FLOATING BREAKWATERS PREDICTING THEIR PERFORMANCE

BRADLEY J. MOREY

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BY

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ABSTRACT

Current environmental and financial restrictions on harbour developments dictate that alternatives to traditional fixed rubble mound and caisson breakwaters are required. The most common solution is the *Floating Breakwater*, a concept which utilizes reflection, dissipation, and/or transformation to reduce incident wave energy. The design and construction of these for exposed coastal regions present major engineering challenges.

The primary objective of this thesis was to develop a comprehensive design rationale to enable the designer predict local wave climate (exceedance probabilities, design spectra), optimize a breakwater design (structural parameters, mooring systems) and estimate costs. To facilitate this a number of aspects were reviewed including methods utilized in predicting the wind-wave climate in fetch limited regions, design criteria for inner harbour wave climates, and performance prediction techniques. Based on this review the author developed a simplified deterministic approach to performance prediction based on dimensional and regression analysis of model test data. This information was combined into a computer simulation to predict the local wave climate, optimize the floating breakwater size, design the mooring system, and determine the cost effective solution.

ACKNOWLEDGEMENTS

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No amount of gratitude can repay my wife, who has made many sacrifices so that I may continue and complete my studies. This thesis is dedicated and truly belongs to her.

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LIST OF SYMBOLS

α = regression constant

β_i = regression coefficient

ΔX = structure displacement (m)

ε = error term

ε_i = residual

ε_S = smearing estimate

ρ_W = density of water (1025 kg/m^3)

B_S = structure width (m)

C_C = mooring cable cost per length (\$/m)

C_T = transmission coefficient

D_S = structure draft (m)

D_W = average water depth over fetch (m)

F_1 = leeward anchorline's pretension (N)

F_2 = seaward anchorline's pre-tension (N)

F_3 = leeward anchorline's force (N)

F_4 = seaward anchorline's force (N)

F_A = applied external force (N)

g = gravitational constant (9.81 m/s^2)

H_C = hollow caisson costs (\$/m)

H_I = incoming wave height (m)

H_S = significant wave height (m)

H_T = transmitted wave height (m)

K_W = wave number

L_C = mooring cable length (m)

L_W = wave length (m)

M_S = structure mass per length (kg/m)

n = number of data points

N_M = number of mooring lines

P_U = probability of wind speed U (0-1)

R_L = location amplification ratio

R_T = stability amplification ratio

T_{lim} = limiting duration (s)

T_P = peak wave period (s)

T_S = structure sidesway period (s)

T_W = wave period (s)

U = predicted storm wind speed (m/s)

U_{10} = wind speed at 10 m elevation (m/s)

U_A = adjusted wind speed (m/s)

U_M = mean wind speed (m/s)

U_T = wind speed adjusted for stability (m/s)

U_W = wind speed overwater (m/s)

U_Z = predicted storm wind speed (m/s)

Y = dependent variable

Y_1 = vertical distance (m)

X = fetch (km)

X_1 = original horizontal distance (m)

X_2 = new horizontal distance (m)

X_E = effective fetch (km)

X_i = independent variable

W_C = submerged anchorline unit weight (N/m)

Z = anemometer height (m)

Chapter 1

INTRODUCTION

1.1 Overview

Current environmental and financial restrictions on harbour developments dictate that alternatives to traditional fixed rubble-mound and caisson breakwaters are essential to the future of coastal engineering. The *Floating Breakwater* is one such alternative, a concept which utilizes reflection, dissipation and/or transformation to reduce wave energy and therefore attenuating incident waves to an acceptable level.

Floating breakwaters can act as the primary source of wave protection or supplemental protection where partial shelter is afforded by other barriers such as reefs, shoals and traditional fixed structures. Installation sites include small craft harbours, marinas, yacht clubs, aquacultural facilities, industrial waterfronts, and recreational areas. During ecological emergencies, marine construction, military applications, and special social/recreational events a temporary structure could prove very beneficial.

With respect to fixed rubble mound structures, floating breakwaters possess a number of distinct advantages. These include lower capital cost, shorter construction time, suitability for deep water sites, minimal impact on water circulation and marine habitat, accommodation for a variety of bottom conditions, and effective performance where large tidal variation exists. Some disadvantages include the limitation to short fetches, shorter service life (10 - 20 years) and a portion of the incident wave is transmitted.

The engineering involved in the design of floating breakwaters for exposed coastal regions present major challenges. In particular, the forecasting of wave heights and periods in a fetch limited environment and predicting the performance of a given floating breakwater system are especially rigorous. With recent advances in the use of probabilistic computer models, this analysis can be completed in a more cost and time effective manner.

1.2 Scope

The objectives of this thesis allow for an overall assessment of floating breakwater technology in an effort to develop a probabilistic computer model suitable for preliminary design purposes. These objectives include the following:

- A review of the state of the art in floating breakwater technology. This includes a discussion of the classification of floating breakwaters, a summary of mooring systems, and an analysis of existing installations.

- Review of methods utilised in predicting the wind-wave climate in fetch limited regions in an effort to develop a deterministic technique to produce the data required to assess floating breakwater performance.

- Review of current floating breakwater performance prediction techniques and develop a simplified deterministic model capable of estimating the structural and mooring parameters.

- Develop a probabilistic software model to estimate the local wind-wave climate, evaluate floating breakwater performance, optimize the structural parameters, and estimate the costs.

A demonstration of the programs capabilities is included by way of a case study. A current installation site at Dildo, Newfoundland was evaluated to determine the local wind-wave climate, optimize the floating breakwater system, and determine the most cost-effective solution.

Chapter 2

INTRODUCTION

2.1 Overview

The first documented example of a floating breakwater was recorded in 1811. Proposed by General Bentham, the Civil Architect of Her Majesty's Royal Navy in Great Britain, the structure was to provide shelter for the British fleet at Plymouth. The system was to consist of 117 wooden, triangular floating frames, each 18.3 m in length, 9.2 m in width and 9.2 m in height, moored with iron chains (Readshaw, 1981).

In 1841, this issue of floating breakwaters was again raised by Captain Taylor of Her Majesty's Royal Navy. He proposed that treated floating wooden timber sections of 4.9 m draft and 2.1 m freeboard when connected to piles would provide a measurable degree of protection (Readshaw, 1981).

Once again in 1842, reference to "Reid's" floating attenuation system was included in the Civil Engineers and Architects Journal (1842). The breakwater (Figure 2.1) consisted of an arched timber frame, with a sloping timber ramp on which the waves would break. The width of the structure was 6.1 m, timbers 0.6 m square were used for framing, and the total length of a single unit was 18.3 meters. The sloping beach angled downward at a 35° angle with a projected depth of 4.6 m. Iron chain was utilized as mooring lines, although problems with the mooring arrangement were anticipated. There is no record of Reid's breakwater ever being built.

In a 1905 presentation to the Royal Dublin Society entitled "On Floating Breakwaters", Joly proposed two floating breakwater concepts (Figures 2a and 2b) for the east coast of Ireland (Joly, 1905). The designs were to take advantage of the added mass of water enclosed within the walls of the systems, providing a relatively stiff and unresponsive structure to the ocean waves. Although this system was never constructed, it roused interest in the concepts of floating breakwaters.

Further experience was reported in 1941 at Lysekil, Sweden, where a 120 m long floating concrete breakwater was built for a small crafts harbour (WCHL, 1981). The catamaran system was constructed from rectangular concrete sections 4.5 m square. Indications are that during its service the system performed satisfactorily.

Only minimal efforts were expended on floating breakwaters until the Normandy Invasion of World War II in 1948, when a floating system was required to protect the off-loading area for soldiers and supplies. Referred to as the "Bombardon", the system (Figure 2.3) was of a simple crucifix cross-section 61 m in length with overall cross dimensions of 9 m by 9 m (Jellet, 1948). The system was designed to withstand waves with heights of 3 m and periods in the range of 5 to 6 s. Preliminary, full scale, trial sections were tested indicating an efficiency in the range of 50 - 70% (Todd, 1948).

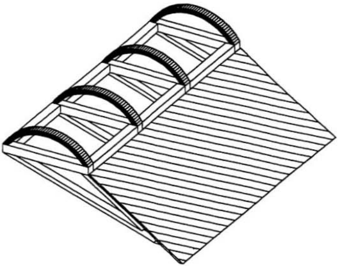


Figure 2.1: Reid's Floating Breakwater

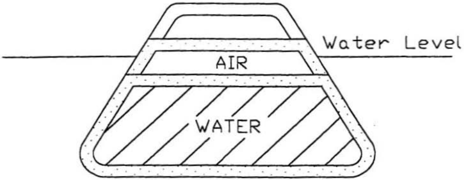


Figure 2.2a: Joly's Floating Breakwater

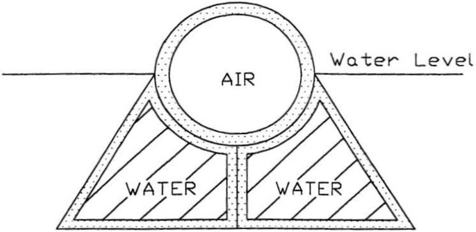


Figure 2.2b: Joly's Floating Breakwater

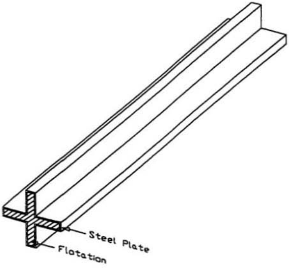


Figure 2.3: Bombardon Floating Breakwater

During the 1970's, there was an explosion of the technology in which a large number of floating breakwaters were constructed to protect marinas and small craft facilities. The number and variety of designs increased dramatically and in 1974, the first conference on floating breakwaters was held at the University of Rhode Island (Kowalski, 1974). In 1981, a second floating breakwater conference was held at the University of Washington (Nece and Richey, 1981).

There have been three significant publications concerning floating breakwaters in the past 15 years. The first was by Lyndell Z. Hales, published in 1981. Sponsored by the U.S. Waterways Experiment Station (WES) in Vicksburg, Mississippi, this study provided detailed information pertaining to a literature review (142 references), a description and classification of floating breakwater configurations, and a detailed description of previous model investigations.

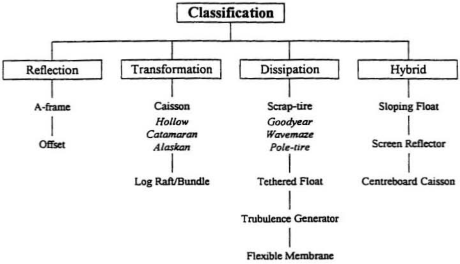
The second was conducted by the Western Canadian Hydraulics Laboratory (WCHL), also published in 1981. Sponsored by the Canadian Department of Fisheries and Oceans, this study also provided a detailed literature review (266 references), description and classifications of breakwaters and detailed descriptions of previous model investigations. One important aspect covered in this report was a summary of current field installations around the world.

The most recent and extensive study was conducted by the Ocean Engineering Research Centre (OERC) at Memorial University of Newfoundland. A joint project sponsored by the Department of Fisheries and Oceans, Public Works and Government Services Canada, and Fisheries, Food and Agriculture Newfoundland. A database of more than 800 references, an extensive survey of floating breakwater sites worldwide, and a summary of previous experimental results were the most significant results of the study. These reports have summarized current breakwater technology to a considerable extent.

2.2 Classification

Floating breakwater systems reduce incident wave heights through the conversion of wave energy via reflection, transformation and dissipation. These energy reduction methods can act in a singular nature or in a combination of one, two, or all three modes. The author suggests that the most effective approach for classification would involve separating systems by these three methods of wave attenuation, as shown in Table 2.1.

Table 2.1: Floating Breakwater Classification



2.2.1 Reflection

Reflective breakwaters utilize large vertical or inclined surfaces to reflect incoming wave energy back out to sea. Efficiency is most sensitive to incident wave height and period, depth and angle of the reflecting surface and the overall structure stability. Adequate stability is provided by a very stiff mooring system. In some cases, wave energy may be transformed into secondary wave trains (Eastern Designers and Company Limited (EDCL), 1991). Typical design concepts include the *A-Frame* and *Offset*.

The A-frame (Figure 2.4) was developed and has been used extensively in British Columbia. A centerboard (timber) has been combined with stabilizers (steel, plastic, or wood) and framing members (steel) to develop a large moment of inertia as an alternative to increasing mass (Richey and Nece, 1974). Benefits of this system include its light weight and simplistic design. The drawbacks are corrosion of steel frames, damage to ends through collisions with other modules causing loss of buoyancy, and a high cost.

2.2.2 Transformation

Transformation breakwaters convert incident wave energy through induced motion response into secondary wave trains of various heights and periods. Highest efficiencies occur when these secondary transmitted wave trains are out of phase with the incident waves. Attenuation is influenced by mass, natural periods of motion, and structure width to wave length (WCHL, 1981). The degree of restraint afforded by the anchoring system is not as crucial to performance as the reflective breakwaters (EDCL, 1991). Concepts grouped under this attenuation method include *Caissons* and *Log Rafts/Bundles*.

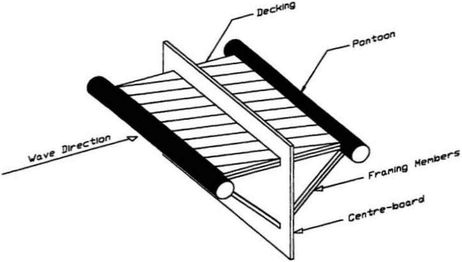


Figure 2.4: A-Frame Floating Breakwater

A typical caisson design is the Alaskan (Figure 2.5), a double pontoon system constructed from concrete and polystyrene foam. The two large pontoons are held in position using a series of braces to provide additional stiffness and floatation. This design is currently used in several harbours along the Alaskan coast.

2.2.3 Dissipation

Dissipative breakwaters convert wave energy into heat, sound, turbulence or friction by breaking waves on sloping surfaces or against structural members. Efficiency is governed primarily through geometry and mooring restraints (WCHL, 1981). These have limited use in attenuating waves of any significant height but have been used extensively in

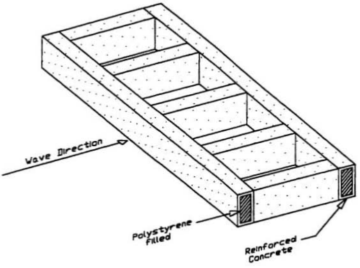


Figure 2.5: Alaskan Floating Breakwater

attenuating wind generated chop (EDCL, 1991). Systems included in this classification are *Scrap Tires, Tethered Float, Flexible Membranes*, and *Turbulence Generators*.

The most common of these designs is the Goodyear (Figure 2.6), which uses a modular building-block design of 18 tires bound with flexible belting with overall length, width and height dimensions of 2.0 by 2.2 by 0.8 m respectively. Each unit is laid out in a 3-2-3-2-3-2-3 combination and held together with unwelded open-link galvanized chain. Units are connected to form a breakwater that is 3, 4 or 5 units in width. Additional floatation is provided by cast-in-place foam positioned in the crowns of tires. Although these breakwaters are cost effective, they need to be 1 to 1.5 times the design wave length and consequently utilize considerable space.

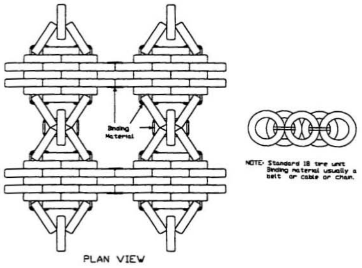


Figure 2.6: Goodyear Scrap Tire Floating Breakwater

2.2.4 Hybrid

The three wave attenuation mechanisms (reflection, transformation, and dissipation) are to some degree incorporated into each floating breakwater. Some systems rely heavily on a combination of these. These breakwaters are hybrid, applying wave attenuation mechanisms simultaneously and/or successively to be effective. The most common systems include *Sloping Float*, *Screen Reflector*, and *Centerboard Caisson*.

One of these systems, the screen reflector (Figure 2.7), first dissipates incoming wave energy on a inclined porous surface. Energy which passes by and through this surface are reflected by a wall. Although the structure does provide better protection, the costs are usually higher

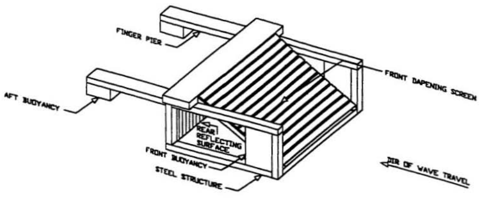


Figure 2.7: Screen Reflector Breakwater

2.3 Nomenclature

As this design classification indicates, there are a significant number of floating breakwater systems. The nomenclature associated with these systems can often imply a different meaning for each type, and a generic set of descriptive terms which relate to *Structural*, *Mooring*, and *Wave* parameters were required. The author has adopted and modified nomenclature from various sources, primarily Kowalski (1974) and Gaythwaite (1988). These parameters, which relate to *Structural, Mooring* and *Wave* characteristics are depicted in Figure 2.8, and further defined in Table 2.2.

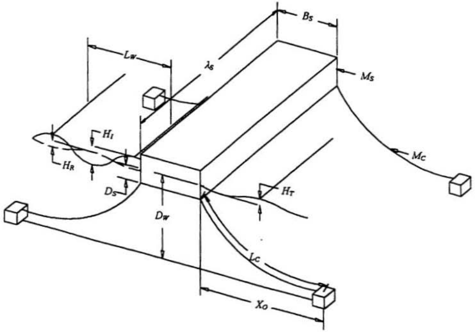


Figure 2.8: Floating Breakwater Nomenclature

Table 2.2: Floating Breakwater Nomenclature

Structural	Mooring	Wave
B_S - *width*	L_C - *chain length*	H_I - *incoming height*
λ_S - *length*	M_C - *chain mass/length*	H_R - *reflected height*
D_S - *draft*	D_W - *water depth*	H_T - *transmitted height*
M_S - *mass/length*	X_O - *horizontal distance*	L_W - *wave length*
T_S - *sidesway period*		T_W - *wave period*

2.4 Mooring Systems

A mooring system for a typical floating coastal structure generally falls under two categories; fixed and cable. In a fixed system, piles develop holding capacity by mobilizing the lateral soil pressure and skin friction in the surrounding seafloor material. Their advantage is an ability to resist both uplift and lateral loads. Unfortunately, they require specialized installation equipment and their underwater installation, particularly in deep flowing water, is very expensive. In addition, their design requires detailed geotechnical data to full pile depth, which is difficult and expensive to obtain (Tsinker, 1986).

In a cable system (Figure 2.9), the structure is connected to a series of anchors to hold the structure in position while allowing some vertical and horizontal movement. For the above noted reasons, the cable system is predominant in floating coastal structures. A review of the cable system components is included in the following sections.

2.4.1 Anchors

Anchors can be broadly classified according to their primary mode of developing lateral resistance. There are three basic categories, which include drag embedment, dead-weight, and direct embedment.

Drag embedment anchors consists of a number of components which include the shank, shackle, fluke, crown, and stock. The most commonly utilized types include the Navy Stockless, AC-14, Danforth, and LWT. Each of these anchors have associated with them a "holding efficiency", defined as the ratio of holding power to anchor weight. This holding power is generated by activating shear stresses within the soil in which the anchors

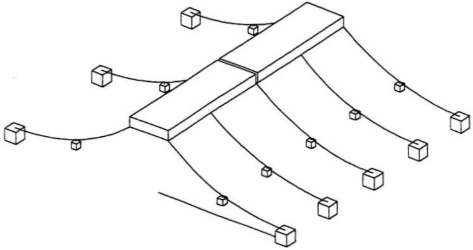


Figure 2.9: Cable Mooring System

are embedded (Gaythwaite, 1990). This is governed by the mass of soil displaced (a function of fluke area and penetration depth) and soil properties. Detailed geotechnical information is necessary to accurately determine the type and mass of anchor. In situations where the anchor placement is crucial great care must be taken to ensure there are no obstructions on or beneath the sea floor.

The dead-weight or gravity anchor depends on submerged weight and friction to provide vertical and horizontal resistance. When the block penetrates the sea floor there is added resistance from suction in cohesive soils and active pressure in granular soils. Additional lateral resistance can be obtained utilizing short needle piles or projections cast into the blocks to activate soil resistance. The anchor shape also helps to penetrate the bottom and gain passive resistance within the soil (Gaythwaite, 1990). Minor information pertaining to the type of soil is required to adequately estimate soil friction.

Direct embedment anchors are forcibly driven into the bottom by pile hammers or propellants. Stake piles are driven beneath the floor and can be accurately located while other anchors are propelled to a certain depth and are set by pulling on the anchor which opens the fluke developing resistance (Gaythwaite, 1990). Again, some detailed geotechnical information is necessary to accurately determine the size of anchor and charge. Any obstructions on or just beneath the seafloor could cause severe problems.

2.4.2 Cables

Cables are usually classified by material which include *synthetics*, *wire rope* and *chain*. In the case of deep water, it is a common practice to utilize chains near the attachments, and synthetic lines in between. In the case of shallow water, chain has been the material of choice.

The main types of synthetic ropes include nylon, dacron, polypropylene, polyethylene, and Kevlar. Nylon is notable for its elasticity and energy absorption properties while Dacron is attractive for its modest elasticity and range of available sizes. Specially constructed plaited, braided, and pre-stretched dacron covered with protective polyvinylchloride or polyethylene jackets are also available. Two of the more common types include "Nolaro" and "Mylar". Synthetic lines have demonstrated considerable potential for short-term operations, due to their ease of handling. These types of materials are not commonly used for long term floating structure installations.

Standard wire rope is constructed from very strong, tough, durable steel combining great strength with high fatigue resistance. They consist of wires wound into strands and strands wound into ropes. The most common type is a regular left lay, where the winding of the wires into strands and the strands into rope oppose each other, thus preventing the rope from unlaying under tension. In marine applications, a electrolytic zinc-plating (two to three times the normal coating) is necessary to protect the wire from rapid weight loss and diameter reduction. Due to corrosion, these are not normally used in a salt water environment, especially where high currents are present (Skop, 1988).

Chain is available in many grades, types, and materials. Cast-steel stud link chain is recommended for shallow water anchoring as it prevents tangling and twisting. Chain has several advantages over the other cable types. These include the ability to be connected to at any point along its length, high abrasion resistance, longevity, and its ability to provide energy absorption from the weight and catenary shape. Galvanizing is usually recommended to increase corrosion resistance. Another approach involves using oversized chain for corrosion and utilize the additional weight to deepen the catenary improving energy absorption characteristics (Miller, 1974).

2.4.3 Hardware

Various types of shackles, swivels, centre rings, and terminations are required to join the major components of a cable mooring system together. This miscellaneous hardware experiences the greatest fatigue and as a result forms the weakest link in the cable system. In many design situations the strength of the mooring system is limited by the capacity of these connections and splices (Miller, 1974). There are other accessories including springs, clump weights and drag plates (Werner, 1988), which act to increase tension in the mooring lines, thereby reducing breakwater motions and improving wave attenuation. Clump weights also increase the catenary to provide deeper clearance for passing vessels.

2.4.4 Patterns

The typical cable mooring arrangement is the spread pattern (Figure 2.10). In most systems, it is commonplace to provide more lines and heavier anchors on the seaward side, usually double the leeward side. Another unique feature involves the placement of lines at the ends of the arrangement at an angle between 45 to 60 degrees (Western Canadian Hydraulics Limited, 1981). These provide additional restrictions to movement in the lateral directions.

In the majority of situations, and where possible, a scope of 2.5 or 3.0 to 1.0 should be utilized. This serves several functions which include the minimizing of line length and costs, transfers the majority of the force to the anchor in a horizontal direction, and increases the overall performance of the anchors as well as the structure.

With respect to the arrangement of the lines, there a number of options. The most utilized are the crossed non-connected, crossed connected, and uncrossed. The lines are usually connected with a centre ring. An evaluation of the site and type of breakwater will usually determine an ideal configuration. For example crossed lines provide deeper clearance for passing ships and require less space on the sea floor.

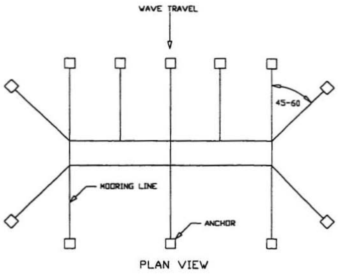


Figure 2.10: Spread Mooring Pattern

2.5 Prototype Installations

There have been over 150 floating breakwaters installed world-wide in countries such as Australia, New Zealand, Great Britain, Italy, Japan, Norway, Switzerland, United States, and Canada. A survey conducted by OERC identified a total of 135 installations (Table 2.3). Most of the owners/operators indicated a degree of satisfaction in the range of 50 to 65 percent but indicated a demand for increased efficiency.

The majority of the problems were related to the inadequacy of mooring systems to reduce motion and hold the breakwater in place. The concepts suggested for future investigations were the caisson, a-frame, screen reflector, and centreboard caisson.

Table 2.3: Installation Type Summary

Classification	*Type*		***Number***
Reflection	*A-Frames*		*5*
	Offset		*16*
Transformation	*Caissons*		
		Hollow-Centred	*35*
		Catamaran	*1*
		Alaskan	*4*
	Rafts/Bundles		*7*
		Rafts	*3*
		Bundles	*14*
Dissipation	*Scrap Tires*		
		Goodyear	*32*
		Pole-Tire	*4*
		Wave Maze	*3*
	Tethered Float		*1*
	Turbulence Generator		*2*
	Flexible Membrane		*1*
Hybrid	*Inclined/Sloping Float*		*2*
	Screen Reflector		*1*
	Centreboard Caisson		*0*
Other			*11*
Total			*135*

Chapter 3

WIND WAVE PREDICTION

When predicting locally generated wave heights and periods in fetch limited regions, two important factors must be considered. Firstly, the designer must be able to predict wind velocities (direction and speed) representative of the location. Secondly, the designer must be able to predict the wave regime based on wind speed, duration, fetch, and water depth.

3.1 Wind Climate Analysis

To facilitate any probabilistic analysis, a designer must have statistics for the wind direction and speed. There are several convenient sources of information available, published by environmental agencies such as Environment Canada. This data is presented in terms of percentage occurrence and mean hourly wind speed by direction and month.

The data presented by Environment Canada provides a probability distribution of wind direction by month, which can easily be converted into a cumulative or exceedance distribution. A commonly used cumulative distribution for wind speed prediction is the Rayleigh distribution. Typical cumulative distributions for 5.0, 7.5 and 10.0 m/s mean wind speeds are shown in Figure 3.1, and can be represented mathematically by Equation 3.1 (Resio and Vincent, 1977).

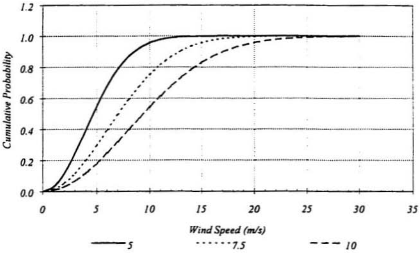


Figure 3.1: Rayleigh Cumulative Probability Distributions for Windspeed

$$P_U = 1 - exp\left[-\frac{\pi}{4}\left(\frac{U}{U_M}\right)^2\right] \tag{3.1}$$

where; U = predicted storm wind speed (m/s)

U_M = mean wind speed (m/s)

P_U = probability of wind speed U (0-1)

The mean wind speed estimated from this equation must be corrected for several factors which describe the atmospheric layer above the waves. These include elevation, stability, location, and drag.

3.1.1 Elevation Correction

When winds are not measured at the 10 m elevation, the wind speed must be adjusted accordingly. If the observed elevation does not exceed 20 m, which is usually the case, a simple approximation (Equation 3.2) can be used (SPM, 1984).

$$U_{10} = \left(\frac{10}{Z}\right)^{\frac{1}{7}} U_z \tag{3.2}$$

where; U_{10} = wind speed at 10 m elevation (m/s)

Z = anemometer height (m)

U_Z = predicted storm wind speed (m/s)

3.1.2 Stability Correction

When the air-sea temperature difference is zero, the boundary layer has neutral stability, and no corrections are needed. When the difference is positive, the layer is stable and the wind speed is less effective in causing wave growth. If the air is at a lower temperature than the layer, then there is increased wave growth. Resio and Vincent (1977) defined an amplification ratio (R_T) to account for the effects of air-sea temperature. An effective wind speed is obtained by Equation 3.3, where R_T is determined from Figure 3.2. In the cases where there is insufficient data, an amplification ratio of 1.1 should be used.

$$U_T = R_T \; U_{10} \tag{3.3}$$

where; U_T = wind speed adjusted for stability (m/s)

R_T = stability amplification ratio

U_{10} = wind speed at 10 m elevation (m/s)

3.1.3 Location Correction

Overwater wind data is often not available, then data from nearby environmental stations can be obtained. These winds can be translated to overwater winds if they are the result of the same pressure system. An effective wind velocity can be calculated by Equation 3.4, where the location ratio (R_L) is obtained from Figure 3.3. When the fetch is less than 15 km, a location ratio of 1.1 is recommended (Resio and Vincent, 1977).

$$U_W = R_L \; U_T \tag{3.4}$$

where; U_W = wind speed overwater (m/s)

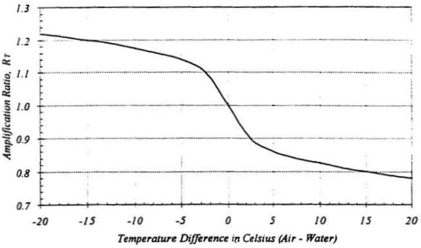


Figure 3.2: Stability Ratio, R_T (Resio and Vincent, 1977)

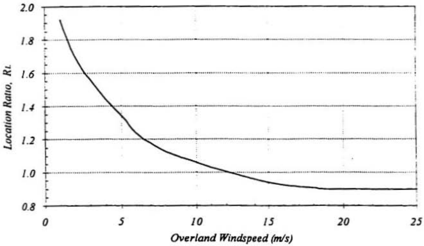


Figure 3.3: Location Ratio, R_L (Resio and Vincent, 1977)

R_L = location amplification ratio

U_T = wind speed adjusted for stability (m/s)

3.1.4 Drag Correction

After the above corrections have been made to the wind data, the wind speed is converted to a wind-stress factor by Equation 3.5 (SPM, 1984). This accounts for the drag over the surface of the water, relating the non-linear relationship between wind stress and wind speed.

$$U_A = 0.71 \left(U_W\right)^{1.23} \tag{3.5}$$

where; U_A = adjusted wind speed (m/s)

U_W = wind speed overwater (m/s)

These approximations and adjustments reduce biases in wind data and provide a means of obtaining information where adequate measurements are not available. However, the collection of over water wind data is preferable, even if collected for a short period, it is of value in relating over land wind data to over water values.

3.2 Wave Climate Prediction

Prediction of wave height and period is normally done through the process of hindcasting, which utilises historical wind data to develop a wave climate based upon the available wind speed, water depth, fetch and duration. The waves at prospective breakwater sites are primarily the result of locally generated winds, as fetches do not exceed 10 km. There are a number of formulae that can be applied for wave prediction, most notably those produced for deep and shallow water waves as described in the Shore Protection Manual (1984).

In recent years, there have been formulae developed which can be utilized to predict wave heights and periods depending on whether the site is duration or fetch limited, and which also provides a smooth transition between deep to shallow water, The most recent series of equations have been developed by Silvester and Hsu (1993). Discussed in detail below, the equations are a major improvement over the techniques used in the Shore Protection Manual.

The Silvester and Hsu prediction process initiates with the estimate of the critical or limiting duration (Equation 3.6) which indicates the time required for fully developed seas to occur for a specific wind speed and fetch. For example, a wind speed of 20 m/s combined with a 3 km fetch and 6 m water depth requires 17,379 s (4.8 hrs) for a full arisen sea.

$$T_{lim} = 3078 \sqrt[3]{U_A \; X^2} \qquad \textbf{(3.6)}$$

where; T_{lim} = limiting duration (s)

U_A = adjusted wind speed (m/s)

X = fetch (km)

3.2.1 Fetch Limited Wave Prediction

When the given duration is longer than the limiting duration the wave growth is limited by the fetch. The significant wave height and peak period can be estimated directly from Equations 3.7 and 3.8. For example, the duration is 18,000 s for a wind speed of 20 m/s, fetch of 3 km, and a water depth of 6 m, which is less than the limiting duration of 17,379 s. Therefore, the designer can apply these equations directly resulting in a significant wave height and period of 0.58 m and 2.44 s respectively.

$$H_S = 0.026 U_A^2 \tanh\left[3.326\left(D_W U_A^{-2}\right)^{3/4}\right] \sqrt{\tanh\left\{\frac{0.422\, X U^{-2}}{\tanh^2\left[3.326\left(D_W U_A^{-2}\right)^{3/4}\right]}\right\}} \tag{3.7}$$

$$T_P = 0.846 U_A \tanh\left[1.789\left(D_W U_A^{-2}\right)^{3/8}\right] \sqrt[3]{\tanh\left\{\frac{0.402\, X U^{-2}}{\tanh^3\left[1.789\left(D_W U_A^{-2}\right)^{3/8}\right]}\right\}} \tag{3.8}$$

where;

H_S = significant wave height (m)

T_P = peak wave period (s)

D_W = average water depth over fetch (m)

U_A = adjusted wind speed (m/s)

X = fetch (km)

3.2.2 Duration Limited Wave Prediction

In this case the given duration is less than the limiting duration. This results in an effective fetch (Equation 3.9) that is less than the actual fetch. This effective fetch is substituted into Equations 3.7 and 3.8 and the wave height and period calculated. For the same example, the duration is reduced to 10,800 s, which is less than the limiting duration of 17,379 s. An effective fetch of 1.47 km is calculated from Equation 3.9, which corresponds to a wave period and height of 0.41 m and 1.93 s respectively.

$$X_E = \left(\frac{T_{lim}}{3078 \sqrt[3]{U_A}} \right)^{3/2} \tag{3.9}$$

where; X_E = effective fetch (km)

T_{lim} = limiting duration (s)

U_A = adjusted wind speed (m/s)

One of the shortcomings in previous wave prediction techniques for floating breakwater installations has been to characterise the locally generated wind-wave climate by a particular design wave, such as that produced by a 50 year storm event. The difficulties in defining acceptable limits have often resulted in unrealistic designs and costs. The author has utilised a more rigorous probabilistic approach to define the appropriate wave regime. In chapter 5, the author describes how these formulae were adopted into a computer model to conduct such a probabilistic design.

Chapter 4

PERFORMANCE ANALYSIS

Once the wave regime has been predicted, the next crucial step involves determining the true performance of the floating breakwater under a specific set of conditions. There have been numerous attempts to develop a prediction technique, several of which have met with moderate success. A second key aspect of performance analysis relates to the acceptable wave climate the designer aims to achieve. These aspects of performance analysis have been investigated to determine a simplified deterministic approach to performance analysis.

4.1 Prediction Techniques

There are a number of techniques that were employed by designers to predict performance of numerous breakwater types. The primary approaches include previous experience, analytical methods, numerical methods, field trials, and laboratory testing. These are discussed in some detail in the subsequent sections.

4.1.1 Previous Experience

There does exist significant information on various types of systems currently in use. Unfortunately this information is often biased due to what many refer to as the "human factor". Owners and operators who have observed their system in the field indicate the performance to be 20-25% more effective than what would be measured (Richey and Nece, 1984). As a result, this type of information can be very misleading and should not be used for performance prediction purposes.

4.1.2 Analytical Methods

Floating breakwaters were first investigated utilizing approximations consisting of idealized forms of wave barriers (rigid structure fixed near the surface). An expression for the transmission coefficient (Equation 4.1) was developed by Macagno (1954). Wiegel (1960) further investigated this model with prime consideration given to wave power transmission (time rate of energy propagation) beneath a thin plate (Equation 4.2).

$$C_T = \left[1 + \left(\frac{k_w \ B_S \ \sinh\ (k_w \ D_w)}{2 \ \cosh\ (k_w \ D_w \ - \ k_w \ D_S)}\right)^2\right]^{-\frac{1}{2}} \tag{4.1}$$

$$C_T = \sqrt{\frac{2 \ k_w \ (D_w \ - \ D_S) \ - \ \sinh\ (2 \ k_w \ D_w \ - \ 2 \ k_w \ D_S)}{2 \ k_w \ D_w \ + \ \sinh\ (2 \ k_w \ D_w)}} \tag{4.2}$$

where; B_S = structure width (m)

D_S = structure draft (m)

D_W = water depth (m)

C_T = transmission coefficient

K_W = wave number

These models do not account for any effect which the motion of the structure can impart on wave transmission. In an attempt to incorporate motion, Carr (1951) assumed the transmission coefficient for a rectangular cross section anchored in shallow water could be predicted from linear wave theory, hydrostatic pressure distributions, linear damping and the sidesway component of motion (Equation 4.3). The motion was characterized by the wave and sidesway period.

$$C_T = \left[1 + \left(\frac{\pi M_S}{\rho_W \; L_W \; D_W} \right)^2 \left(\frac{T_W^2}{T_S^2} \right)^2 \right]^{-\frac{1}{2}} \tag{4.3}$$

where; C_T = transmission coefficient

L_W = wave length (m)

M_S = structure mass per length (kg/m)

T_S = structure sidesway period (s)

T_W = wave period (s)

4.1.3 Numerical Methods

Detailed computer models have recently been employed. These utilize simple rectangular shapes where the wave interaction phenomena can be linearized and simplifying assumptions taken to reduce the complexity of the problem. Numerical prediction normally requires at least two steps of analysis; "hydrodynamic" and "body-response". While the body response analysis can be solved utilizing standard methods of numerical approximation, the hydrodynamic analysis is more complicated. A number of numerical models for the hydrodynamic analysis has been developed in recent decades. These have been reviewed and evaluated by several specialists in the field (Bando and Sonu, 1986) to determine the most computationally economic and effective method. The most highly recommended technique is the Hybrid Green Function Method (HGFM), proposed by Ijima and Yoshida (1975).

4.1.4 Field Trials

Full scale tests are very expensive and time consuming, and current methods of recording and measuring waves have not been completely verified. There have been a number of full scale sections tested, and indications are that field efficiencies are generally higher than efficiencies predicted by laboratory or numerical methods.

It has been suggested that this is due to the variability of the wave surface and angle of attack. When a wave encounters the structure, there are two important factors to consider. Firstly, the effective width of the structure is increased as the angle of attack increases. It has been well established that an increase in width will increase attenuation.

Secondly, the force on the structure must vary when both wave surface and angle of attack are constantly changing. This variability reduces translation of the structure which has also been shown to increase attenuation.

4.1.5 Laboratory Testing

Developing and conducting a series of comprehensive tests to evaluate performance has been used very often. Unfortunately, this tends to be very costly and time consuming. In addition, there are a finite number of test conditions which are available. As a result, the test results are only valid when considering the same structure exposed to similar conditions. Despite these shortcomings, it is this approach which holds the most promise for performance prediction.

4.2 Dimensional Regression Technique

As mentioned previously, inadequate information on performance of prototype installations exists and undertaking physical, numerical and full scale section investigations is not considered feasible in the initial stages of design. Of these prediction methods, the extrapolation of results from previous model studies is the most reasonable approach.

The dimensional regression technique the author employed involved four basic steps. First, various breakwater systems were analyzed to ascertain what parameters were important in the attenuation process. Second, dimensional analysis was conducted to isolate the parameters into a series of dimensionless ratios, relevant in describing the wave attenuation process. Third, available model test data was collected and converted into dimensionless ratios developed in step two. Fourth, a regression analysis was conducted to find the most appropriate deterministic model. These steps are described in detail in the subsequent sections.

4.2.1 Parametric Review

The primary factor in evaluating floating breakwater performance is the Transmission Coefficient (C_T), defined as the ratio of transmitted wave height (H_T) to incoming wave height (H_I). Prior parametric studies have utilized the ratios of structure width to wave length (B_S / L_W), wave height to wave length (H_I / L_W), structure draft to water depth (D_S / D_W), and water depth to wave length (D_W / L_W) are most commonly utilized. In early analytical formulae, the ratio of structure mass to water density, wave length and water depth ($M_S / \rho_W L_W D_W$) was considered important.

These terms relate to two aspects of breakwater performance, the structural and wave parameters. A crucial aspect not covered adequately or even at all in previous studies concerns the geometric stiffness characteristics of the mooring system (K_M), a measure of the force required to displace the breakwater 1 unit from its original or neutral position.

Geometric Stiffness

For an accurate deterministic model to be developed, a method to evaluate the geometric stiffness was required. Typically, mooring systems consist of a series of chains to hold the structure in position. These chains form a natural catenary shape and require a series of detailed calculations to determine the appropriate forces and corresponding displacements. There were two primary sources of information identified ((Berteaux, 1976), (Tsinker, 1986)) from which Equations 4.4 to 4.9 were developed. The terms used in these equations are further clarified by Figure 4.1.

$$X_2 = X_1 - \Delta X \tag{4.4}$$

$$F_1 = F_2 = \frac{W_C \; X_1^2}{2 \; Y_1} \tag{4.5}$$

$$L_C = X_1 \left[\frac{(X_1^2 + Y_1^2)^{0.5}}{X_1} + \frac{W_C^2 \; X_1^5}{24 \; F_2^2 \, (X_1^2 + Y_1^2)^{1.5}} \right] \tag{4.6}$$

$$F_4 = \sqrt{\frac{W_C^2 \; X_2^5}{24 \left(X_2^2 + Y_1^2\right)^{1.5} \left[L_C - \left(X_2^2 + Y_1^2\right)^{0.5}\right]}} \tag{4.7}$$

$$F_A = F_4 - F_2 \tag{4.8}$$

$$K_M = N_M \left(\frac{F_A}{\Delta X}\right) \tag{4.9}$$

where; X_1 = original horizontal distance (m)

X_2 = new horizontal distance (m)

ΔX = structure displacement (m)

F_1 = leeward anchorline's pretension (N)

F_2 = seaward anchorline's pre-tension (N)

W_C = submerged anchorline unit weight (N/m)

Y_1 = vertical distance (m)

L_C = length of mooring line (m)

F_3 = leeward anchorline's force (N)

F_4 = seaward anchorline's force (N)

F_A = applied external force (N)

N_M = number of mooring lines

K_M = mooring stiffness (N/m)

Consider a simplified scenario, a floating breakwater 30 m in length has 6 mooring lines attached to the system. Three of these are connected to the leeward side while the other lines are connected to the seaward side to form three pairs of lines (N_M). The is 20 mm diameter with a unit weight (W_C) 78.5 kN/m. The water depth (Y_1) is 10 m and the lines are placed with a scope of 3 to 1 (X_1 is three (3) times Y_1) to make X_1 30 m. If we arbitrarily select a change in position (ΔX) of 0.2 m we can apply equations 4.4 to 4.9, which indicate it would take 5kN to move the 30m structure 0.2m, for a stiffness of 25kN/m.

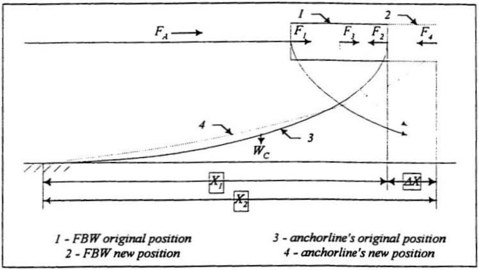


Figure 4.1: Catenary Mooring System

4.2.2 Dimensional Analysis

The resulting basic functional Equation (4.10) contains eight variables. Since this system is a surface dominated phenomenon, the viscous and inertia effects have been considered negligible and excluded from the analysis.

$$H_T = \phi(H_I, L_W, D_W, B_S, D_S, M_S, K_M) \qquad \textbf{(4.10)}$$

where; H_T = transmitted wave height (m)

H_I = incoming wave height (m)

L_W = wave length (m)

D_W = water depth (m)

B_S = structure width (m)

D_S = structure draft (m)

M_S = structure mass per unit length (kg/m)

K_M = mooring stiffness (N/m)

After a thorough review of dimensional analysis techniques, the method of synthesis was adopted. The reasoning for doing so was this technique uses linear proportionality's (length dimensions) to develop a dimensionally homogeneous equation. Based on Equation 4.10, six of the eight terms already have linear dimensions (m), while the remaining two terms of mass per unit length (M_S) and geometric stiffness (K_M) include dimensions of mass (kg) and time (s).

In order to develop linear proportionality's, the synthesis technique allows you to add the terms relating to gravity (g) and water density (ρ_W) where necessary to develop linear terms. The structure unit mass term (units of kg/m) was modified by dividing with the water density (units of kg/m^3) and taking the square root of the term to arrive a linear proportionality (m). The stiffness term (units of kg/s^2) is a little more complex, as it contains both mass (kg) and time (s). The modification involves dividing by both the gravitational constant (units of m/s^2) and water density (units of kg/m^3) then taking the square root to arrive at a linear proportionality. These terms then replace the structure unit mass and stiffness in Equation 4.10 to arrive at the final functional Equation 4.11.

$$0 = \phi\left(H_T,\ H_I,\ L_W,\ D_W,\ B_S,\ D_S,\ \left(\frac{M_S}{\rho_w}\right)^{\frac{1}{2}},\ \left(\frac{K_M}{g\ \rho_w}\right)^{\frac{1}{2}} \right) \qquad \textbf{(4.11)}$$

where;

H_T = transmitted wave height (m)

H_I = incoming wave height (m)

L_W = wave length (m)

D_W = water depth (m)

B_S = structure width (m)

D_S = structure draft (m)

M_S = structure mass per unit length (kg/m)

K_M = mooring stiffness (N/m)

g = gravitational constant (9.81 m/s^2)

ρ_W = density of water (1025 kg/m^3)

The final step in the process, is to create a series of dimensionless parameters, by dividing each term by one of the others. This process allows for the modification of any parametric term by dividing or multiplying as well as squaring or cubing. The final basis for the selection of the terms should be based on the particular system under investigation and should reflect the understanding of the process involved. The most common parameters have been shown in Equation 4.12, and is considered representative for a variety of floating breakwater types. Other, more unique systems may not be completely explained by the results of the analysis. As a result, additional parameters would be required to analyze the system. Provided the above steps are maintained, the parameters derived from a separate analysis would still be valid.

$$C_T = \frac{H_T}{H_I} = \phi\left(\frac{H_I}{L_W}, \frac{D_W}{L_W}, \frac{B_S}{L_W}, \frac{D_S}{D_W}, \frac{M_S}{\rho_W L_W^2}, \frac{K_M}{g\rho_W L_W^2}\right) \tag{4.12}$$

where;

H_T = transmitted wave height (m)

H_I = incoming wave height (m)

L_W = wave length (m)

D_W = water depth (m)

B_S = structure width (m)

D_S = structure draft (m)

M_S = structure mass per unit length (kg/m)

K_M = mooring stiffness (N/m)

g = gravitational constant (9.81 m/s^2)

ρ_W = density of water (1025 kg/m^3)

4.2.3 Model Analysis

Now that we have our dimensionless terms, the next step was to collect all available model test data and arrange the results with respect to these terms. The author has considered one of the most common design types, the caisson. A thorough review of numerous model studies conducted on caissons was collected and summarized in Appendix A. A summary plot of the data is shown in Figure 4.2, which plots the transmission coefficient (C_T) as a function of the wave period (T_P).

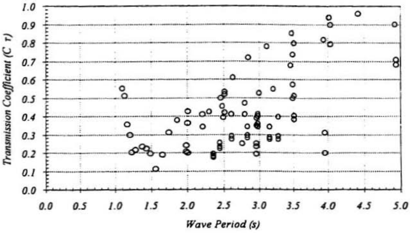


Figure 4.2: Caisson Data Summary

4.2.4 Multiple Regression

In multiple regression the objective is to build a probabilistic model that relates the dependent variable (in this case C_T) to one or more of the predictor terms as depicted in Equation 4.13. This implies that each of the predictor terms have a linear relationship with the dependent variable.

$$Y = \alpha + \beta_1 X_1 + \beta_2 X_2 + \ldots \beta_i X_i + \varepsilon \quad \textbf{(4.13)}$$

where; Y = dependent variable

α = regression constant

β_i = regression coefficient

X_i = independent variable

ε = error term

When environmental variables (wave height, wave length, wind velocity) are present as the independent variables, the relationships are not normally linear. To correct for this non-linearity, the data must be transformed by utilizing intrinsically linear functions, which include exponential, power, and logarithmic relationships (Table 4.1).

Table 4.1: Intrinsically Linear Functions (Devore, 1987)

Type	*Function*	*Transformation*	*Linear Form*
Exponential	$y = \alpha \exp^{\beta x}$	$y' = \ln(y)$	$y' = \ln(\alpha) + \beta x$
Power	$y = \alpha x^{\beta}$	$y' = \log(y)$, $x' = \log(x)$	$y' = \log(\alpha) + \beta x'$
Logarithmic	$y = \alpha + \beta \log(x)$	$x' = \log(x)$	$y' = \alpha + \beta x'$

When log( ) appears, either a base 10 or base e logarithm can be used.

The most widely used intrinsically linear function is the power model. As shown in Table 4.1, this involves taking the logs of both the dependent and independent variables, then proceeding with the regression. These single variable regression analysis techniques can easily be extended to include multiple regression. Once complete the linear form of the model can be converted to a simple power relationship as shown in Equation 4.14.

$$Y = \alpha\, X_1^{\beta_1}\, X_2^{\beta_2} \ldots X_i^{\beta_i}\, \varepsilon \tag{4.14}$$

where; Y = dependent variable

α = regression constant

β_i = regression coefficient

X_i = independent variable

ε = error term

One of the problems with the transformation involves the error term (ε), which in the transformed model represents the median of the error, not the mean. Therefore, the error term or smearing estimator must be determined for the prediction to be representative. According to Devore (1987) this is achieved by averaging the exponentials of the residuals in the transformed model (Equation 4.15).

$$\varepsilon_s = \frac{\sum_{i=1}^{n} exp(\varepsilon_i)}{n} \tag{4.15}$$

where; ε_S = smearing estimate

ε_t = residual

n = number of data points

When conducting the regression, the author utilized Microsoft Excel and conducted a stepwise forward/backward substitution analysis. This procedure starts off with no predictors in the model and considers fitting in turn with carrier X_1, X_2, and so on. The variable which yields the largest absolute *t-ratio* (a measure of the influence of the parameter) enters the model provided the ratio exceeds the specified constant, which has a standard value of 2 (t_{in}). Suppose carrier X_1 entered the model, then models with (X_1, X_2),(X_1, X_3),...,(X_1, X_i) are considered in turn. The term which coupled with X_1 that has the highest *t-ratio* is then added to the model. After each addition, the previous terms are examined to ensure that their *t-ratio* also exceeds 2, and if one of the previous terms no longer have a *t-ratio* which exceeds 2 it is discarded.

The principal behind the forward/backward substitutions, is that a single variable may be more strongly related to the dependent variable than either of the two or more variables individually, but in combination of these variables may make the single variable subsequently redundant. While in most situations these steps will identify a good model, there is no guarantee that the best or even a nearly best model will result. Close scrutiny should be given to data sets for which there appear to be strong relationships between some of the potential carriers.

4.2.5 Results

After an exhaustive analysis of the transformation methods, the power model was adopted. The reasoning being aptness of the model, as the others did not adequately describe the behaviour of the wave attenuation process. Based on the stepwise forward/backward regression, the first term to enter was the relative width (B_S/L_W) parameters with a *t-ratio* of -13.78. The next two terms to enter the relationship were the mooring parameter ($K_M/g\,\rho_W L_W^2$) and relative depth parameter (D_W/L_W) with *t-ratio's* of -2.41 and 2.44 respectively. The regression analysis was continued on the remaining three parameters but, the *t-ratio's* of these terms were all less than 2 and not included in the relationship. The resulting linear mathematical relationship is shown in Equation 4.16.

$$ln\left(\frac{H_T}{H_I}\right) = -\,2.097 + 0.151\,ln\left(\frac{D_W}{L_W}\right) - 0.437\,ln\left(\frac{B_S}{L_W}\right) - 0.192\,ln\left(\frac{K_M}{g\,\rho_W\,L_W^2}\right) \quad \textbf{(4.16)}$$

where;

H_T = transmitted wave height (m)

H_I = incoming wave height (m)

D_W = water depth (m)

L_W = wave length (m)

B_S = structure width (m)

K_M = mooring stiffness (N/m)

g = gravitational constant (m/s^2)

ρ_W = density of water (kg/m^3)

To verify aptness of the model the author used two diagnostic plots. The first is a plot of predicted versus measured C_T (Figure 4.3), while the second is a plot of standardized residuals against the predicted C_T (Figure 4.4). The less the scatter in these plots the better the fit of the model. If there are trends in the residuals either upward or downward the aptness of the model has to be questioned. With respect to the plots in Figures 4.3 and 4.4, there are no obvious trends so Equation 4.16 is valid. More information, on model aptness and interpretation can be found in Devroe (1987).

The next step is to convert the linear relationship (Equation 4.16) into that of a power relationship. Utilizing a smearing estimator of 1.025, calculated by Equation 4.15, the resulting regression constant becomes 0.126. The remaining regression coefficients become the exponents of each of three parameters indicated by Equation 4.16. The final, more useful, power relationship is shown in Equation 4.17, which provides a reasonable representation of the wave attenuation process for a hollow caisson floating breakwater. A plot of the measured versus predicted C_T is shown in Figure 4.5.

$$C_T = 0.126\left(\frac{D_W}{L_W}\right)^{0.151}\left(\frac{B_S}{L_W}\right)^{-0.437}\left(\frac{K_M}{g\,\rho_W\,L_W^2}\right)^{-0.192} \qquad \textbf{(4.17)}$$

where; C_T = transmission coefficient

D_W = water depth (m)

L_W = wave length (m)

B_S = structure width (m)

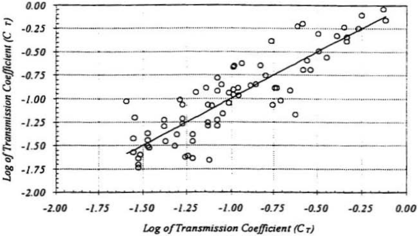


Figure 4.3: Measured versus Predicted C_T (Logarithmic Model)

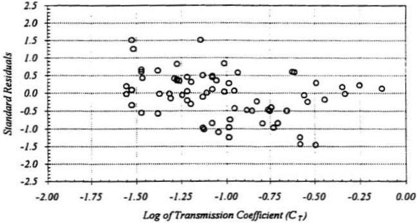


Figure 4.4: Standardized Residuals versus C_T (Logarithmic Model)

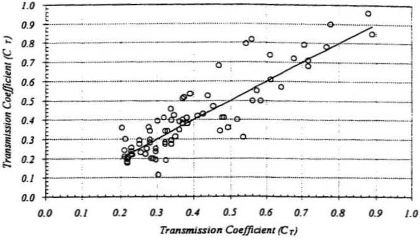


Figure 4.5: Measured versus Predicted C_T (Power Model)

K_M = mooring stiffness (N/m)

g = gravitational constant (m/s^2)

ρ_W = density of water (kg/m^3)

A review of the mechanics of the hollow caisson is required to ensure that the parameters derived from the regression analysis are valid. The caisson utilizes transformation to attenuate incoming wave energy. This requires two important aspects; a sufficient width (B_S) and mooring stiffness (K_M) so that the structure is relatively unresponsive to the passage of waves. Hence, the regression equation is consistent with this theoretical analysis. The third term is relative depth (D_W / L_W), which is a direct indication of whether the structure is in deep, transitional or shallow waters. In shallow water depths, more energy is present near the surface which increases the caisson performance, hence the addition of the term is reasonable.

The exponents of the relative terms are an indication of the sensitivity of the attenuation, as well as the effect of increasing or decreasing the numerators, in this case water depth (D_W), structure width (B_S), and mooring stiffness (K_M). The most sensitive term is the relative width (B_S / L_W) ratio, with an exponent of -0.437. The negative exponent implies that as the structure width (B_S) increases relative to the wave length (L_W), the transmission coefficient (C_T) decreases, improving performance. A 50% increase in width (B_S) causes a corresponding decrease in the transmission coefficient (C_T) by 16%.

The same is true of the mooring stiffness (K_M) ratio, with an exponent of -0.192. This results in only a 7% decrease in the transmission coefficient (C_T) for a 50% increase in the stiffness (K_M). The third and final ratio of relative depth (D_W / L_W) has an exponent of 0.151, which causes a 6% increase in the transmission coefficient (C_T) for a 50% increase the water depth relative (D_W) to the wave length (L_W).

4.3 Performance Criteria

A crucial design aspect for floating breakwaters is to provide a set of performance criteria that the structure must be designed to achieve. A number of studies on this criteria have been conducted. The most useful of these include a recent study conducted on behalf of the Small Craft Harbours (SCH) branch of the Department of Fisheries and Oceans (DFO) by Eastern Designers Limited (1991). They recommended the adoption of exceedance or threshold values (Table 4.2).

Table 4.2: SCH Exceedance Criteria (Eastern Designers Limited, 1991)

Wave Period (s)	*Significant Wave Height (m) Not to be exceeded once*		
	in 50 years	*per year*	*per week*
$2 > T_P$	-	-	0.30
$2 < T_P < 6$	0.60	0.30	0.15
$T_P > 6$	0.60	0.30	0.15

A more recent study conducted by Atria Engineering (Fournier, 1993) based their exceedance or threshold values on the dominant vessel class, as well as significant wave height and frequency of occurrence (Table 4.3).

Table 4.3: Atria Engineering Exceedance Criteria (Fournier, 1993)

Location	*STACAC class*	*Length (m)*	*Threshold H_S (m)*	*Percentage Occurrence*
Service Loading	1 2 & 3	0 - 10.7 10.7 - 19.8	0.3 0.4	1.0 - 2.5%
Mooring Basin	1, 2 & 3	0 - 19.8	0.5	1.0 - 2.5%

The most recent study conducted by the Ocean Engineering Research Centre (Morey and Cammaert, 1995) combined not only these two studies but, incorporated other studies around the world. The resulting exceedance criteria (Table 4.4) were broken down into two categories, recreational and commercial. This criteria incorporates a significant wave height (H_S), peak wave period (T_P), and hours of exceedance per year.

Table 4.4: OERC Exceedance Criteria (Morey and Cammaert, 1995)

Performance Criteria	*Harbour Type*	
	Recreational	*Commercial*
H_S *(m)*	*0.30*	*0.35*
T_P *(s)*	≤ 4	≤ 4
hrs / % exceedance per year	*88 hrs / 1%*	*88 hrs / 1%*

The importance of these criteria can be seen when investigating the wave regime. These specify exactly the inner harbour requirements for a specific installation. Utilizing this data and the deterministic formulae presented for both wave prediction and performance analysis, it becomes a probabilistic process to define the existing wave climate in terms of percentage exceedance and to evaluate a floating breakwater to determine its efficiency. In Chapter 5, the author describes how these formulae were adopted into a computer model to conduct such a probabilistic analysis.

Chapter 5

PROBABILISTIC MODEL

One of the shortcomings in previous prediction and performance analysis techniques for floating breakwater installations has been to characterize the locally generated wind-wave climate by a particular design wave (50 year event). The author has utilized a more rigorous probabilistic approach and Monte Carlo simulation to define the appropriate wave regime (Chapter 3). This allows the designer to take advantage of breakwater performance models developed by the author as well as established wave regime criteria, which until this time have not been utilised (Chapter 4).

5.1 Wind - Wave Climate

The author suggests a probabilistic approach through Monte Carlo simulation to allow the designer to take advantage of established wave attenuation criteria. The approach (Figure 5.1) incorporates the years (50), months (6-12), and storm events (240) simulated to completely describe the entire locally wind generated wave climate. The

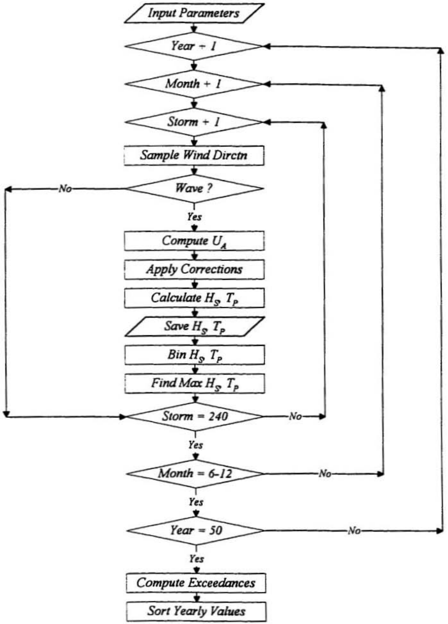


Figure 5.1: Wind-Wave Prediction Algorithm

details of the process are discussed in the subsequent sections but, in general it involves randomly generating the wind direction and speed, calculating the corresponding wave heights and periods, then binning and sorting the respective waves to arrive at an exceedance and extremal distribution.

5.1.1 Wind Climate

The first step in the prediction involves generating the storm wind direction. This is accomplished by utilising directional probabilities that are converted to a discrete cumulative distribution. A random number is generated between 0 and 100 and checked against the discrete distribution to determine the respective direction. Once predicted, a check on the fetch is made to ensure a wave can be generated. In situations where the fetch is 0 km, the routine is bypassed and a new storm event is initiated.

The storm wind speed is predicted based upon the Rayleigh distribution, re-arranged to predict wind speed based on a randomly generated probability and known mean wind speed (Equation 5.1). This speed is corrected for elevation, stability, location and drag then utilised to predict the significant wave height and peak period.

$$U_Z = \sqrt{\frac{-4\,U_M^2\,\ln(1 - P_U)}{\pi}} \tag{5.1}$$

where; U_Z = predicted storm wind speed (m/s)

U_M = mean wind speed (m/s)

P_U = probability of wind speed U (0-1)

5.1.2 Wave Climate

The wave climate is predicted based on the steps outlined in Section 3.2. The first step is to calculate the critical duration, which is compared to the given duration of 10,800 s (3 hrs). This duration is based on a review of detailed wind records for two maritime sites (Holyrood, NF and Chetticamp, NS) conducted by the author and estimating the durations of constant wind speeds, then taking the average. Although this may vary from site to site, this value is representative of locally wind generated wave conditions.

Once the limiting condition is established the representative significant wave height and wave period for the storm event is calculated, and are promptly saved to file. These values are then binned, which refers to counting one for each interval the wave height exceeds. For example, with bins at 0.1 m intervals a 0.25 m wave height would count 1 in the 0.0-0.1 m, 0.1-0.2 m, and 0.2-0.3 m bins. In addition, the wave height is compared with previous heights in order to determine the maximum conditions in that year.

5.1.3 Probabilistic Summary

At the completion of the simulation, the binned wave height numbers are divided by the total number of simulations to determine the cumulative exceedance probabilities. A secondary objective of the simulation is to compute the maximum yearly wave heights. These heights are then sorted to determine the appropriate wave height and periods for the structural (15 year) and mooring (average and 50 year) system designs.

5.2 Performance Analysis

The specific type of breakwater selected by the designer can be optimized for site specific conditions by utilizing the exceedance distribution as well as the performance prediction equations. To accomplish this, the algorithm (Figure 5.2) is broken down into three distinct algorithms including geometric stiffness, structure parameters, and performance prediction. Once the designs are selected, a cost comparison is required to select the optimal configuration. These steps are discussed in more detail in the subsequent sections.

5.2.1 Geometric Stiffness

As discussed in Chapter 4, the stiffness is a function of several parameters, which include the water depth, scope, and rode mass per unit length. This section of the algorithm utilizes the water depth at the site combined with four (4) sizes of chain including 13, 20, 25, and 32mm diameter to estimate the stiffness under 0.3m of movement for a 30m structure comprised of three (3) 10m units.

The process employs Equations outlined in Section 4.2.1, which dictate how the stiffness is to be calculated. Once a stiffness for a given water depth and chain diameter has been calculated, the next step is to proceed with selecting the dimensions of the breakwater and predicting performance.

5.2.2 Structural Parameters

Depending upon the type of structure being considered, the program initiates with the smallest recommended dimensions of the system. These dimensions are based on an extensive review of floating breakwater installations conducted by Morey and Cammaert (1995).

The key is to predict performance for each combination of mooring stiffness and structure parameters, then using this prediction to calculate the exceedance probabilities for the transmitted wave heights (H_T). If a combination of structure size and mooring stiffness satisfies the exceedance criteria (1% < 0.3m), the parameters are stored. The same process is then repeated for a second, third, etc., mooring stiffness and the parameters stored if the exceedance criteria is satisfied.

Once all the possible combinations of mooring stiffness and structure parameters have been examined, a final cost comparison is conducted. This involves estimating both the structural and mooring costs. These are based on analysis of typical concrete caisson costs summarized by Morey and Cammaert (1995). The resulting mathematical model is shown in Equation 5.1.

$$H_C = \frac{600(9.4\,B_S + 91) + 8\,L_C\,C_C}{30} + 96 \tag{5.1}$$

where; H_C = hollow caisson costs ($/m)

B_S = structure width (m)

L_C = mooring cable length (m)

C_C = mooring cable cost per length ($/m)

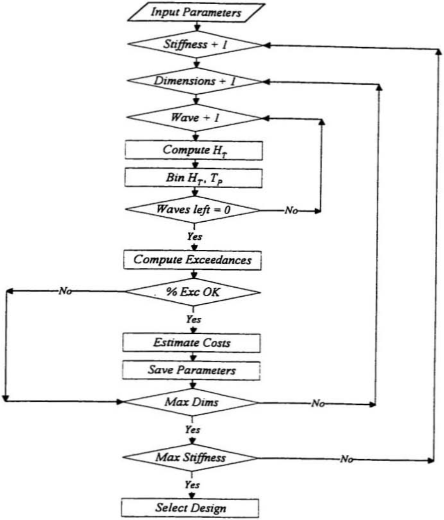


Figure 5.2: Performance Analysis Algorithm

5.3 Example Simulation

To better illustrate this probabilistic approach, consider the following example of Dildo, a site located in Trinity Bay, Newfoundland. A review of the site was conducted by the author to ascertain the necessary directional probabilities, mean wind speeds, fetches, and water depths in each of the 16 directions (Table 5.1).

Table 5.1: Dildo Data Summary

Direction	*Discrete Probability %*	*Cumulative Probability %*	*Mean Wind Speed m/s*	*Fetch km*	*Water Depth m*
N	4.7	4.7	0.0	0.0	0
NNE	2.4	7.1	0.0	0.0	0
NE	2.9	10.0	0.0	0.0	0
ENE	2.4	12.4	0.0	0.0	0
E	4.3	16.7	0.0	0.0	0
ESE	5.6	22.3	0.0	0.0	0
SE	5.8	28.1	0.0	0.0	0
SSE	5.0	33.1	6.4	2.8	20
S	6.9	40.0	6.2	5.7	30
SSW	8.4	48.4	6.5	3.4	30
SW	14.7	63.1	6.9	2.8	30
WSW	11.8	74.9	0.0	0.0	0
W	8.5	83.4	0.0	0.0	0
WNW	4.4	87.8	0.0	0.0	0
NW	4.7	92.5	0.0	0.0	0
NNW	5.6	98.1	0.0	0.0	0

Based on the information it was anticipated that the winds of greatest concern blow out of the SSE through SW directions for a total probability of 35% of the time. A wind - wave simulation was performed using the software developed by the author, based on a design period of 50 years, an eight (8) month operating season, and a 11 m water depth at the proposed location of the breakwater. The results of the analysis include an exceedance distribution (Table 5.2) as well as typical design storm conditions (Table 5.3).

Table 5.2: Exceedance Distribution

H_S	%TTL	%WVE
> 0.0	35.0	100.0
> 0.1	23.7	67.6
> 0.2	16.9	48.2
> 0.3	11.4	32.7
> 0.4	7.2	20.6
> 0.5	4.1	11.8
> 0.6	2.1	6.1
> 0.7	1.0	2.9
> 0.8	0.4	1.3
> 0.9	0.2	0.5
> 1.0	0.1	0.2
> 1.1	0.0	0.1
> 1.2	0.0	0.0

Table 5.3: Design Storm Conditions

	Speed	Period	Height
Year	m/s	s	m
avg	9.2	1.28	0.24
1	18.5	2.53	0.85
5	19.0	2.57	0.88
10	19.9	2.63	0.93
15	28.9	2.65	1.09
25	21.6	2.74	1.02
50	27.5	3.08	1.34

On review of the exceedance distribution, the critical value to note is 0.70 m, which a probability of 1%. In order for a breakwater to be effective it would be necessary for the structure to attenuate this wave from 0.7 to 0.3 m or a reduction of 57%. Some of the other important values include the average and 50 year conditions with heights of 0.24 and 1.34 m and periods of 1.28 and 3.08 s respectively. These are utilized in selecting the mooring system components which include the rodes, anchors, and connections. The final value is the 15 year event with a height and period of 1.09 m and 2.65 s respectively. This parameter is used to verify the safety factors in the structural design.

Once this analysis is completed the next process involves the optimization of the breakwater with respect to the costs of the system. The results of the analysis are shown in Table 5.4, which indicate the most cost effective system based on the combination of mooring stiffness, structure draft and structure width. The resulting structure should have a draft of 1 m, width of 3 m, and use 13 mm diameter chain in the mooring system.

Table 5.4: Breakwater Cost Optimization

Stiffness ***kN/m***	***Draft*** ***m***	***Width*** ***m***	***Chain*** ***mm***	***Costs*** ***$/m***
18.6	*1*	*3*	*13*	*1,034*
31.6	*1*	*4*	*16*	*1,319*
48.7	*1*	*3*	*19*	*1,278*
76.8	*1*	*3*	*25*	*1,432*

Chapter 6

CONCLUSIONS

Current environmental and financial restrictions on harbour developments dictate that alternatives to traditional fixed rubble-mound and caisson breakwaters are essential to the future of coastal engineering. The *Floating Breakwater* is one such alternative, a concept which utilizes reflection, dissipation and/or transformation to reduce wave energy and therefore attenuating incident waves to an acceptable level.

Floating breakwater systems reduce incident wave heights through the conversion of wave energy via reflection, transformation and dissipation. These energy reduction methods can act in a singular nature or in a combination of one, two, or all three modes. The author suggests that the most effective approach for classification would involve separating systems by these three methods of wave attenuation

The analysis of floating breakwater systems was divided into two distinct stages. The first was the evaluation of the wind generated wave climate through the use of recently developed empirical and theoretical formulae. This involves a combination of the wind speed and duration, fetch, and water depth. These simplified formulae provide an accurate yet simple deterministic approach that can with some modification be adapted into a computer algorithm.

The second stage involves performance prediction of a given breakwater size in combination with its mooring system. This involved a detailed analysis of existing techniques as well as the collection and analysis of existing breakwater performance. The Author employed well established dimensional analysis and regression techniques to determine a representative deterministic model to evaluate performance. The model involved parameters relating to the structure, mooring system, site bathymetry, and the wave climate.

The deterministic models developed by the author provides a unique ability by which exiting guidelines on acceptable wave climates. These guidelines recommend the use of a 0.30m threshold wave height combined with an exceedance probability of 1%. The final aspect of the probabilistic approach was the development of a computer algorithm.

The program developed by the author is unique, in that it provides the designer with a detailed analysis of the wave climate and then optimizes the breakwater based on the site specific criteria. The wave climate module predicts two key items which include wave exceedance probabilities and the extremal distribution. Each individual wave predicted in this stage is applied to the prediction equations and a transmitted exceedance distribution developed. When the exceedance criteria is met, an estimate of the system costs is developed and the most effective system selected.

A case study based on a Newfoundland site, located in Trinity Bay, was conducted and the most cost effective system based on several combinations of mooring stiffness, structure draft and structure width selected. The resulting structure would have a draft of 1 m, width of 3 m, and use 13 mm diameter chain in the mooring system and cost $1,034 in comparison to the other systems which ranged between $1,200 and $1,400.

Overall, the techniques developed and applied by the Author serve well to provide the breakwater designer with an effective means of determining the approximate size and mooring system for a given type of breakwater.

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APPENDIX A

CAISSON TEST DATA

This appendix includes relevant data collected from a series of model experiments conducted on various types of floating caisson breakwaters. A more detailed discussion of these experiments can be found in Morey and Cammaert (1995).

Each of the parameters included in the following table(s) form the basis of the parametric analysis discussed in this thesis. These parameters are as follows:

ID = model test identification

H_T = transmitted wave height (m)

H_I = incoming wave height (m)

L_W = wave length (m)

D_W = water depth (m)

B_S = structure width (m)

D_S = structure draft (m)

M_S = structure mass per unit length (kg/m)

K_M = mooring stiffness (N/m)

ID	H_T	H_I	L_W	D_W	B_S	D_S	M_S	K_M
1A	0.126	0.648	8.64	6.40	7.11	0.90	6399	45472
1A	0.165	0.701	9.34	6.40	7.11	0.90	6399	45472
1A	0.235	0.803	10.71	6.40	7.11	0.90	6399	45472
1A	0.257	1.025	13.67	6.40	7.11	0.90	6399	45472
1A	0.323	0.940	12.54	6.40	7.11	0.90	6399	45472
1A	0.399	1.164	15.52	6.40	7.11	0.90	6399	45472
1A	0.502	1.261	16.81	6.40	7.11	0.90	6399	45472
1A	0.738	1.440	19.22	6.40	7.11	0.90	6399	45472
1B	0.118	0.648	8.64	6.40	7.11	1.21	8603	45472
1B	0.156	0.701	9.34	6.40	7.11	1.21	8603	45472
1B	0.224	0.893	11.91	6.40	7.11	1.21	8603	45472
1B	0.239	1.025	13.67	6.40	7.11	1.21	8603	45472
1B	0.263	0.940	12.54	6.40	7.11	1.21	8603	45472
1B	0.320	1.164	15.52	6.40	7.11	1.21	8603	45472
1B	0.347	1.261	16.81	6.40	7.11	1.21	8603	45472
1B	0.551	1.441	19.22	6.40	7.11	1.21	8603	45472
1C	0.121	0.691	8.64	6.40	7.11	1.92	13651	45472
1C	0.189	0.747	9.34	6.40	7.11	1.92	13651	45472
1C	0.212	1.094	13.67	6.40	7.11	1.92	13651	45472
1C	0.234	0.857	10.71	6.40	7.11	1.92	13651	45472
1C	0.297	1.003	12.54	6.40	7.11	1.92	13651	45472
1C	0.353	1.242	15.52	6.40	7.11	1.92	13651	45472
1C	0.393	1.345	16.81	6.40	7.11	1.92	13651	45472
1C	0.618	1.537	19.22	6.40	7.11	1.92	13651	45472
2A	0.051	0.245	6.13	13.71	9.15	1.62	14823	16310
2A	0.192	0.549	13.72	13.71	9.15	1.62	14823	16310
2A	0.195	0.976	24.40	13.71	9.15	1.62	14823	16310
2A	1.033	1.519	37.97	13.71	9.15	1.62	14823	16310
2B	0.059	0.245	6.13	13.71	9.15	1.83	16745	16310
2B	0.214	0.549	13.72	13.71	9.15	1.83	16745	16310
2B	0.303	0.976	24.40	13.71	9.15	1.83	16745	16310
2B	1.078	1.519	37.97	13.71	9.15	1.83	16745	16310
3A	0.128	0.353	6.22	3.05	3.05	0.61	1861	16310
3A	0.224	0.654	13.86	3.05	3.05	0.61	1861	16310
3A	0.275	0.531	9.84	3.05	3.05	0.61	1861	16310
3A	0.340	0.681	19.06	3.05	3.05	0.61	1861	16310
3A	0.705	0.890	25.42	3.05	3.05	0.61	1861	16310
3B	0.062	0.311	6.22	9.00	3.05	0.61	1861	30290
3B	0.104	0.305	7.63	9.00	3.05	0.61	1861	30290
3B	0.175	0.412	8.24	9.00	3.05	0.61	1861	30290
3B	0.199	0.555	13.86	9.00	3.05	0.61	1861	30290
3B	0.258	0.549	12.20	9.00	3.05	0.61	1861	30290

ID	H_T	H_I	L_W	D_W	B_S	D_S	M_S	K_M
3B	0.294	0.551	9.84	9.00	3.05	0.61	1861	30290
3B	0.421	0.572	19.06	9.00	3.05	0.61	1861	30290
3C	0.083	0.197	9.84	14.63	3.05	0.61	1861	34060
3C	0.100	0.244	12.20	14.63	3.05	0.61	1861	34060
3C	0.185	0.462	13.86	14.63	3.05	0.61	1861	34060
3C	0.389	0.681	19.06	14.63	3.05	0.61	1861	34060
3C	0.397	0.722	16.05	14.63	3.05	0.61	1861	34060
3C	0.761	0.847	25.42	14.63	3.05	0.61	1861	34060
3C	0.779	0.813	30.50	14.63	3.05	0.61	1861	34060
4A	0.278	0.610	9.63	7.62	3.66	1.07	3916	34060
4A	0.295	0.563	14.08	7.62	3.66	1.07	3916	34060
4A	0.536	0.674	19.26	7.62	3.66	1.07	3916	34060
4B	0.210	0.535	9.73	7.62	4.88	1.07	5222	34060
4B	0.228	0.556	13.90	7.62	4.88	1.07	5222	34060
4B	0.260	0.635	10.58	7.62	4.88	1.07	5222	34060
4B	0.445	0.655	18.71	7.62	4.88	1.07	5222	34060
4B	0.696	0.851	24.33	7.62	4.88	1.07	5222	34060
5	0.017	0.151	3.78	3.50	1.80	0.30	540	6500
5	0.019	0.094	2.35	3.50	1.80	0.30	540	6500
5	0.022	0.101	2.52	3.50	1.80	0.30	540	6500
5	0.027	0.137	3.43	3.50	1.80	0.30	540	6500
5	0.027	0.116	2.91	3.50	1.80	0.30	540	6500
5	0.027	0.090	2.24	3.50	1.80	0.30	540	6500
5	0.028	0.126	3.15	3.50	1.80	0.30	540	6500
5	0.030	0.084	2.10	3.50	1.80	0.30	540	6500
5	0.032	0.168	4.20	3.50	1.80	0.30	540	6500
5	0.041	0.080	2.00	3.50	1.80	0.30	540	6500
5	0.042	0.076	1.89	3.50	1.80	0.30	540	6500
5	0.059	0.189	4.73	3.50	1.80	0.30	540	6500
5	0.082	0.216	5.39	3.50	1.80	0.30	540	6500
5	0.108	0.252	6.30	3.50	1.80	0.30	540	6500
5	0.124	0.302	7.56	3.50	1.80	0.30	540	6500
5	0.189	0.378	9.45	3.50	1.80	0.30	540	6500
5	0.264	0.433	10.82	3.50	1.80	0.30	540	6500
5	0.363	0.504	12.60	3.50	1.80	0.30	540	6500
5	0.472	0.605	15.12	3.50	1.80	0.30	540	6500
5	0.643	0.756	18.90	3.50	1.80	0.30	540	6500
5	0.948	1.008	25.20	3.50	1.80	0.30	540	6500
5	1.361	1.512	37.80	3.50	1.80	0.30	540	6500