COASTAL DEFENCE STRUCTURES IN NSW, AUSTRALIA

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ABSTRACT - 100 Years of Experience In a High Energy Wave Climate This paper uses the examples of Coffs Harbour breakwaters and the training wall head on the Richmond River at Ballina to outline some of the developments in coastal engineering design and construction over the last century. Most of the 63 major structures along the New South Wales (NSW) coastline are similar. Recent developments in data collection and analysis as well as the evolution of physical modelling techniques used for coastal structure design at the NSW Department of Public Works and Services' Manly Hydraulics Laboratory (MHL) are discussed using these two projects. The use of computational models in conjunction with physical models and the introduction of new variables such as wave obliquity and groupiness into breakwater design are discussed. The need to evaluate the performance of artificial units such as Accropodes, concrete cubes and Hanbars for primary armour at breakwater heads due to restrictions on the availability of quarry armour and available construction techniques is briefly addressed.

1. Introduction

The first European settlement of NSW at Sydney in 1788 was by sea. The first public works in the new colony included landing steps and wharves constructed in Sydney Cove. The involvement of Public Works in coastal engineering extended along the coastline with the growth of the new colony. To a very large extent, the need for these works to improve navigation and trade formed the basis for the development of coastal engineering practice in NSW.

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Coastal engineering in NSW has gone through a series of stages which are identifiable in the public (coastal engineering) works and government programs of the time. These included:

- From 1788 to the 1950s the emphasis on coastal engineering was for the improvement of navigation for the coastal trade, including the development of major ports, construction of wharves, dredging of navigation channels and construction of shipbuilding and repair facilities. By the 1950s the coastal trade had all but ceased to exist and could not compete with the expanded rail and road network. The major ports of Newcastle, Sydney, Port Kembla and Botany Bay were established and have continued to develop the capacity to handle larger ships and containers.
- From the 1950s to the present there has been an emphasis on the use of all but the major ports for the fishing industry. From 1959 to 1966 Public Works constructed nine fishing ports under the fishing ports program. This program was augmented in 1976 and further works included wharves, facilities, entrance works, breakwaters, boat harbours and mini-ports.
- From the mid-1970s to the present there has been an increasing awareness of the coastal environment and its fragile nature and the need to preserve the natural environment and the recreational amenity it provides. This increased awareness has coincided with a boom in the tourist industry, together with an unparalleled increase in coastal development. The number of registered vessels in NSW increased from 12,000 in 1979 to 90,000 in 1989 and 160,000 in 1997 (NSW Waterways pers. comm.) Coastal engineering focus has changed to beach protection works, beach regeneration and the provision of tourist and recreational infrastructure. These works have been supported by government programs since the mid-1970s.

2. The Coastal Structure as an Asset

The NSW Government recently undertook an asset appraisal of breakwaters and river entrance training walls, including aspects on the history of design, construction and performance over a period of nearly 100 years. A total of 63 structures which are 22,600 m in length (Figure 1) were appraised. The figure also shows the location of 12 rock quarries from which armour was obtained for construction. The study (MHL648 1993) placed the value of the structures at \$550 million.

MHL has been closely linked with many of the designs of the structures on the NSW coastline through its data collection and physical modelling capabilities which have been built up over a period of nearly 40 years. Some of the original designs as constructed (with irregular maintenance) have withstood wave energy over the last 100 years. In designing a coastal structure a number of different factors have to be considered by the engineer. These include the practicalities of construction, the short-and long-term requirements of the construction, the availability of materials, environmental factors and the hydraulic criteria. After arriving at a preliminary design or, more commonly, a number of options the design is usually optimised using either a physical or numerical model or both.

3. Some Examples of Historical Developments in Coastal Engineering Structures

For over three decades both numerical and physical modelling have been found to complement each other and have been used in investigations such as on the Coffs Harbour breakwater layout (MHL110, 1966). To illustrate the evolution in design philosophy examples of the training walls at Ballina on the Richmond River (Figure 2) and Coffs Harbour eastern breakwater (Figure 3) are discussed. These two structures are examples of a training wall and a breakwater at a harbour. Most of the 63 major structures along the NSW coastline are in these two categories. This section also describes recent developments in data collection and analysis as well as the evolution of physical modelling techniques used for coastal structure design at MHL. The paper also traces the rationale for using computational models in conjunction with these physical models.

3.1 Richmond River Entrance

The Richmond River entrance was charted by Captain Henry Rous in August 1828 while inspecting the coast between Brisbane and Port Macquarie. His records include a note on manoeuvres for getting inside the tricky entrance bar.

"The sandbar at the entrance to the Richmond River was notorious - it shifted across the 6,000 ft wide entrance sculpting mazes of sand which discouraged ships" (Coltheart 1997). The bar was regularly dredged from 1878. By the turn of the century the Public Works Department maintained a fleet of 39 dredges working along the NSW coast.

A plan to train the entrance was drawn up by the Chief Engineer of Public Works, Sir John Coode. In 1889 the entrance was completely blocked by natural sand shoals so that produce had to be carted overland to the Clarence River to the south. Work on the entrance training was commenced that year comprising internal training walls and the southern breakwater. Figure 2 shows a design cross-section developed and presented for the Richmond River breakwaters. The design consisted essentially of the placement of a pile of rocks to form the structure, as shown on this original design drawing.

Rock for the northern breakwater was quarried locally at Pilot Point while rock for the southern breakwater was punted from a quarry at Rileys Hill, 19 miles upstream.

In the spring of 1891 it is reported that "*the channel again broke through the southern spit, moving in one day a distance of nearly 3,000 feet.*" (Coltheart 1997). A training wall was under construction at west Ballina on the north arm upstream. A larger training wall on the southern side was added, enclosing a reclamation site of 63 acres. Dynamite was used to break up the shoals and two dredges were deployed working 16 hours a day to clear the shoals. This was unsuccessful as the southerly weather continued to wash sand around the southern breakwater and into the entrance. Work on the northern breakwater was stopped in 1899 and for more than two years the total workforce of 200 men was deployed night and day on the southern breakwater to try to stop the sand ingress.

By May 1901 the southern breakwater had reached a length of 7,542 feet and the night force recommenced work on the northern breakwater. By mid-1902 the

southern breakwater was 800 feet short of the design length and the northern breakwater 1,000 feet short. Several time the ends had to be restored as they were flattened by storms. A new quarry was opened to provide larger armour stone. A locomotive was used to haul the blocks along the northern wall while four horses hauled the tip trucks carrying rock from 1 ton to 20 tons along one mile of track to the ends of the breakwater.

The entrance works had totally reformed the entrance, reconfiguring the channels with the dredges reclaiming swamps and tidal flats and creating new channels. Dredging and bank protection works were ongoing to maintain the navigability of the river. Vessels with a draft of 10 feet could steam 63 miles up river and, in 1904, 300 steamers and two schooners crossed the Richmond River bar.

In the 1930s the storm damage required constant repairs and unemployment relief funds were used in setting 30 ton concrete blocks onto the southern wall. Five dredges worked regularly in the river during the 1930s. The northern breakwater was damaged and repaired in 1947 and the southern wall again in 1949.

By the 1960s dredging was only used to maintain a 6 foot minimum depth over the bar for smaller vessels.

In contrast, the upgrading of the Richmond River entrance training walls today included intensive numerical assessment, modelling to determine inshore wave conditions, and physical model testing of various primary armour materials in a random wave basin at MHL. The modern design is based on one of the most thorough and longest wave databases used anywhere in the world.

3.2 Coffs Harbour

Coffs Harbour is situated on the mid-north coast of NSW approximately 600 kilometres north of Sydney. The area was settled in the 1840s by timber loggers and the harbour developed as a timber trading port.

The Coffs Harbour jetty was completed and opened to shipping in August 1892. It was severely damaged in storms two years later. The port had the highest net tonnage in NSW and was the busiest on the north coast averaging 399 ships per year between 1909 and 1924. Ship visits reduced to 26 in 1960 and to seven by 1970. The last freighter to use the jetty was in 1973. The jetty has been subsequently shortened to its original length and restored, primarily as a tourist feature. Figure 3 indicates the period in time that construction at Coffs Harbour took place. It is seen that construction of the breakwater took place from 1918 to 1974.

In 1912 the decision was taken to construct an artificial harbour by constructing breakwaters at Coffs Harbour to protect the jetty. Work began immediately on the causeway connecting the mainland to South Coffs Island and this effectively intersected the alongshore drift past the islands and to the northern beaches. A quarry was established on the island to provide rock for the proposed eastern and northern breakwaters which enclosed a harbour of 217 acres. Construction of the breakwater private contract commenced in 1915 and the contract was cancelled and taken over by Public Works in 1917.

Work on the eastern breakwater which commenced in 1918 was washed away by heavy storms in 1919 and 1920. The northern wall was two thirds complete when heavy storms in 1921 destroyed the end and washed away five tipping wagons leaving the railway track suspended in mid air. The northern wall was finally connected to Muttonbird Island in 1924. Continual maintenance was required as the wall was low enough to be overtopped in storms. In 1925 storms levelled 400 feet of the northern wall which had to be reconstructed.

Storm damage was a major cost factor on the eastern wall construction. Following major damage in 1925 it was noted that "the quarry could not provide stone big enough to withstand the seas" (Coltheart 1997) and use of concrete blocks with a toe of 100 tonne blocks was recommended for the seaward face. In gales during 1937, 110 feet of the wall was demolished and had to be replaced. The wall was completed in 1939 and the concrete capping in 1943. Major repairs required upgrading of the railway line and carriages in 1953.

Major impacts on the coastal processes in the study area have resulted from the harbour construction. The breakwaters have intersected the estimated south to north littoral drift of 75,000 m³/annum resulting in accretion of Boambee Beach to the south and Jetty Beach within the harbour. At the present time, following many years of lower than average wave energy conditions the beaches north of Coffs Harbour are still in a depleted condition.

4. Historic Aspects of Data Collection for Coastal Engineering Design

The necessity to use accurate site specific data for coastal engineering construction has been acknowledged for over a century. Data has also been used to evaluate the performance of a structure after it was constructed and to refine and improve coastal engineering design. Some of the first evidence available on the collation of wave data for Coffs Harbour is the documentation of wave heights at the seaward end of the wharf (Figure 3) during rough seas on six occasions from December 1936 to July 1954 (MHL186 1974).

An example of continuous wave data recording prior to the use of data buoys is obtained from wave pole records as reported by White (MHL109 1966) taken half a mile from the coast at Nobbys Head, Newcastle in 1965 (Figure 4). A significant step in wave data analysis on the NSW coastline was taken when wave hindcasts were calculated for Port Kembla for the years 1950 to 1964 by Stone and Foster (MHL109 1966). The results of these analyses are indicated in Figure 5 and compare well with present day hindcast wave data. Long-term continuous offshore wave measurements were begun in Sydney in 1974 and have continued since, using a network of seven wave data buoys. Increased accuracy in extreme wave height (Goda et al 1988) and storm duration (Sobey 1997) analysis are of interest to the coastal engineer. Analysis techniques have been developed over the last three decades to make maximum use of short time series. The relevance of long-term wave measurement is seen in the variations obtained for the 100-year wave height for Port Kembla (Figure 6). Presently, MHL measures wave height and directions using a number of methods such as a network of wave data buoys (Kulmar 1995). These include a directional buoy in Sydney (Figure 6); radar imagery; aerial photography; electromagnetic current meters; hindcasting and satellite imagery. Whilst the data buoy network is a

continuous measurement program, the other methods of measurement were used for specific projects. The NSW measured wave climate represents one of the longest continuous measured wave data sets anywhere in the world.

4.1 An Example in the Change in Requirements for Data

The requirements for data to aid coastal engineering design projects have changed over the last few decades. This can be illustrated by the equation used to obtain armour size. Many design equations have been developed to obtain armour size. Of these equations those suggested by Irribarren (1938), Hudson (1958) and Van der Meer (1987) have been the most accepted and utilised.

Iribarren's equation for stability (1938):

$$\frac{H}{\Delta D_{n50}} = \frac{\cos \alpha - \sin \alpha}{K^{1/3}}$$
(1)

Hudson's equation for stability (1958):

$$\frac{\text{Hs}}{\Delta D_{n50}} = (K_D \cot \alpha)^{1/3}$$
(2)

Van der Meer's equations for stability (1987):

For plunging waves:

$$\frac{H_s}{\Delta D_{n50}} * \sqrt{\xi_z} = 6.2 P^{0.18} \frac{S}{\sqrt{N}}$$
(3A)

For surging waves:

$$\frac{H_{s}}{\Delta D_{n50}} = 1.0P^{-0.13} \frac{S}{\sqrt{N}}^{0.2} \sqrt{\cot \alpha \xi^{P}}$$
(3B)

where:

ξz	=	surf similarity parameter	Р	=	permeability coefficient of structure
α	=	slope angle	$S = A/D_{n50}^2$	=	damage level
Δ	=	relative mass density	Α	=	erosion level in cross-section
Dn ₅₀	=	nominal diameter of a stone	Ν	=	number of waves
W ₅₀	=	50 percentile value of mass distribution	H_1H_s	=	wave height
			K ₁ K _D	=	coefficient of damage

Variables such as storm duration or number of waves (N), permeability coefficient of the structure (P) and surf similarity parameter (ξ) have been included in design criteria to obtain the mass of primary armour over the last decade. Water level and therefore the influence of breaking waves was included in Hudson's physical modelling regime though not in his formulae. Variables such as wave obliquity (Galland 1994) and wave groupiness (Funke 1979), though not included in the design formulae, have to be considered in the finalisation of designs when using physical models. To this end MHL has included the analysis of wave groups during storm and calm conditions (Jayewardene 1993) and the influence of wave obliquity on primary armour (Jayewardene 1997) in its research and development programs.

5. A Short History of Physically Modelling Coffs Harbour Breakwaters

Two physical models were carried out on Coffs Harbour, one in 1966 and the most recent investigation in 1998. The main aims of the 1966 model were to provide improved berthing at the existing jetty (Figure 3) and improved anchorage conditions for the existing fishing fleet. The 1998 model attempted to simulate damage conditions to the eastern breakwater head where thirteen 40 tonne blocks were displaced during a storm in May 1997. Although the aims of the two investigations were different, it is instructive to compare the techniques of wave generation and data collection used in these models to obtain an understanding of the material changes that have taken place in coastal engineering design.

5.1 Wave Data

Hindcast wave data was used to obtain the deep water design wave for the 1966 model (MHL 109 1966). Extreme wave analysis was carried out to obtain design wave data for the 1998 model (MHL 914 1997). Regular waves were used in the early model whereas long crested irregular waves were used in the more recent model. Irregular waves have been used in physical models since the late 1970s and the advantages over regular wave modelling are well documented in the literature (Graveson 1974). In the mid-1980s the importance of wave grouping in harbour layout and the importance of correct reproduction of these groups particularly for ship movement investigations was established (Sand 1983, Bowers 1988). This is probably the reason for the observation made at the conclusion of the 1966 study that "Reproduction of the surge motion by detailed observation of the prototype were not successful in the model" (MHL110 1966). Accurate wave height (Figure 7) and wave direction (Figure 8) data during the May 1997 storm in conjunction with numerical modelling (Figure 9) aided in establishing that relatively long period waves (15-17s) of relatively low return period (1-2 year) resulted in the damage of the head. This damage was reproduced accurately in the physical model (Figure 10).

5.2 Water Level Data

Table 1 indicates some water levels used in physical model investigations carried out at MHL for previous projects. In addition to the 90 years of tidal data recorded at Fort Denison, Sydney a network of tide recorders established in 1984 has enabled MHL to predict tides utilising 69 tidal constituents (MHL591 1991). The ability to predict tides has in turn enabled MHL to estimate anomalies accurately and, hence, obtain accurate return periods for extreme water level statistics (MHL591 1991). The water level recorder at Coffs Harbour was able to record the extreme water levels during the May 1997 storm and helped to accurately simulate storm conditions in the physical model. The database generated by the MHL network has monitored some extreme storms such as the one that occurred in May 1974 when waves up to 12 metres associated with large tides and storm surges affected the NSW coastline over several days. A decade ago this storm was adopted as a design storm. With improved tools for extreme event analysis (AWACS 1992) it has been shown by Monte Carlo simulation techniques (Goda 1988) that this storm was more extreme than the 100-year event. This resulted in considerable lowering of crest heights of

MHL Report No.	Year	Site	Design Wave Height/Period	Design Water (Low Water)	Head Slope	Recommended Size	Comment
110	1966	Coffs Harbour	18 ft/10 s	6 ft		10.5 to 12 tonne	Physical 3D model
112	1966	Wollongong Harbour	25 ft/10 s	6 ft		15 tonne	Physical 3D model
256	1978	Bellambi	2 m/7 and 10 s	2.5 m	1:1.5	10.5 to 12 tonne	Physical 3D model
261	1979	Port Kembla revetment	5.4 to 7.1 m/10 s	3.5 m and 4 m	1;1.5	15 tonne	Physical 2D model
272	1979	Port Kembla sea wall	5.5 m/10 and 14 s	4 m	1:2	12 tonne Hanbars 7 tonne dolos	Physical 2D model
301	1980	Eden breakwater	4 to 6 m/6 to 15 secs	to 3.2 m	1:1.5	15 tonne Hanbars	Physical 2D model
782	1996	Oblique wave tests	6.3 m/12 s	2.4 m	1:1.5	10 tonne rock	Physical 3D model
795	1996	Ballina	6.3 m/12 s	2.4 m	1:2.2	suggested to 57 tonne 15 tonne Accropode	Physical 2D model
860	1997	Ballina	6.3 m/12 s	2.4 m	composite		Physical 3D model
897	1997	Ballina	6.3 m/12 s	2.4 m	composite 15 tonne Accropode, antifer cubes, Hanbars		Physical 3D model
941	1998	Coffs Harbour eastern breakwater	7.4 m/12 s 5.4 m/15 s	2.4 m	composite	40 tonne cubes 28 tonne Hanbars 22 tonne Accropodes	Physical 3D model

structures such as the Sydney parallel runway (AWACS 1992) that have been constructed recently.

Table 1 Design Data - Physical Models

6. Physical Model of Ballina Training Wall Head

Historical records indicate that 30 tonne artificial concrete cubes were used instead of rock armour in the early 1930s on the Ballina Heads. Recent physical modelling carried out at MHL (MHL860 1997) compared the performance of artificial armour units such as Accropodes, Antifer cubes and Hanbars (a NSW patented unit) with that of rock armour. Table 2 indicates the values for the mass of primary armour using a coefficient of damage (K_d) obtained from testing carried out by Foster and Gordon (1973). Their tests were conducted using regular waves, whereas present day (MHL) tests use Pierson Moskowitz spectra with spectral peak period and wave height associated with extreme event analysis. Reflection analysis (Mansard 1980) carried out in the flume to accurately assess the incident wave height and numerical modelling using REFDIF (a numerical refraction/diffraction model) was utilised to obtain inshore wave conditions which were replicated in the physical model.

7. Numerical Modelling Used in Conjunction with Physical Models

Constant updating and improvement of short and long wave computational models in the last three decades has significantly extended the design situations for which they can be used. These include the use of directional wave spectra, bed friction, flow separation and recirculation near jetties, radiation damping near harbour entrances and diffraction through breakwater gaps. However, in cases such as the simulation of the May 1997 storm at Coffs Harbour, nearly all the damage was caused by wave overtopping and instabilities caused by flows in the secondary armour. These are better simulated in a properly scaled physical model with gravitational and inertial forces dominating frictional forces (Cornett 1995). Conditions at the water depth corresponding to the long crested irregular wave generator were obtained for the 100-year event by using the numerical model REFDIF. A similar approach was used when comparing the performance of artificial armour units such as Hanbars, Accropodes and Antifer cubes (MHL897 1997).

	Hu	0						
Chainage (m)	Wave Height (m)	Damage	Armour Size M50 (tonnes)	Kd 30° 60°		Armour Size (single layer) 30° 60°		Actual Placed Contract
325	4	5-10	7.6	4	5.5	7.1	5.1	5
		10-15	5.7	5.5	8.5	5.1	3.2	
								\downarrow
390	4.5	5-10	10.8	4	5.5	10.1	7.3	7.5
		10-15	8	5.5	8.5	7.3	4.7	
								\downarrow
440	5	5-10	14.9	4	5.5	13.8	10.1	10
		10-15	11.1	5.5	8.5	10.1	6.4	
								\downarrow
490	5.5	5-10	19.8	4	5.5	18.	13	10
		10-15	14.8	5.5	8.5	13	8	↓ ↓
								Ch520
540	6	5-10	25.7	4	5.5	23.8	17.3	15-18
		10-15	19.2	5.5	8.5	17.3	11.2	
								↓ ↓
590	6.5	5-10	32.6	4	5.5	30.3	22	Head
		10-15	24.4	5.5	8.5	22	14.2	

 Table 2 Comparison of armour size on Ballina breakwater using Hudson's method, oblique wave testing and actual placed armour

Table 2 indicates values for the mass of primary armour using coefficient of damage (K_d) values obtained from testing carried out by Foster and Gordon (1973). These values were compared with those obtained by using Van der Meer's breakwater design criteria (Van der Meer 1988). The results obtained from Hudson's formula and from oblique wave testing are shown in Table 2.

8. Methods of Construction and Quarrying

Up to the mid-1970s the construction methods used at both Ballina and Coffs Harbour generally involved the sourcing of local armour rock and random placement by tipping. Natural armour stone was sourced on site from adjacent coastal headlands and nearby purpose quarries. In most instances these stones were tipped from specially constructed tipping frames on rail wagons and later from modified truck bodies. To increase the primary armour mass, concrete armour units ranging from 10 to 38 tons were cast and then tipped on training wall slopes. Smaller units up to 2 tons were frequently cast on top of the wall and pushed over the crest by bulldozers. The lack of control resulted in haphazard positioning of the units and poor interlocking. Where rail systems were adopted for armour placement (at Ballina and Coffs east breakwater) the crests were capped with concrete to support the loads

imposed by the construction plant and armour unit. The concrete caps often deteriorated as the structure settled. Current construction techniques have emphasised the need to optimise armour size by careful design utilising both single layer and double layer, placed and tipped primary armour (MHL860, MHL897 1997). Cost considerations have also favoured single layer placement of even rock armour as tipping of rock results in significantly increased material costs. Other factors relevant to single layer placement of rock are a reduction in environmental cost of quarry utilisation, difficulty in sourcing large armour and the increasing distance and cost of material transport.

Physical modelling has enabled the designer to evaluate the influence of wave grouping, wave obliquity and permeability of the structure. It has also aided in comparing the performance of the single layered Accropode unit with units that have already been placed on structures on the NSW coastline such as the Hanbar, dolos and rectangular cube.

Many of the NSW coastal structures are located adjacent to popular holiday towns and villages and hence provide waterway recreational facilities. Presently construction and repair is regulated from the planning approval stage to the on-site placement of material, and must fulfil the mandatory requirements of NSW occupational health and safety legislation.

9. Conclusions

Manly Hydraulics Laboratory has been closely linked with many of the designs of the structures on the NSW coastline through its data collection and physical modelling capabilities which have been built up over a period of nearly 40 years. Some of the original designs as constructed have withstood wave energy over the last 100 years which, by design standards, should have failed. However recently introduced design variables such as wave obliqueness offer quantitative explanations for this stability. This paper has used the examples of Coffs Harbour breakwaters and the Ballina training wall head construction and repair to outline some of the developments in coastal engineering design and construction in NSW over the last century. An example of the design equations for primary armour size was used to indicate the influence of new variables such as permeability, storm duration and wave obliqueness on training wall construction at Ballina. The present day focus on the awareness of and need to care for a fragile environment which provides many recreational amenities is emphasised. The establishment and maintenance of extensive water level and wave databases is shown to increase design wave height and wave periods used in physical model investigations. The return period for design water levels has been established. The paper highlights the shortcomings of physical modelling when tools such as the proper generation and control of wave groupiness were unavailable.

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11. References

Australian Water and Coastal Studies 1992, Parallel Runway Combined Probability of Occurrence of Water Levels and Waves, AWACS Report No. 92/20.

Bowers E.C, Wave Grouping and Harbour Design, NATO ASI Series 1988, Lisbon Portugal, pp 391-416

Coltheart, L. 1997 Between Wind and Water, Hale & Iremonger

Cornett, A 1995 PhD Thesis, Technical Report HYD-TR-005, National Research Council, Canada

Department of Public Works N.S.W. Head Office File No. R 1017/7, Coffs Harbour Jetty Range 1948 -1963.

Foster D.N., A.D. Gordon 1973, *Stability of Armour Units Against Breaking Waves*, Proc 1st Australian Conference on Coastal and Ocean Engineering Sydney, Australia

Funke E.R, Mansard E.P.D, 1979 On the Synthesis of Realistic Sea States in a Laboratory Flume, Report LTR-HY-66-1979 National Research Council Canada pp 1-54

Galland J.C 1994, Rubble Mound Breakwater Stability Under Oblique Waves, Proc ICCE 1994

Goda Y 1988, On the Methodology of Selecting Design Wave Height, Proc 21st ICCE, Malaga, Spain, 1988.

Graveson, H. 1974, *Model Tests with Directly Reproduced Nature Wave Trains*, 14th ICCE Copenhagen, Denmark

Jayewardene, I.F.W. et al 1993, Analysis of Wave Groupiness, 11th Australasian Conference on Coastal and Ocean Engineering, Queensland, Australia, pp 99-103

Jayewardene, I.F.W. 1997, Young, T Laboratory Measurement of Oblique Irregular Waves on a Rubble Mound Breakwater, 13th Australasian Conference on Coastal and Ocean Engineering, Christchurch, New Zealand, pp 197-202

Kulmar, M. 1995, *Wave Direction Distributions off Sydney, New South Wales*, 12th Australasian Conference on Coastal and Ocean Engineering, Melbourne, Australia, pp 197-202

Manly Hydraulics Laboratory 1966, Report on Wave Conditions at Shellharbour, MHL109

Manly Hydraulics Laboratory 1966, Coffs Harbour Model Investigation, MHL110

Manly Hydraulics Laboratory 1974, Coffs Harbour Historical Data Review MHL186

Manly Hydraulics Laboratory 1991, Storm Surges Monitored Along the NSW Coast MHL591

Manly Hydraulics Laboratory 1992, NSW Breakwater Asset Appraisal Pilot Study, MHL636.

Manly Hydraulics Laboratory 1993, Breakwater Asset Appraisal, MHL648

Manly Hydraulics Laboratory 1996, Laboratory Measurement of Oblique Irregular Waves on a Rubble Mound Breakwater, MHL Report No. 782 (draft).

Manly Hydraulics Laboratory 1997, Ballina Accropode Breakwater 3D Basin Testing MHL860

Manly Hydraulics Laboratory 1997, Coffs Harbour Eastern Breakwater Historical Storm Analysis, MHL914

Mansard. E, 1980, The Measurement of Incident and Reflected Spectra Using a Least Square Method, Proc 17th Int. Conf on Coastal Engineering pp 154-172

Sand S.E. et al, 1983, Group Bounded Long Waves In Physical Models, Ocean Engineering Vol 10, pp 261-294

Sobey, R.J. and Orloff L.S. 1995, Triple Annual Maximum Series in Wave Climate Analysis, Coastal Engineering 26 (1995) pp135-151

Van der Meer, J.W 1988, Deterministic and Probabilistic Design of Breakwater Armour Layers, Journal of Waterway Port and Coastal Engineering Jan 1988



Figure 1 Asset Appraisal Study indicating Location of Rubble Mound Structures on the NSW Coast (MHL648)





Figure 2 Sketches of the Richmond River Entrance Southern Breakwater Structure (Burrows 1904-05) (Coltheart 1997)

COMPARISON OF WAVE HEIGHT AND PERIOD REPORTED BY NOBBYS SIGNAL STATION, AND MEASURED OFF WAVE POLE BY NEWCASTLE DISTRICT OFFICE





	Extreme Wave Height Analysis Port Kembla (m)				
Years	10 yr event	20 yr event	100 yr event		
1975-77	6.20	7.30	7.75		
1975-80	7,05	8,35	8.95		
1975-83	7.10	8.60	9.20		
1975-86	6.80	8.05	8.55		
1975-89	7.20	8.50	9.10		
1975-92	7.55	8.95	9.60		
1975-97	7.30	8.60	9 20		





Figure 5 Return Periods for Hindcast Waves (1965) (MHL109)



Figure 7 Coffs Harbour Water Level and Offshore Wave Height Data) (May 1997 Storm) (MHL914)

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Figure 8 Coffs Harbour Offshore Wave Direction Data (May 1997 Storm) (MHL914)



Figure 10 Replicating May 1997 Storm Damage in a Physical Model