Floating breakwater performance

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Thesis -
Project Report

FLOATING BREAKWATER PERFORMANCE

BY

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SCHOOL OF CIVIL ENGINEERING

MASTER OF ENGINEERING SCIENCE

8·909G PROJECT REPORT

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THE UNIVERSITY OF NEW SOUTH WALES

FLOATING BREAKWATER PERFORMANCE

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MICHAEL A. THACKRAY

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MICHAEL A. THACKRAY BE CEng MICE MIEAust

JANUARY 1984

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ACKNOWLEDGEMENTS

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These include the Coastal Council of New South Wales, whose generous assistance in providing a research grant for the project is gratefully acknowledged.

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Michael A Thackray

January 1984

SYNOPSIS

Floating breakwaters provide an alternative form of wave barrier to conventional rubble mound and caisson breakwaters in low to moderate wave climates with relatively short wave periods.

This report reviews the literature on floating breakwaters and discusses existing and potential uses of floating breakwaters in Australia.

It also contains results of a field programme which measured the prototype performances of two breakwaters in Sydney Harbour and compared them with those predicted using physical models. The programme was carried out using low cost wave measuring equipment specially developed for this study.

LIST OF SYMBOLS

H_I	-	Incident Wave Height
$_T$	-	Transmitted Wave Height
C_T	-	Coefficient of Transmission (= H_T/H_I)
T	-	Wave Period
L	-	Wave Length
B	-	Breakwater Beam
D	-	Water Depth
d	-	Breakwater Draft
F	-	Mooring Force per unit length
H_I/L	-	Wave Steepness

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1. INTRODUCTION

The magnificent network of coastal waterways in New South Wales is one of the State's most valuable natural assets. These waterways provide sheltered deep-water ports for commercial shipping, safe areas for recreational boating, excellent fishing grounds for commercial and amateur fishermen, and ideal conditions for oyster growing, as well as being attractive destinations for tourists and holidaymakers.

With these waterways, Australia's high standard of living, and a climate conducive to year-round outdoor activities, the rapid growth in popularity of recreational boating in NSW experienced in recent years may be expected to continue.

However, the development of new facilities is, in general, failing to keep pace with demand because of the shortage of suitable areas of waterfront land required for boat ramps, marinas, parking areas and related onshore facilities.

For non-trailerable craft, which require moorings or marina berths, there is an acute shortage in NSW especially in popular boating areas such as Sydney Harbour and Pittwater. Here the bays and harbours, particularly those providing any natural protection, are close to saturation point with inefficient swing moorings, and few sites are suitable for the development of modern marina facilities. This is due to the generally steep topography of the land and the proliferation of residential developments which have left little vacant land available for onshore facilities.

Figure 1.1 is from a recent report by the author for the Boating Industry Association of NSW (Ref 12), and illustrates the widening gap between supply and demand for mooring facilities in the State, particularly in the Sydney and Pittwater areas.

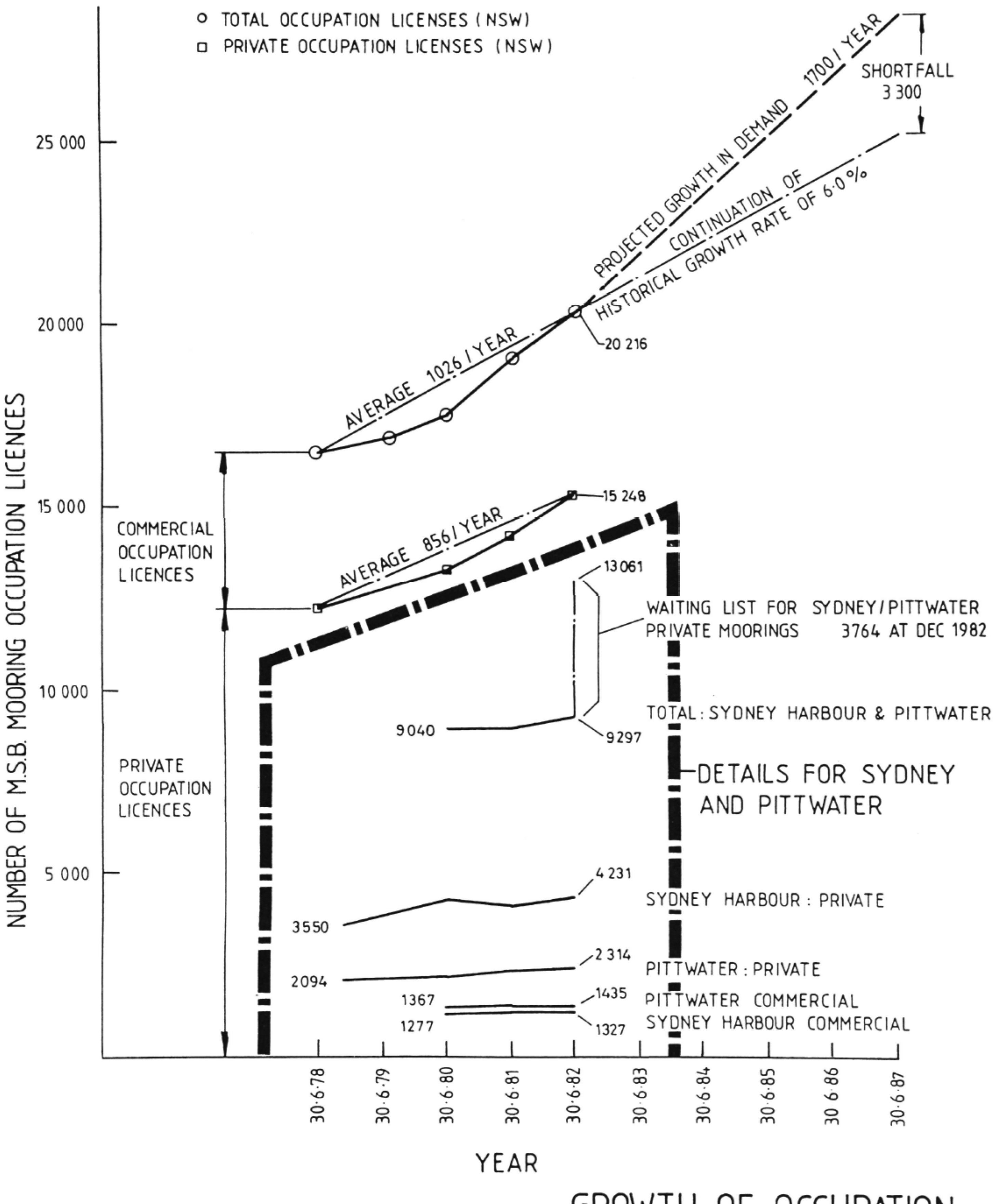
Since presentation of that report to the Government, early in 1983, several interdepartmental committees have been established to consider marina development and related matters, including planning and design, approval procedures and land tenure for leased waterfront sites.

One likely outcome of these moves is that relatively exposed sites will be considered for marina and other recreational boating developments, and these could require artificial wave protection in the form of floating breakwaters. If this type of protection can be successfully provided, the benefits to the boating public and the community generally will be significant.

One other important area where this type of protection may be beneficial is in estuaries where the conflict between recreational boating and oyster farming is resulting in significant losses to the oyster industry. Passing boat waves move the oysters around in their trays, which unless raked regularly allow the oysters to be washed out to die on the seabed.

If floating breakwaters can be shown to be satisfactory in terms of cost and performance, the benefits to the oyster farmer and consumer as well as the boating public could prove to be significant.

Figure 1.1



GROWTH OF OCCUPATION LICENSES IN N.S.W.

This report surveys the literature on floating breakwaters, describes several installations in Australia, gives details of a performance measurement programme for two floating breakwaters in Sydney and considers potential applications for floating breakwaters in Australia.

2. FLOATING BREAKWATERS - CHARACTERISTICS, PERFORMANCE PARAMETERS AND DESIGN

2.1 CHARACTERISTICS

A floating breakwater may be defined as **a moored structure which floats at or near the water surface and causes incident wave energy to be dissipated by reflection, turbulence or any other means, and thus causes wave heights to be reduced on its leeward side.**

The concept of floating breakwaters has long been recognised as an alternative means of providing artificial harbours in areas having a low to moderate wave climate, ie where wave heights do not exceed (say) 2.5 m and periods do not exceed 4 to 5 seconds. The main advantages of this type of breakwater over the more conventional rubble mound and caisson type structures are as follows:

- economy of material - The breakwater floats at water level, where it has its greatest effect in suppressing surface wave activity by reflecting and dissipating wave energy. Its economic advantage over fixed structures increases with water depth.

- mobility - As it floats, it can be towed into position for any length of time and towed elsewhere when required. This allows floating breakwaters to be used either in a permanent role, such as protection of mooring facilities, or a temporary role, such as protection of construction operations including dredging and pipe-laying in exposed locations.

- speed of construction - Using prefabricated units, a floating breakwater can be installed in a fraction of the time required for structures built in situ, including rubble mound and piled slatted breakwaters. One benefit of reduced installation time is that the probability of damage due to adverse weather is greatly reduced.

- lack of dependence on bottom conditions - As the breakwater is supported by flotation, no detailed geotechnical investigation is required beyond a general examination of the seabed and typical profile to establish the optimum type of mooring system required. Provided suitable anchoring is possible, poor bottom conditions do not affect the structure.

- minimal environmental impact - In order to prevent wave overtopping, and to cater for tidal conditions, rubble mound and caisson breakwaters must be constructed with crest/deck level several metres above High Water Level. The resulting visual impact can often be considered unsatisfactory. However, floating breakwaters are usually constructed with freeboard of less than one metre, and their visual impact is therefore much less. Floating breakwaters also have virtually no effect on currents which flush the sheltered area and thus enable water quality to be maintained. Rubble mound and caisson breakwaters are also permanent structures and can only be removed at great expense, and this can limit the options for redevelopment/expansion of the harbour. By comparison floating breakwaters are easily removed and relocated and thus the changes to the original environment may be reversed if required.

- berthing efficiency - Pontoon type floating breakwaters have the advantage over rubble mound breakwaters that they can be used to provide berths for vessels and be incorporated in the overall layout of marinas as main walkways, thus making maximum use of available waterways.

The disadvantages of floating breakwaters include:

- limited operational life - It is estimated that the typical operational life of a pontoon type floating breakwater is less than 30 years, whilst overseas experience indicates that the life of a floating tyre breakwater is less than 10 years. At the end of its life, such a structure would require either complete replacement or major structural repairs.

- relatively high maintenance costs - Due to the cyclic nature of the loadings imposed on breakwater modules, connections and the anchoring system, regular inspection and replacement of components is essential. As much of this work involves expensive underwater work, the cost is generally much higher than for routine maintenance of above water structures. Floating breakwaters also collect marine growth which may need to be removed periodically to maintain adequate freeboard.

- limited performance range - Floating breakwater performance is inversely related to wave height and period, within the approximate limits given above. Site conditions must therefore be carefully assessed to ensure that the desired level of performance can be achieved.

- severe consequences of failure - The most likely causes of failure of a floating breakwater are the dragging or snapping of the anchor system, and the failure of module connections. Either way, the modules may be released to become floating battering rams in a crowded mooring area. At the same time, wave action within the harbour is increased as the breakwater barrier is removed, and the possibility of severe damage to moored vessels and harbour facilities is greatly increased. By comparison, failure of rubble mound or caisson breakwaters is usually localised, and the mode of failure (say by slumping of a rubble mound or overturning of a caisson) does not usually result in total removal of the wave barrier, so partial protection is still provided.

2.2 PERFORMANCE PARAMETERS

From the literature reviewed (See **Section 3**) it is apparent that the following dimensionless parameters are the most significant in determining floating breakwater performance:

- L/B, the ratio of wave length to breakwater width, known as "relative width".

- H_I/L, the ratio of incident wave height to wave length, known as "wave steepness".

- d/D, the ratio of breakwater draft to water depth, known as "relative depth".

- H_R/H_I, the ratio of reflected wave height to incident wave height, known as "C_R" the coefficient of reflection.

- The stiffness of the mooring system which affects the breakwater's dynamic response to incident waves.

Breakwater performance is generally measured using H_T/H_I, the ratio of transmitted wave height to incident wave height, known as C_T, the "transmission coefficient".

The above parameters are illustrated in **Figure 2.1.**

Following observations during the field tests on Sydney harbour (See **Section 5),** an energy loss coefficient has been defined as follows:-

By considering wave energy, the following relationship may be derived:

$$H_I^2 = H_T^2 + H_R^2 + \text{Energy Loss Function}$$

$$\therefore \quad H_T^2/H_I^2 + H_R^2/H_I^2 + \frac{\text{Energy Loss Function}}{H_I^2} = 1.0$$

$$\therefore \quad (C_T)^2 + (C_R)^2 + C_L = 1.0$$

where C_L may be considered as an "energy loss" coefficient.

For a particular breakwater, the relative magnitudes of C_T, C_R and C_L indicate the mode of energy dissipation. For example, a low C_T and C_R value gives a high C_L value, indicating a high degree of turbulence, whilst a high C_R value indicates a high degree of wave reflection and low turbulence.

2.3 DESIGN

The first step in the design of a floating breakwater for a particular site is to determine the maximum permissible wave height inside the breakwater, ie, H_T, the transmitted wave height. This will usually be dictated by the type of vessel and berthing system. The following **Table 2.1** gives recommended Canadian criteria for a small craft harbour used exclusively by pleasure boats (Ref 27), and appears appropriate for Australian conditions.

TABLE 2.1 SMALL CRAFT HARBOURS - ACCEPTABLE WAVE HEIGHTS

Wave Direction Relative to Vessel	Peak Wave Period (S)	Significant Wave Height (m)		
		1 in 50 year	Once per Year	Once per week
Head Sea	$2 > T$	-	-	-
Head Sea	$2 < T < 6$	0.6	0.3	0.15
Head Sea	$T > 6$	0.6	0.3	0.15
Beam Sea	$2 > T$	-	-	-
Beam Sea	$2 < T < 6$	0.23	0.15	0.08
Beam Sea	$T > 6$	0.23	0.15	0.08

(Note: "Once per week" indicates that this wave height should not be exceeded more than 10% of the time).

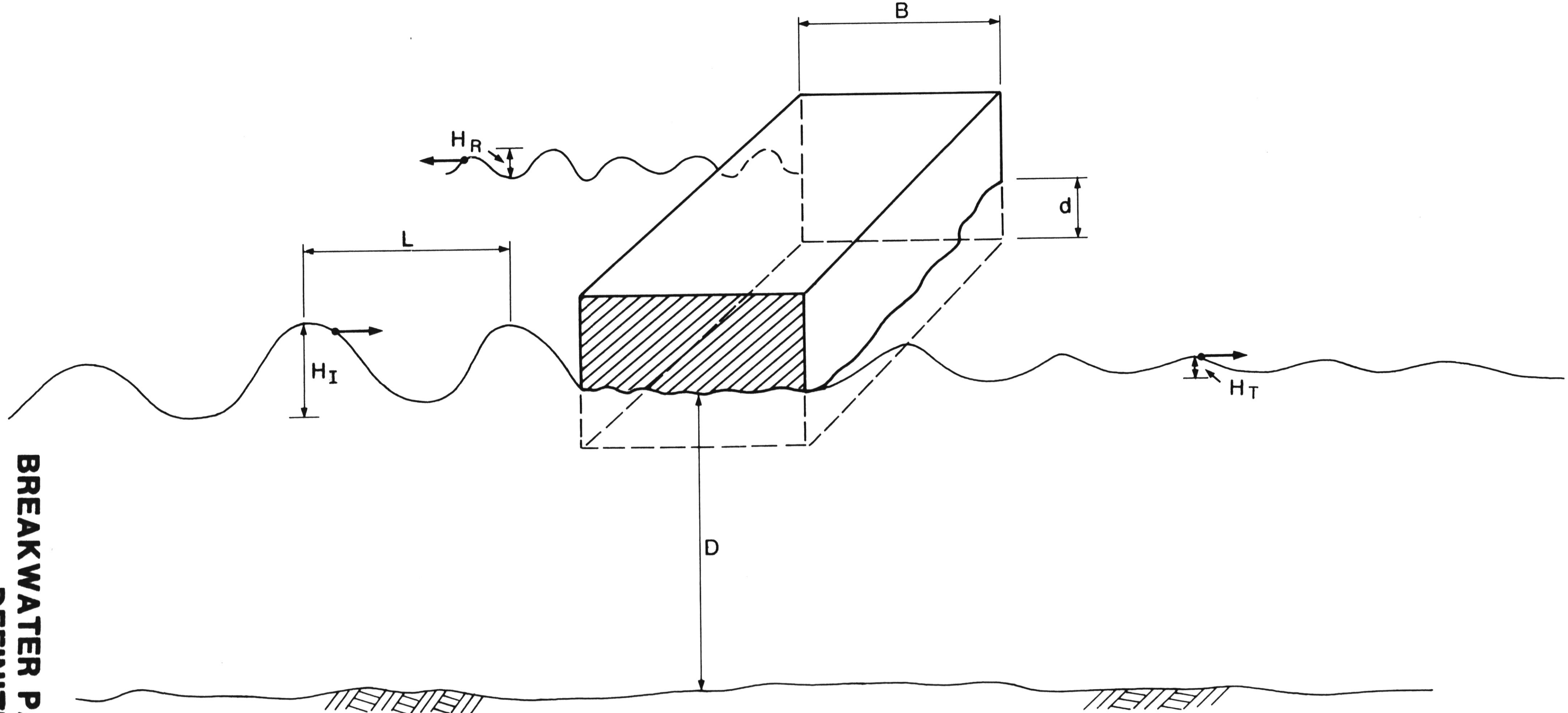


Figure 2.1

BREAKWATER PARAMETERS - DEFINITION SKETCH

Having determined the acceptable transmitted wave height, it is necessary to determine the incident wave heights likely to be experienced at the site.

Typically, there is little or no site-specific wind or wave data available, and it is necessary to investigate the wave climate at the site. Obviously the best way, if funds permit, is to establish a data collection programme for at least 12 months to allow for seasonal variations.

This involves the installation of a wave recording device such as a wave rider buoy or a wave pole attached to a pile or other structure. Since neither of these provides data on wave direction, it is advisable to install an anemometer nearby which records both wind strength and direction, provided it can be located to minimise local effects.

Analysis of the records thus obtained provides a correlation between local wind and waves. It then becomes possible to detect the presence of other waves such as long period ocean swells generated by offshore disturbances which may also affect the site.

Correlation of the wind record with the nearest official weather station is also possible for the data collection period. Assuming reasonable correlation is found, it is possible to analyse the long-term weather record for the station, and build up a relatively reliable picture of the long-term wind and wave climate at the site.

For situations where this approach is not practical, a study of fetch characteristics applying hindcasting techniques will provide a reasonable indication of the prevailing wave climate provided that the site is not subject to outside influences such as ocean swell effects. A recent paper by C L Vincent **(Ref 23)** updates the methods described in the Shore Protection Manual. **(Ref 29).**

Having established the incident wave height for a selected return interval, and knowing the allowable transmitted wave height, it is possible to determine the transmission coefficient required. However there are several other criteria to be satisfied, namely:

- That all structural elements, including the breakwater sections, connections and mooring system, can perform satisfactorily under normal conditions as well as during the worst storm that might reasonably be expected to occur during the life of the structure and it is suggested that this could have a return interval of 50 years (pontoon breakwater) or 20 years (floating tyre breakwater).

- That the costs of construction and maintenance be within reasonable limits to ensure the economic viability of the project.

It should be emphasised that the accurate assessment of wave period is of major importance. For deep-water waves where water depth exceeds 50% of the wave length (and most floating breakwater sites come into this category), $L = 1.56\ T^2$.

Therefore, a relatively small increase in wave period from, say, 4.0 to 4.5 seconds increases the wave length from 25.0 m to 31.6 m. Model test results for a 7 m wide catamaran floating breakwater (Ref 25) indicate that this could increase the transmission coefficient from 22% to 54%, and could result in unacceptable conditions inside the breakwater.

2.4 CONCLUSIONS

It is concluded that

- The key performance parameter is the transmission coefficient C_T which is dependent mainly on the relative width (L/B), wave steepness (H_I/L) and relative depth (d/D).

- It is critical that wind and wave climate at the site be accurately assessed, since if the wave period experienced by the prototype is significantly greater than estimated, transmitted waves could exceed acceptable limits.

- The breakwater should be designed to withstand the worst storm that might reasonably be expected to occur during its working life, as well as being able to resist the fatigue loads on connections and moorings imposed by constant movement of the system.

- It is desirable to carry out a site specific wind and wave data collection programme for at least 12 months. However, appropriate hindcasting techniques do provide a suitable basis for design provided

  that the site is not subject to outside influences such as long period ocean swell penetration.

- Measurement of reflected wave heights also allows determination of the energy loss coefficient C_L, which provides an indication of the mode of energy dissipation for a particular breakwater.

3. FLOATING BREAKWATERS - LITERATURE SURVEY

3.1 TYPES OF FLOATING BREAKWATER

Hales **(ref 23)** refers to the identification of 60 different floating breakwater configurations which may be categorised into 10 basic types (See **Figure 3.1** for sketches) as follows:

(i) The pontoon floating breakwater

- generally prismatic in shape
- section may consist of single or multiple pontoons
- double pontoon type combines large mass with large radius of gyration and may also be used as a floating pier for access and/or cargo unloading
- most basic types well documented by experimental and prototype testing
- most floating breakwaters in use are of this type.

(ii) The sloping float breakwater -

- generally has inshore end resting on sea bed, seaward end floating with anchors at both ends,
- US Navy and US Corps of Engineers are currently investigating characteristics and possible use of 27 m long steel pontoons which are standard US military equipment
- experimental results available. Prototype being tested in USA (July 1982).

(iii) Scrap-tyre floating breakwater

- 3 basic structural types - "Wave Maze", "Goodyear Module" and "Pole-Tyre", all well documented by experimental and field testing
- "Goodyear Module" prototype has been tested extensively and system has been successfully used in many locations, is under consideration at others.

(iv) A-Frame arrangement floating breakwater -

- consists of a pair of horizontal cylinders at the water surface supporting a vertical wave wall in the centre
- utilises locally available timber in areas such as Canada, parts of USA in order to minimise cost
- model and prototype performance well documented

- vertical wall section induces relatively high mooring forces
- system was first used at Lund, British Columbia in 1965.

(v) Tethered float breakwater -

- consists of a large number of buoyant spherical floats tethered at or below the water surface
- attenuation mainly by drag and turbulence
- no prototype results published.

(vi) Porous walled breakwater -

- designed to reduce mooring forces by creating turbulence
- no prototype results published.

(vii) Hydraulic breakwater -

- releases a high velocity jet of water near the surface to encourage energy dissipation by wave breaking
- no prototype results published
- generally considered very expensive to operate because of power requirements

(viii) Flexible membrane floating breakwater -

- flexible wide floating blanket of rubber sheets with a second layer some distance below the surface
- 2 types - bag and blanket
- both require large area of water to be covered relative to wave length
- no prototype results published.

(ix) Turbulence generator floating breakwater -

- consists of thin horizontal plate(s) designed to cause wave breaking and thus dissipate wave energy
- includes the "Seabreaker" and "Harris and Sutherland" breakwaters developed in the UK, both of which are well documented in model and prototype form
- this type also includes parabolic beaches, which may be hinged on the seabed or freely floating, however no prototype results for parabolic beaches yet published.

(x) Energy peak dispersion floating breakwater -

- a staggered front is presented to the incoming wave so that the sections of dimension half-wavelength cause the pressure forces to be out of phase by 180° thus reducing the mooring forces

- one type is the offset floating breakwater, which reduces mooring forces by offsetting sections of the seaward face by one half of maximum wave length

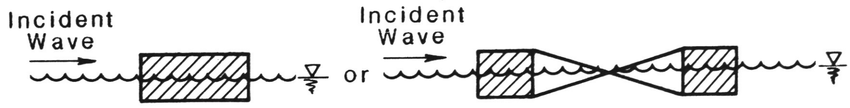
- no prototype results availble for the offset breakwater

- another type is the Bowley Wave barrier, an array of modular mooring buoy-type structures whose response to incident waves sets up a train of reflected waves which trigger wave breaking and reduce transmitted wave energy

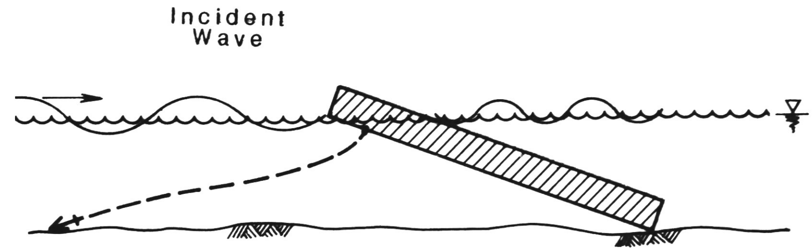
- no prototype results available for the Bowley Wave barrier.

Figure 3.1

(i) PONTOON

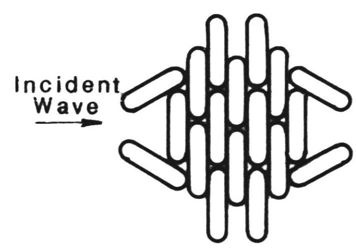


(ii) SLOPING FLOAT

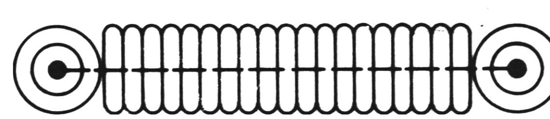


(iii) SCRAP TYRE

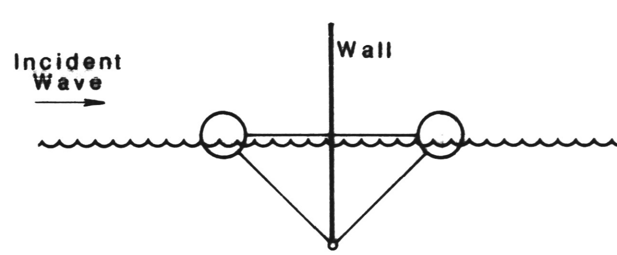
(a) "Goodyear Module"



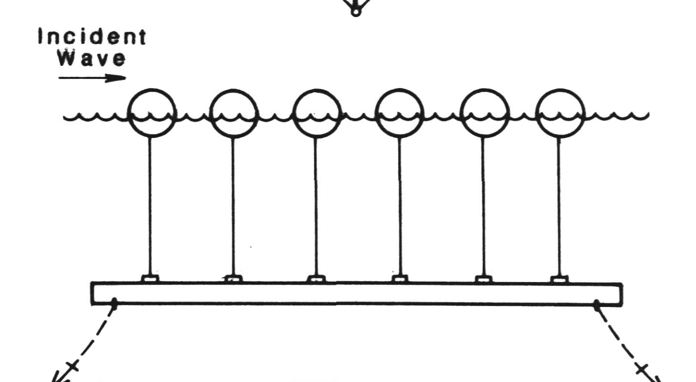
(b) Pole Tyre



(iv) A FRAME

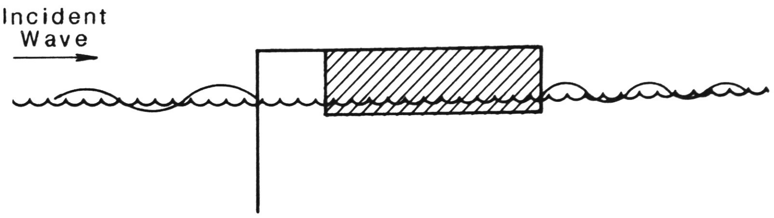


(v) TETHERED FLOAT

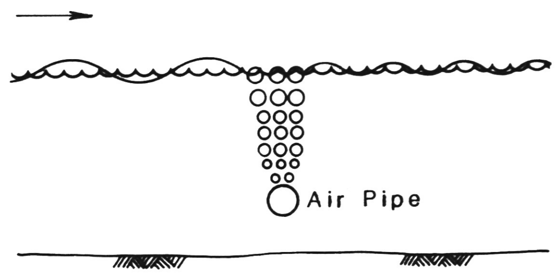


BREAKWATER TYPES

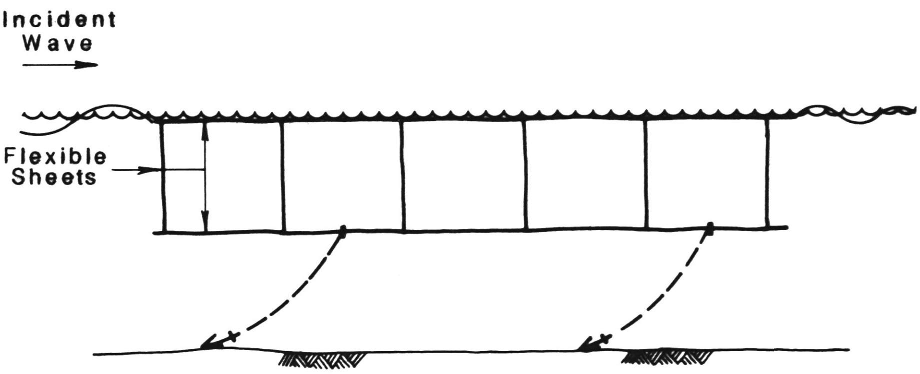
(vi) POROUS WALLED BREAKWATER



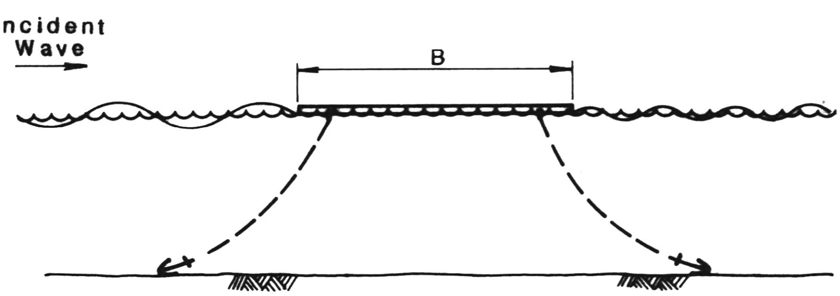
(vii) HYDRAULIC BREAKWATER



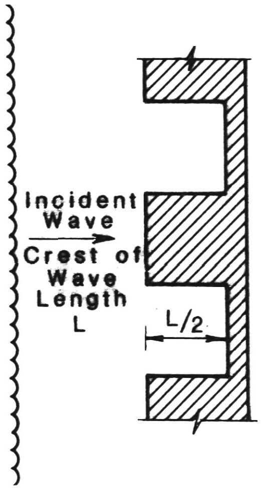
(viii) FLEXIBLE MEMBRANE



(ix) TURBULENCE GENERATOR
FLOATING BREAKWATER



(x) ENERGY PEAK DISPERSION
BREAKWATER



3.2 FIELD INSTALLATIONS

The following table gives details of a range of floating breakwater installations in Australia, the United Kingdom, Canada, New Zealand and the United States.

TABLE 3.1 - SUMMARY OF FLOATING BREAKWATER INSTALLATIONS

Type	Location	Sect Dimensions (L x B x D) (m) Water Depths	Material	Year Instd.	Approx Cost/m* A$ 1982	Remarks
Pontoon	Embarcadero Washington State, USA (ref 3.6)	61 x 4.27 x 1.15 Low water depth (Lwd) 6	RC deck and sides Polystyrene core. (Exposed on bottom and lower 0.25 m of sides)	1974	1750 to 2240	Max fetch breakwater = 1.8 km Chain connections between units caused deadeye failure soon after installation and were replaced by neoprene covered wire rope. No report since 1974 available
	Port Orchard Washington State USA (ref 3.6)	i) 457 x 3.66 x 0.9 ii) 137 x 2.44 x 0.9 Lwd 10.7	Lightweight concrete Styrofoam core.	1974	1120	No problems with b/water up to 1975. Early wind storm snapped 4 piles. Reported that boat wakes were not attenuated to any degree
Pontoon	North Haven, Adelaide, Sth Australia	90 x 2.4 x 0.76 Lwd 3.0	Concrete unit with reinforced deck, unreinforced sides and base, polystyrene core and timber walings. (Standard Monier Rocla through bolted Marina unit).	1981	495	Very short fetch wind waves (H = 0.6 m) inside a new Marina Basin. Preliminary indications are that it will provide satisfactory wave protection for a site having very limited exposure
Pontoon with Outrigger/ Wave Breaker on Seaward Side	Spit Bridge, Sydney, NSW (ref 7)	146 x 4.5 x 1.4 Lwd 15	RC concrete box section 1.03 x 1.37 m with steel outrigger 450 dia 2 m to seaward.	1968	1520	This system has operated successfully up to the present time (June 83) with no evidence of major structural problems. Corrosion of fittings and damage to rubber buffers have required regular attention. Some reinforcement corrosion becoming evident on seaward side, but it is not affecting serviceability
Tandem Pontoons and Suspended Keel	Birkenhead Point, Sydney NSW (personal communication)	170 x 5.0 x 1.4 Lwd 6	Steel fibre reinforeed concrete, skin polystyrene core. Galvanised steel framing supports 3.0 m long keel timbers between two rows of standard marina pontoons	1979	1560 (b/water) + 440 (mooring piles. Total 2000	Anchored by piles. Fetch length 3.0 km in NE/NW directions. B/water performance has been satisfactory, although structural damage of units and pile guides needs continual maintenance. Similar single row of pontoons (no keel) facing 1.2 km SW fetch suffered extensive damage within 12 months of installation and windward section is now unusable (Nov 82)

* Based on assumed inflation of 10% per annum and A$1 = US$1.1

Type	Location	Sect Dimensions (L x B x D) (m) Water Depths	Material	Year Instd.	Approx Cost/m* A$ 1982	Remarks
Open Rectangular Post-tensioned Box Sections	Tenakee, Alaska (ref 3.6)	91 x 6.4 x 1.5 Lwd 9	Lightweight reinforced concrete (0.1 m thick), Styrofoam Core	1972	3015	Performance generally satisfactory during first two years (last report 1974). Fetch 8 km
	Sitka, Alaksa (ref 3.6)	279 x 6.4 x 1.5 Lwd 3.0 - 10.0	As above	1973	2740 (cf $3040/m Est for 1980 by McLarens	Performance generally satisfactory in first year (last report 1974). Exposed to ocean swell and 4 km wind fetch
Catamaran type Pontoon b/water	Royal Sydney Yacht Squadron, Kirribilli, NSW (ref 25)	45 x 7.0 x 2.5 + 45 x 5.0 x 2.5	Pontoon units 11.5 x 1.2 x 2.5 m reinforced concrete box sections with polystyrene core. Units joined by steel cross-bracing. 7.0 m section for primary SE fetch, 5.0 m section for secondary NE fetch	1982	Contract price 5600 (using anchors)	Effective fetch 2.35 km (ESE). Hsig = 1.31 m, Ts = 3.95s Hmax = 2.4 m, Tmax = 4.0s for 30.9 m/s wind speed. Mooring forces: Fmax = 6164 N/m (wind wave only)
Catamaran Type Unit	Friday Harbour Washington State USA (ref 3.6)	276 x 7.6 x 1.6 Lwd 9-13.7	Polyolefin Pontoons (3 x 3.1 x 1.5 m) supporting timber decking	1972	2270 (cf $2300 (1980) by McLarens)	10% of plastic pontoons failed during first storm.
'A' Frame Unit	Lund Harbour, British Columbia, Canada (ref 3.6)	110 x 7.5 x 5.5 Lwd 15 Tidal range 6	Welded Steel Frame Creosoted timber wave barrier	1965	3200 cf 2500 (1980) by McLaren	No protective coating used on steelwork. Fetch length 11.5 km. No major failures during first 10 yers. Module connections have caused minor problems.
Floating Rubber Tyre Breakwater	Lyttleton Harbour New Zealand (ref 4)	45.5 x 15.7 x 1 tyre depth Lwd 2.4	Synthetic belt scrap tyres (5000 no)	1979	Estimate Aug 1981 550	4.5 m length was a test section kept in place for 17 months until Feb 1981. Estimate based on a 300 m prototype to be built. Total annual inspection and maintenance costs est at $11000. Assumes use of local dry dock for removing growth by high pressure hoses at two year intervals for each 6 x 50 m section
Rubber Tyre/ Floating Aluminium Tube B/water	Holmes Harbour, Washington State USA (Ref 3.6)	134 x 9 x 1 tyre depth (modules 12.2 m long) Lwd 6-24	Foam filled aluminium tubes threaded through scrap rubber car tyres	1979	390	Exposed to a 30 km indirect fetch (not in a straight line from b/water) Seaward end of b/water began to fail and longitudinal/transverse tube connection had failed within 12 months of installation

Type	Location	Sect Dimensions (L x B x D) (m) Water Depths	Material	Year Instd.	Approx Cost/m* A$ 1982	Remarks
Horizontal Plate + Outrigger on Inshore Side ('Seabreaker')	Developed in UK (No commercial installations documented) (ref 17)	Variable Prototype 40 x 7.3 x 2.0	Steel frame (protective coating not specified) + polyurethane foam buoyance blocks coated with GRP	Prototype built 1971	Not known	
Horizontal wave Barrier ('Harris' Floating B/water)	Ardyne Point Scotland (ref 15)	55 x 18 x 0.5	Timber	1975	Not known	Build to protect temporary access bridge to deep sea oil platform under construction 200 m offshore
	Rhu Marina on Clyde River Glasgow Scotland (ref 9.10.15)	420 x 10 x 0.5 (draft) x 1287 T displacement water depth 8 m	Prestressed concrete with polystyrene core	1977	1975 Dock & Harbour Authority est cost of conventional floating b/waters as $2920/m	Suffered major damage in 1979 during first major storm, due to long period waves. Unofficial reports indicate the marina is still operational
Tethered Float B/water System	Channel Islands California USA (proposed) (ref 18)	1.5 m dia hollow steel spacing, 25 rows of 50 spheres floating just below sea surface	Steel spheres tethered to concrete anchors in an array, chained to sea bed moorings	Was proposed to be done in 1975	Est at 10% of cost of conventional b/water	Theoretical performance is promising but no prototype performance data currently available (Sept 1982)

3.3 SURVEY OF MODEL AND FIELD TEST PROGRAMMES

The following **Table 3.2** summarizes model and field testing of the performance characteristics of selected types of floating breakwater.

TABLE 3.2 SURVEY OF MODEL AND FIELD TESTING

Type and Researcher	Reference	Details of Research	Type of Study	Conclusions Reached
Single Pontoon - Carver	(Ref 23) p.53	2D and 3D model tests for East Bay Marina, Olympia Harbour, Washington, USA. Sections: . 12' x 97' x 5' rect sect (3.66 x 29.3 x 1.5 m) . 12' x 96' x 5' + 3.5' (1.07 m) vert plate on lower leading edge . 16' x 96' x 5' rect sect Studied effects of: . Relative width (B/L) . Angle of wave attack . Linear vs concave vs convex layout. Wave heights 1.5' x 3.5 ' (0.45 m tol 0.5 m). Periods 2.5 s to 4.5 s.	1:10 Model Study	Wider unit superior only for B/L 0.3. Best results achieved was CT = 0.375 for W/L = 0.5. Also found that improvement in attenuation gained by altering angle of wave attack - dropped sharply as wave period increased, eg CT = 0.45 for 90°, 2.5 s, CT = 0.27 for 75°, 2.5 s, CT = 0.30 for 60°, 2.5 s, CT = 0.78 for 90°, 3.5 s, CT = 0.64 for 75°, 3.5 s, CT = 0.73 for 60°, 3.5 s. No significant advantages in using convex/concave over linear layout.
Single Pontoon - Araki & Chujo	(Ref 24)	Design and development programme for IHI 'L' and 'S' type pontoon units using catenary chain and damping wgt mooring systems. Investigated: . Relative width effects. Steel durability. Mooring forces. No details given of prototype dimensions, costs or anchor systems.	Tank tests and field testing of prototype	Claimed to have succeeded in developing: . A system with excellent performance, . A comprehensive design method. Data on mooring design. . An emergency self submerging system. No other data yet published to confirm these claims - 1 year service test for type 'S' pontoon in very exposed location described as satisfactory but no performance dta given. (Hmax = 11.4 m, est Hsig = 3 to 4 m). Earlier transmission results for T = 0.5 to T = 20.0 s were slightly better than tank test results, but wave steepness unknown

Type and Researcher	Reference	Details of Research	Type of Study	Conclusions Reached
Double Pontoon – Davidson (1971)	(Ref 23)	Oak Harbour Marina, Washington, USA. Max wave height = 2.0' (0.6 m) T_{max} = 3.5 s. Desired max height 0.5' (0.15 m). Therefore reqd CT = 0.25. Section tested: 42.5 x 10.0 x 7.2 ft (13.0 x 3.0 x 2.2 m). Wooden modules moored using anchors and later piles. Also tested anchor forces.	1:10 Scale Model	Section inadequate for 0.6 m waves for periods exceeding 2.5 s, for both anchored and piled moorings in water depth of d = 9.0 m. For 0.8 m waves and and period of 3.5 s, H_T = 0.45 and C_T = 0.56. For T = 3.55, H_I = 0.9 m, Peak Mooring Force = 44.7 kn/m
"Alaska" Type Double Pontoon – Christensen & Richey (1974)	(Ref 3, 6, 23)	1. Tenakee Springs B/water Alaska, USA. Studied incident and transmitted wave spectra for fully instrumented prototype. Also studied mooring forces – Incident wave range 0 – 0.3 m only	Field measurements	For T 3.0 s, $C_T < 0.5$. For T 2.0 s, $C_T < 0.3$. System comprises concrete elements post tensioned together to form open rectangular modules with overall beam of 6.4 m
		2. Sitka Floating B/water Similar to above	Field measurements	As above
Double Pontoon – Christensen & Richey (1974)	As Above	3. Friday Harbour – Similar study using polyolefin flotation tanks and wooden deck pontoon 25' wide (7.6 m)	Field measurements	C_T 0.3 for T $\leq$ 2.5 s C_T = 1.0 for T = 4.0s Moorings forces measured were significantly less than those predicted from theory or tank test results
Single pontoon with outrigger Foster and Stone (1968)	(Ref 7)	Spit Bridge Marina. Design/investigation of floating b/w attenuation characteristics and mooring forces.	Model Study	Determined that for HI = 0.67 m, T = 2.6 s. CT of 0.45 achieved using 9 m x 2.0 m x 1.35 m hollow concrete pontoon with 450 dia steel tubular outrigger 2.5 m to windward
Double Pontoon – HD Pite (Univ of NSW WRL – 1980)	(Ref 25)	Royal Sydney Yacht Squadron Marina Design/Investigation of attenuation characteristics & mooring forces	Model Study	For 7 m beam catamaran found that CT increase from 0.2 for T = 2.0 s to 0.5 for T = 4.0 s, to 0.8 for T = 5.0 s. Peak and average mooring forces evaluated

Type and Researcher	Reference	Details of Research	Type of Study	Conclusions Reached
Double Pontoon - Araki & Chujo	(Ref 24)	Design and development programme for IHI 'LH' type pontoon units using catenary chain mooring. A prototype was installed at Katura Bay Japan, with testing due for completion Sept 1979. Prototype dimensions and details not stated but est from photos as 8 m high x 6 m wide. Also investigated anchor forces and durability of components.	Tank tests and field testing of prototype	Tank tests (scale not stated) showed this to be almost as efficient as the 'L' type unit using damping weight moorings. Considered by researchers as suitable for deep water use. Field trial performance results not yet available (August 1982)
Tubulence Generator B/water - Harris & Sutherland	(Ref 5, 23)	Design and development programme for "Harris & Sutherland" floating b/water. Investigated wave attenuation and mooring forces.	Model and field testing of small prototype	Achieved 70% to 80% attenuation for L/B = 1.5. Found mooring forces proportional to solid area facing incident waves and approx 2% of dead-weight of structure
Sloping Float B/water - Raichlen (1978)	(Ref 23)	Test progrmame for use of Standard US Army & Navy barges as floating b/waters, having leeward end submerged or resting on seabed, windward end floating and tethered by a seaward anchor. Investigated attenuation characteristics and mooring (27 x 8.5 x 1.5 m) long prototype	Model tests	Test results indicate that for a water depth of less than 9.0 m, CT of less than 0.5 is achievable for wave periods up to 7.0 s using a 27 m long barge unit
Wave Maze Scrap Tyre Floating B/water - Kamel & Davidson (1968)	(Ref 23)	Investigated wave steepness and relative width effects on wave attenuation. Also determined mooring forces.	Model testing	For $0.01 < HI/L < 0.04$, CT is significantly higher than for $0.05 < HI/L < 0.07$. CT = 0.31 for B/L = 3.0. For B/L = 3.0, mooring force = 13% of horizontal force exerted on vertical wall by reflected waves (ref. Webber)

Type and Researcher	Reference	Details of Research	Type of Study	Conclusions Reached
"Goodyear Module" Scrap Tyre Floating B/water - Giles & Sorensen (1978/9) Giles & Erkert (1979)	(Ref 23)	Investigated wave transmission and mooring forces	Prototype scale tank tests	Determined a satisfactory design curve for CT vs B/L. (Note: Designers are warned that B/L = 1.40 is an upper limit and that 12 modules (x 2.13 = 25.6 m) is the maximum width applicable to this data (Prof Hales p 128).
"Goodyear Module" Scrap Tyre Floating B/water - Bushell & Penney	(Ref 4)	Investigated suitability for protection of small craft harbour at Lyttleton, New Zealand in 1979/80 using test section 45.5 m x 15.7 m	Prototype field tests	For max 900 mm, 3.0 s waves, observed CT was 0.33 to 0.44 using 15.7 m x 45.5 m test section
"Pole-Tyre Scrap Tyre Floating B/water - Harms & Bender (1978)	(Ref 4)	Compared performance of "Goodyear Module B/W' and Pole Tyre" B/W	Scale model tests	Found that a narrower breakwater is possible using pole-tyre system but that mooring forces are higher. (Note: Results giving CT vs B/L relationship assume wave steepness of HI/L = 0.04).

3.4 CONCLUSIONS

The following conclusions have been drawn from the material reviewed:

- There are certain key parameters, such as relative width (B/L), wave steepness (H_I/L) and relative depth (d/D) which may be used to define basic relationships between the large number of variables in the complex dynamic system associated with a floating breakwater. These are discussed in more detail in Section 2.2.

- Little data is available on mooring forces, forces developed at connections between modules, and the forces developed in modules by wave action. As these must be known for the worst conditions which the system must withstand, this appears to be a major shortcoming in the research to date. It is however, apparent that the stiffness of the mooring system is important to the overall performance of a particular installation.

- Data on wave attenuation performance under storm conditions is also scarce, and it is extremely difficult to determine the likely wave climate inside the breakwater under such conditions. There are severe limitations on the acceptable wave climate in a marina, due to the varying responses of different sizes and types of craft. As the consequences of excessive movement of vessels and snapped mooring lines can be extreme in crowded mooring areas, this is an area which requires significant research if floating breakwaters are to be used with confidence for the protection of moored vessels in exposed locations.

- Although it is possible to predict behaviour based on theoretical dynamic models, and the level of understanding is improving as the sophistication of analytical methods increases, the most productive approach appears to be an empirical one based on physical modelling, experience and prototype performance measurements.

- Data on maintenance costs is almost non-existent, which makes investment decision analysis (say comparing the long term costs of a floating breakwater with those of a rubble mound alternative) extremely unreliable.

- In North America, the most common and economical types of floating breakwater are the pontoon and rubber tyre type. Floating tyre breakwaters are the cheapest to install but they have a high failure rate and a life expectancy of less than 10 years.

- Floating tyre breakwaters, possibly as a result of low construction and investigation budgets, have suffered frequent mooring failures.

- Pontoon type floating breakwaters appear to be the most cost-effective design where life expectancy exceeds 10 years.

- The major problem experienced in North America with all types of floating breakwaters has been failure of module connections.

- The upper wave period limit of effectiveness for most floating breakwaters is between 3 and 5 seconds.

4. FLOATING BREAKWATERS IN AUSTRALIA

4.1 SPIT BRIDGE MARINA, NSW

The floating breakwater installed at the Spit Bridge Marina in 1968 is believed to have been the first in Australia. This followed severe damage to the inshore floating marina during strong westerly winds. The breakwater was developed by Sinclair Knight & Partners Pty Ltd with model testing being carried out by the University of NSW Water Research Laboratory **(Ref 7).** The breakwater has been designed to attenuate 0.67 m (2.6 s) waves to 0.30 m in 110 km.hr $^{-1}$ winds.

The site locations and fetch are shown in **Figure 4.1** and the general arrangement of the pontoons, tubular steel outriggers and anchor system is shown in **Figure 4.2.**

On 27 September 1981, Sydney experienced strong westerly winds with a peak gust velocity of 165 km.hr $^{-1}$. **Photographs 1, 2** and **3** were taken during this event and clearly demonstrate the degree of protection provided by the breakwater to the moored vessels.

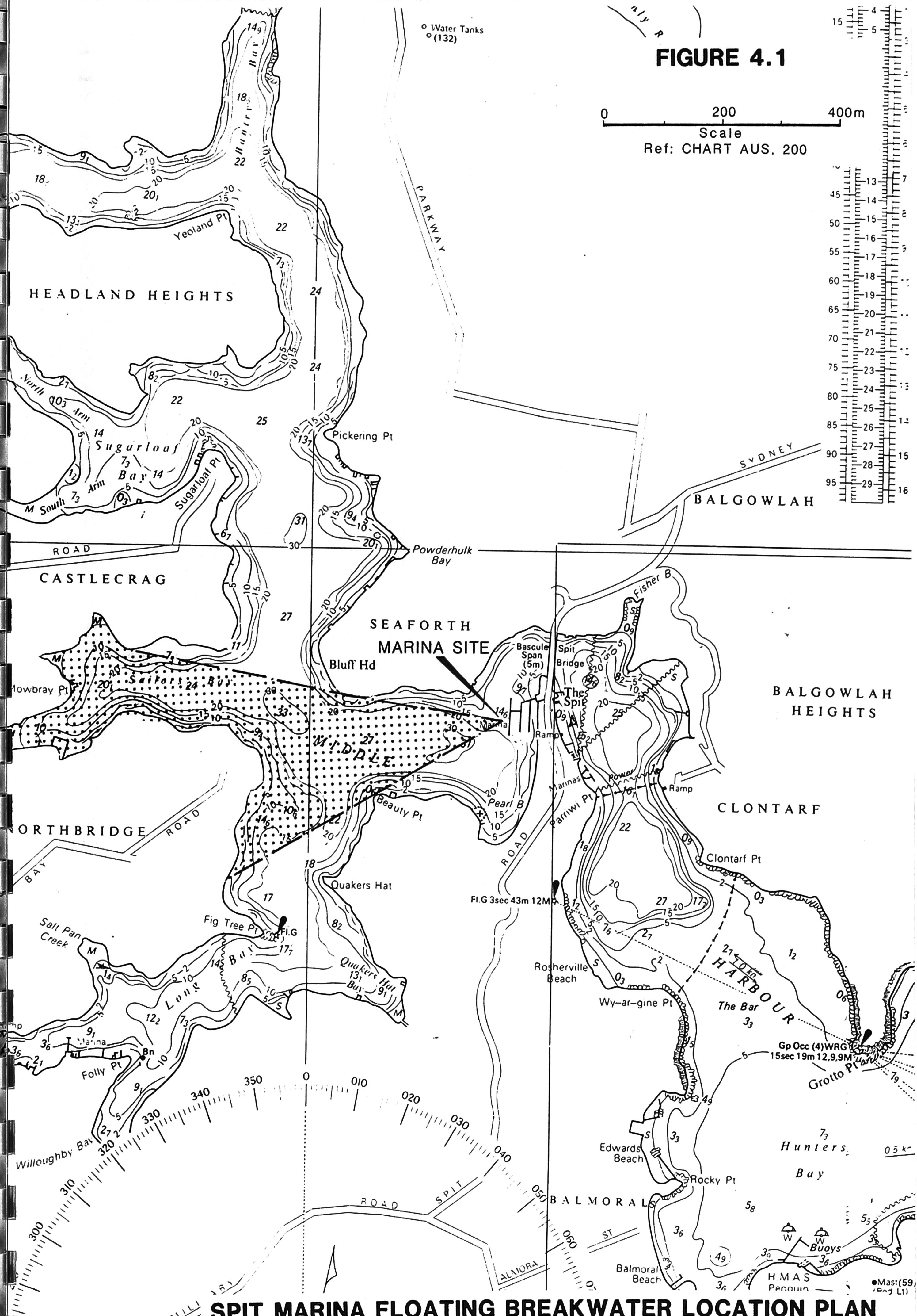
The breakwater units are hollow concrete pontoons with typical dimensions 9.35 x 2.03 x 1.37 m, with wall, deck and bottom thicknesses of 89 mm. Reinforcement of all surfaces, including two internal bulkheads, is a single layer of 335 wire fabric with additional 12 mm bars at concentrated load points. Typical dry weight is 14 tonnes.

450 mm diameter hollow steel outriggers approximately 9.0 m long are located approximately 2 m to seaward of the pontoons, supported on two 100 mm dia steel struts at each end of the pontoon. All steelwork is protected by coal tar epoxy and appears to be in reasonable condition.

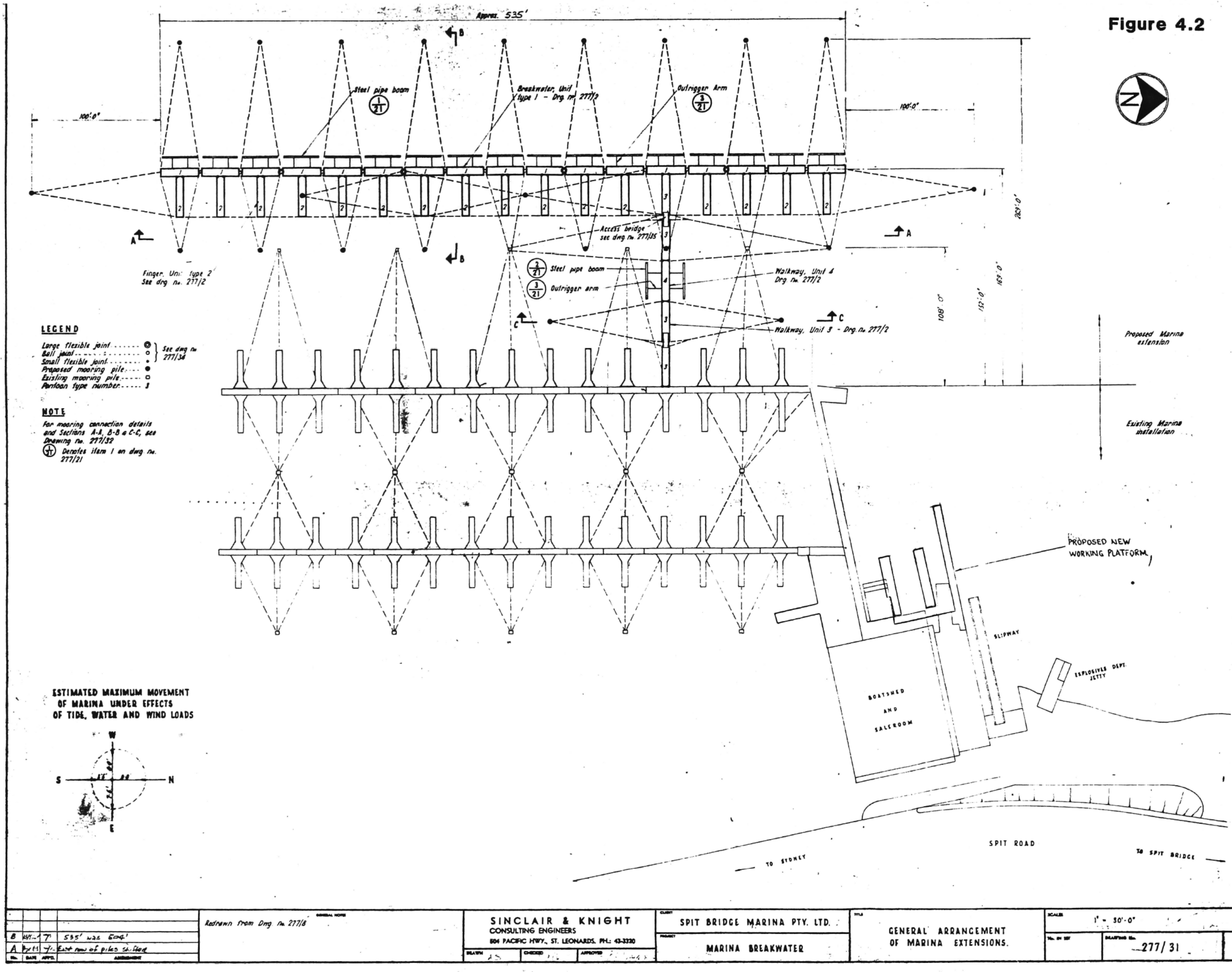
The mooring system is a combination of 19 mm chain, 12 mm polypropylene rope and drag weights, shackled to 300 mm stake piles jetted into the seabed. Still water tension in the system is approximately 2 kN.

Connections between pontoons consist of rubber "donuts" to resist compression loads, and chains to resist tension loads, thus providing a totally articulated system as seen in **Photograph 4.**

FIGURE 4.1



SPIT MARINA FLOATING BREAKWATER LOCATION PLAN



Approx. 535'
100'-0"
100'-0"
Steel pipe boom
Breakwater, Unit type 1 - Drg. no. 277/2
Outrigger Arm
A
B
Finger, Unit type 2
See drg. no. 277/2
Access bridge
see drg. no. 277/35
Steel pipe boom
Outrigger arm
Walkway, Unit 4
Drg. no. 277/2
Walkway, Unit 3 - Drg. no. 277/2
C
229'-0"
169'-0"
152'-0"
108'-0"
Proposed Marina extension
Existing Marina installation
LEGEND
Large flexible joint
Ball joint
Small flexible joint
See dwg no. 277/34
Proposed mooring pile
Existing mooring pile
Pontoon type number
NOTE
For mooring connection details and Sections A-A, B-B & C-C, see Drawing no. 277/32
Denotes item 1 on dwg no. 277/21
PROPOSED NEW WORKING PLATFORM
SLIPWAY
EXPLOSIVES DEPT. JETTY
BOATSHED AND SALEROOM
ESTIMATED MAXIMUM MOVEMENT OF MARINA UNDER EFFECTS OF TIDE, WATER AND WIND LOADS
W
S
N
E
SPIT ROAD
TO SYDNEY
TO SPIT BRIDGE
B 535' was 604'
A East row of piles shifted
Redrawn from Drg. no. 277/8
SINCLAIR & KNIGHT
CONSULTING ENGINEERS
504 PACIFIC HWY., ST. LEONARDS. PH.: 43-3330
SPIT BRIDGE MARINA PTY. LTD.
MARINA BREAKWATER
GENERAL ARRANGEMENT OF MARINA EXTENSIONS.
1" = 30'-0"
277/31

Figure 4.2

PHOTOGRAPH NO. 1

Spit Bridge Marina Floating Breakwater
27 September, 1981, During Severe Westerly Winds

(Peak gust 165 km hr^{-1}). Note degree of turbulence between tubular outrigger and windward face of pontoon.

PHOTOGRAPH NO. 2

Spit Bridge Marina, 27 September, 1981

Note low transmitted waves behind front row of boats moored to the floating breakwater.

PHOTOGRAPH NO. 3

Spit Bridge Marina, 27 September, 1981

Note the degree of protection provided by the floating breakwater under severe conditions.

PHOTOGRAPH NO. 4

Spit Bridge Marina
20 May, 1983

Note the greater degree of corrosion of steelwork on the seaward (RH) side, and the high degree of articulation permitted by the module connection system.

4.2 BIRKENHEAD POINT MARINA, DRUMMOYNE, NSW

The second floating breakwater believed to have been installed in Australia was completed in 1979 at Birkenhead Point Marina on the Parramatta River at Drummoyne. It is believed no model tests were carried out for this breakwater which consists of two rows of marina pontoons side by side with vertical timber slats suspended between them to form a keel. This breakwater (See **Photograph No 5)** is located at the northern end of the marina and faces the north-east and north-west fetches (see **Figure 4.3).** Reports indicate that it performs satisfactorily as a wave attenuator, although cracking in many units indicates some structural problems. The breakwater is anchored by stub lengths of concrete piles cast into large diameter concrete filled tubular steel piles cut off some distance below water level.

The severity of the wave climate at this site is indicated by the failure of the single row of pontoons facing the south-west fetch **(Photograph 6).** In addition to wind waves, the marina suffered major damage to finger/walkway connections in the first 12 months after installation due mainly to passing boat wakes, with the result that its capacity was reduced from 220 to 110 berths.

PHOTOGRAPH NO. 5

**Birkenhead Point Marina Floating Breakwater
May, 1983**

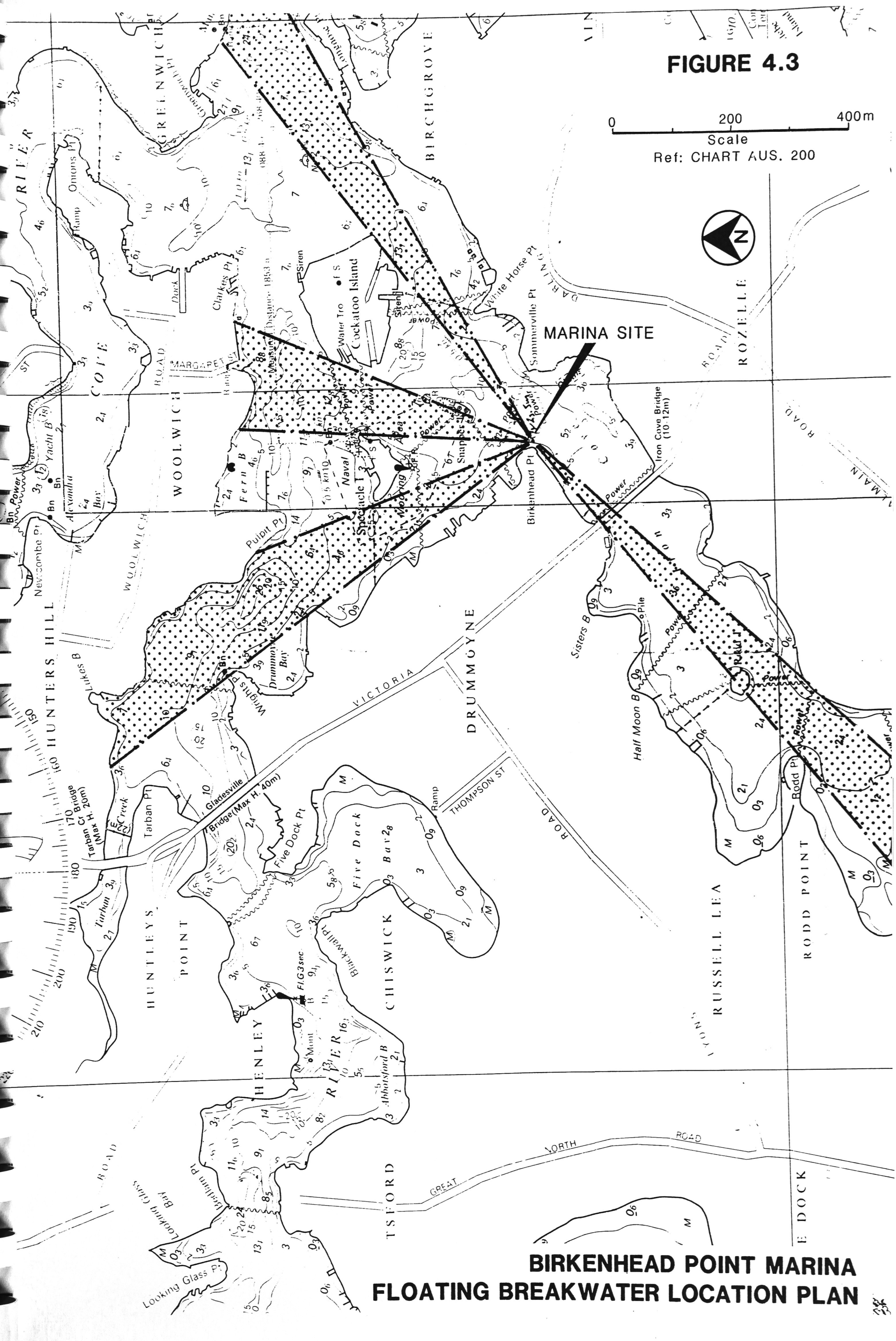
Note tops of timber keel slats suspended between two rows of pontoon units facing NE and NW fetches.

PHOTOGRAPH NO. 6

Birkenhead Point Marina, May, 1983

Note damage to single row of pontoons facing exposed SW fetch. This section of marina was decommissioned within 12 months of installation, but left in place to protect adjacent berths.

FIGURE 4.3



BIRKENHEAD POINT MARINA
FLOATING BREAKWATER LOCATION PLAN

4.3 NORTH-HAVEN MARINA, ADELAIDE, SA

The North Haven development in Adelaide is the most ambitious marina project undertaken in Australia to date, and details of the massive scale of development are shown in **Figure 4.4.**

In 1980, following completion of the purpose-built harbour, the first marina at North Haven was installed for the Crusing Yacht Club of South Australia.

As no other on-water development had taken place at that time, the marina was exposed to a series of winter North-Westerly gales which resulted in wind waves estimated at up to 0.75 m in height within the harbour. Following minor damage to several pontoons during installation, the contractor elected to construct a temporary floating breakwater. This consisted of a row of 2.4 m wide throughbolted marina units secured by anchors and this proved effective in preventing further damage. It is reported that the breakwater, seen in **Photograph No 7,** is to remain a permanent fixture within the harbour.

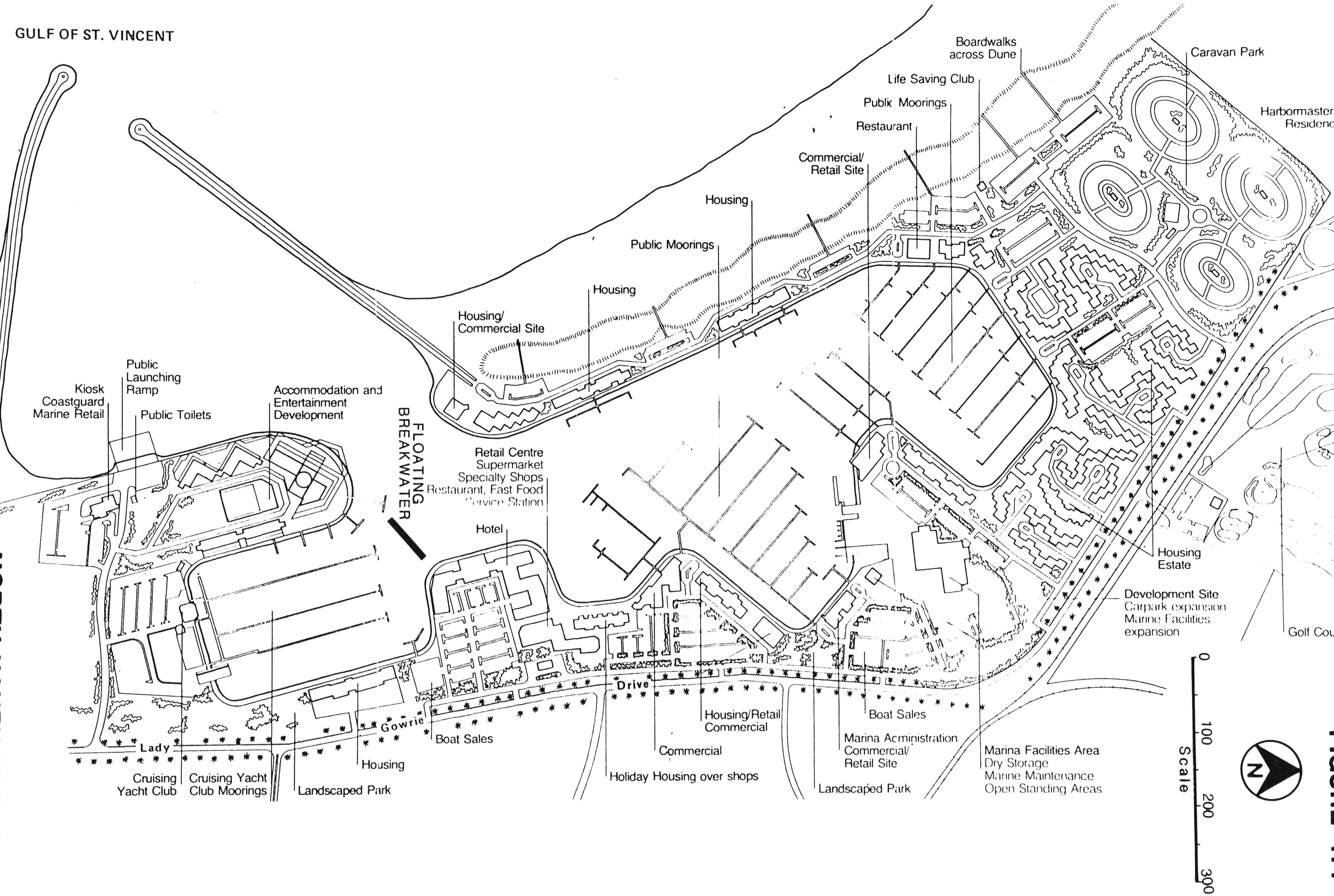


FIGURE 4.4

NORTH HAVEN HARBOUR

PHOTOGRAPH NO. 7

North Haven Marina, 1982. Looking South-East showing floating breakwater protecting new marina.

Note rubble revetment along water's edge which may have contributed to local wind wave activity by wave reflection.

4.4 ROYAL SYDNEY YACHT SQUADRON, LAUNCHING FACILITY KIRRIBILLI, NSW

Themost recently completed floating breakwater (as at January 1984) is that for the Royal Sydney Yacht Squadron, which occupies an exposed site at Kirribilli on the northern shores of Sydney Harbour **(see Figure 4.5).** For a sustained wind of 30.9 ms $^{-1}$ from the south-east, it is estimated **(Ref 25)** that the maximum wave height at the site could be up to 2.36 m. Using various methods, the significant wave height and period are estimated as follows:

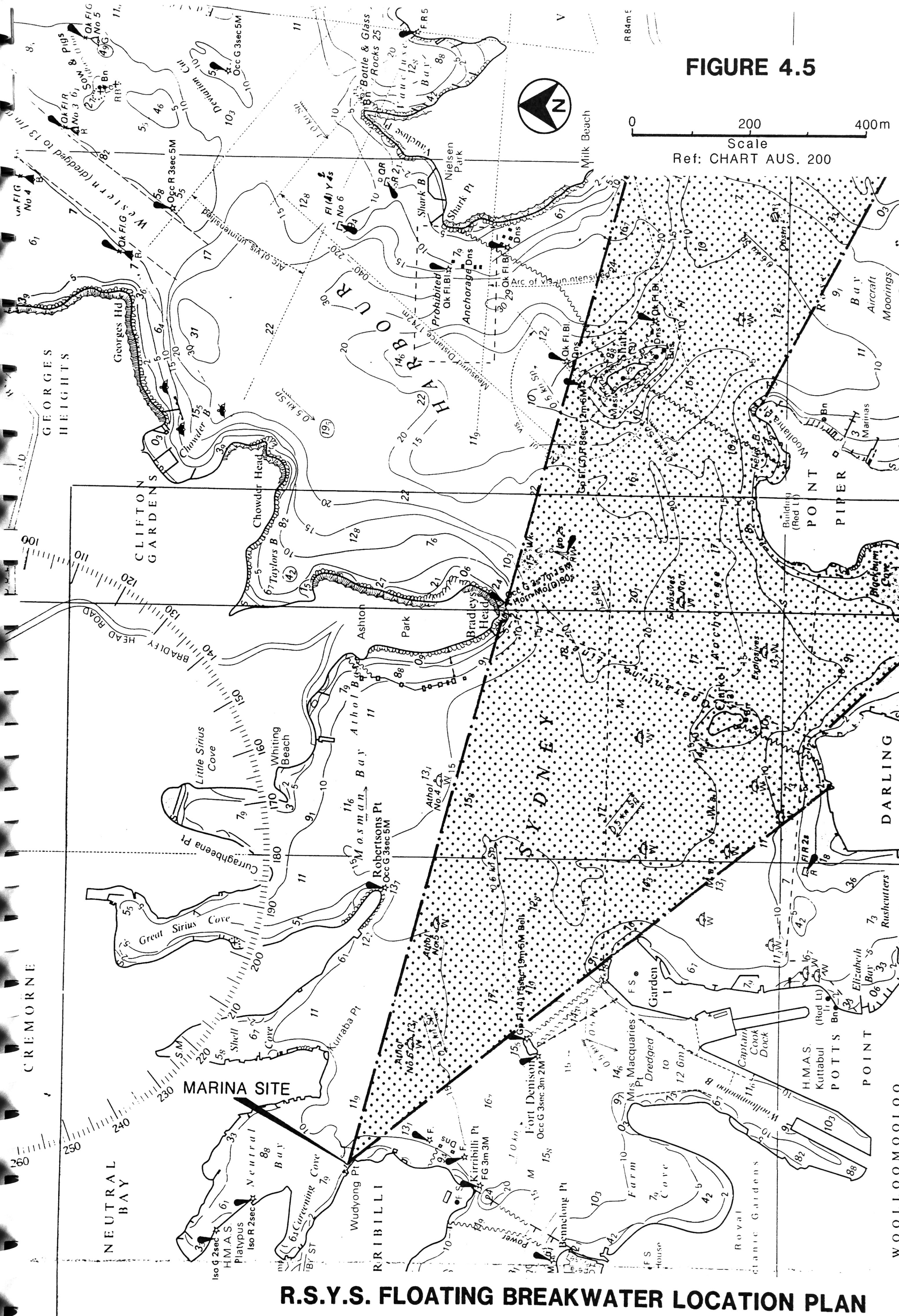
H_S = 0.40 m to 1.31 m (Average 0.82 m)
T_S = 2.1 s to 3.89 s (Average 2.9 s)

The 90 m long breakwater is laid out in an "L" shape with one leg 7 m wide facing the south-east, and the other 5 m wide facing the less exposed north-east fetch **(see Photograph 8).** It is a catamaran-type structure and consists of two rows of concrete encased polystyrene floats 1.2 m wide x 2.5 m deep, connected by transverse steel frames.

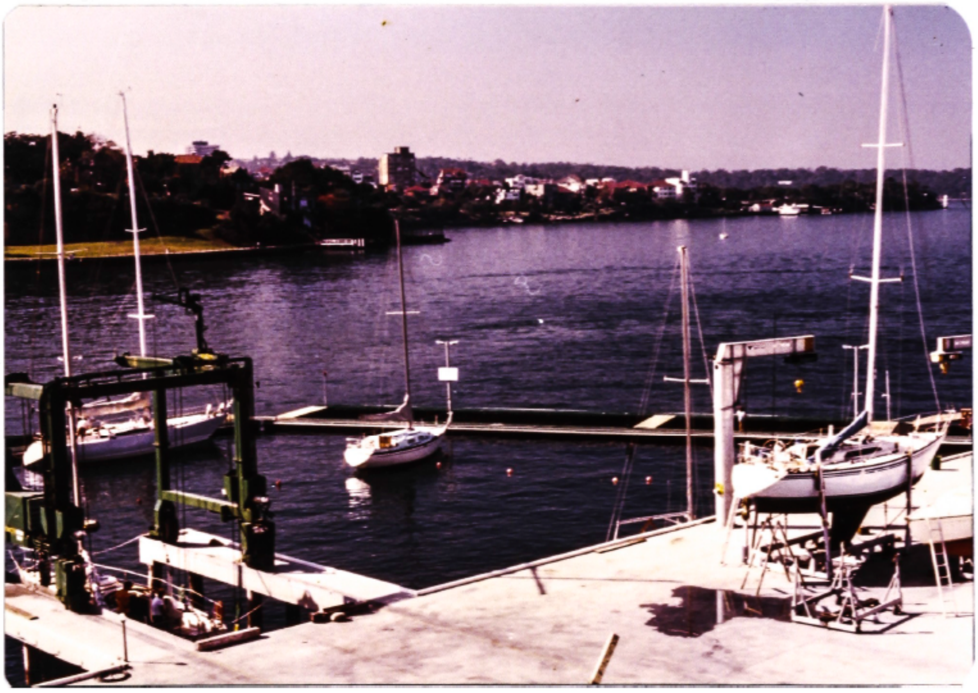
In each row, 11.5 m long units are connected by longitudinal steel channels fixed by bolts into cast-in threaded nylon inserts.

The breakwater is anchored using polypropylene mooring lines connected to concrete mooring blocks, and the lines carry drag plates to stiffen the mooring system. It was completed in late 1982, and is considered to be effective in protecting the Club's yacht launching facility containing two light cranes and a 30 T capacity mobile boat hoist. No permanent moorings are permitted, although the breakwater is also used for temporary berthing.

FIGURE 4.5



R.S.Y.S. FLOATING BREAKWATER LOCATION PLAN



PHOTOGRAPH NO. 8

Royal Sydney Yacht Squadron Floating Breakwater
13 May, 1983

Note 7m wide catamaran type floating breakwater facing SE fetch, protecting boat launching/recovery facility, and providing temporary berths for visitors.

4.5 PROPOSED FLOATING TYRE BREAKWATER AT GEELONG

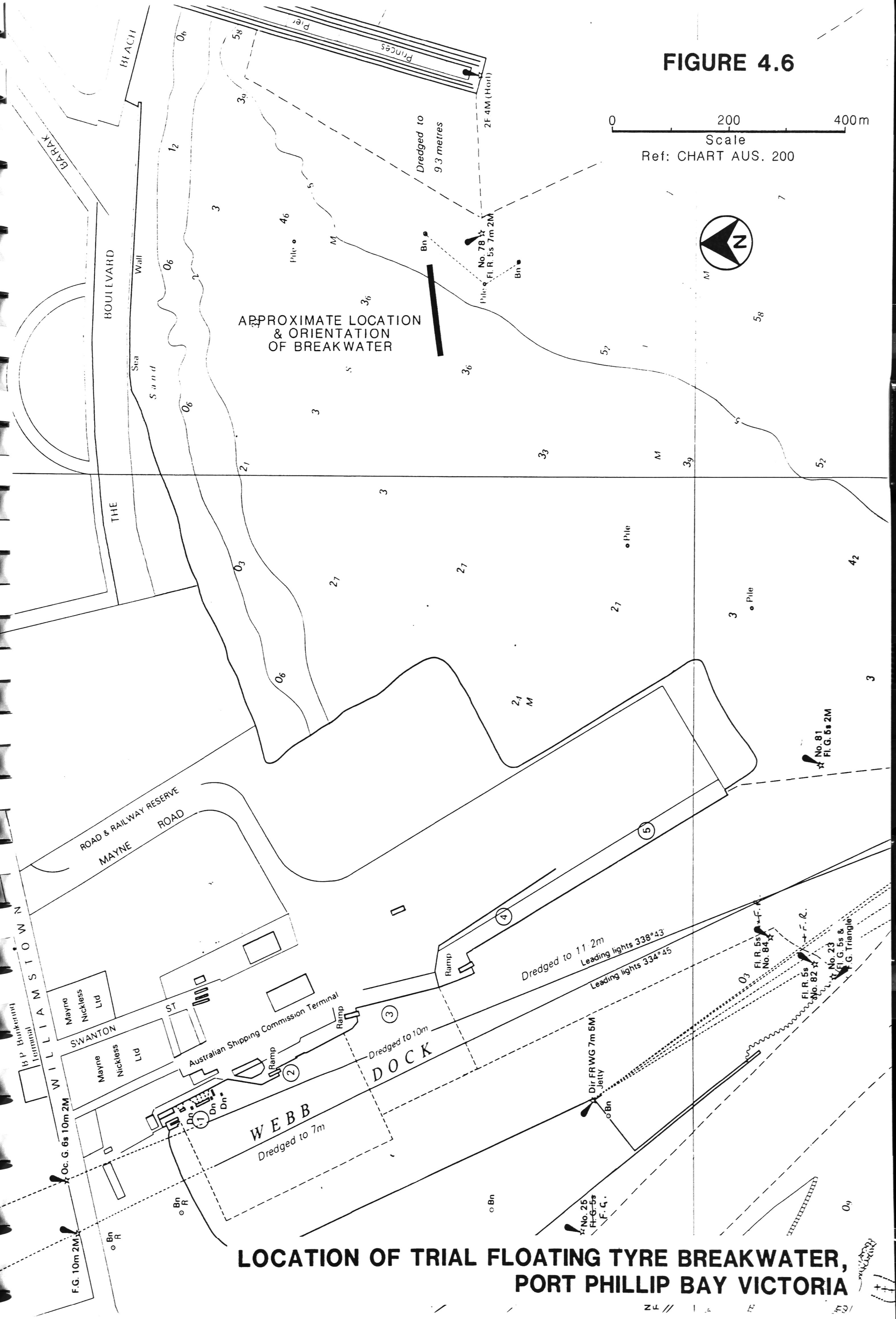
The Public Works Department of Victoria is assessing a floating tyre breakwater for use at Corio Bay, Geelong, South-West of Melbourne. Preliminary investigations indicated that the wave climate is more severe than would normally be considered appropriate for this type of breakwater, and a trial section 90 x 40 m has been constructed using a modified "Goodyear Module" configuration.

The trial section, constructed in 1982, was towed to a relatively exposed site at the Northern end of Port Phillip Bay (See **Figure 4.6)** and Waverider buoys were installed in October 1983 to measure incident and transmitted wave heights for ten minutes every two hours over a six month period.

During an inspection of the trial section by the author in January 1984, it was reported that the modified layout, consisting of wooden poles threaded through tyres along the edges of the breakwater, was more effective in transferring mooring loads to the modules than the conventional configuration. Previously, modules had been failing regularly at the mooring connection points. It was also reported that failure of stainless steel bolts securing the tyre strapping has made regular detailed inspection essential. Such inspections are slow as all connections are underwater, and therefore subject to fouling by marine growth, making the job of identifying a failure and labelling it for later repair extremely difficult.

With such a wide section of breakwater, it was also apparent that anything other than straight sections would be very difficult to install and maintain because of the problems of connecting adjacent sections at the corners.

FIGURE 4.6



LOCATION OF TRIAL FLOATING TYRE BREAKWATER, PORT PHILLIP BAY VICTORIA

4.6 POTENTIAL APPLICATIONS OF FLOATING BREAKWATERS IN AUSTRALIA

As stated in **Section 1,** floating breakwaters may permit the development of relatively exposed marina sites for recreational craft, particularly where more conventional forms of wave protection such as fixed breakwaters are too expensive or are unacceptable for other reasons such as unsuitable environmental impact, excessive depths or poor bottom conditions.

However, there are also other potential uses for this type of wave protection in Australia, including the following:-

- **Oyster farming.** Johnson **(Ref 28)** estimates that, in the Georges River area, some oyster growers face an annual average damage bill of up to $5 000 due to excessive wave conditions. Wind waves and passing boat wakes cause major problems for growers by moving oysters around in their trays in the inter-tidal zone. If not raked regularly, the oysters are washed out and die on the seabed. In some areas, movement of bed materials by boat wakes has buried oyster leases alongside river channels.

  Many farmers have therefore erected crude fixed breakwaters, which are unsightly at low tide and ineffective at very high tides. An alternative could be the use of cheap floating breakwaters to dissipate the wave energy and reduce the high labour costs involved in raking the oysters, and repairing the wave damage to the racks.

- **Commercial fishing ports.** Fishing in most areas off the Australian coast is largely seasonal as the major fleets move from port to port following the fish migratory patterns. Development of every port used is therefore not justified, however improvement of selected ports using floating pontoon breakwaters which can also provide additional short term berths and survive the worst storms may be worthwhile, particularly in areas which are already partially sheltered.

- **Defence applications.** Because of Australia's extensive coastline, many parts of which are inaccessible except by sea, the establishment of beach landing points suitable for military cargo handling operations can be a major obstacle. The development of a modular, robust and compact floating breakwater suitable for naval/military use would greatly increase the safety and flexibility of ship to shore operations. Research into this aspect of floating breakwaters has been under way in the USA for many years.

4.7 CONCLUSIONS

All five breakwaters discussed have been or will be installed to protect marina facilities. This confirms the opinions expressed in **Section 1** in that floating breakwaters have already made a major contribution to the development of recreational boating facilities in Australia.

It is concluded that this will continue to be the case, as it is apparent that the pace of such development in Australia is increasing, with the first installation in 1968, others in 1979 and 1980 followed by one in 1982 and the Port Phillip field trials in 1982/83.

It is also concluded that the pontoon type is preferred to the floating tyre type in Australia, with 80% being single/multiple pontoons.

It is further concluded that there are opportunities for the development of floating breakwaters for other purposes including oyster farm wave protection, commercial fishing port development and use by Australia's defence forces.

5. FLOATING BREAKWATER FIELD TESTING PROGRAMME

5.1 INTRODUCTION

Although the floating breakwater installations at the Spit Marina and Birkenhead Point are generally considered to perform satisfactorily, it was apparent that no detailed wave or breakwater performance measurements had been carried out at either site prior to 1981. This project was therefore conceived with the idea of developing inexpensive, portable equipment for simultaneous measurement of incident and transmitted waves to obtain reliable performance data.

5.2 EQUIPMENT

Previous efforts by others to assess floating breakwater performance typically involved expensive equipment such as Wave Rider buoys. This level of expenditure was considered unjustified in view of the moderate wave climate (typically less than 1.0 m) anticipated during testing.

Because of the relatively deep water at the sites, (Spit Marina water depth is approximately 15 m at low water at the breakwater), it was decided to try a floating system incorporating a capacitance wire supported on a spar buoy which could be submerged using a "tension-leg" anchor system, as shown in **Figure 5.1** and **Photograph 9.**

The equipment was designed and fabricated by UNSW Water Research Laboratory staff at Manly Vale before static calibration testing was carried out in a variable level tank.

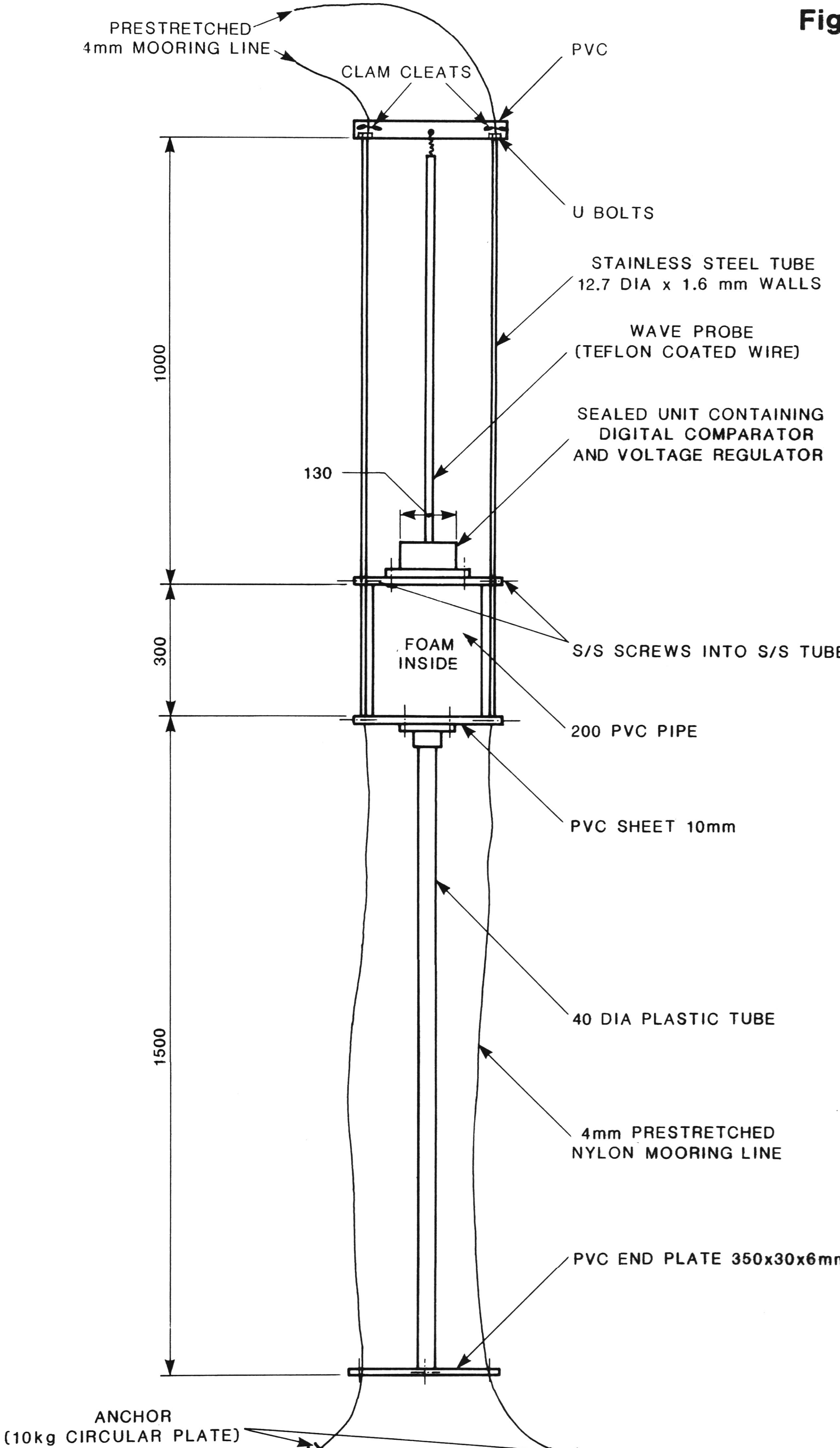
One major problem encountered was the sealing of electronic circuitry carried on the spar buoy. Initially, melted beeswax was used, but this proved ineffective. Eventually, a proprietary brand of re-entrant encapsulant resin was used successfully.

The total cost of materials for each of the spar buoys was approximately $200.00.

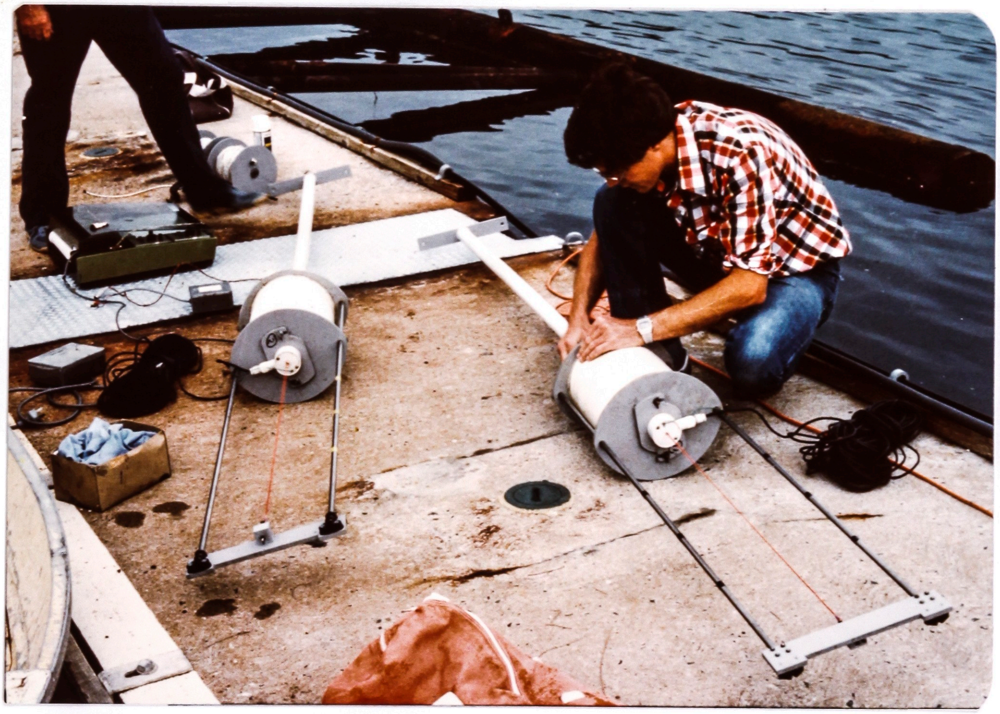
Following static calibration, the two spar buoys were placed side by side in the Laboratory's wave flume to enable dynamic calibration to be carried out. For this test, the buoys were connected to the dual channel Northrup and Johnson chart recorder, and waves of varying height and period were run through the flume, and charts produced for each spar buoy. At the same time, visual observations of crest and trough height for particular waves (usually the second and third in a train to enable easy identification on the chart) were made, and these were compared with the chart measurements to obtain calibration factors for each spar buoy.

A field trial was then carried out on Manly Dam to test the effectiveness of the anchor system and to identify any problems with the equipment. The anchors used consisted of 20 kg circular steel plates with a welded lug for fastening the 4 mm prestretched mooring line. When these anchors were tested, their weight, combined with the suction of the mud bed, made recovery extremely difficult. Their weight was then reduced to 10 kg and they have since proved effective and easier to handle.

Figure 5.1



SPARBUOY



PHOTOGRAPH NO. 9

Assembly of Spar Buoys, Spit Bridge Marina
20 May, 1983

Note markings on stainless steel tubes at 200 mm spacing for on-site calibration checks.

During this test, the need to lay out mooring lines carefully to avoid underwater tangles also became obvious. Small mooring line winch drums, suitable for fixing into rowlock holes were later fabricated and proved effective in preventing line tangles.

5.3 FIELD TESTS - ROYAL SYDNEY YACHT SQUADRON BREAKWATER

Field tests commenced on 20 January 1983 at Royal Sydney Yacht Squadron's recently completed floating breakwater. Wind was Force 4 to 5 from the south-east and the conditions were ideal for typical wind chop measurements, with the occasional passing ferry providing some higher, longer period waves. However, the high degree of reflection of wind waves by the breakwater produced a very confused wave pattern at the incident wave buoy, and it was not possible to identify the pure incident wave height. Due to interference from waves reflected off the seawall underneath the suspended deck, a confused wave pattern was also observed at the transmitted wave buoy, and it was concluded that it would be better to carry out measurements on calm days using readily identifiable waves from passing vessels to obtain incident and transmitted wave height measurements.

On 13 May 1983, wind conditions at the Royal Sydney Yacht Squadron site were initially calm, with a light western wind (Force 1 to 2) developing during the morning. Passing ferries and commercial harbour traffic provided easily identifiable wave trains, which enabled the chart to be marked as the leading wave passed the incident wave buoy, (see **Photograph No 10)** reflected off the breakwater and passed the incident wave buoy again, and as the transmitted wave passed the inshore wave buoy. A typical event is shown in **Figure 5.2.**

The field results **(Table 5.1 & Figure 5.3)** indicate that the model test results were conservative, as the highest C_T value measured in the period range 1.7 to 3.6 seconds was 0.28, compared with 0.50 for the model.

(Note: A Tucker-Draper analysis **(Table 5.2)** carried out on the wind wave record for the first day of testing indicated a C_T of 0.34 for a 5 minute wave record having a significant period of 1.8 seconds. Although this indicates reasonable breakwater performance, it must be remembered that both incident and transmitted wave heights contain reflected wave components. As these were not included in the model test results, it is therefore not appropriate to compare this result with the model test results.)

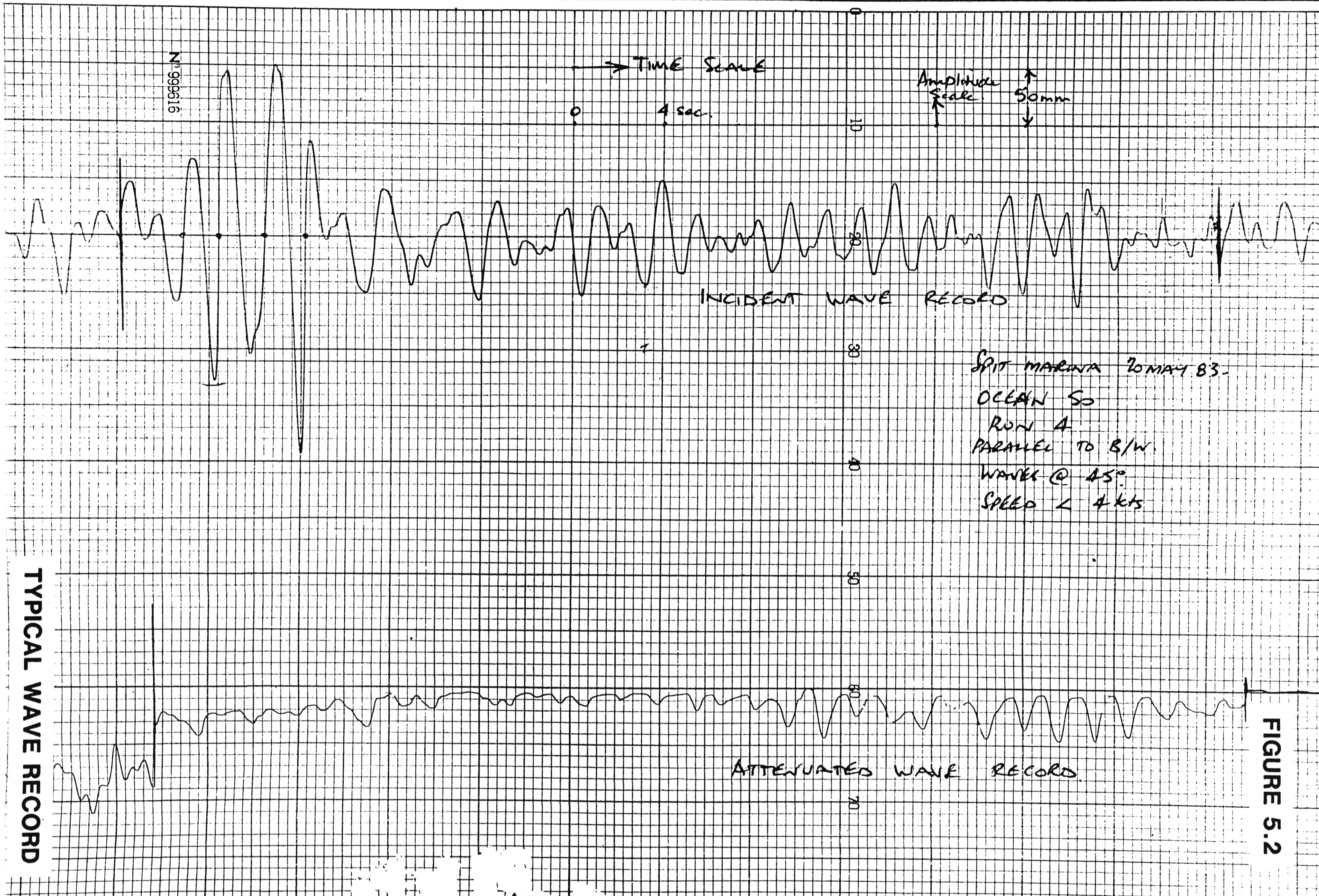
The lack of consistent C_R and C_L values obtained highlights the difficulties in identifying the reflected wave height on the wave record, although the four results for C_R in the 0.47 to 0.58 range are considered indicative of this breakwater and are high compared with the Spit Marina results.

It is of interest to note that there was negligible difference in breakwater performance when the approach angle changed from 90° to 45° to the face of the breakwater. The range of wave periods achieved during the site measurements for groups of waves was 1.7 to 3.6 s. As these were produced by a typical cross-section of commercial vessels navigating on Sydney Harbour, it appears that 4.0 seconds is a reasonable upper limit for design purposes under such conditions.

PHOTOGRAPH NO. 10

Royal Sydney Yacht Squadron, 13 May, 1983

Installation of incident wave spar buoy, following tensioning of the anchor liner using clam cleats at the top of the stainless steel to submerge the buoy to a fixed level. The spar buoy's positive buoyancy positions it between the seaward and inshore anchors and holds it in position.

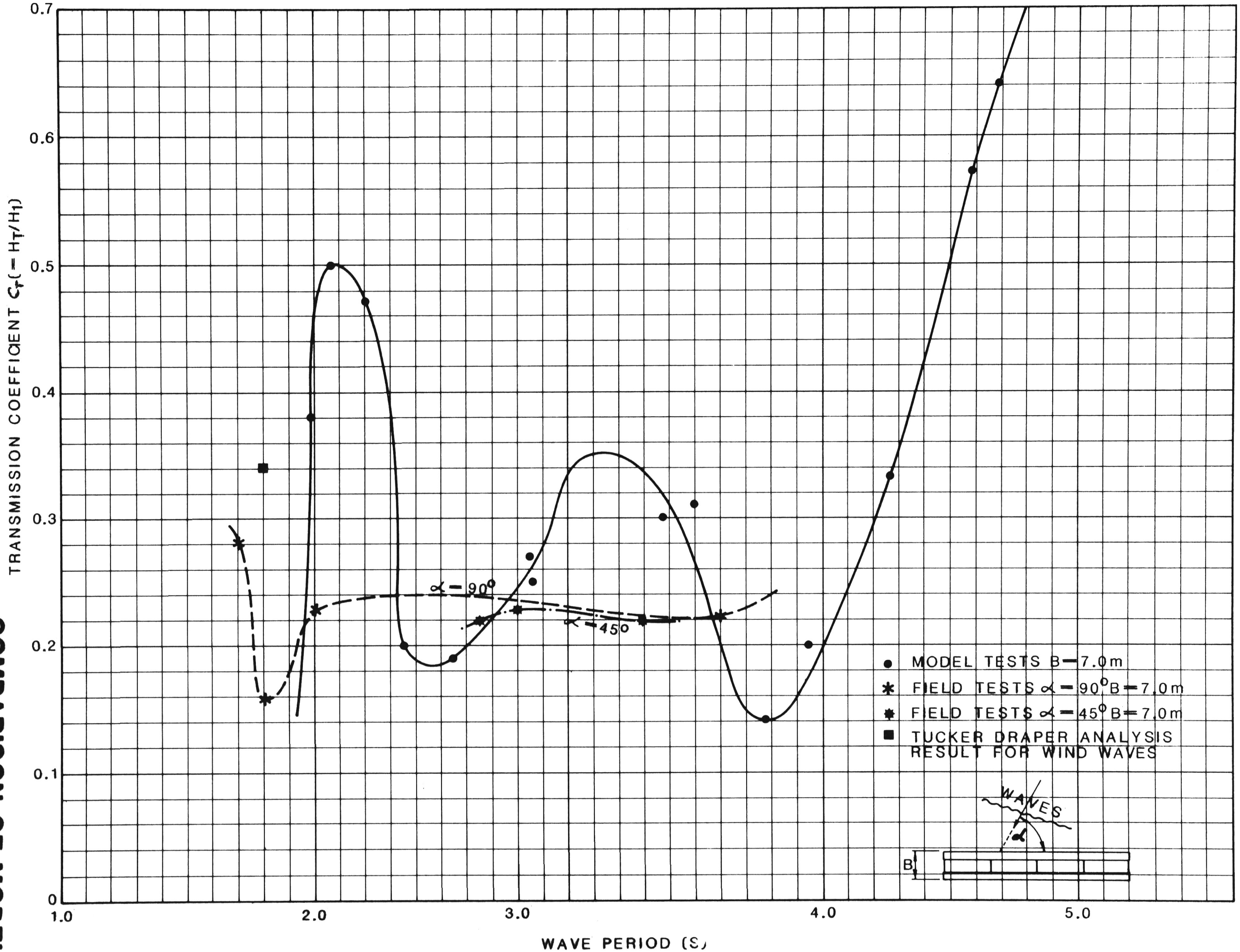


TYPICAL WAVE RECORD

FIGURE 5.2

SUMMARY OF RESULTS - RSYS 13 MAY 1983 **TABLE 5.1**

TIME	EVENT	APPROACH ANGLE TO 7 m b/w	PERIOD T	H_I (mm)	H_T (mm)	C_T (Observed)	(x 0.864) (Corrected)	REMARKS	H_R	C_R	C_L
0938	Inbound Ferry	90°	1.7	224	72	0.32	0.28	No 1 Buoy (Offshore)	208	0.93	0.06
0947	Long Swell	45°	2.8	196	52	0.265	0.23	$\frac{(H_{obs})}{(H_{rec})_1}$ = 1.108 from flume calibration	N/A	-	-
0948	Long Swell	45°	2.65	188	48	0.255	0.22	No 2 Buoy (Inshore) $\frac{(H_{obs})}{(H_{rec})}$ = 0.957	N/A	-	-
0950	Outbound Ferry	20°	2.6	280	96	0.34	0.29	$C_T = \frac{H_T}{H_I} = \frac{(H_T)rec \times 0.957}{(H_I)rec \times 1.108}$	152	0.54	0.62
1002	Inbound Ferry	90°	1.8	256	48	0.19	0.16		148	0.58	0.64
1013	Outbound Ferry	45°	3.3	380	100	0.26	0.22	$= \frac{(H_T)}{(H_I)} \times \underline{0.864}$	180	0.47	0.73
1025	Inbound Police Launch	90°	3.6	400	80	0.25	0.22		190	0.48	0.73
1033	Inbound Ferry	90°	2.0	352	96	0.27	0.23		N/A		
1035	Outbound Navy Launch	0°	2.6	520	152	0.29	0.25		N/A		



COMPARISON OF MODEL
vs PROTOTYPE PERFORMANCE
FOR ROYAL SYDNEY YACHT SQUADRON
FLOATING BREAKWATER

FIGURE 5.3

TABLE 5.2

TUCKER DRAPER ANALYSIS

ROYAL SYDNEY YACHT SQUADRON 20 JAN 1983
LENGTH OF RECORD = 5 MIN

WAVE RECORDER	No 1	No 2
MAX CREST HT ABOVE MWL (A) (mm)	268	108
MAX TROUGH DEPTH BELOW MWL (B) (mm)	224	88
NO OF CRESTS (N_C)	233	-
WAVE HEIGHT (C = A + B) (mm)	492	196
NO OF ZERO CROSSINGS (N_Z)	168	182
TUCKER-DRAPER FACTOR	0.59	0.59
SIGNIF WAVE HEIGHT (H_S) (mm)	290	115
BUOY CALIBRATION FACTOR	1.10	0.95
CORRECTED H_S VALUE (mm)	319	109
ZERO CROSSING PERIOD T_Z	$\frac{300}{N_Z} = 1.795$	-

$\therefore$ TRANSMISSION COEFFICIENT C_T = 0.34

5.4 FIELD TESTS - SPIT BRIDGE MARINA FLOATING BREAKWATER

On 20 May 1983, calm conditions were experienced at the Spit Marina floating breakwater. However, as this site has few commercial vessels passing during the week, it was necessary to enlist the assistance of a 15 m cruiser to obtain suitable waves (see **Photographs 11** and **12).** Other large vessels also passed intermittently and a reasonable range of results was recorded.

Figure 5.4 shows the model test results for a 1.8 m wide pontoon with various combinations of outrigger baffle and anchor line weights. These indicate that for waves parallel to the breakwater having periods of up to 3.1 s, a C_T value of less than 0.45 should be achieved. As the design criteria was C_T=0.45 for 2.6 s waves, the design was considered satisfactory.

However, construction considerations resulted in the use of tubular steel outriggers with a vertical drag plate, and a reduction of the overall width to 4.5 m.

The field results **(Table 5.3 & Figure 5.4)** indicate that the prototype's wave attenuation performance is satisfactory with a maximum recorded C_T value of 0.38.

The C_R and C_T values, when compared with the RSYS results, confirm that the outrigger at the Spit results in a lower degree of reflection and a higher degree of turbulence than the RSYS breakwater. This is borne out by observations.

It is noted that the wave direction appears to have a significant effect on wave transmission, with waves at 45° being attenuated much more than those parallel to the breakwater. It is considered likely that the extra attenuation is due to the presence of finger pontoons and moored vessels which provide additional reflective surfaces for wave energy dissipation.

PHOTOGRAPH NO. 11

Spit Bridge Marina Floating Breakwater
20 May, 1983

Ocean 50 motor cruiser assisting in the production of waves at 45° to breakwater.

PHOTOGRAPH NO. 12

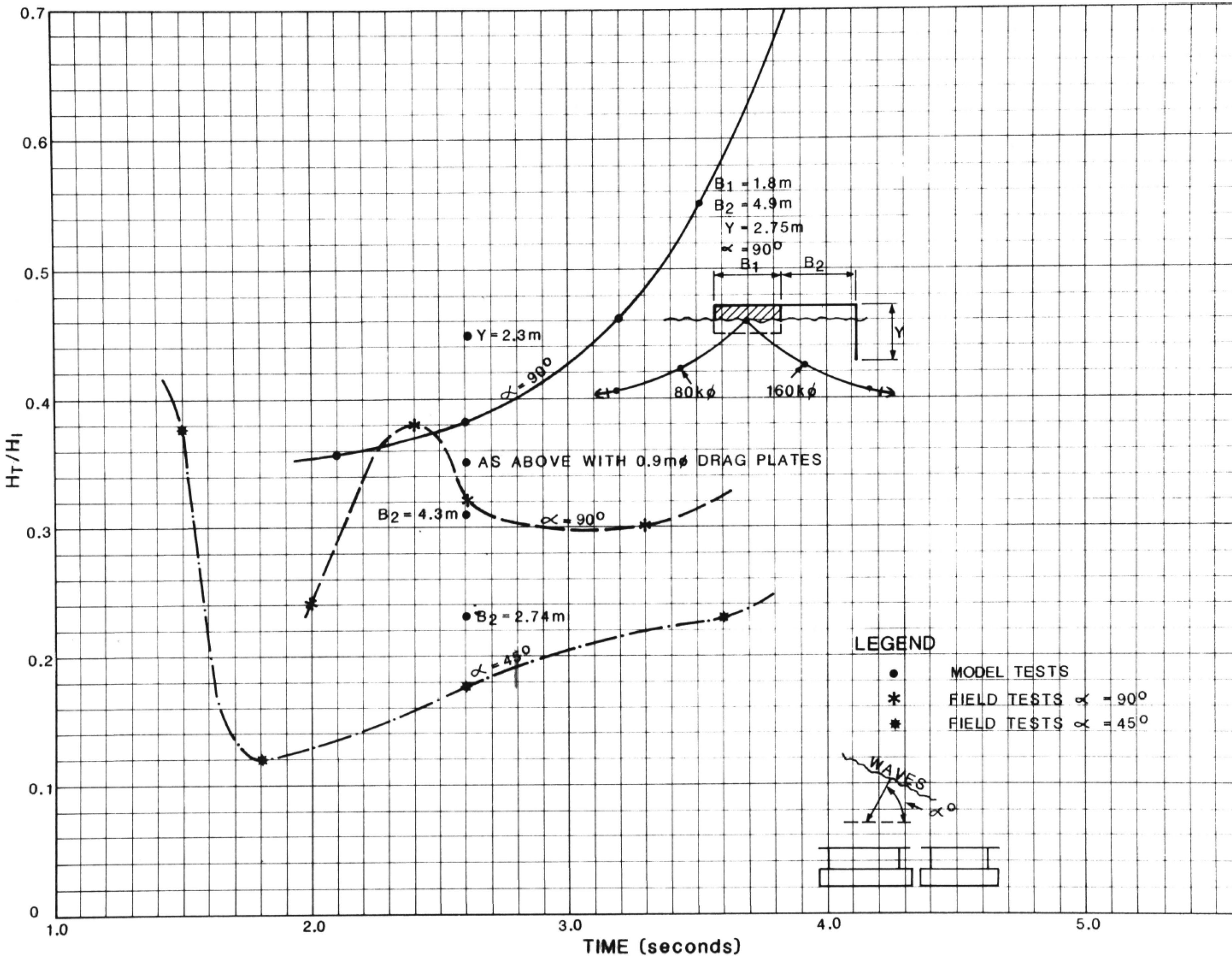
Spit Bridge Marina Floating Breakwater
20 May, 1983

45° waves striking the breakwater following a run by the Ocean 50 motor cruiser.

SUMMARY OF RESULTS - SPIT BRIDGE 20.5.83

TABLE 5.3

TIME	EVENT	ANGLE OF APPROACH	T	H_I	H_T	$C_T = H_T/H_I$ OBSERVED	C_T CORRECTED	REMARKS	H_R	C_R	C_L
1110	Random waves from west	90°	2.0	48	10 mm	0.21	0.24	No 2 buoy offshore Correction $\frac{Hobs}{H_{rec}}$ = 0.96 for incident wave	-	-	-
1112	Small Runabout	45°	1.5	60	20	0.33	0.30	No 1 Buoy Inshore $\frac{Hobs}{H_{rec}}$ = 1.108	40	0.67	0.46
1115	Ocean 50 Run 1	45°	3.6	800	160	0.20	0.23		212	0.27	0.87
1120	Ocean 50 Run 2	45°	2.6	320	50	0.16	0.18	∴ C_T Corrn factor = $\frac{0.957}{1.108}$ = $\underline{0.864}$	96	0.30	0.88
1130	Ocean 50 Run 3 (6 kts)	90°	2.6	320	88	0.275	0.32	eg $\frac{\text{1120 hrs}}{\text{T = 2.6S}}$ (Hi)rec = 320 mm	72	0.225	0.94
1132	Ocean 50 Run 4 (4 kts)	45°	1.8	280	28	0.10	0.12	∴ (Hi)actual (No 2) = 0.957 x 320 = $\underline{307.2\text{ mm}}$ HT = 50 mm	56	0.200	0.95
1135	Ocean 50 Run 5	90°	3.3	456	120	0.26	0.30	(HT) actual = 1.108 x 50 = 55.4 mm	56	0.12	0.89
1140	Capt Cook Ferry	90°	2.4	192	64	0.33	0.38	$C_T = \frac{55.4}{307.2} = 0.18$	76	0.40	0.73



COMPARISON OF MODEL vs PROTOTYPE PERFORMANCE SPIT BRIDGE MARINA FLOATING BREAKWATER

Figure 5.4

5.5 CONCLUSIONS

The following conclusions have been reached:

- The prototype performances of both the Royal Sydney Yacht Squadron floating breakwater and the Spit Bridge Marina floating breakwater appear to confirm the results of the model testing carried out for each, and indicate that the model test results may be slightly conservative.

- The range of wave periods and wave heights experienced in the field on a particular day is limited by weather conditions and the types and speeds of craft in the vicinity. Therefore confirmation of the performance over the range of wave heights and periods tested in the laboratory would be a lengthy process and would depend on favourable weather conditions, particularly for the design storm case.

- The relevance and importance of properly conducted model testing for floating breakwaters is therefore confirmed, since a wide range of conditions can be tested in the laboratory quickly and economically for different breakwater types and configurations.

- Although no field measurement of mooring forces was carried out in this study, no mooring failures have been reported at either site and it is therefore reasonable to assume that the designs for both installations are adequate for the conditions encountered to date. However, the importance of regular underwater inspections cannot be over emphasised since any failures in these components could result in very serious damage.

- Determination of reflection and energy loss coefficients using reflected wave height measurements can be used to identify the mode(s) of energy dissipation for a particular breakwater.

- The equipment developed during this investigation was inexpensive to fabricate, worked satisfactorily and could be used with confidence by appropriately qualified personnel for the following tasks in areas having a low to moderate wave climate:

  - floating or fixed breakwater performance measurements
  - confirmation of wave height and period predictions in a particular location under given wind conditions
  - short duration wave data collection (anchor tension must be adjusted at not more than hourly intervals due to tidal variation in water levels).

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