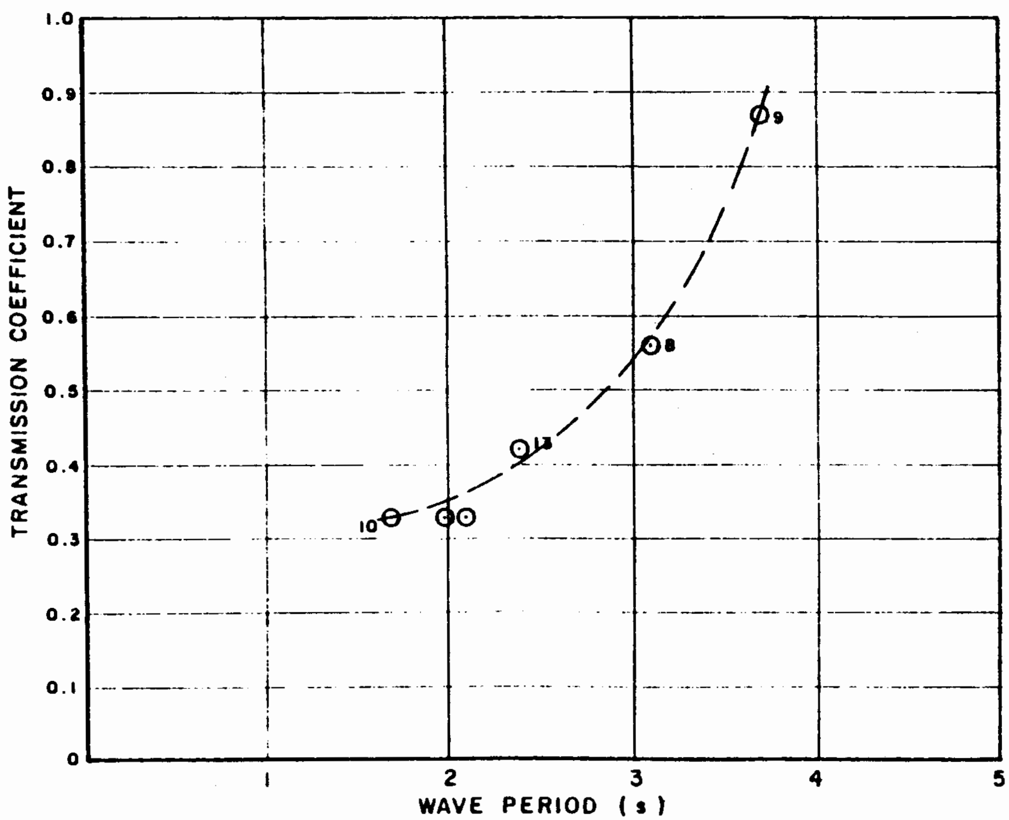
SCALE 1:24
REGULAR WAVES

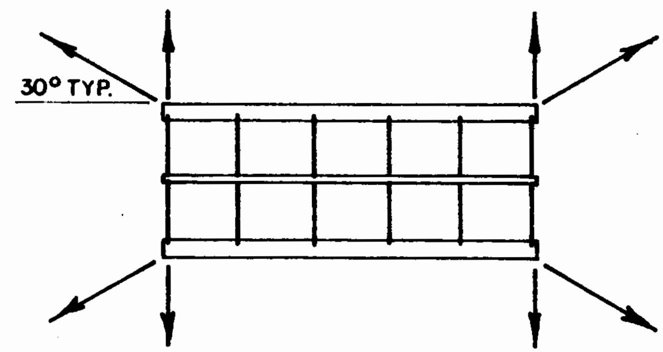
TEST CONDITIONS

WAVE PERIOD	1.7 to 3.7 s
WAVE HEIGHT	.46 to 1.8 m (Not available for all tests)
DEPTH OF WATER	12.2 m
WAVE STEEPNESS	0.08 to .13
RELATIVE DEPTH	.57

MOORING DETAILS

Moored with chain, unit mass = 16.4 kg/m. Diameter and initial tension not available. Scope of mooring line = 2.5. Eight mooring lines, one at each corner and 2 each side used to moor breakwater.





PLAN

NOTE

Numbers on graph show — percent wave steepness

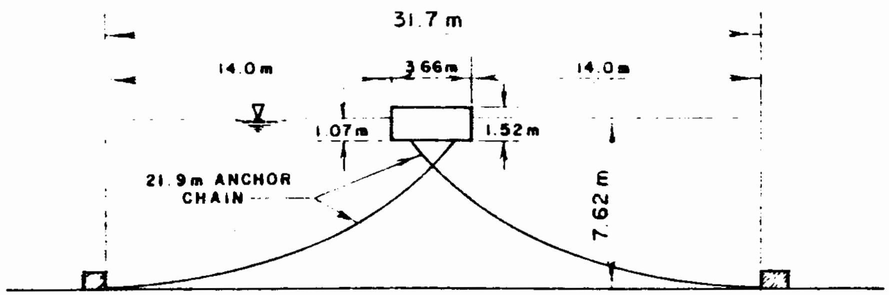
FLOATING BREAKWATER DESIGN

WCHL, 1966, INVERTED A-FRAME

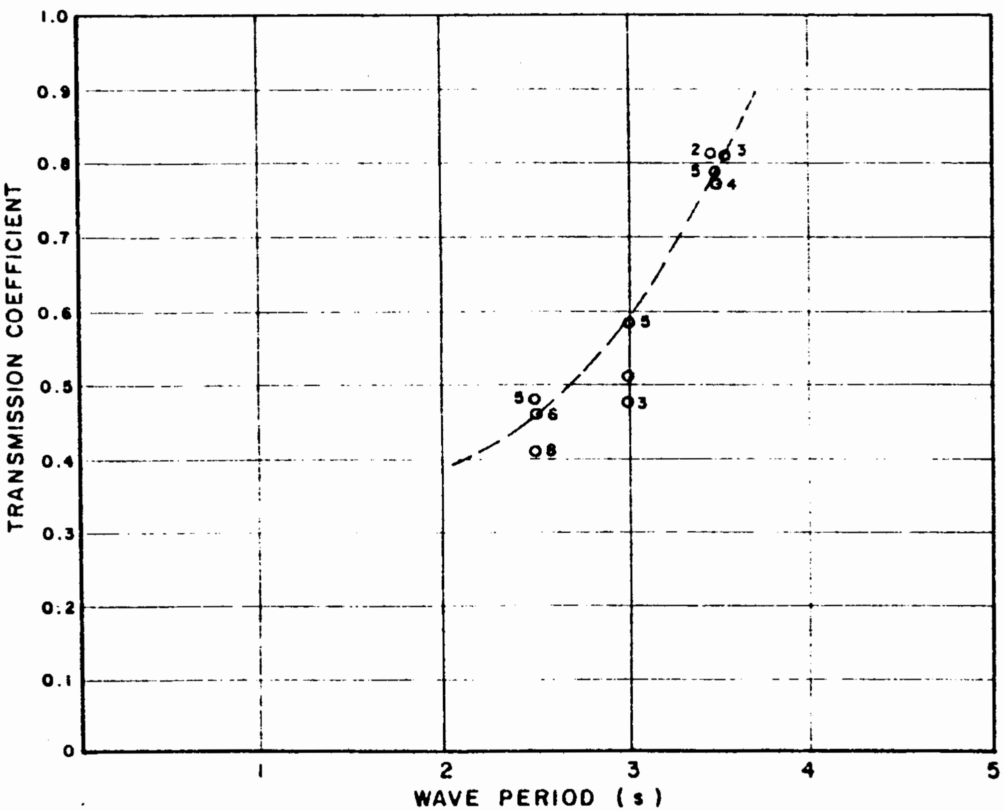
WESTERN CANADA HYDRAULIC LABORATORIES LTD.

FIGURE C8

bcil 6638 A-WCH



END VIEW



SCALE 1:10
REGULAR WAVES

TEST CONDITIONS

WAVE PERIOD	2.5 to 3.5 s
WAVE HEIGHT	0.5 to 1.1 m
DEPTH OF WATER	7.6 m
WAVE STEEPENSS	0.02 to 0.08
RELATIVE DEPTH	0.4

NATURAL PERIODS

HEAVE	4.6 s)
ROLL	4.0 s) Including effect of
PITCH	3.0 s) mooring lines
SWAY	10.5 s)

MOORING DETAILS

Moored with 22 m of 21.2 kg/m chain, link diameter = 30 mm. Mooring chains crossed underneath breakwater. Details required to calculate initial tension in mooring lines provided in figure above.

NOTE

This breakwater also tested in 3 dimensional model at 1:10.

Numbers on graph show — percent wave steepness

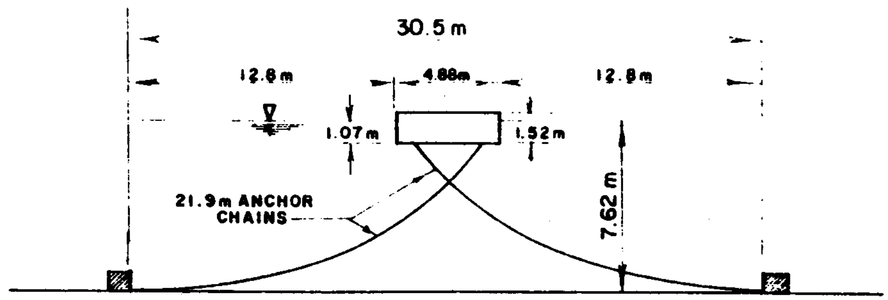
FLOATING BREAKWATER DESIGN

CARVER, 1979, CAISSON

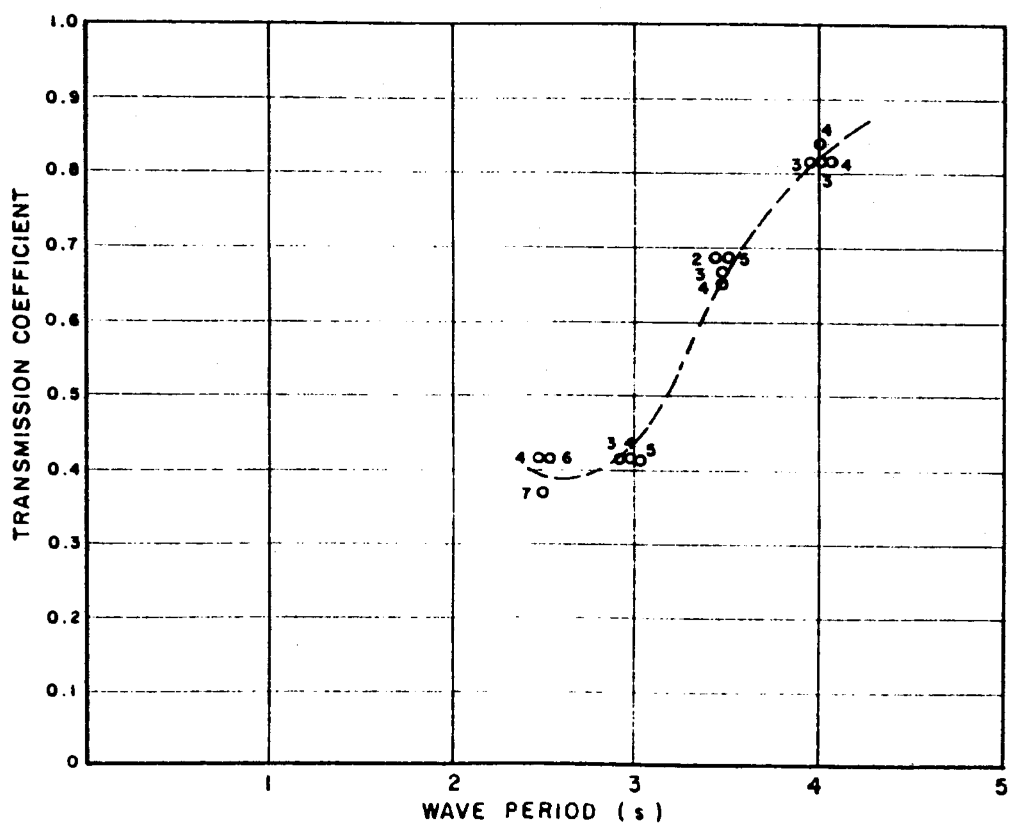
FIGURE C9

WESTERN CANADA HYDRAULIC LABORATORIES LTD.

bc-l 6638 A-WCH



END VIEW



SCALE 1:10
REGULAR WAVES

TEST CONDITIONS

WAVE PERIOD	2.5 to 4.0 s
WAVE HEIGHT	0.5 to 1.1 m
DEPTH OF WATER	76 m
WAVE STEEPNESS	0.02 to 0.08
RELATIVE DEPTH	7.0:31

NATURAL PERIODS

HEAVE	6.3)
ROLL	3.1) Including effect of
PITCH	2.5) Mooring lines
SWAY	13.1)

MOORING DETAILS

Moored with 22 m of 21.2 kg/m chain, link diameter = 30 mm. Mooring chains crossed underneath breakwater. Details required to calculate initial tension in mooring lines provided in figure above.

NOTE

Numbers on graph show — percent wave steepness

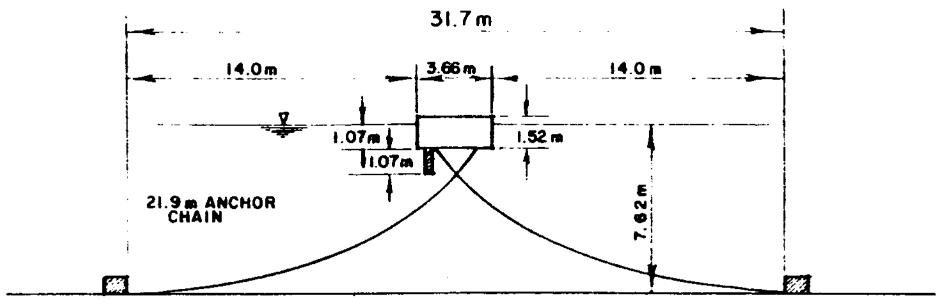
FLOATING BREAKWATER DESIGN

CARVER, 1979, CAISSON

FIGURE C 10

WESTERN CANADA HYDRAULIC LABORATORIES LTD.

bc:l 6638 A-WCH



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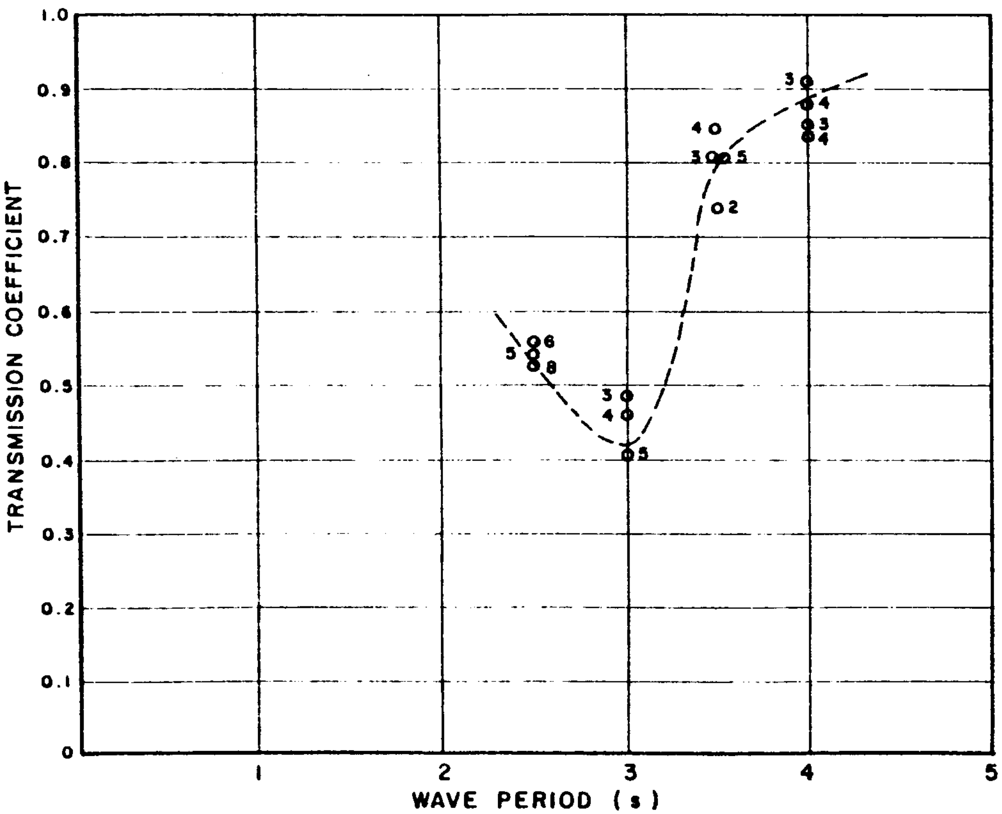


SCALE 1:10
REGULAR WAVES

TEST CONDITIONS

WAVE PERIOD	2.5 to 4.0 s
WAVE HEIGHT	0.5 to 1.1 m
DEPTH OF WATER	7.6 m
WAVE STEEPNESS	0.02 to 0.08
RELATIVE DEPTH	0.4

NATURAL PERIODS

HEAVE	5.8 s)
ROLL	3.7 s) Includes effect of
PITCH	3.7 s) mooring lines
SWAY	11.8 s)

MOORING DETAILS

Moored with 22 m of 21.2 kg/m chain, link diameter = 30 mm. Mooring chains crossed underneath breakwater. Details required to calculate initial tension in mooring lines provided in figure above.

NOTE

Numbers on graph show — percent wave steepness

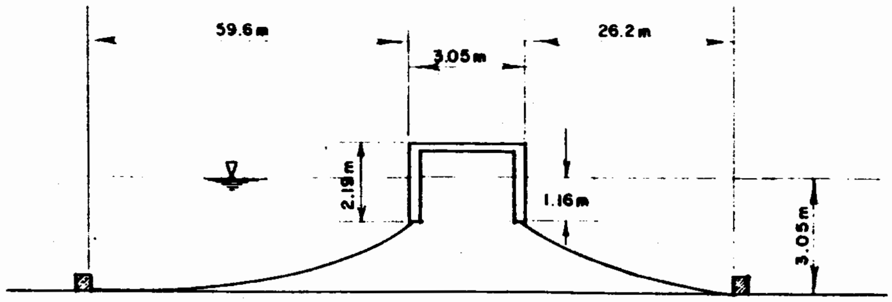
FLOATING BREAKWATER DESIGN

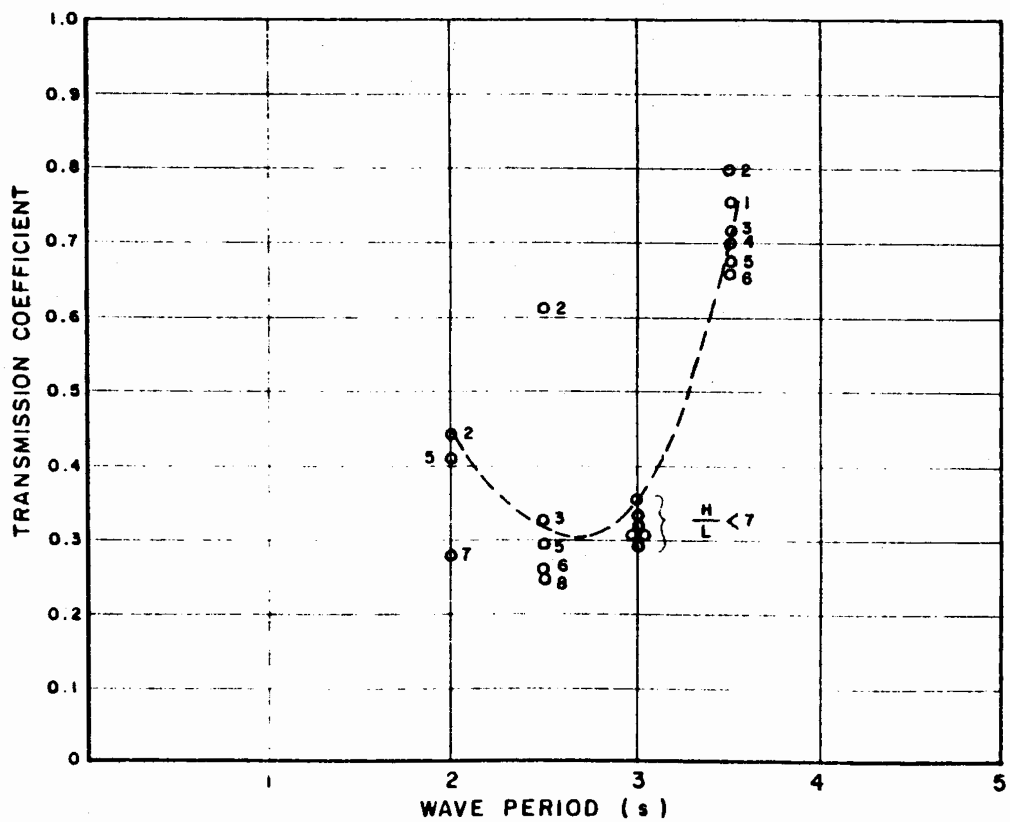
CARVER, 1979, CAISSON

FIGURE C11

WESTERN CANADA HYDRAULIC LABORATORIES LTD.

bcl 6638 A-WCH





MODEL SCALE 1:10
REGULAR WAVES

TEST CONDITIONS

WAVE HEIGHT	0.2 to 1.5 m
WAVE PERIOD	.2 to 3.5 s
DEPTH OF WATER	3.0 m
WAVE STEEPNESS	0.008 to 0.09
RELATIVE DEPTH	0.19 to 0.49 (depth = 3.0 m)

MOORING DETAILS

Moored with chain. Two chains each side. Weight of chain not provided.

Initial tension in mooring chain : 9.8 kN (depth = 9.0 m)
0 N (depth = 3.0 m)

Length of chain 61 m seaward side
27 m harbour side

NOTE

Dynamic properties of breakwater were modelled. Roll period of model = 7.1 s. Not clear that this period includes the effect of the moorings.

Numbers on graph show — percent wave steepness

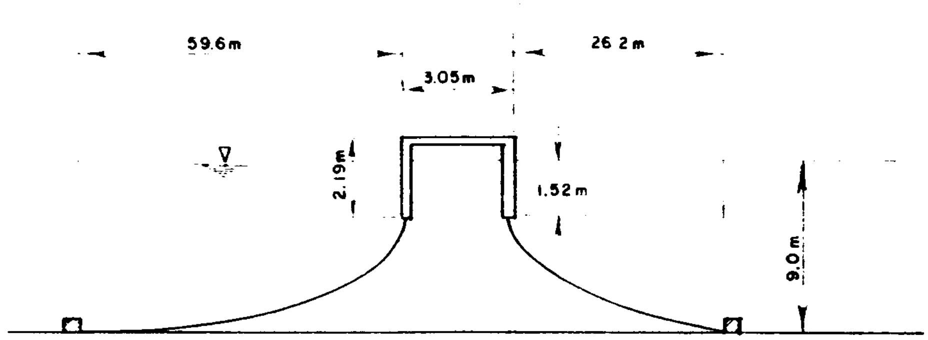
FLOATING BREAKWATER DESIGN

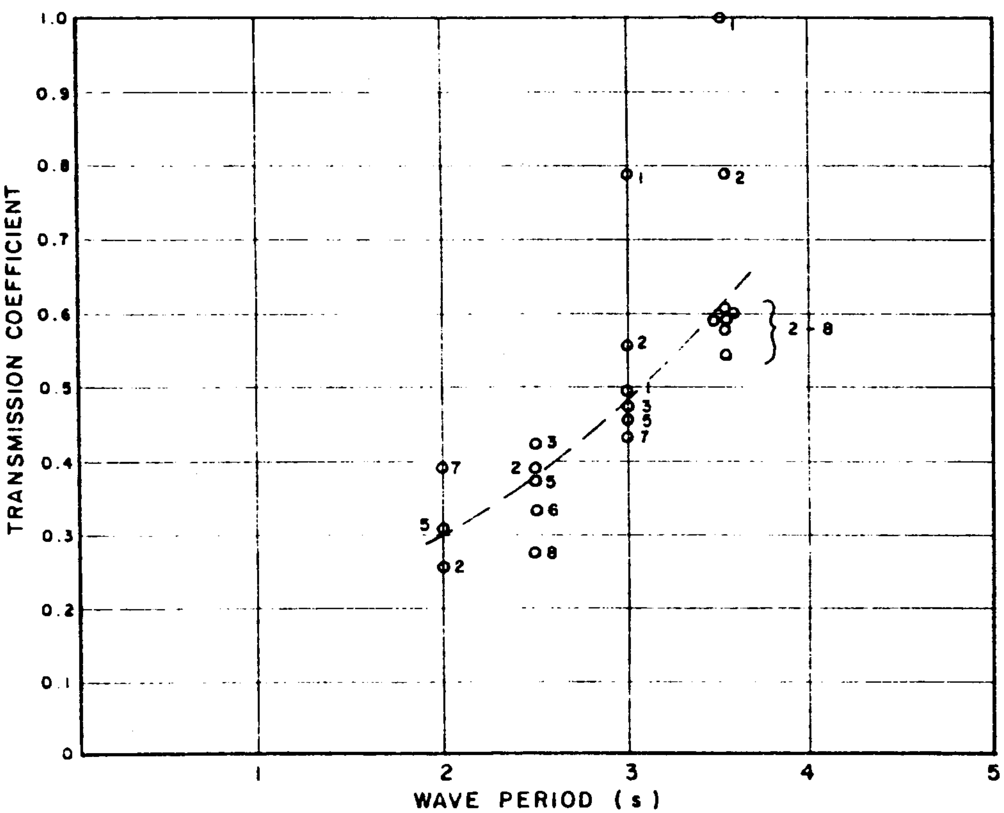
DAVIDSON, 1971, CAISSON

FIGURE C12

WESTERN CANADA HYDRAULIC LABORATORIES LTD.

bc-l 6638 A-WCH





MODEL SCALE 1:10
REGULAR WAVES

TEST CONDITIONS

WAVE PERIOD	2.0 to 3.5 s
WAVE HEIGHT	0.2 to 1.5 m
DEPTH OF WATER	9.0 m
WAVE STEEPNESS	0.008 to 0.09
RELATIVE DEPTH	0.47 to 1.43

MOORING DETAILS

Moored with chain. Two chains each side. Weight of chain not provided.

Initial tension in mooring chain 9.8 kN.

Length of chain 61 m seaward side
27 m harbour side

NOTE

Dynamic properties of breakwater modelled.

Roll period of model = 7.1 s

Not clear that this period includes the effect of the moorings.

Numbers on graph show —— percent wave steepness

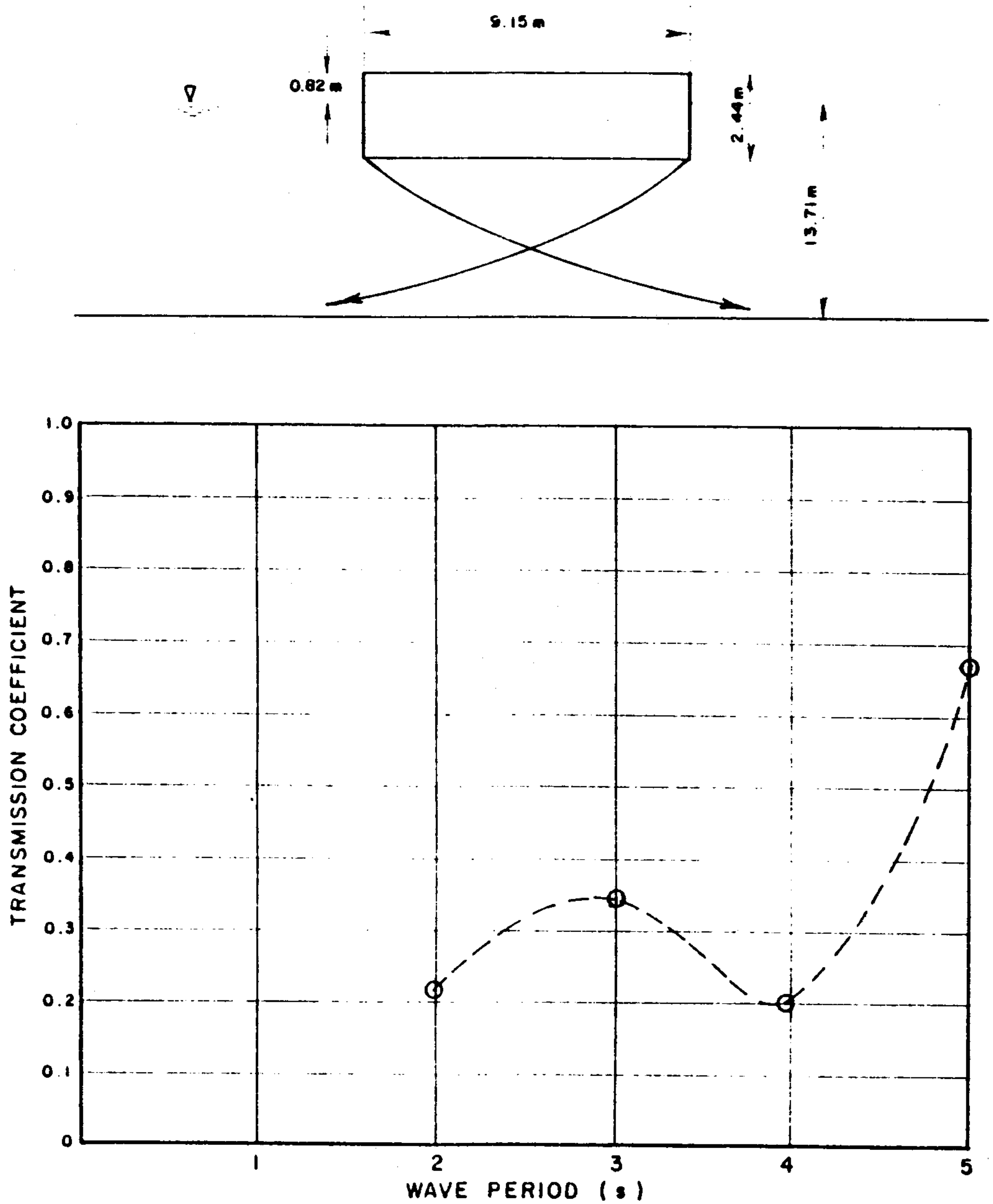
FLOATING BREAKWATER DESIGN

DAVIDSON, 1971, CAISSON

FIGURE C13

WESTERN CANADA HYDRAULIC LABORATORIES LTD.

bcil 6638 A-WCH



SCALE 1:30
REGULAR WAVES

TEST CONDITIONS

WAVE HEIGHT	0.2 m to 1.6 m
WAVE PERIOD	2.0 to 5.0 s
DEPTH OF WATER	13.7 m
WAVE STEEPNESS	0.04 (constant)
RELATIVE DEPTH	0.35 to 2.2

MOORING DETAILS

Breakwater moored with chain. Details of chain, initial tension and scope not available. Chains crossed underneath breakwater.

NOTE

Model construed from Aluminum plate to model reinfoirced concrete

Heave period	2.4 s
Roll period	4.4 s

This breakwater was also tested with no bottom panel, however, the draft was insufficient, air was lost, and the breakwater sank.

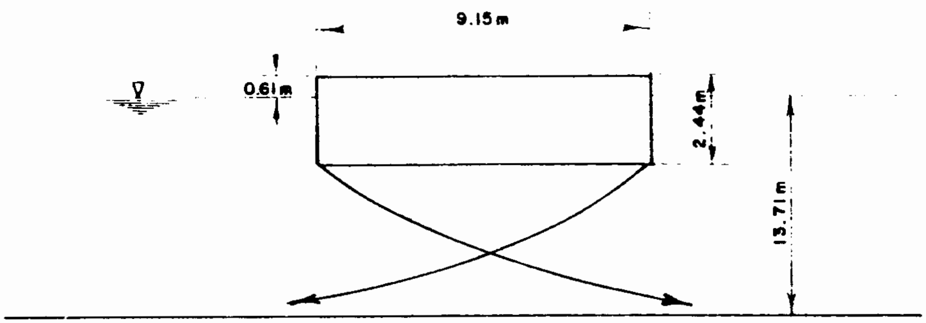
FLOATING BREAKWATER DESIGN

WCHL, 1975, CAISSON

WESTERN CANADA HYDRAULIC LABORATORIES LTD.

FIGURE C14

bc-16638 A-WCH



SCALE 1:30
REGULAR WAVES

TEST CONDITIONS

WAVE PERIOD	2.0 to 5.0 s
WAVE HEIGHT	0.24 to 1.6 m
DEPTH OF WATER	13.7 m
WAVE STEEPNESS	0.40 (constant)
RELATIVE DEPTH	0.35

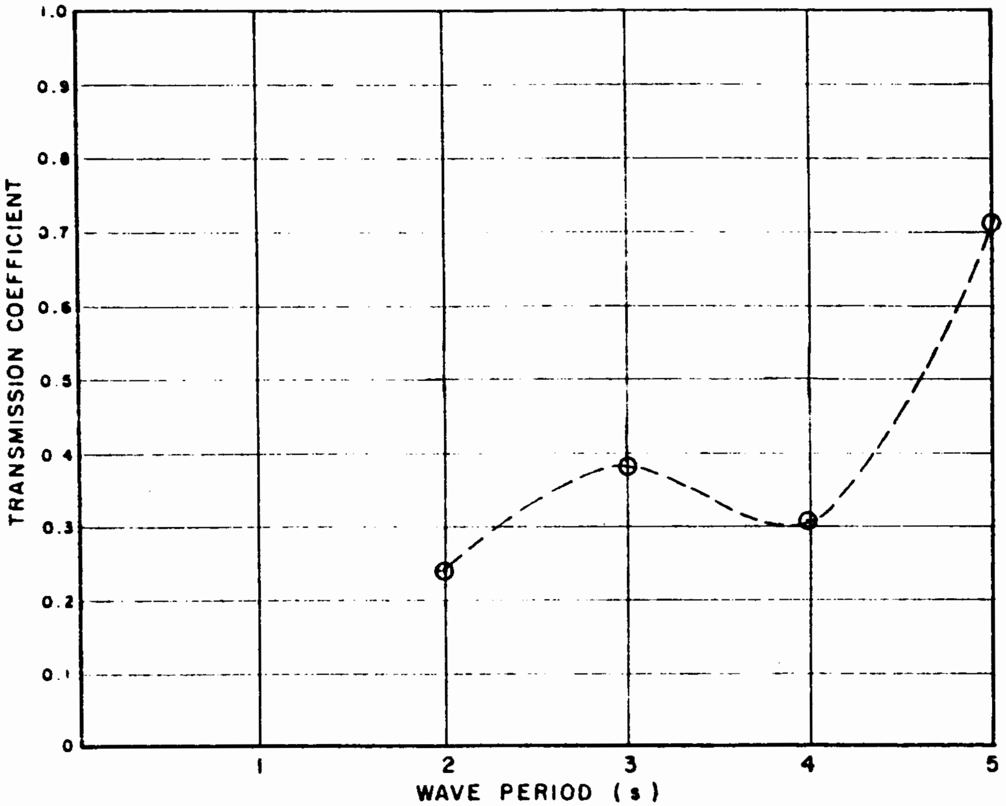
MOORING DETAILS

Moored with chain. Details of chain, initial tension, and scope not available. Chains crossed underneath breakwater.

NOTE

Model constructed from plexiglass to model reinforced concrete. Model was tested with and without water in the interior of breakwater.

Heave period	3.0 s
Roll period	3.7 s



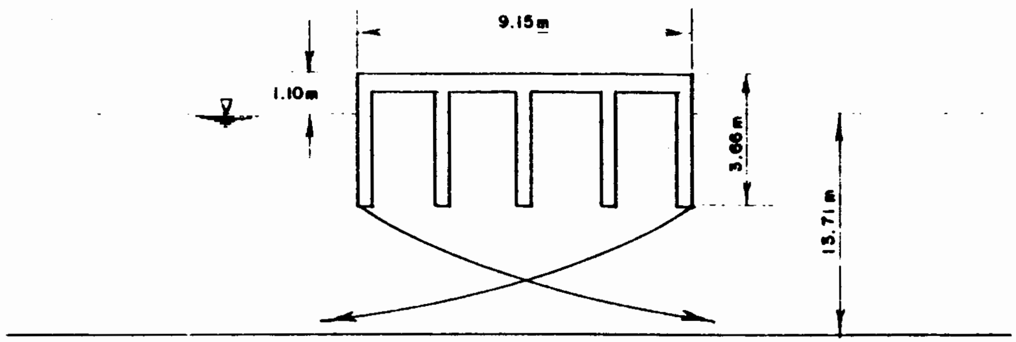
FLOATING BREAKWATER DESIGN

WCHL, 1975, CAISSON

WESTERN CANADA HYDRAULIC LABORATORIES LTD.

FIGURE C15

bc-16638 A-WCH



SCALE 1:30
REGULAR WAVES

TEST CONDITIONS

WAVE PERIOD	2.0 to 5.0 s
WAVE HEIGHT	0.24 to 1.6 m
DEPTH OF WATER	13.7 m
WAVE STEEPNESS	0.04 (constant)
RELATIVE DEPTH	0.35

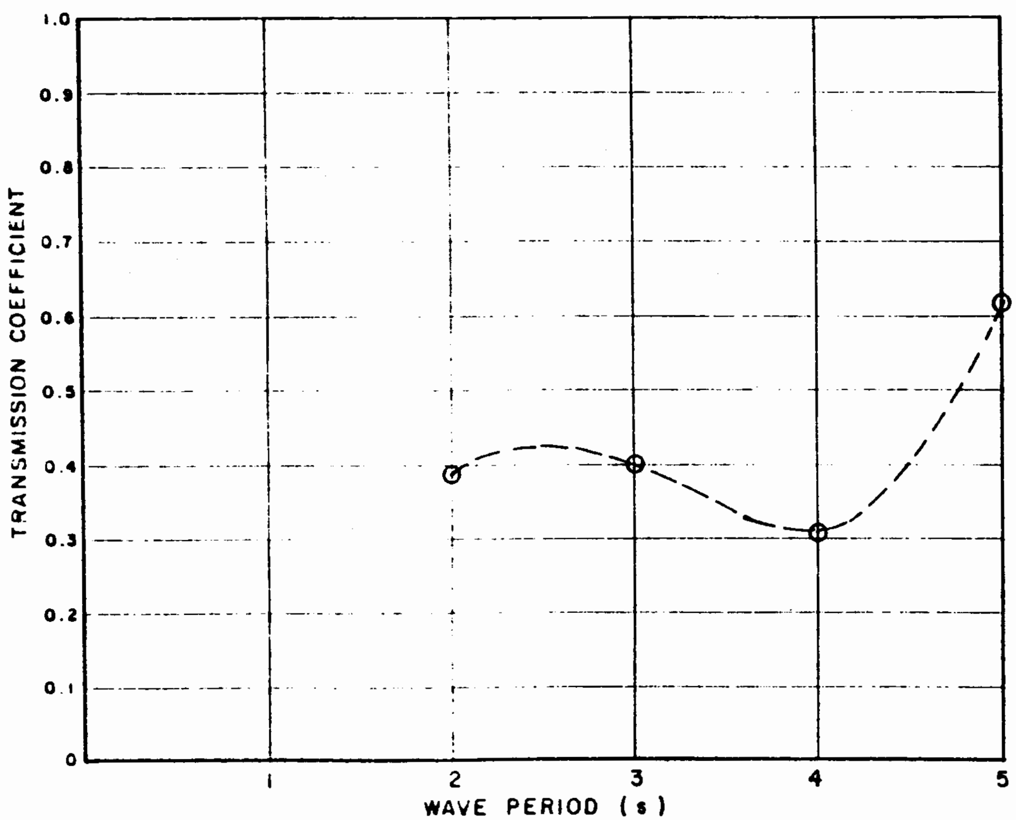
MOORING DETAILS

Moored with chain. Details of chain, initial tension and scope not available. Chains crossed underneath breakwater.

NOTE

Model constructed from Aluminum plate to model reinforced concrete.

Heave period	3.6 s
Roll period	-



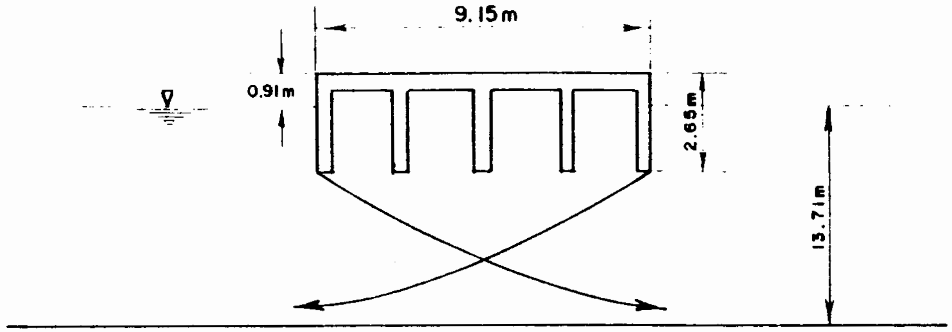
FLOATING BREAKWATER DESIGN

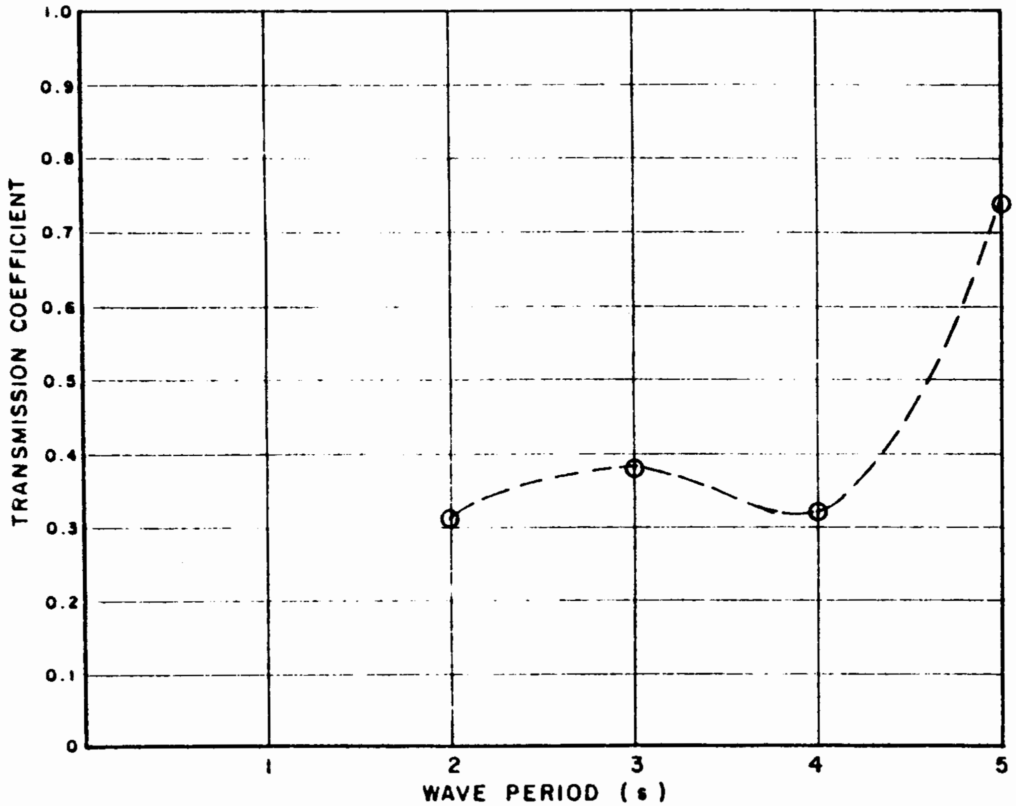
WCHL, 1975, CAISSON

FIGURE C16

WESTERN CANADA HYDRAULIC LABORATORIES LTD.

bc-16538 A-WCH





SCALE 1:30
REGULAR WAVES

TEST CONDITIONS

WAVE PERIOD	2.0 to 5.0 s
WAVE HEIGHT	0.24 to 1.6 m
DEPTH OF WATER	13.7 m
WAVE STEEPNESS	0.04 (constant)
RELATIVE DEPTH	0.35

MOORING DETAILS

Moored with chain. Details of chain initial tension and scope not available. Chains crossed underneath breakwater.

NOTE

Model constructed from Aluminum plate to model reinforced concrete.

Heave period	2.2 s
Roll period	-

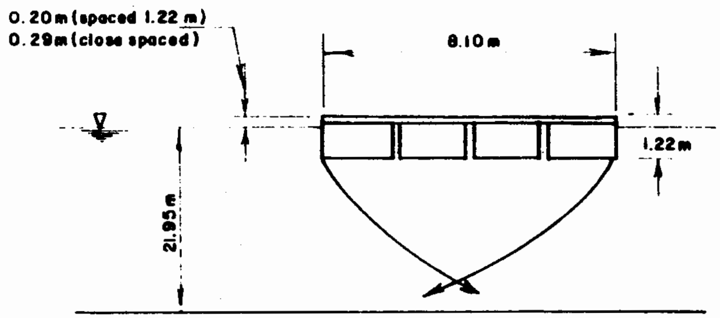
FLOATING BREAKWATER DESIGN

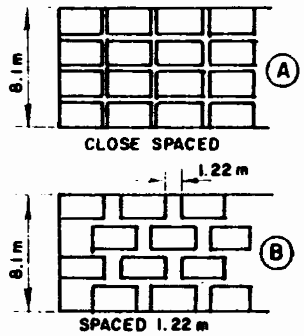
WCHL, 1975, CAISSON

FIGURE C17

WESTERN CANADA HYDRAULIC LABORATORIES LTD.

bc-16638 A-WCH





UNDERSIDE PLAN VIEW

SCALE 1:24
REGULAR WAVES

TEST CONDITIONS

WAVE PERIOD	2.0 to 5.0 s
WAVE HEIGHT	0.3 to 1.5 m
DEPTH OF WATER	22 m
WAVE STEEPNESS	0.04 (constant)
RELATIVE DEPTH	0.5

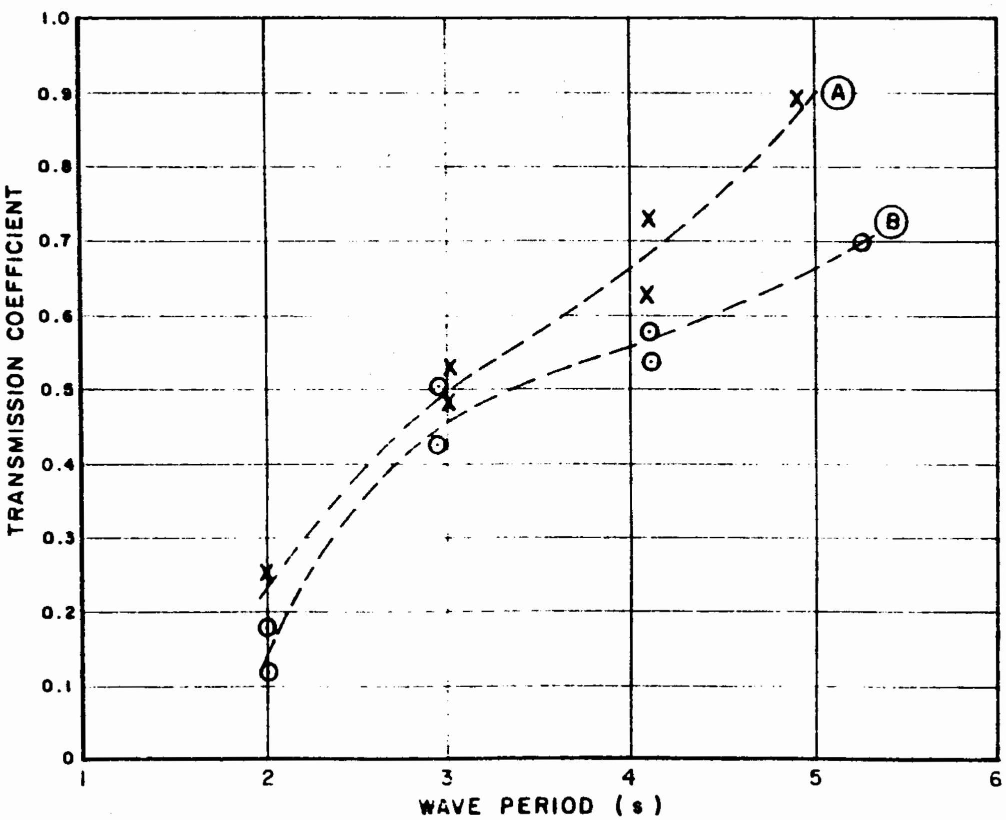
MOORING DETAILS

Moored with 66 m of nylon line. Diameter mass and initial tension of mooring line not available. Mooring lines crossed underneath breakwater.

NOTE

Natural periods determined without mooring lines.

	Closed Spaced	Spaced 1.22 m
Heave	4.5	4.2
Roll	2.3	3.8



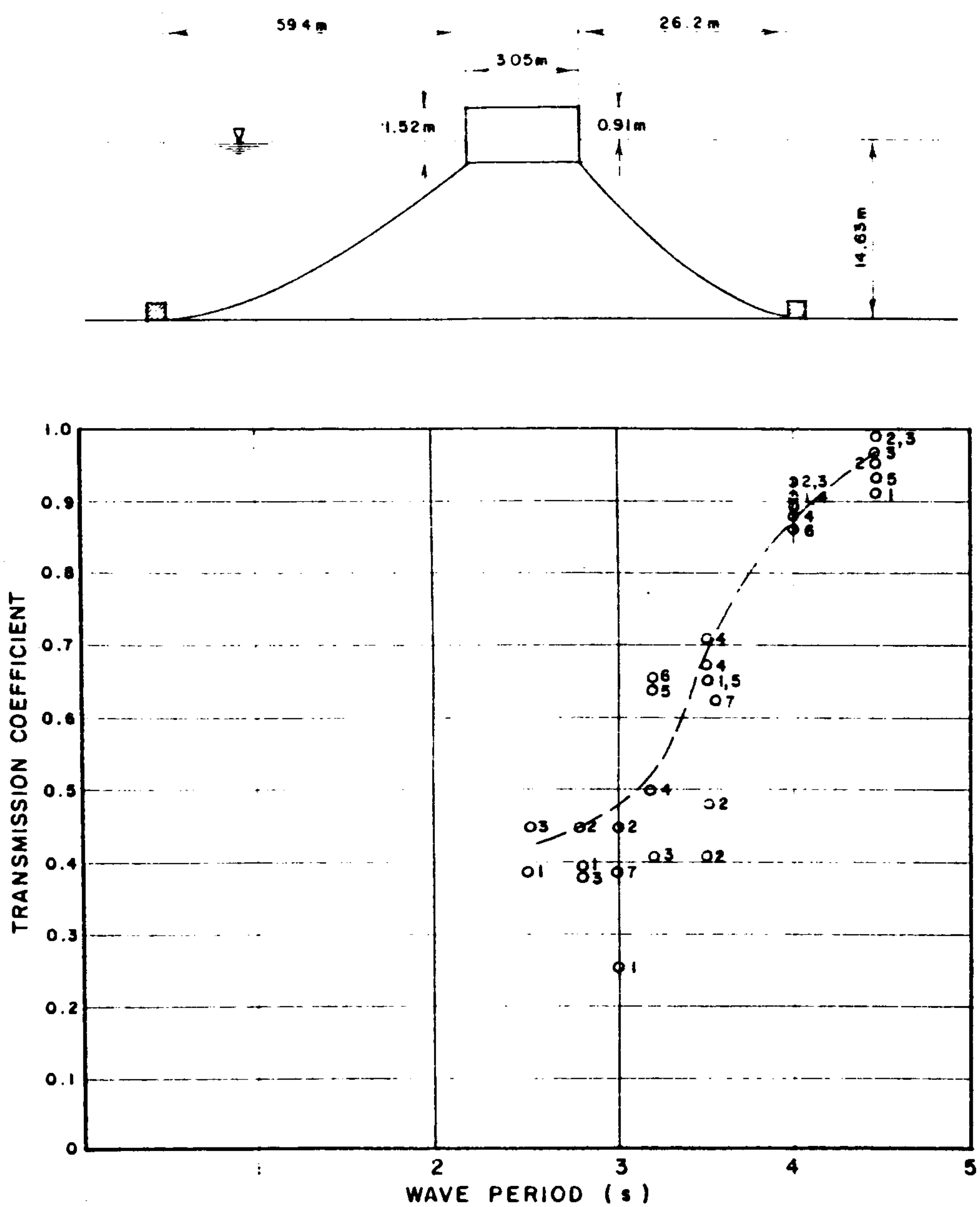
FLOATING BREAKWATER DESIGN

WCHL, 1979, CAISSON

WESTERN CANADA HYDRAULIC LABORATORIES LTD.

bcl 6638 A-WCH

FIGURE C18



SCALE 1:16
REGULAR WAVES

TEST CONDITIONS

WAVE PERIOD	2.5 to 4.5 s
WAVE HEIGHT	0.13 m to 1.5 m
DEPTH OF WATER	14.6 m
WAVE STEEPNESS	0.005 to 0.07
RELATIVE DEPTH	0.5

MOORING DETAILS

Moored with 2 mm (model) double link galvanized chain. Mass not available. Length approximately 22 m. (Not confirmed in report). Initial tension not available.

NOTE

Model constructed of 12.5 mm plywood with 6.4 mm steel plates screwed on inside of all sides. Natural periods measured:

Heave	3.0 s) With mooring
Roll	9.0 s) chains

Numbers on graph show — percent wave steepness

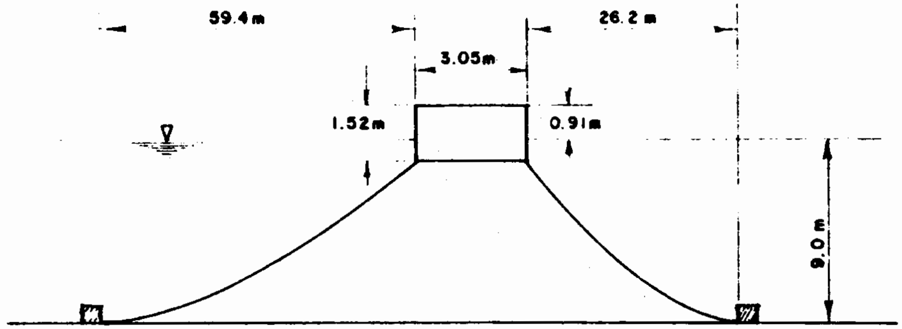
FLOATING BREAKWATER DESIGN

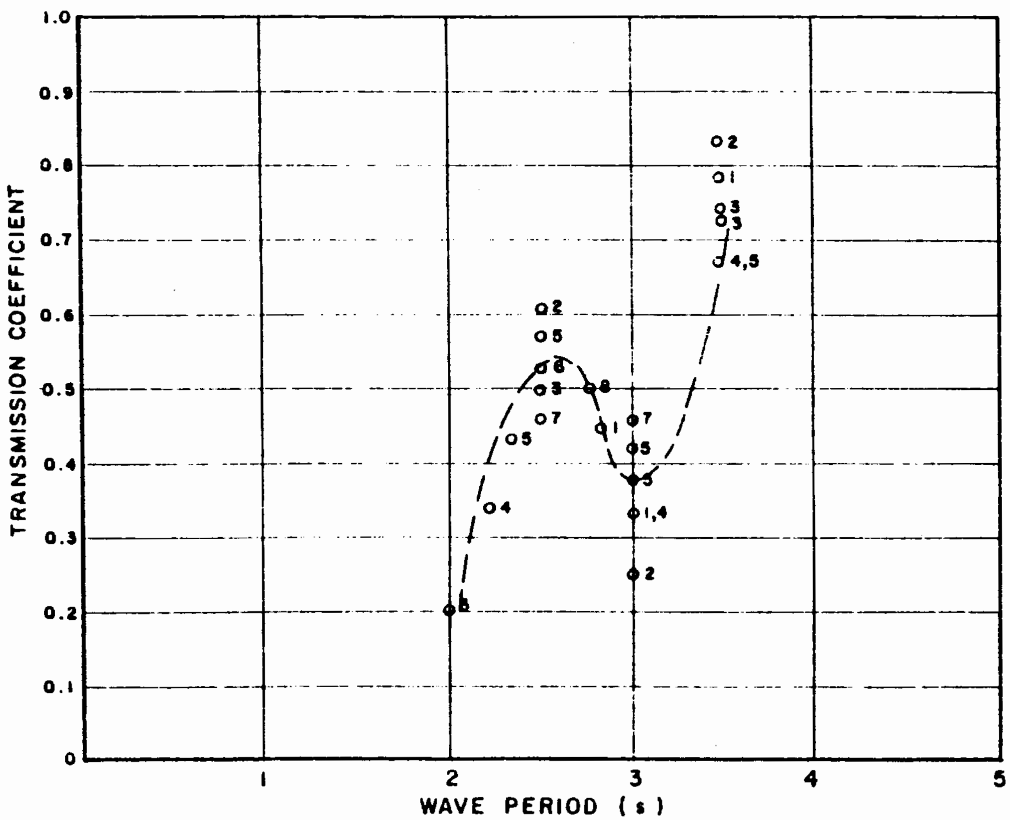
NECE & RICHEY, 1976, CAISSON

FIGURE C19

WESTERN CANADA HYDRAULIC LABORATORIES LTD.

bcil 6638 A-WCH





SCALE 1:10
REGULAR WAVES

TEST CONDITIONS

WAVE PERIOD	2.0 to 3.5 s
WAVE HEIGHT	.15 to .98 m
DEPTH OF WATER	9.0 m
WAVE STEEPNESS	0.003 to 0.078
RELATIVE DEPTH	0.17

MOORING DETAILS

Moored with 2 mm (model) double link galvanized chain. Mass not available. Length approximately 22 m. (Not confirmed in report). Initial tension not available.

NOTE

Model constructed of 12.5 mm plywood with 0.4 mm steel plate fastened on inside of all surfaces. Natural periods measured:

Heave	3.0 s) with mooring
Roll	9.0 s) chains

Numbers on graph show — percent wave steepness

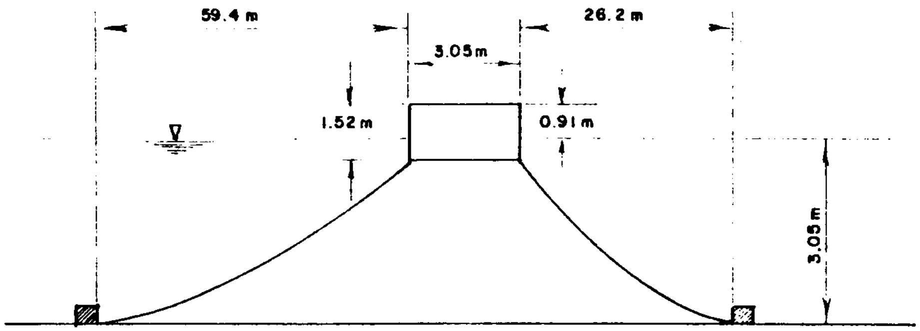
FLOATING BREAKWATER DESIGN

NECE & RICHEY, 1976, CAISSON

FIGURE C20

WESTERN CANADA HYDRAULIC LABORATORIES LTD.

bc-l 6638 A-WCH



SCALE 1:10
REGULAR WAVES

TEST CONDITIONS

WAVE PERIOD	2.0 - 4.0 s
WAVE HEIGHT	.12 - 1.0 m
DEPTH OF WATER	3.0 m
WAVE STEEPNESS	0.007 to 0.089
RELATIVE DEPTH	0.16 to 0.5

MOORING DETAILS

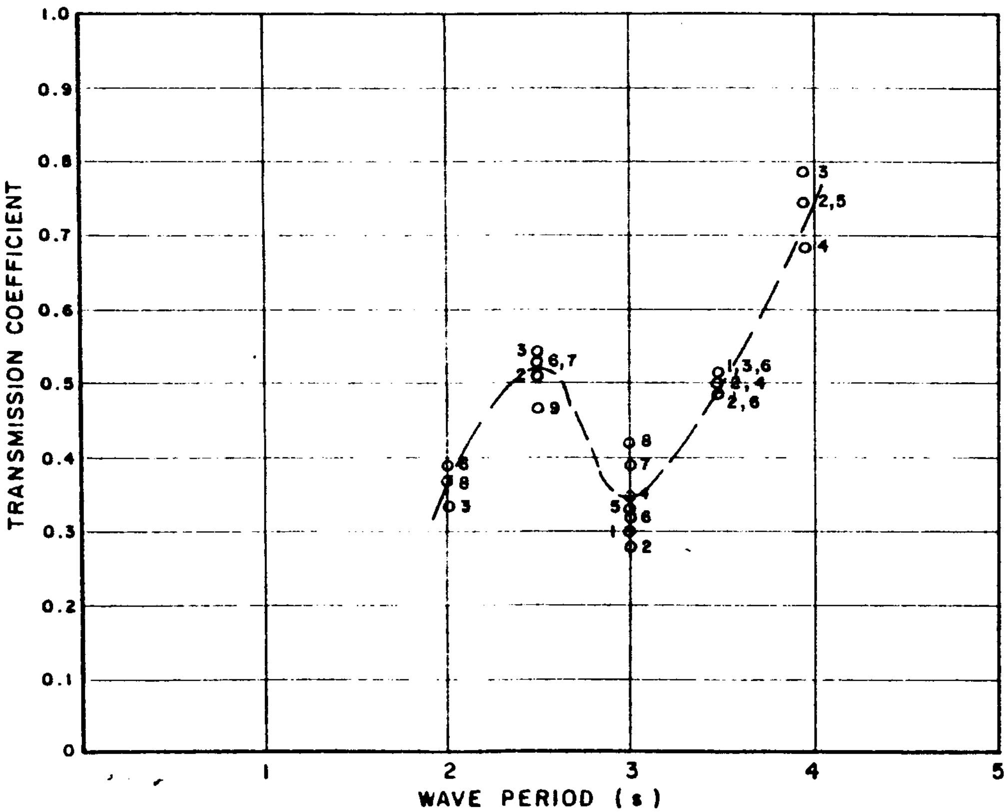
Moored with 2 mm (model) double link galvanized chain, mass not available. Length approximately 22m. (Not confirmed in report). Initial tension not available.

NOTE

Model constructed of 12.5 mm plywood with 6.4 mm steel plate fastened to inside surfaces. Natural periods measured:

Heave	3.0 s) with mooring
Roll	9.0 s) chains

Numbers on graph show — percent wave steepness



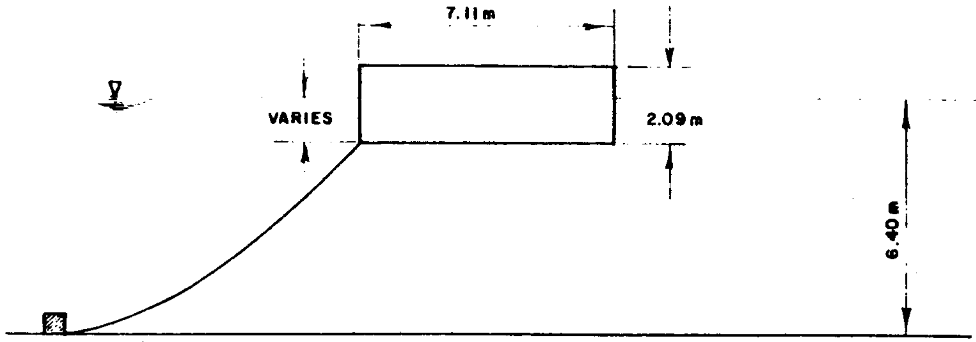
FLOATING BREAKWATER DESIGN

NECE & RICHEY, 1976, CAISSON

FIGURE C21

WESTERN CANADA HYDRAULIC LABORATORIES LTD.

bcl 6638 A-WCH



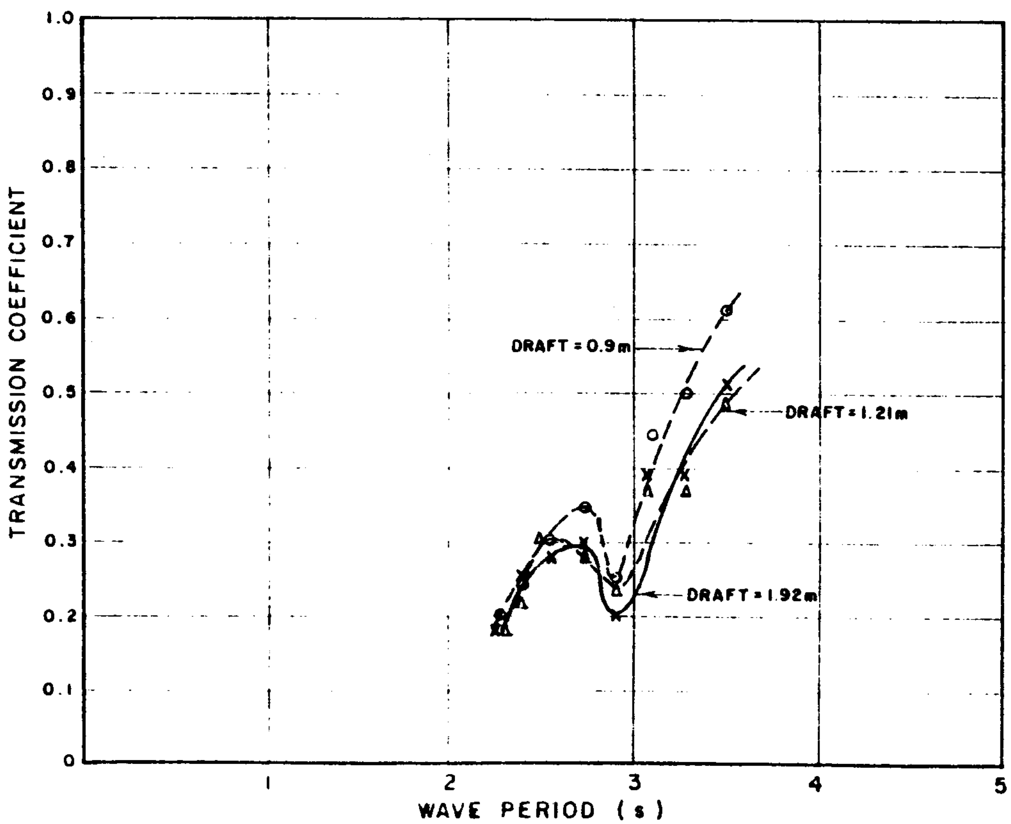
SCALE 1:14
REGULAR WAVES

TEST CONDITIONS

WAVE PERIOD	2.3 to 3.5 s
WAVE HEIGHT	not available
DEPTH OF WATER	6.4 m
WAVE STEEPNESS	Maintained between 0.4 and 0.7
RELATIVE DEPTH	0.34

MOORING DETAILS

Moored with 1.5 mm (model) stainless steel cable. A spring with a linear spring constant connected the cable to a force meter. Stiffness not available.
Initial tension = O
Scope not available.



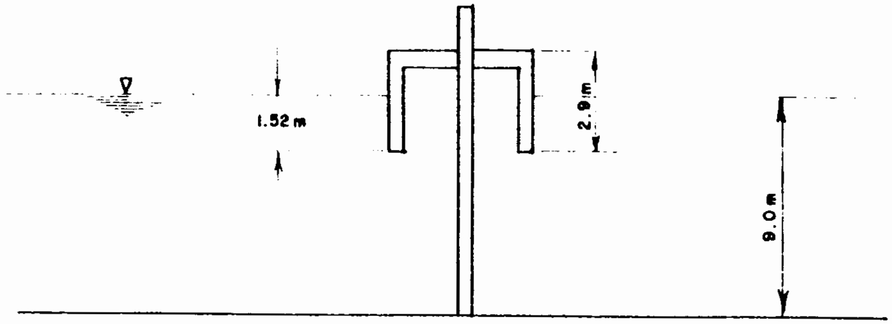
FLOATING BREAKWATER DESIGN

OFUYA, 1968, CAISSON

FIGURE C22

WESTERN CANADA HYDRAULIC LABORATORIES LTD.

bc-16638 A-WCH



MODEL SCALE 1:10
REGULAR WAVES

TEST CONDITIONS

WAVE PERIOD	2 to 3.5 s
WAVE HEIGHT	0.2 to 1.5 m
DEPTH OF WATER	3.05 m
WAVE STEEPNESS	.008 to .09
RELATIVE DEPTH	0.19 to 0.49

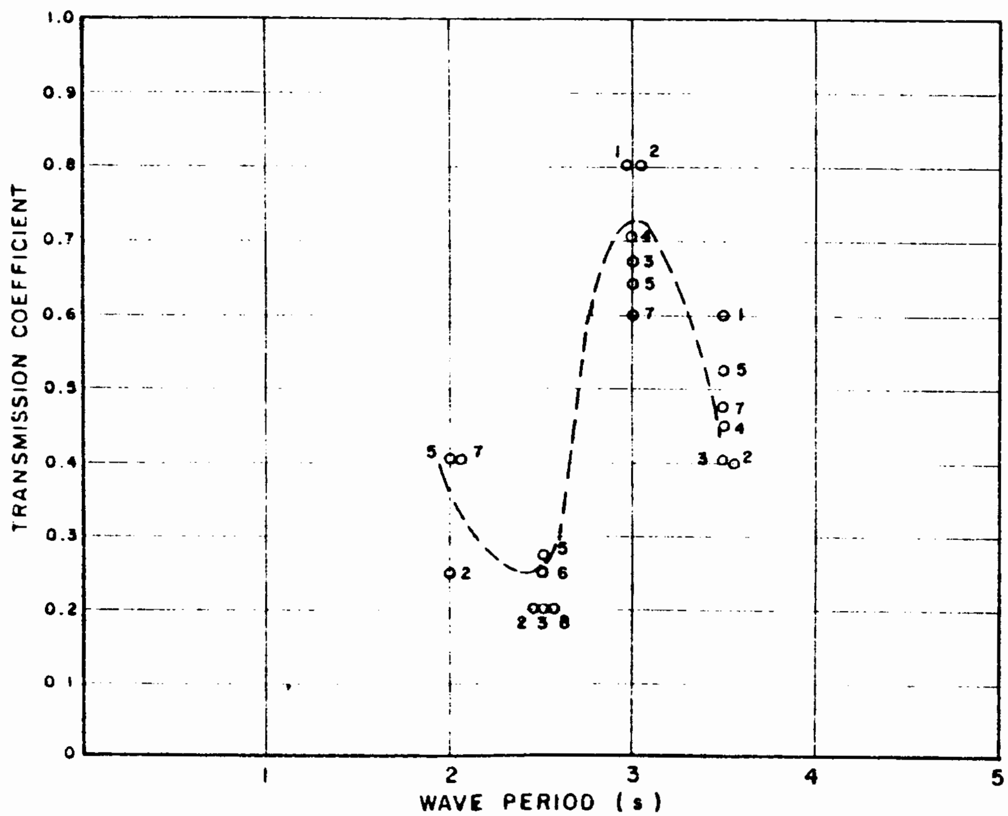
MOORING DETAILS

Moored by a .5 m diameter pile at centre line of breakwater.

NOTE

Breakwater was observed to bind on the pile.

Numbers on graph show — percent wave steepness

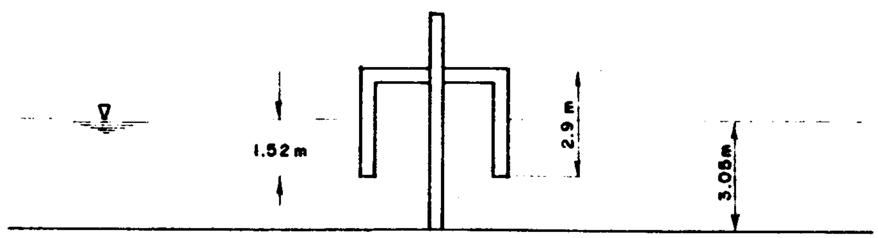


FLOATING BREAKWATER DESIGN

DAVIDSON, 1971, CAISSON

FIGURE C23

WESTERN CANADA HYDRAULIC LABORATORIES LTD.



MODEL SCALE 1:10
REGULAR WAVES

TEST CONDITIONS

WAVE PERIOD	2 to 3.5 s
WAVE HEIGHT	0.2 to 1.5 m
DEPTH OF WATER	3.05 m
WAVE STEEPNESS	.008 to .09
RELATIVE DEPTH	0.19 to 0.49

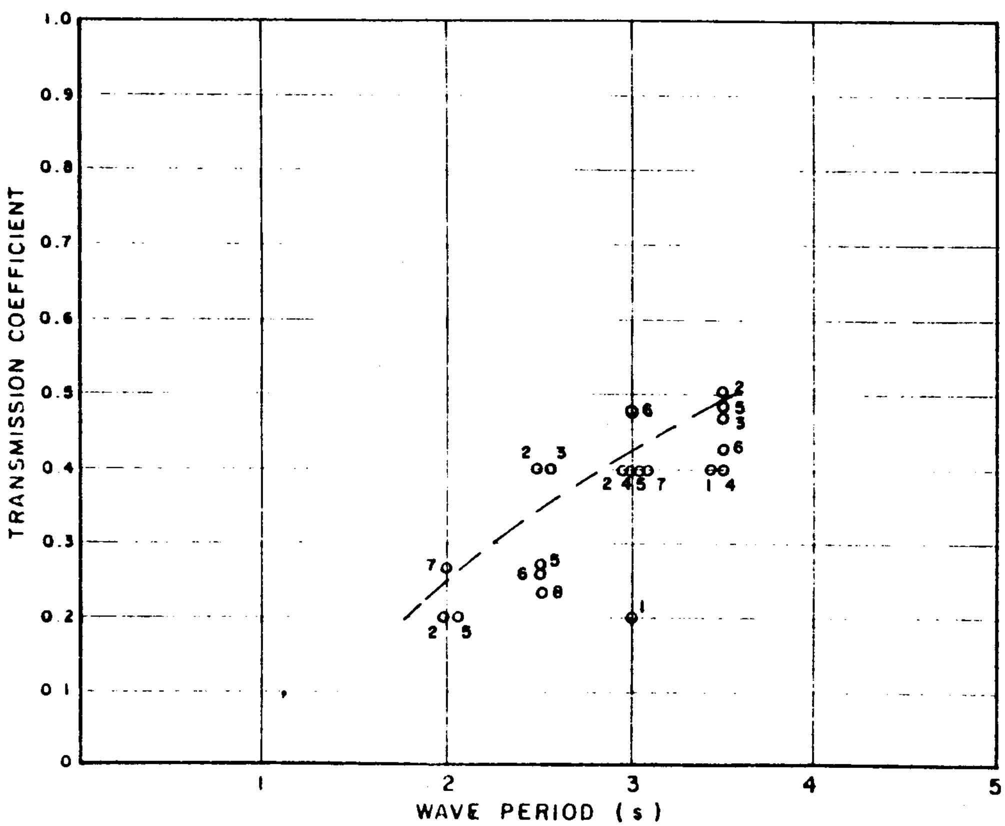
MOORING DETAILS

Moored by a .5 m diameter pile at centre line of breakwater.

NOTE

Breakwater was observed to bind on the pile.

Numbers on graph show — percent wave steepness



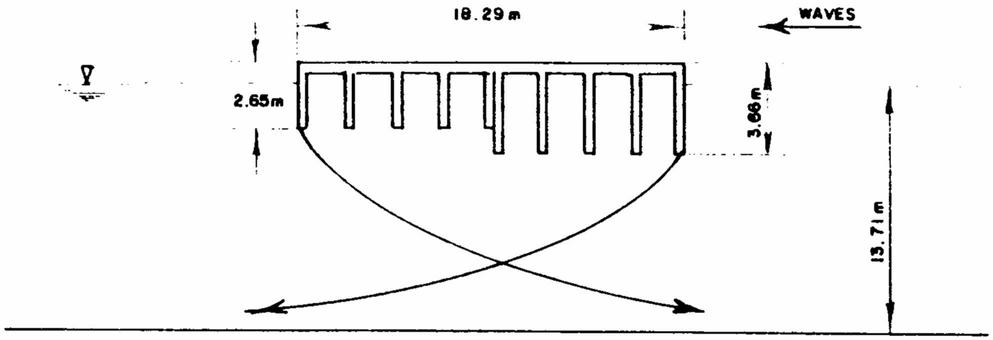
FLOATING BREAKWATER DESIGN

DAVIDSON, 1971, CAISSON

FIGURE C24

WESTERN CANADA HYDRAULIC LABORATORIES LTD.

6-16038 A-WCH



SCALE 1:30
REGULAR WAVES

TEST CONDITIONS

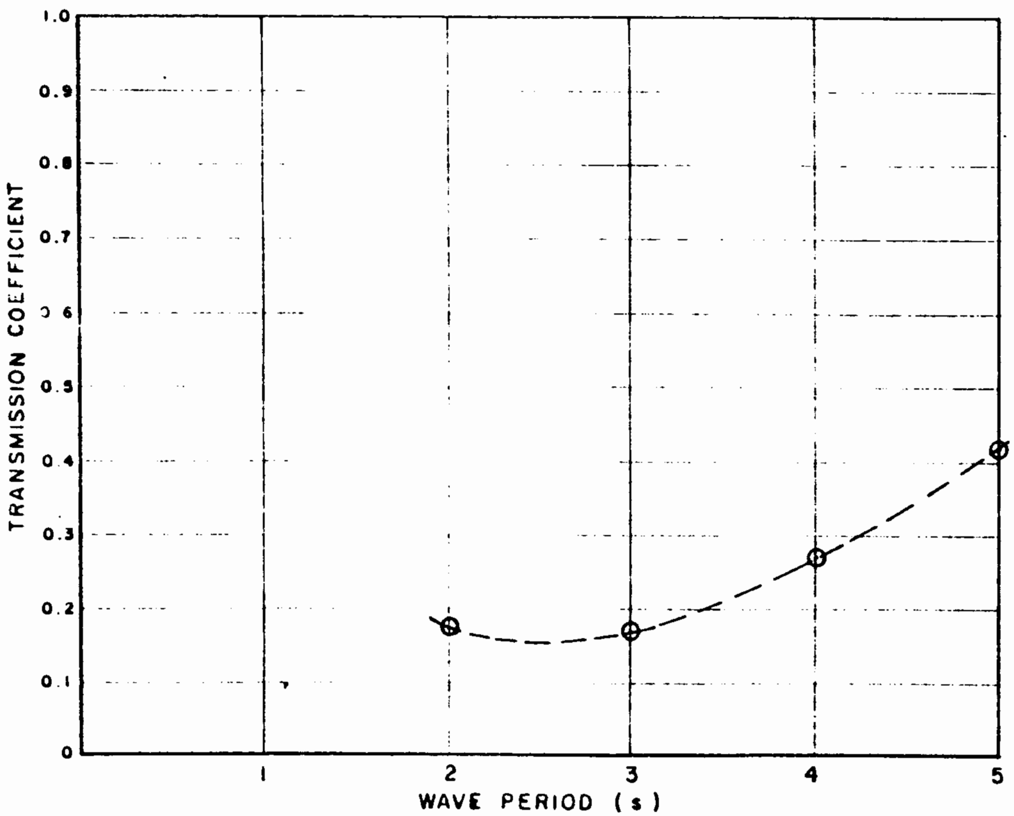
WAVE PERIOD	2.0 to 5.0 s
WAVE HEIGHT	0.24 to 1.6 m
DEPTH OF WATER	13.7 m
WAVE STEEPNESS	0.40 (constant)
RELATIVE DEPTH	0.35

MOORING DETAILS

Moored with chain. Details of chain initial tension and scope not available. Chains crossed underneath breakwater.

NOTE

Model constructed from Aluminum plate to model reinforced concrete. Draft and natural periods not available.



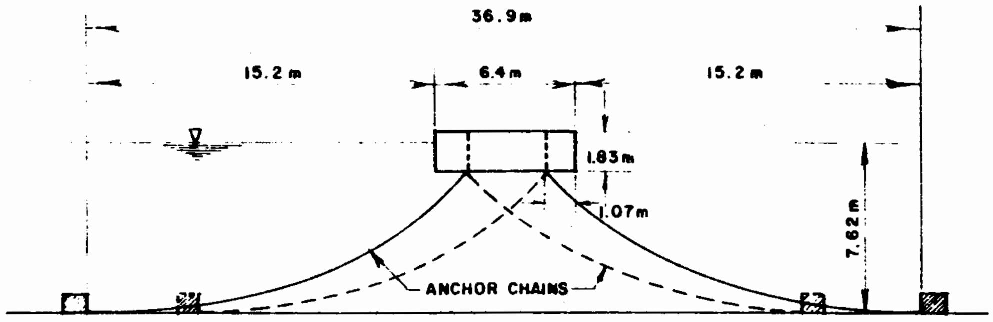
FLOATING BREAKWATER DESIGN

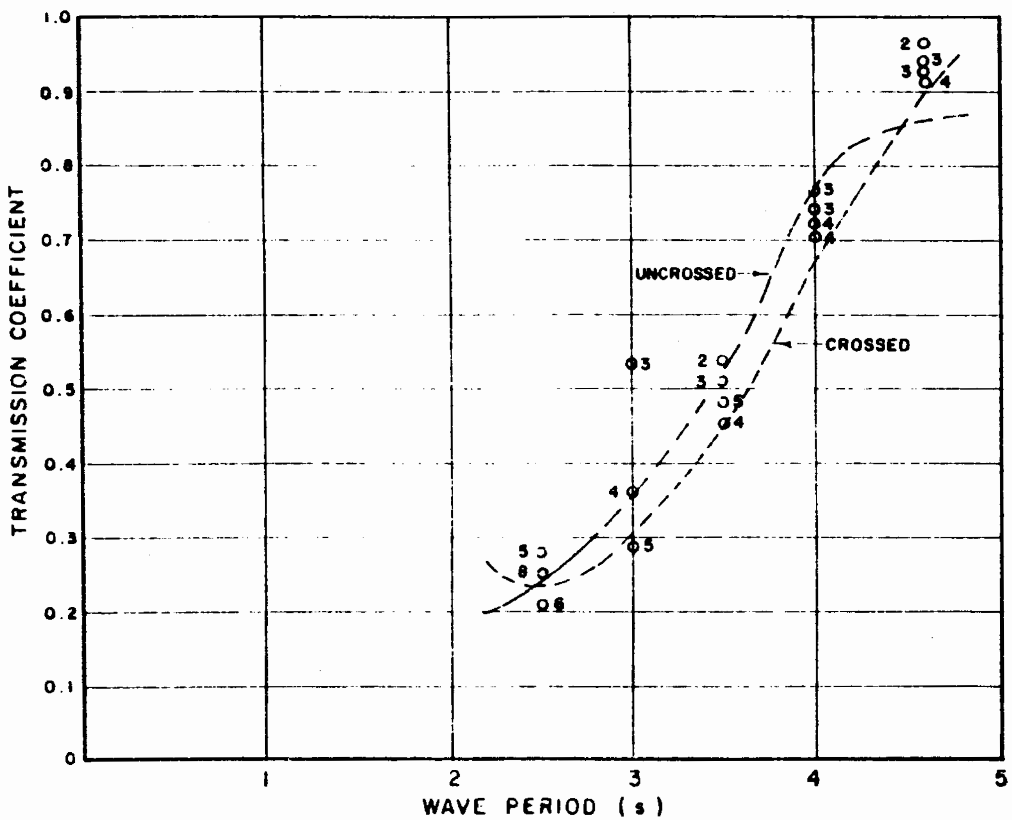
WCHL, 1975, CAISSON

WESTERN CANADA HYDRAULIC LABORATORIES LTD.

bc-16638 A-WCH

FIGURE C25





SCALE 1:10
REGULAR WAVES

TEST CONDITIONS

WAVE PERIOD	2.5 to 4.5 s
WAVE HEIGHT	0.46 to 1.1 m
DEPTH OF WATER	7.6m
WAVE STEEPNESS	0 .02 to 0.08
RELATIVE DEPTH	0.4

MOORING DETAILS

Moored with 22 m of 21.2 kg/m chain, link diameter = 30 mm. Details required to calculate initial anchor tension provided in figure above.

Breakwater tested with crossed and uncrossed mooring lines.

NOTE

Dynamic properties of breakwater modelled

Heave 6.3 s
Roll 3.1 s
Pitch 2.5 s
Sway 13.1 s

(includes effect of moorings. No
(change for crossed or uncrossed
(lines.

Numbers on graph show — percent wave steepness

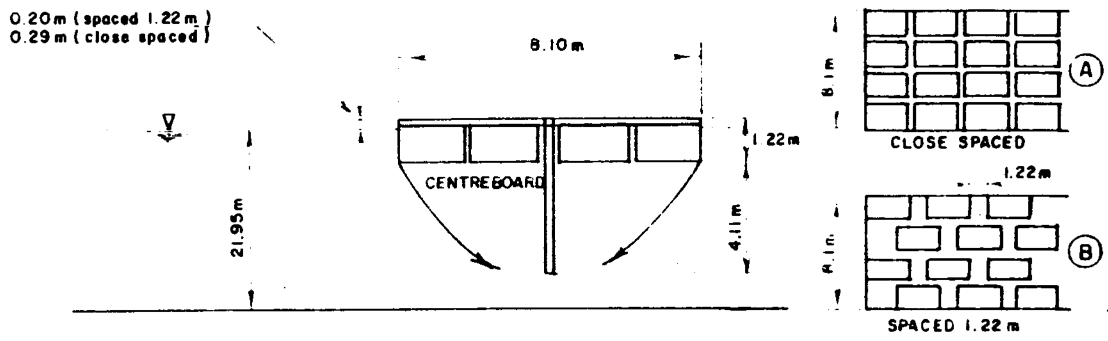
FLOATING BREAKWATER DESIGN

CARVER, 1979, PONTOON

WESTERN CANADA HYDRAULIC LABORATORIES LTD.

bc-16638 A-WCH

FIGURE C26



UNDERSIDE PLAN VIEW

SCALE 1:24
REGULAR WAVES

TEST CONDITIONS

WAVE PERIOD	2.0 s to 5.0 s
WAVE HEIGHT	0.3 m to 1.5 m
DEPTH OF WATER	22 m
WAVE STEEPNESS	0.04 (constant)
RELATIVE DEPTH	0.5

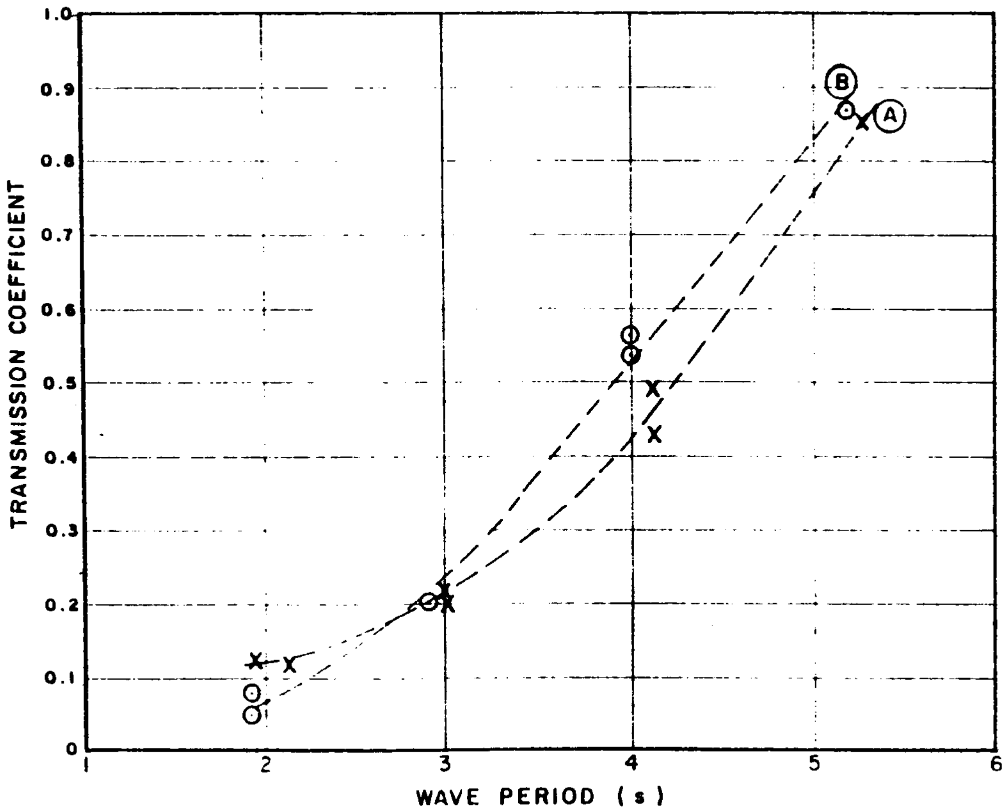
MOORING DETAILS

Moored with 66 m of nylon line. Diameter, mass and initial tension of mooring line not available. Mooring lines crossed underneath breakwater.

NOTE

Natural periods determined without mooring lines

Close spaced	Spaced 1.22 m
Heave 4.9 s	Heave 4.6 s
Roll 3.9	Roll 3.0 s



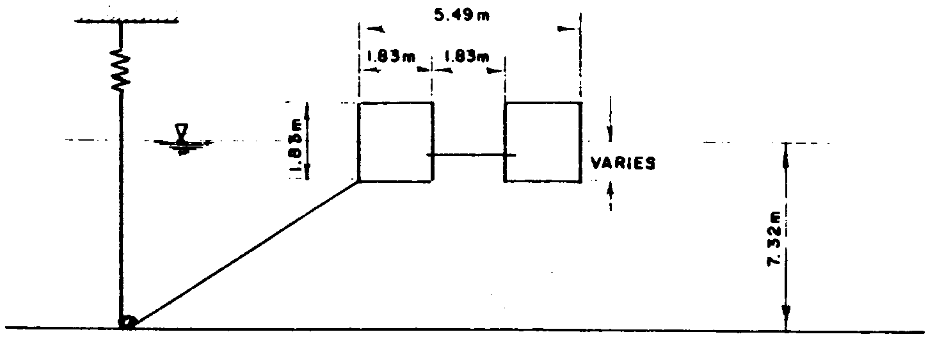
FLOATING BREAKWATER DESIGN

WCHL, 1979, PONTOON

WESTERN CANADA HYDRAULIC LABORATORIES LTD.

FIGURE C27

bci16638 A-WCH



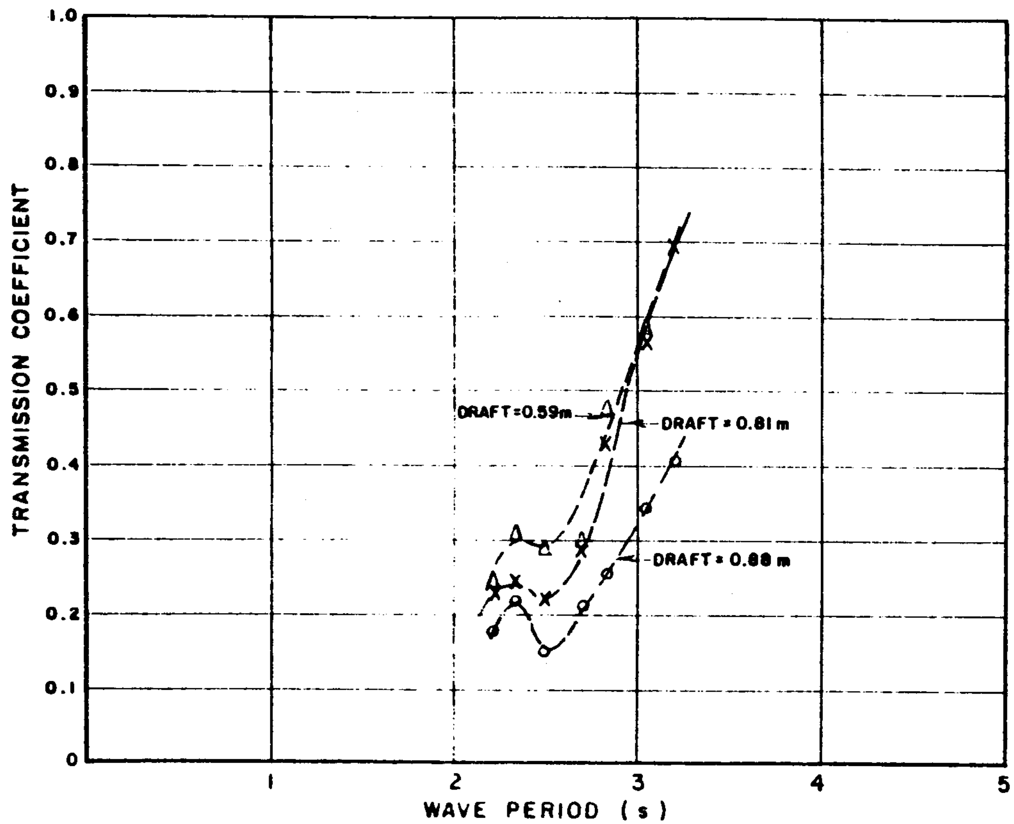
SCALE 1:12
REGULAR WAVES

TEST CONDITIONS

WAVE PERIOD	2.2 to 3.0 s
WAVE HEIGHT	Not available
DEPTH OF WATER	7.32 m
WAVE STEEPNESS	0.04 to 0.06
RELATIVE DEPTH	0.46 to 0.96

MOORING DETAILS

Moored with 1.5 mm (model) stainless steel cable. A spring with a linear spring constant connected cable to a force meter.
Initial tension = 0 Scope not available.



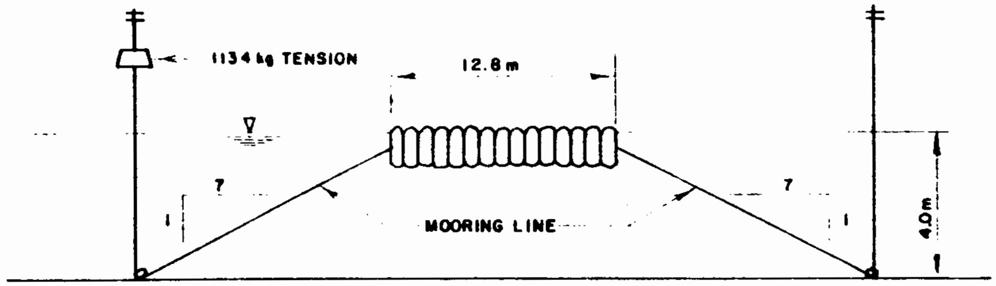
FLOATING BREAKWATER DESIGN

OFUYA, 1968, PONTOON

FIGURE C28

WESTERN CANADA HYDRAULIC LABORATORIES LTD.

bc-l 6638 A-WCH



SCALE 1:1
REGULAR WAVES

TEST CONDITIONS

WAVE PERIOD	2.9 s to 8.3 s
WAVE HEIGHT	0.3 m to 1.32 m
DEPTH OF WATER	2m and 4m
WAVE STEEPNESS	0.006 to 0.1
RELATIVE DEPTH	0.08 to 0.34

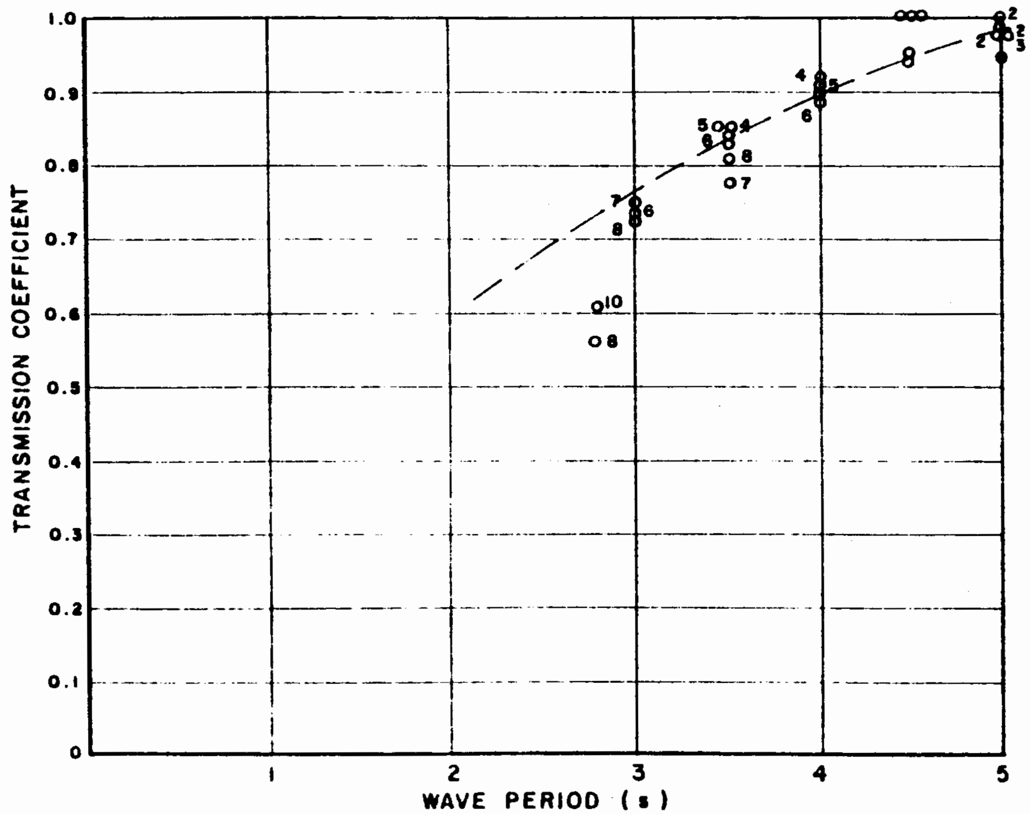
MOORING CONDITIONS

Breakwater moored with 6 m of open link chain and an unknown length of 4.8 mm stainless steel cable on the seaward side. Landward mooring cable was 2.4 mm stainless steel cable. Initial tension not available.

NOTE

Limitations of the wave generator resulted in a wide range of steepness over the range of periods tested. Steepness not held constant.

Numbers on graph show — percent wave steepness



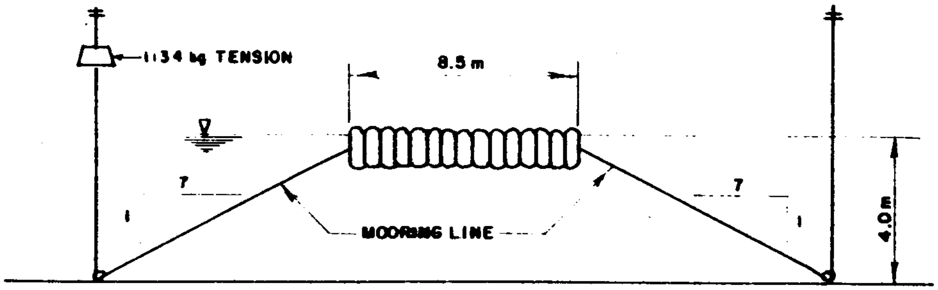
FLOATING BREAKWATER DESIGN

GILES & SORENSON, 1978, FLOATING TIRE

FIGURE C29

WESTERN CANADA HYDRAULIC LABORATORIES LTD.

bcil 6638 A-WCH



SCALE 1:1
REGULAR WAVES

TEST CONDITIONS

WAVE PERIOD	2.8 to 8.3 s
WAVE HEIGHTS	0.3 to 1.3 m
DEPTH OF WATER	4 m
WAVE STEEPNESS	0.01 to 0.1
RELATIVE DEPTH	0.08 to 0.34

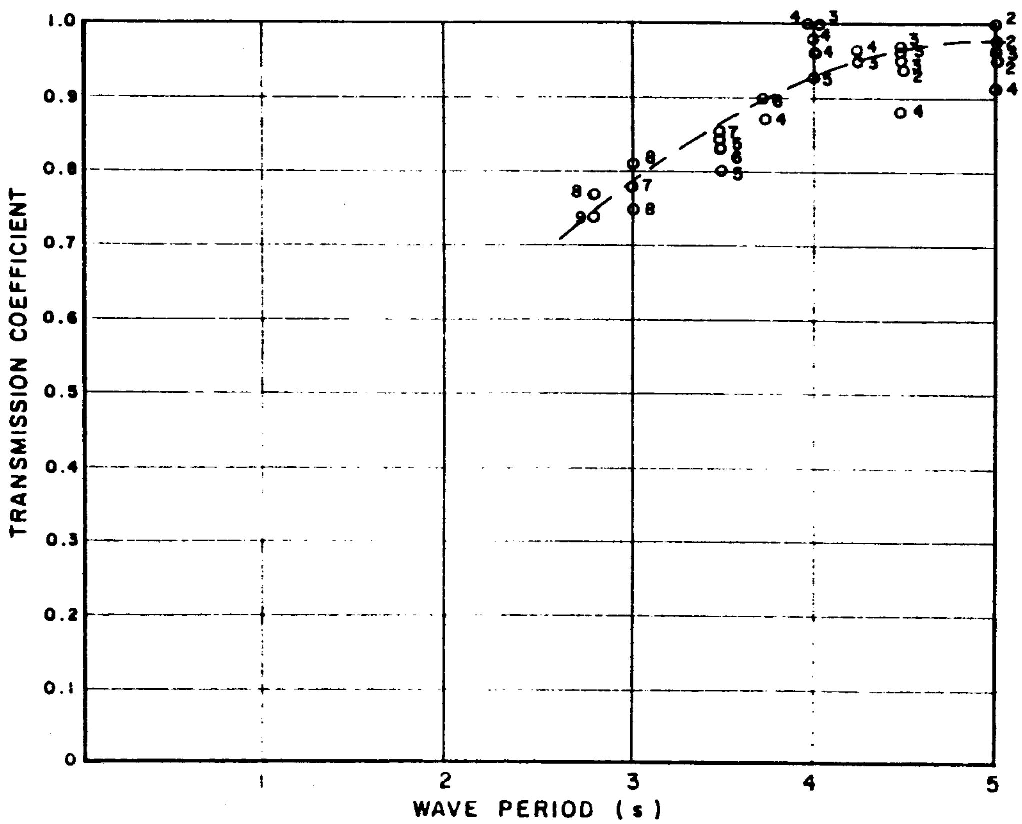
MOORING DETAILS

Moored with 6 m of open link chain and unspecified length of 4.8 mm diameter stainless steel cable on seaward side. Landward 4.8 mm diameter cable only. Initial tension not available.

NOTE

Limitations of wave generator resulted in a wide range of steepness over the range of periods tested.

Numbers on graph show — percent wave steepness



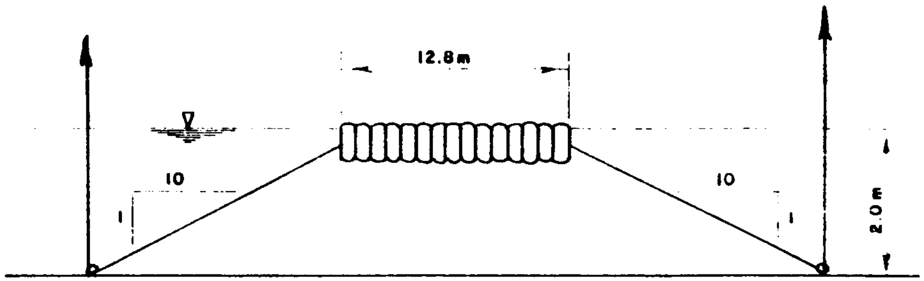
FLOATING BREAKWATER DESIGN

GILES & SORESON, 1978, FLOATING TIRE

FIGURE C30

WESTERN CANADA HYDRAULIC LABORATORIES LTD.

bc-l 6638 A-WCH



SCALE 1:1
REGULAR WAVES

TEST CONDTIONS

WAVE PERIOD	2.6 to 8.0 s
WAVE HEIGHT	.2 to 1.0 m
DEPTH OF WATER	2 m
WAVE STEEPNESS	0.01 to 0.09
RELATIVE DEPTH	0.06 to 0.21

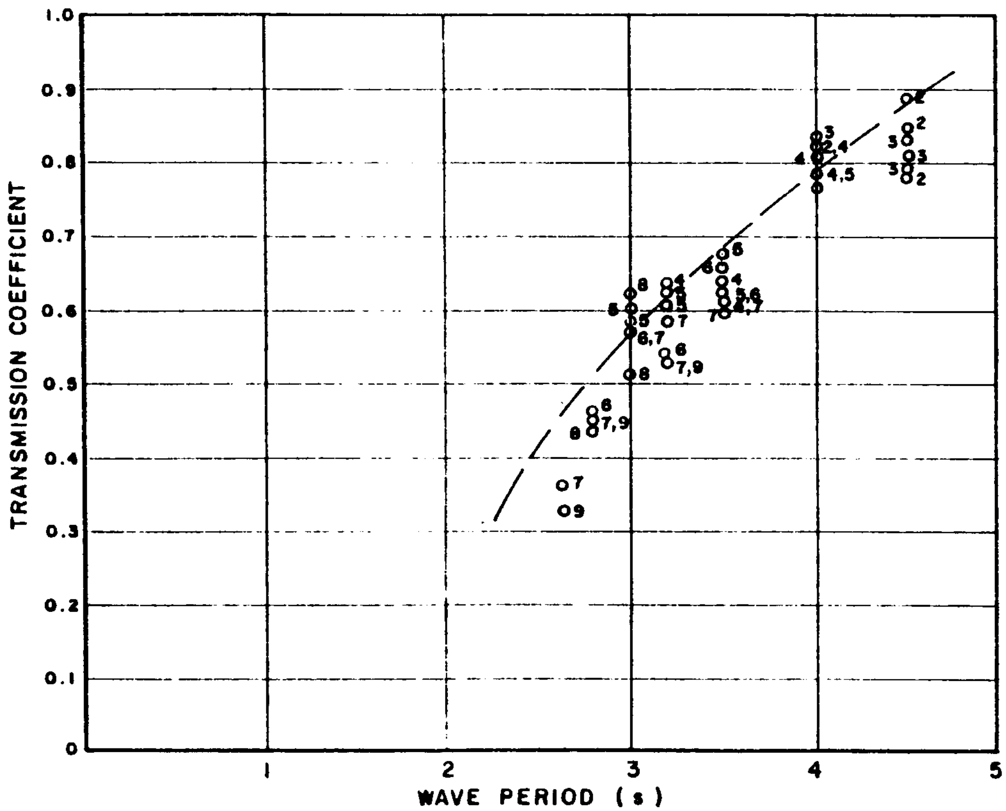
MOORING DETAILS

Moored with 6 m of open link chain and unspecified length of 4.8 mm diameter stainless steel cable on seaward side. Landward cable 4.8 mm diameter cable only. Initial tension not available.

NOTE

Limitations of wave generator resulted in a wide range of steepness over the range of periods tested.

Numbers on graph show — percent wave steepness



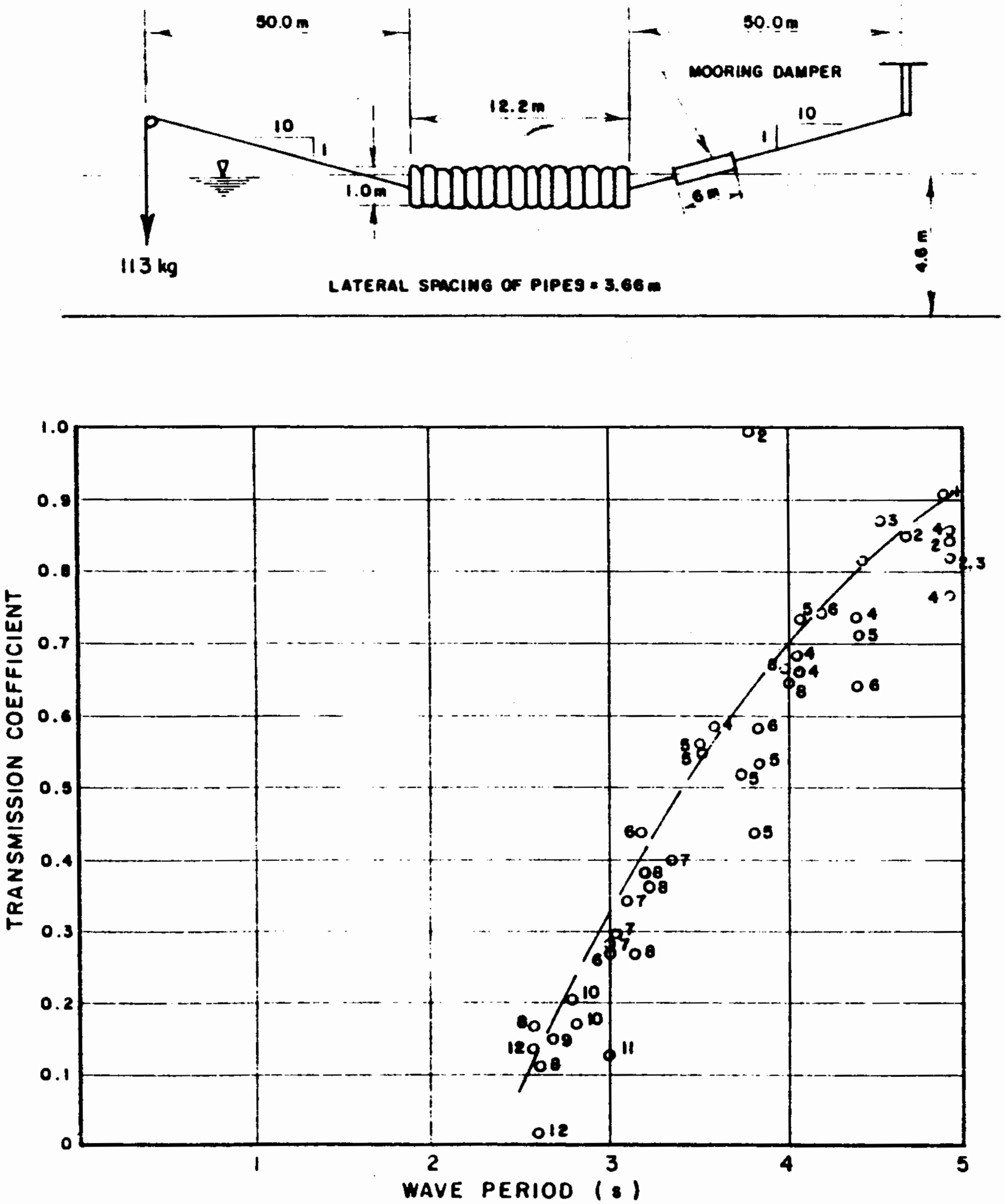
FLOATING BREAKWATER DESIGN

GILES & SORENSON, 1978, FLOATING TIRE

FIGURE C31

WESTERN CANADA HYDRAULIC LABORATORIES LTD.

bcl 6638 A-WCH



SCALE 1:1
REGULAR WAVES

TEST CONDITIONS

WAVE PERIOD	2.6 to 8.0 s
WAVE HEIGHT	0.42 to 1.78 m
DEPTH OF WATER	4.6 m
WAVE STEEPNESS	.01 to .12
RELATIVE DEPTH	.10 to .45

MOORING DETAILS

Moored with 6 mm wire rope both sides. Initial pretension = 113 kg in landward mooring line.

Mooring damper consists of 6 automobile tires connected together with conveyor belting. Damper is connected to pipe (Type 1).

NOTE

Limitations of wave generator resulted in a wide range of steepness over the range of periods tested.

Numbers on graph show — percent wave steepness

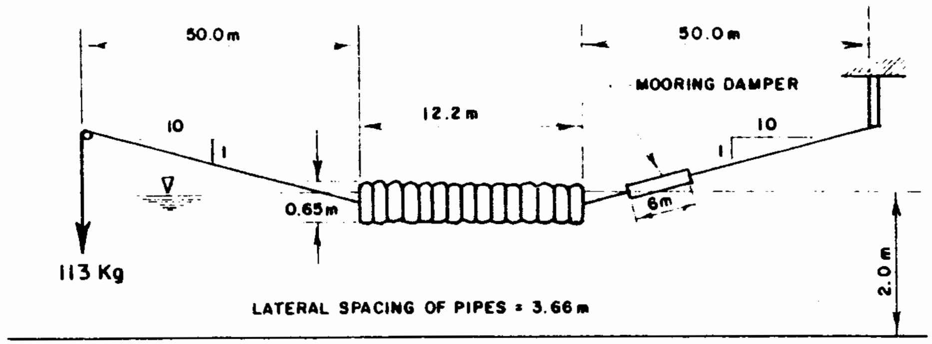
FLOATING BREAKWATER DESIGN

HARMS ET AL, 1980, PONTOON

WESTERN CANADA HYDRAULIC LABORATORIES LTD.

FIGURE C32

bcl 6638 A-WCH



SCALE 1:1
REGULAR WAVES

TEST CONDITIONS

WAVE PERIOD	2.6 to 8.1 s
WAVE HEIGHT	.15 to 1.13 m
DEPTH OF WATER	2.0 m
WAVE STEEPNESS	.004 to 0.1
RELATIVE DEPTH	.06 to .22

MOORING DETAILS

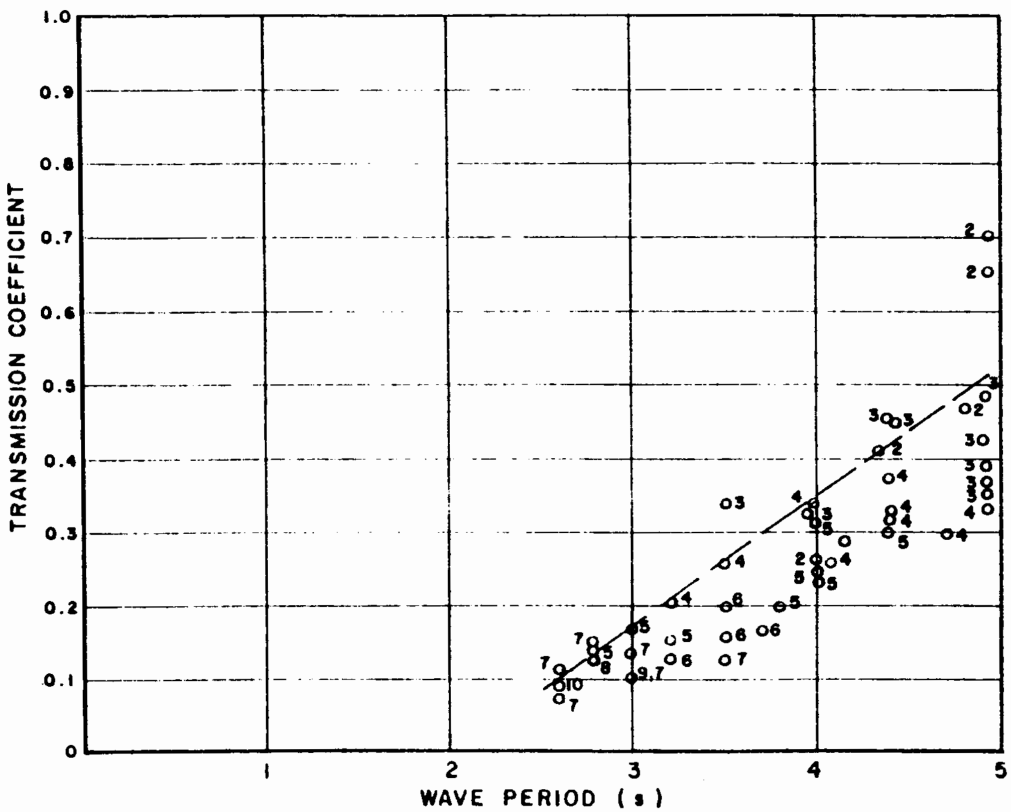
Moored with 6 mm wire rope both sides. Initial pretension = 113 kg in landward mooring line.

Mooring damper consists of 6 automobile tires connected together with conveyor belting. Damper is connected to pipe (Type 1).

NOTE

Limitations of wave generator resulted in a wide range of steepness over the range of periods tested.

Numbers on graph show — percent wave steepness



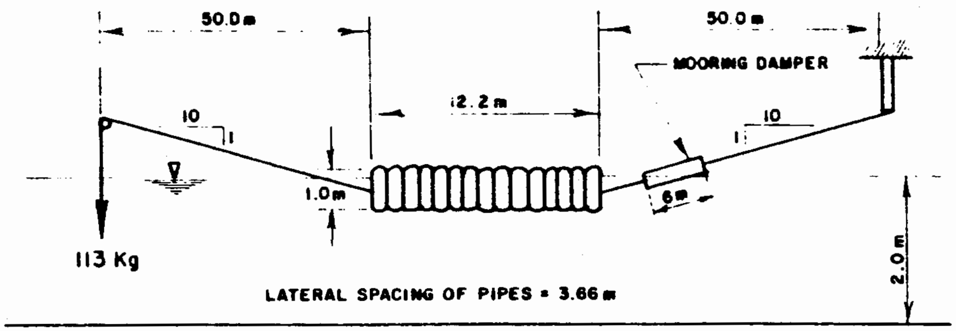
FLOATING BREAKWATER DESIGN

HARMS ET AL, 1980, POLE TIRE

FIGURE C33

WESTERN CANADA HYDRAULIC LABORATORIES LTD.

bc-16638 A-WCH



SCALE 1:1
REGULAR WAVES

TEST CONDITIONS

WAVE PERIOD	2.6 to 8.1 s
WAVE HEIGHT	.26 to 1.1 m
DEPTH OF WATER	2 m
WAVE STEEPNESS	.01 to .09
RELATIVE DEPTH	.06 to .22

MOORING DETAILS

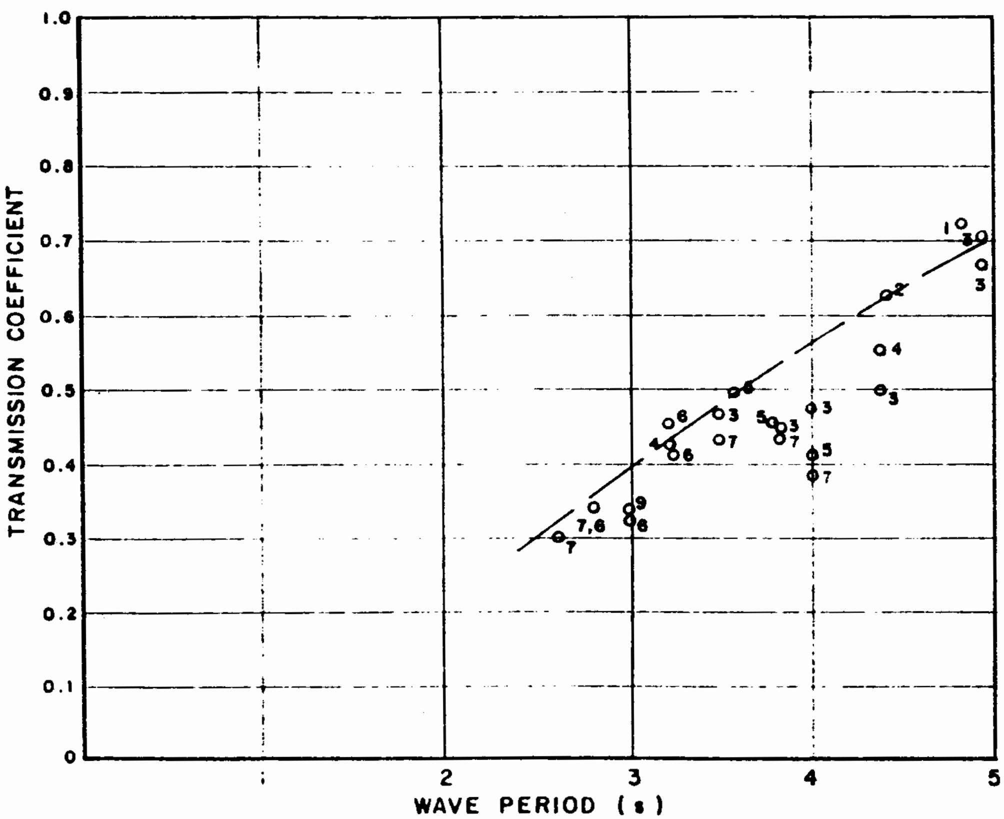
Moored with 6 mm wire rope both sides. Initial pretension = 113 kg in landward mooring line.

Mooring damper consists of a 6 m long loop of conveyor belting connected to tires strung on pipe (Type 3).

NOTE

Limitations of wave generator resulted in a wide range of steepness over the range of period tested.

Numbers on graph show — percent wave steepness



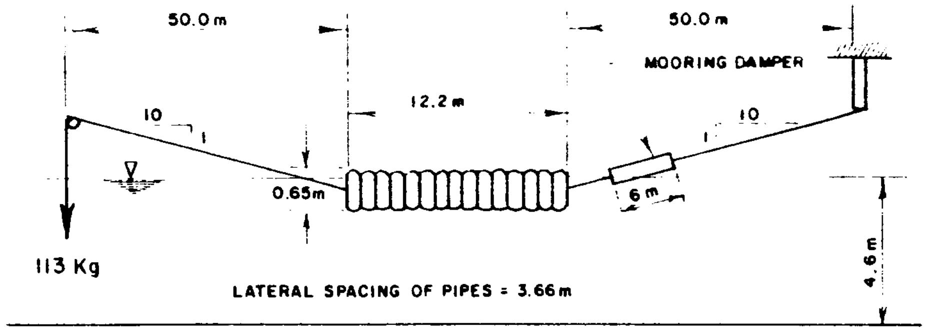
FLOATING BREAKWATER DESIGN

HARMS ET AL, 1980, POLE TIRE

FIGURE C 34

WESTERN CANADA HYDRAULIC LABORATORIES LTD.

bcil 6638 A-WCH



SCALE 1:1
REGULAR WAVES

TEST CONDITIONS

WAVE PERIOD	2.6 to 8.1 s
WAVE HEIGHT	0.3 to 1.3 m
DEPTH OF WATER	4.6 m
WAVE STEEPNESS	.01 to .10
RELATIVE DEPTH	.09 to .43

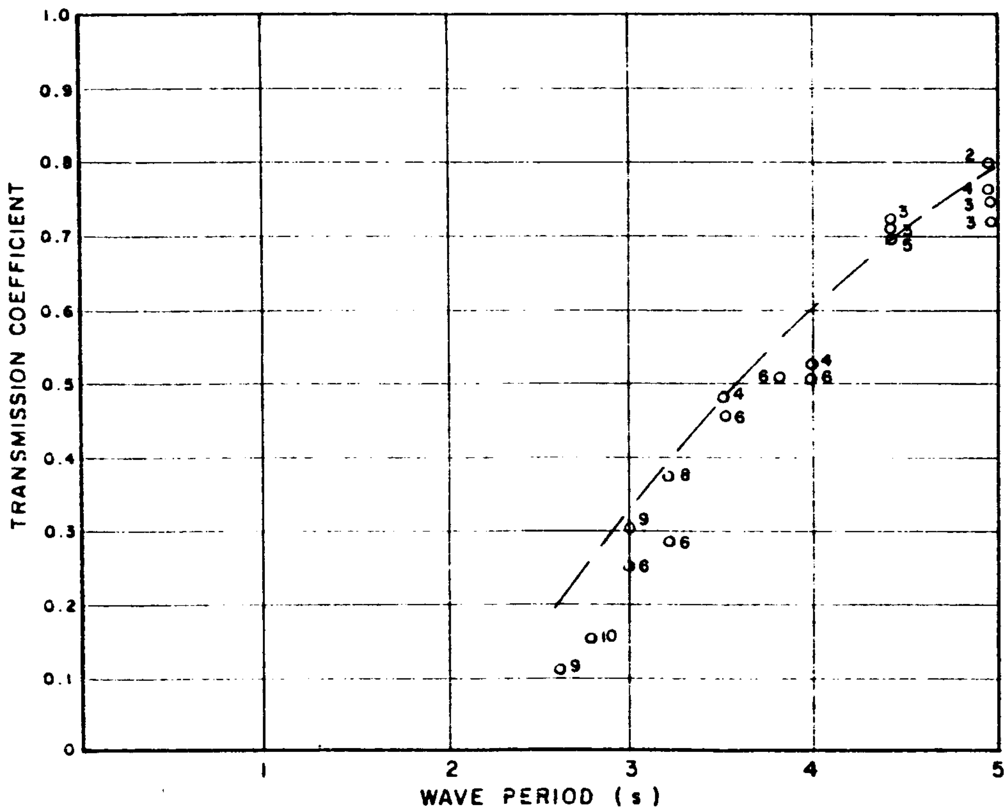
MOORING DETAILS

Moored with 6 mm wire rope both sides. Initial tension = 113 kg in landward mooring line.

Mooring damper consists of a 6 m long loop of conveyor belting connected to tires strung on pipe (Type 3).

NOTE

Limitations of wave generator resulted in a wide range of wave steepness over the range of periods tested.



Numbers on graph show — percent wave steepness

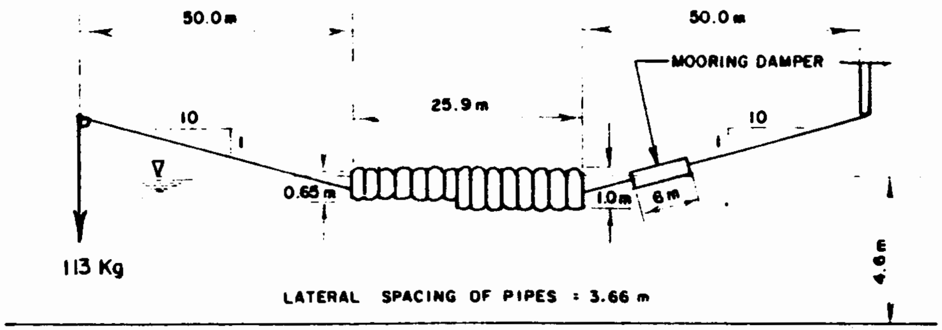
FLOATING BREAKWATER DESIGN

HARMS ET AL, 1980, PONTOON

FIGURE C 35

WESTERN CANADA HYDRAULIC LABORATORIES LTD.

bcil 6638 A-WCH



SCALE 1:1
REGULAR WAVES

TEST CONDITIONS

WAVE PERIOD	2.6 to 8.1 s
WAVE HEIGHT	.28 to 1.3 m
DEPTH OF WATER	4.6 m
WAVE STEEPNESS	.01 to .11
RELATIVE DEPTH	.09 to .43

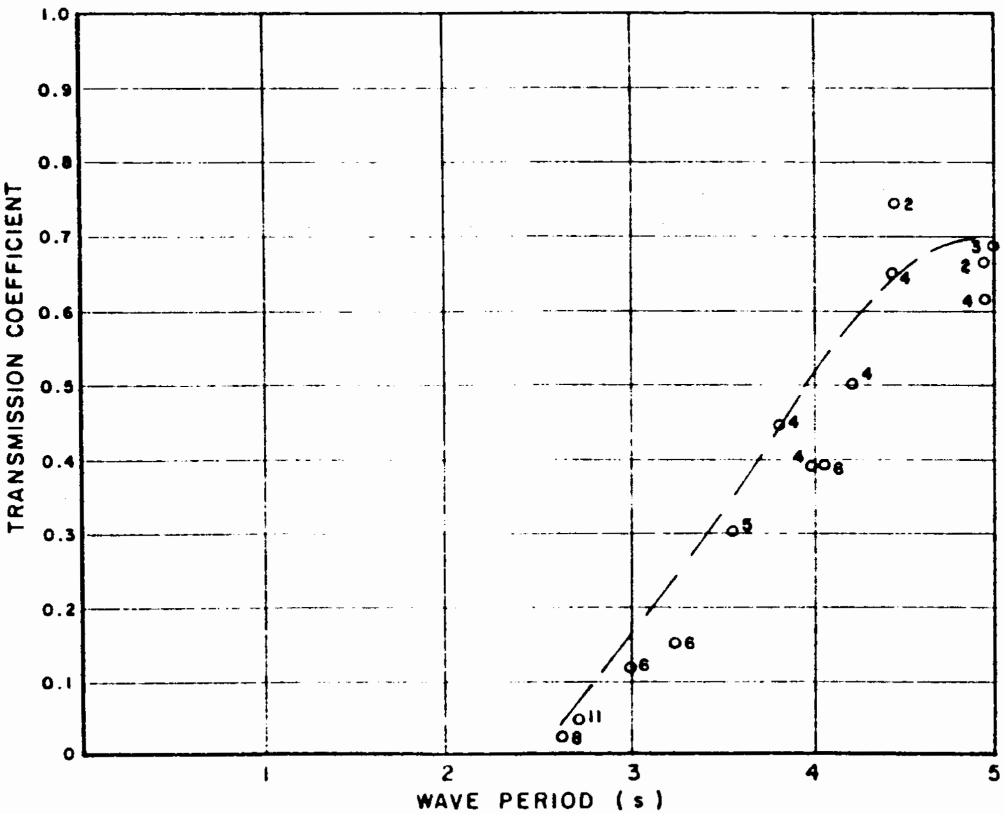
MOORING DETAILS

Moored with 6 mm wire rope both sides. Initial tension = 113 kg in landward mooring line.

Mooring damper consists of a 6 m long loop of conveyor belting connected to tires strung on pipe (Type 2).

NOTE

Limitations of wave generator resulted in a wide range of wave steepness over the range of periods tested.



Numbers on graph show — percent wave steepness

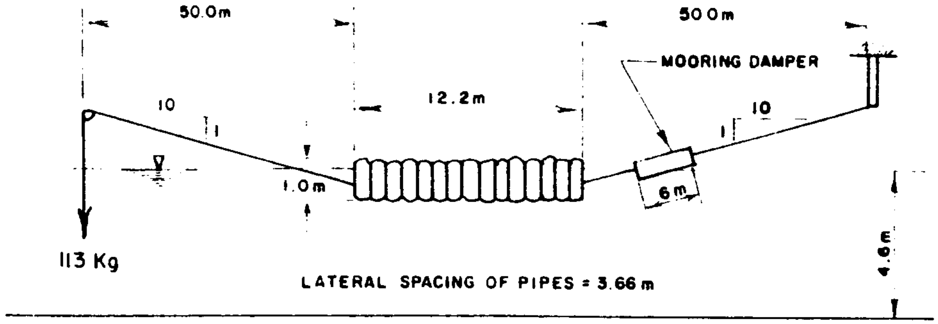
FLOATING BREAKWATER DESIGN

HARMS ET AL, 1980, PONTOON

FIGURE C36

WESTERN CANADA HYDRAULIC LABORATORIES LTD.

bc-l 6638 A-WCH



SCALE 1:1
REGULAR WAVES

TEST CONDITIONS

WAVE PERIOD	2.6 to 8.1 s
WAVE HEIGHT	0.3 to 1.5 m
DEPTH OF WATER	4.6 m
WAVE STEEPNESS	0.01 to 0.11
RELATIVE DEPTH	0.09 to 0.43

MOORING DETAIL

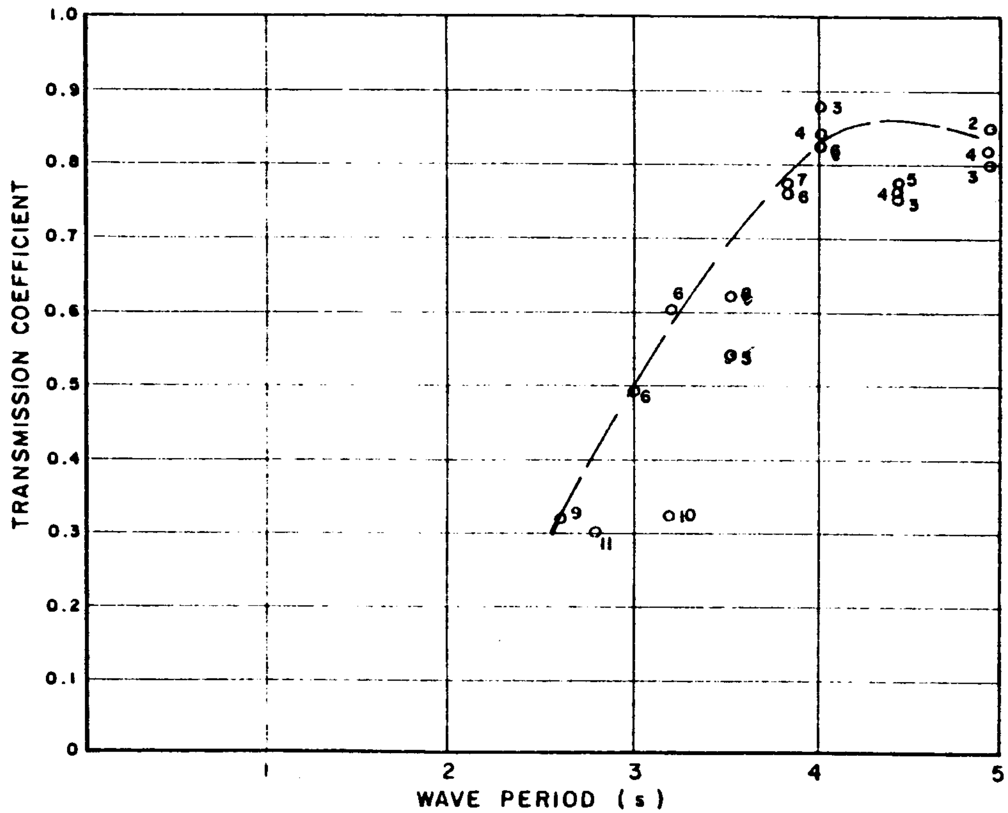
Moored with 6 mm wire rope both sides. Initial tension = 113 kg in landward mooring line.

Mooring damper consists of a 6 m long loop of conveyor belting connected to tires strung on pipe (Type 3).

NOTE

Limitations of wave generator resulted in a wide range of wave steepness over the range of periods tested.

Numbers on graoh show —— percent wave steepness



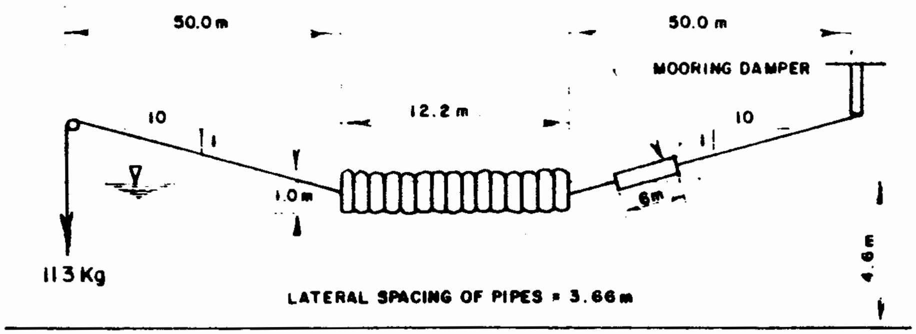
FLOATING BREAKWATER DESIGN

HARMS ET AL,1980, PONTOON

WESTERN CANADA HYDRAULIC LABORATORIES LTD.

FIGURE C37

bc-l 6638 A-WCH



SCALE 1:1
REGULAR WAVES

TEST CONDITIONS

WAVE PERIOD	2.6 to 8.1 s
WAVE HEIGHT	.32 to 1.3 m
DEPTH OF WATER	4.6 m
WAVE STEEPNESS	0.01 to 0.11
RELATIVE DEPTH	0.09 to 0.43

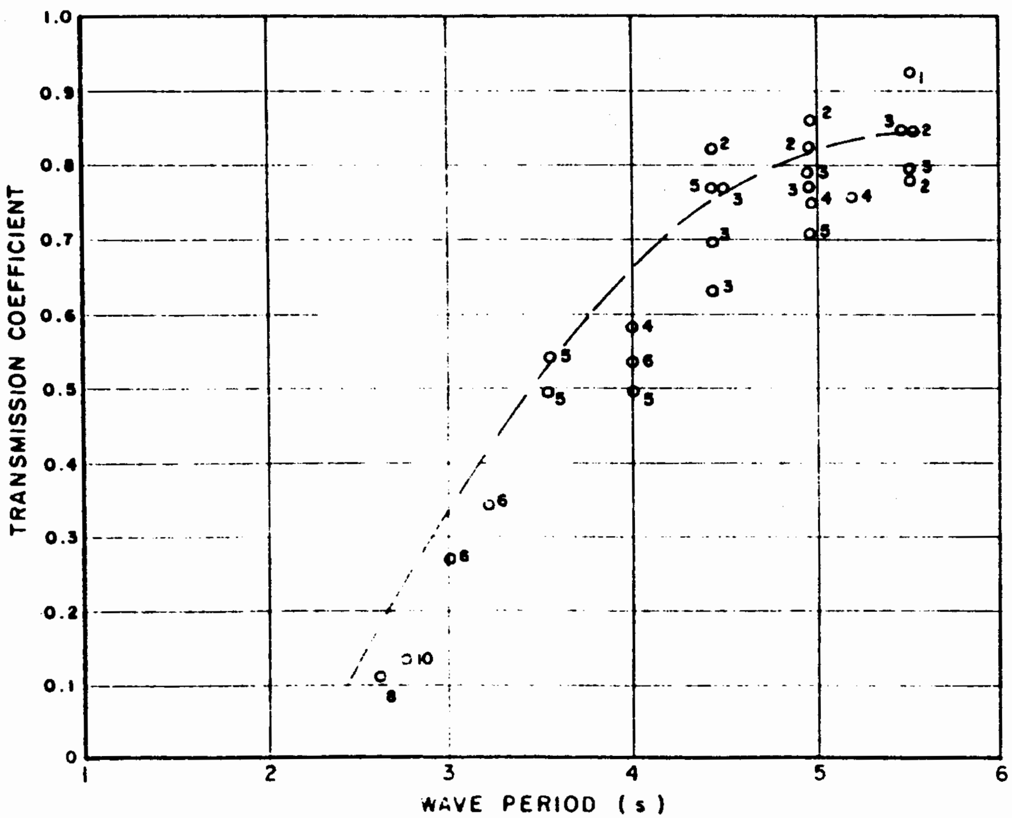
MOORING DETAILS

Moored with 6 mm wire rope both sides. Initial pretension of 113 kg in landward mooring line.

Mooring damper consists of a 6 m long loop of conveyor belt edging. Damper connected to pipes (Type 2).

NOTE

Limitations of wave generator resulted in a wide range of steepness over the range of periods tested.



Numbers on graph show —— percent wave steepness

FLOATING BREAKWATER DESIGN

HARMS ETAL, 1980, PONTOON

FIGURE C38

WESTERN CANADA HYDRAULIC LABORATORIES LTD.

bcl 6638 A-WCH