Wamberal beach terminal Protection structure - physical modelling study

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WRL Technical Report 97/26

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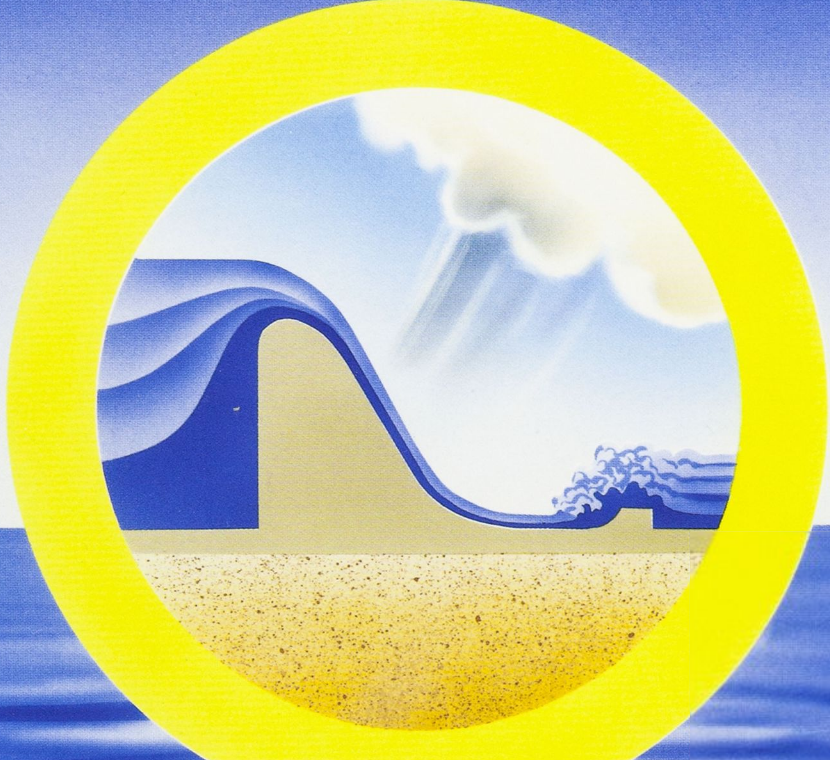

THE UNIVERSITY OF NEW SOUTH WALES
WATER RESEARCH LABORATORY

WAMBERAL BEACH TERMINAL PROTECTION STRUCTURE – PHYSICAL MODELLING STUDY

WRL Technical Report 97/26 October 1997

by

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1. INTRODUCTION

Wamberal Beach is located between Broken Head and Wamberal Point, on the Central Coast of NSW (Figure 1). In 1995 the Council of the City of Gosford adopted a Coastal Management Plan for the open coast beaches of Gosford City. The plan identified erosion hazard zones associated with past storm events and predicted trends. Specifically for Wamberal Beach, the principal strategic option recommended is protection of the narrow dune ridge and associated development, together with action to maintain the beach amenity. Terminal protection by a buried structure accompanied by action to maximise long term maintenance of sand supply to the beach was recommended.

Unisearch,Water Research Laboratory (WRL) was engaged by Council to:

- assess Terminal Protection Structure (TPS) design options;
- undertake detailed design of preferred option;
- prepare Environmental Impact Statement; and
- prepare technical specifications and drawings.

Following the presentation of design options to Council and the general community, a Seabee armoured structure (Figure 2) was selected as the preferred option.

This report details two-dimensional (2D) physical model investigations of the proposed Seabee armoured structure undertaken in the random wave flume at WRL, Manly Vale, using both random and monochromatic waves. Testing was undertaken to determine:

- wave run-up on the Seabee armour;
- optimum structure crest elevation;
- structure over-topping rates;
- Seabee armour stability for a number of combined design storm wave conditions and still water levels (SWL's);
- toe stability for a number of storm wave conditions and SWL;
- destructive testing.

A total of approximately 100 individual tests were performed during the model testing program. All model testing was undertaken at a geometric scale of 1 in 28. Reduced levels refer to Australian Height Datum (AHD). Unless otherwise specified, data and results presented in this report are given in prototype equivalent units.

2. DESIGN WAVE CLIMATE AND WATER LEVELS

A combined storm wave climate and water level study for Wamberal Beach was undertaken by WRL (Couriel et al, 1997), using joint probability analysis ('Monte Carlo'). Briefly, a 26 year record of wave data was obtained from a wave-rider buoy maintained in deepwater offshore of the entrance to Botany Bay, Sydney, NSW. During major storms of May and June 1974, the wave-rider buoy was out of service. Recognised estimates of the significant wave height during these storms (Foster et al, 1975), as well as data from the Port Kembla waver-rider buoy, were used where possible to fill gaps in the data set. Hourly tide data were obtained from the gauge at Fort Denison, Sydney.

From the above analysis, it was determined that depth limited conditions exist at the toe of the proposed TPS structure. Assuming bed scour down to −1.0 m AHD and wave setup equivalent to 10% of the deepwater significant wave height, design water levels were used to determine the maximum depth limited breaking wave height impacting on the structure. Although offshore wave heights are significantly higher, depth limited breaking wave heights govern the structural stability and hydraulic performance of the proposed TPS and, therefore, were adopted for design.

The design breaking wave heights at the structure were transferred (using linear wave theory) to equivalent "offshore" wave heights in 24 m water depth. This is the scaled depth of the deepest section of the wave flume where wave measurement was undertaken during testing.

The combined design wave climates and water levels for differing storm recurrence interval (RI) are summarised below:

5 year average recurrence interval

significant wave height $H_{s(24)}$	m	2.37
zero-crossing wave period T_z	sec	9.0
peak wave period T_p	sec	12.9
storm duration	hrs	1.0
still water level SWL	m AHD	2.4

20 year average recurrence interval

significant wave height $H_{s(24)}$	m	2.53
zero-crossing wave period T_z	sec	9.5
peak wave period T_p	sec	13.6
storm duration	hrs	1.0
still water level SWL	m AHD	2.7

50 year average recurrence interval

significant wave height $H_{s(24)}$	m	2.66
zero-crossing wave period T_z	sec	9.8
peak wave period T_p	sec	14.0
storm duration	hrs	1.0
still water level SWL	m AHD	2.8

100 year average recurrence interval

significant wave height $H_{s(24)}$	m	2.82
zero-crossing wave period T_z	sec	10.1
peak wave period T_p	sec	14.4
storm duration	hrs	1.0
still water level SWL	m AHD	3.0

3. THE PHYSICAL MODEL

3.1 Model Scale

The physical model of the proposed Wamberal Terminal Protection Structure (TPS) was constructed at an undistorted length scale of 1:28. This scale was selected on the basis of the dimensions of the wave flume, availability of model armour units, and to minimise scale effects. The scale ratios derived for the study, as determined by Froudian similitude, were as follows:

Length Ratio	=	L_r			=	28
Time Ratio	=	T_r	=	$(L_r)^{1/2}$	=	5.29
Volume Ratio	=	V_r	=	$(L_r)^3$	=	21952
Discharge Ratio	=	Q_r	=	V_r/T_r	=	4150

3.2 Testing Facilities

2D testing of the proposed Terminal Protection Structure (TPS) was undertaken in the random wave flume at WRL. The flume is 35 m long, 0.9 m wide and 1.4 m deep (Figure 3). Waves are generated by a hydraulically powered piston-type wave paddle capable of generating both monochromatic and random waves. Random waves were generated using a Pierson-Moskowitz spectra.

3.2.1 Wave Measurement

Wave heights in the flume were monitored using three capacitance wave probes. All probes were cabled to a personal computer running programs developed within WRL that permit quick and reliable assessment of wave heights.

Random wave spectra were measured using the three probe array method described by Mansard and Funk (1980). This technique uses a least squares analysis to separate the incident and reflected spectra from the measured co-existing spectra. Probe spacing is indicated in Figure 3.

Monochromatic waves were measured using one of the three probes for direct measurement and the remaining two probes for verification. Monochromatic tests were generally undertaken in short (6 to 8 waves) 'pulse' tests, permitting incident wave heights to be determined directly, prior to the arrival of waves reflected from the structure. Where continuous monochromatic tests were performed, a wave probe mounted on a mobile trolley was moved along the flume for a distance of at least one wave length to capture the envelope of water surface elevations created by incident and reflected waves. The minimum and maximum wave heights were determined (H_{min}, H_{max}). Incident wave height (H_i) and reflection coefficient (K_r) were estimated from:

$$H_i = (H_{max} + H_{min}) / 2$$
$$K_r = (H_{max} - H_{min}) / (H_{max} + H_{min})$$

3.2.2 Wave Run-up Measurement

The wave run-up of surging and broken waves across the Seabee structure was measured by a single inclined capacitance probe, oriented parallel to the front face of the structure. The sensing wire of the probe was maintained within 5 mm of the surface of the structure, ensuring accurate measurement of the upper edge of individual run-up excursions.

3.2.3 Wave Over-topping Discharge

Rates of wave over-topping were quantified by placing a tray on the landward side of the crest of the model structure, and water collected in the tray after each test extracted with a pump. Collected volumes were determined using a graduated flask and column. During extreme wave over-topping events, the tray was bailed by hand during the test run, to ensure no over-spilling of the collection tray. These procedures were used to estimate both average and instantaneous (single wave) wave over-topping rates.

3.3 Seabee Design

The stability of Seabee armour units is governed by blanket theory. Unlike conventional rubble mound structures where the size of stable armour is proportional to the wave height cubed (H^3), the necessary size of Seabee armour is directly proportional to the wave height, resulting in a significantly more slender cross-section. For preliminary design, the stable Seabee unit height (R) is given from blanket theory by (Unisearch, 1997):

$$R = \frac{H_s}{\left(C_B(1-p)(S_r-1)\cot\alpha\right)^{1/3}}$$

where R is the height of armour layer (m)
H_s is the design significant wave height (m)
C_B is the stability coefficient (5.0 for preliminary design)
p is the armour layer porosity
α is the angle of armour slope from horizontal (degrees)
S_r is the relative density of Seabee armour units (kg/m^3).

Preliminary design of the Seabee structure for stability and crest elevation was undertaken during the preparation of alternative TPS options by WRL (Figure 4). A high porosity Seabee unit was selected to minimise wave run-up and wave over-topping, permitting a lower crest elevation while maintaining the required protection to dunes and beach-front property. A single hole concrete unit was adopted, based on the outcomes of a CLP/public meeting. Dimensions of the prototype Seabee units are as follows (refer Figure 5 for definition of parameters):

- R = 800 mm
- D = 800 mm
- d = 480 mm
- p = 45%
- ρ = 2300 kg/m^3

3.4 Model Construction

A false floor was constructed in the wave flume to represent the slope of the Wamberal Beach nearshore profile. For a distance of 10.6 m (300 m prototype) in front of the model structure, a slope of 1 in 33 was constructed from sheets of marine ply. On the seaward end of this slope a 2.4 m section was constructed at a slope of 1:6 down to the floor of the deepest section of the flume. A 0.9 m (model) removable floor section was incorporated immediately in front of the model structure, covering a 150 mm (model) deep wooden box constructed below. This removable section located at the toe of the model structure provided for a solid bed to be used during run-up and over-topping tests, but after removal and filling of the box with sand, permitted toe and structure stability tests to be performed using a mobile bed.

The model test section is shown in Figure 6, representing a typical cross-section through the proposed prototype structure with the beach is at an eroded state due to storm action. A slope of 1V in 1.5H was selected for the seaward face of the structure in order to minimise the impact of the prototype TPS 'footprint' on beach amenity. The following materials were used to construct the model.

3.4.1 Core Material

The core of the structure was formed using sand ($D_{50} = 0.15$ mm) moulded to the correct shape. Fine sand was selected as this more realistically represents the scaled hydraulic conductivity and hence drainage characteristics of the prototype structure during wave run-up.

3.4.2 Filter Cloth

To prevent leaching of the core material in the prototype, a geotextile filter cloth is to be placed between the core and rock underlayer. In the model, this filter cloth was modelled using a finely knitted fabric.

3.4.3 Rock Underlayer

The rock underlayer used in the model was constructed from commercial 10 mm (nominal) road metal. The rock was sieved prior to construction, the grading curve determined for the material shown in Figure 7.

This material corresponds to prototype material of D50 $\approx$ 250 mm. The design underlayer material grading (prototype equivalent units) is also shown in Figure 7.

3.4.4 Seabee Armour

Dimensions of the model Seabee units are as follows:

- R = 28.6 mm
- D = 28.8 mm
- d = 18.5 mm
- p = 45%
- W = 16.6 g
- ρ = 2180 kg/m^3

The units are manufactured from moulded plastic.

3.4.5 Wave Return Wall

Three different designs for the wave return wall were tested. Figure 8 shows the dimensions of each design. *Design A* was constructed from multiple templates cut from marine ply to the correct cross-sectional dimensions, glued to form the required 0.9 m length (i.e. the width of the wave flume). *Design B* and *Design C* were constructed of 50 mm PVC pipe, cut lengthways to the correct dimensions. Figure 9 shows the operation of *Design C* wave return wall under test conditions.

3.4.6 Toe – Gabion Basket and Reno Mattresses

The toe of the Seabee armoured structure was modelled using reno mattresses available from previous model studies. The mattresses (refer Figure 6) were approximately $230\,\text{mm} \times 90\,\text{mm} \times 18\ \text{mm}$ (model) = $6.4\,\text{m} \times 2.5\,\text{m} \times 0.5\,\text{m}$ (prototype), made from sewn fly-screen, filled with appropriately modelled river gravel. The gabion basket at the base of the Seabee armour layer (refer Figure 4) was modelled by doubling the reno mattresses at the base of the Seabee wall to approximate the correct dimensions ($1.0\,\text{m} \times 1.0\,\text{m}$ prototype)

4. TEST PROCEDURE AND RESULTS

4.1 Wave run-up

The model Seabee structure was initially built with a crest elevation of 13 m AHD, and the wave return wall was not placed at the crest. The objective of over-building the Seabee structure and extending the Seabee units right to the crest was to determine maximum run-up elevations.

Random waves were used for all run-up tests. Due to wave generation and data acquisition requirements, a total of 44 individual tests were performed. Each test was run for 102 seconds (9 minutes prototype), and a different random wave signal was used to generate the required wave spectral characteristics being tested.

Eleven tests were performed for each of the 5, 20, 50 and 100 year recurrence interval joint probability design wave and water-level parameters described in Section 3. In this manner, run-up on the model structure was determined for a prototype storm duration of approximately 1.5 hours. The reason for undertaken this number of tests was to ensure that the full spectra of waves incident to the structure were measured.

In Appendix A the details of each test are tabulated. Figure 10 presents the results for mean run-up elevation (Rmean). In this and subsequent figures, the result of each individual test is identified by a unique label. The leading number identifies the corresponding joint probability recurrence interval for design parameters (i.e. 5, 20, 50, 100 years), and the letter identifies the individual test (11 per average recurrence interval, labelled 'a' to 'k' respectively). A second order polynomial is fitted to the data to assist in highlighting the trend. Figure 11 presents the results for significant run-up (Rsig), defined by the mean run-up elevation of the highest 1/3 of run-up events. Figure 12 presents the results for Rs2%, or the mean of the highest 2% of run-up events. The maximum run-up limit (Rmax) recorded during each test run is presented in Figure 13.

Figure 14 summarises the results of all tests for Rmean, Rsig, Rs2% and Rmax. As anticipated, an increase in elevation is associated for each run-up parameter with both increasing still water levels and wave height. Maximum run-up elevations of approximately 12 m AHD were recorded, with mean run-up elevations generally in the range 4.0 to 6.0 m AHD.

4.2 Over-topping

Following the completion of all run-up testing, the model was re-built to determine rates of wave over-topping. The height of the structure was reduced, and each of the three wave return walls were tested in turn. For aesthetic and costs reasons, the objective was to determine the minimum acceptable crest elevation. Random waves were used for the over-topping tests as they provide a more realistic assessment of over-topping discharge.

The *Design A* wave return wall was initially installed, and mean over-topping discharge determined for crest elevations of 10.0m AHD, 9.0m AHD and 8.0m AHD respectively. The crest elevation was lowered for each set of tests by the removal of upper rows of Seabee units and rock underlayer, and the excavation of core material. Data acquisition required testing was undertaken for each return-wall/crest-height combination in individual runs of 136 seconds (model), corresponding to 12 minutes prototype. A set of five tests were undertaken for each wall/height combination, corresponding to 60 minutes (1 hour) prototype design storm conditions. Mean wave over-topping discharge was determined by combining the results of each set of five tests.

Due to the unacceptable performance of the *Design A* wave return wall at a crest elevation of 8.0m AHD, *Design B* and *Design C* were tested, for a crest elevation corresponding to 8.0m AHD. The details of all over-topping test parameters are tabulated in Appendix B. A summary of the results for over-topping discharge are tabulated below in Table 1. These results are presented graphically in Figure 15.

TABLE 1
SUMMARY OF MEAN OVER-TOPPING DISCHARGE TEST RESULTS

Wave Return Wall	Crest elevation (m AHD)	Mean over-topping discharge ($m^3/m/s$)
Design A	10.0	3.93×10^{-3}
Design A	9.0	1.23×10^{-2}
Design A	8.0	2.72×10^{-2}
Design B	8.0	3.66×10^{-3}
Design C	8.0	3.65×10^{-3}

Average rates of wave over-topping discharge increased by an order of magnitude for wave return wall *Design A*, when the crest elevation was reduced from 10.0m AHD to 8.0m AHD. The performance of *Design B* and *Design C* was virtually identical, and at a crest elevation of 8.0 m AHD, relative to *Design A* resulted in a order of magnitude reduction in over-topping discharge.

4.3 Toe Stability

All preceding wave run-up and over-topping tests were performed using the immobile (i.e. fixed) bed at the toe of the structure, as described in section 3.4 (refer Figure 3). During approximately 70 individual tests performed during as part of the fixed bed (wave run-up and over-topping) test program, the toe of the Seabee structure was observed to be stable.

Following completion of the wave run-up and wave over-topping test program, the wave flume was drained, and the 0.9 m long (25 m prototype) removable section of floor immediately in front of the model structure removed. The sand box beneath this floor section was filled with fine sand (same sand used to construct the core of the model). Tests to specifically examine toe stability were then performed using this mobile bed.

4.3.1 Random Waves

Random wave testing was undertaken for the 100 year average RI storm wave height and still water level, with bed scour to −1.0 m AHD at the toe of the structure. A total of five individual tests were performed, for a total equivalent prototype storm duration of 60 minutes. The flume was then drained, and the mobile bed section at the toe and immediately seaward of the structure examined for scour. Figure 16 shows the bed at the completion of these tests. No additional scour was evident, instead some burial of the reno mattress occurred. Large amplitude bedforms were observed to develop seaward of the structure.

To test toe stability for scour beyond the design condition of −1.0 m AHD, a localised scour hole was dug by hand immediately in front of the reno mattress, to a depth of −2.0 m AHD, extending a distance 2.0 m (prototype) seaward of the structure (Figure 17). Random wave tests using the 100 year RI storm wave and still water level were then repeated. Figure 17 also shows the mobile section of bed following completion of these tests and draining of the flume. Rapid infilling of the −2.0 m AHD scour hole was observed. The toe of the Seabee structure remained stable.

4.3.2 Monochromatic Wave Tests

A further series of tests were undertaken using monochromatic waves and a range of water levels. The objective of these tests was to examine toe stability for worst case conditions of repeated waves breaking directly on and immediately seaward of the structure. At the start of each new test, bedforms developed in front of the structure during the previous test were removed, and the bed reformed to a scour elevation of −1.0 m AHD.

The qualitative results of these tests are summaries below in Table 2.

TABLE 2
TOE STABILITY – MONOCHROMATIC TESTS

SWL (m AHD)	Wave period (s)	Wave breaking characteristics	Comment
3.0	12	plunging waves breaking directly on toe	sand moving onshore, burial of toe occurring
3.0	13	plunging waves breaking directly on toe	reno mattress stable, some movement of material inside loose packed gabion basket
3.0	14	plunging waves breaking directly on toe	reno mattress stable, some movement of material in loose packed gabion basket
1.5	12	plunging waves breaking directly on toe	toe stable
1.5	12	spilling waves breaking approx. 100 m seaward	onshore sediment transport and accretion at toe of structure
0.0	14	plunging waves breaking directly on toe	toe stable
0.0	14	plunging waves breaking approx. 100 m seaward	toe stable
0.0	13	plunging waves breaking directly on toe	toe stable
0.0	13	spilling waves breaking approx. 100 m seaward	toe stable
0.0	12	plunging waves breaking directly on toe	toe stable
0.0	12	spilling waves breaking approx. 100 m seaward of	toe stable
4.0 (exceeds design conditions)	12	plunging waves breaking immediately seaward of toe	pronounced bedform development, movement of material in loose packed gabion basket, reno mattress stable
4.0 (exceeds design conditions)	12	plunging waves breaking directly on structure (structure failure)	significant shifting of Seabees in top 5 rows, Seabees stable below SWL, no evidence of additional scour at toe, reno mattress stable, movement of material in loose packed gabion basket. conclusion: toe stable

In summary, monochromatic tests indicate that the toe of the model structure was stable up to and exceeding the condition of armour layer instability (well in excess of design conditions).

4.4 Structure Stability

4.4.1 Design Waves/Water-levels

During the testing of the Wamberal TPS physical model, the Seabee structure was subjected to approximately 10 hours (prototype) of the 100 year RI design wave and water-level conditions, and for a similar combined time period for the 5, 20 and 50 year RI design parameters (refer Sections 2.1 – 2.2). During each phase of the testing program for wave run-up, wave over-topping discharge and toe stability, note was also taken of the stability of Seabee units and secondary underlayer.

In summary, the structure was found to be stable under all design wave and water-level conditions. Minor settling of individual Seabee units was observed, as the rock underlayer settled and compacted to a stable form. Localised loss of core material by seepage was noted beneath the geotextile fabric adjacent to the flume wall, however this was an artefact of the manner in which the model was constructed, rather than the prototype design. The problem was simply solved by sealing the gap between the edge of the geotextile fabric and the wall of the flume.

After running the physical model tests for an extended period of time, a subtle re-adjustment of the entire Seabee structure was noted. At the completion of all run-up, over-topping and stability testing, the slope of the front face had decreased marginally, with rotation occurring around the elevation coinciding with the intersection of the 100 year RI still water level and structure (refer Figure 18). This appeared to have had no effect on structure performance and stability. However, the observation of subtle structural re-adjustment emphasises that the final detailed design must incorporate careful consideration of the manner in which the structure is to be keyed into the existing dune slope. Additional geotechnical investigation/design is recommended.

4.4.2 Destructive Test

The gently sloping (1:33) sea bed in front of the model structure resulted in depth limited wave conditions, effectively preventing the impact of sufficiently large waves to result in structural failure. To determine conditions that would result in failure of the structure, one final test was undertaken. The raised floor of the flume was removed, along with the sand in the mobile section of the flume floor seaward of the toe. With the still water level set at the 100 year RI elevation of 3.0 m AHD, a water depth immediately seaward of structure of 9.0 m was achieved. Using monochromatic waves, the peak 100 year RI wave period of Tp = 14.4 seconds was selected, and then wave height slowly increased up to the point of structure failure. This condition is representative of a scenario where long-term beach recession has occurred along Wamberal Beach, resulting in exacerbated design conditions.

Destruction of the model Seabee structure was initiated at a wave height of approximately 5.5 m measured in 24 m water depth (equivalent to deepwater a wave height of 5.7 m). This compares favourably to the measured wave height of 5.3 m in 24 m water depth (5.5 m deepwater wave height) measured during toe stability testing. Eventual failure of the Seabee armour layer was noted after the still water level was raised 1.0 m above the 100 year RI design conditions to 4.0 m AHD.

5. CONCLUSIONS

Two-dimensional physical model testing has been undertaken in the random wave flume by WRL, to verify and refine the design of the proposed Terminal Protection Structure for Wamberal Beach, NSW. Testing was completed using both random and monochromatic waves, for the 5 year, 20 year, 50 year and 100 year recurrence interval combined storm wave/water-level design conditions. Extreme testing of the structure to the point of destruction was also undertaken.

The completed test program included detailed measurements and observations of:

- wave run-up on the Seabee armour
- structure over-topping discharge
- Seabee armour stability
- structure toe stability.

The principle findings of this physical model study are summarised below:

◊ The results of extensive random wave run-up testing confirm that in the absence of a wave return wall, a crest elevation in the range 11 – 12 m AHD is required.

◊ Wave over-topping tests undertaken using the preliminary design wave return wall (*Design A*) resulted in acceptable over-topping discharge for a crest elevation of 10.0 m AHD, but unacceptable discharge for lower crest elevations. Additional testing of two alternative wave return wall designs (*Designs B* and *C*) indicated acceptable rates of mean over-topping discharge for a reduced crest elevation of 8.0 m AHD.

◊ Both random wave and monochromatic wave testing of the gabion basket and reno mattress toe confirm toe stability for all design wave/water-level conditions. During these tests, local bed scour to −1.0 m AHD was assumed. In addition, the presence of an artificial scour hole down to −2.0 m AHD immediately in front of the reno mattresses, was observed to have no detrimental effect on toe stability.

◊ The stability of the Seabee armour units was observed at all stages of the testing program. The Seabee armoured layer of the TPS structure was found to be stable under all design wave and water-level conditions. Minor settling and re-adjustment of the rock under-layer was noted.

◊ A subtle re-adjustment of the entire Seabee structure was noted after the model testing program had been running for an extended period of time. The front face of the structure had decreased marginally, with rotation occurring around the elevation coinciding with the elevation of the 100 year RI still water level. This appeared to have had no effect on the hydraulic performance or stability of the structure. However, the observation of subtle structural adjustment emphasises that the final detailed design must incorporate careful consideration of the how the structure is to be keyed in to the existing dune slope.

◊ Depth limited conditions seaward of the structure effectively prevented the testing of the model TPS to destruction, under design wave/water-level conditions. By removal of the floor of the wave flume to simulate long term beach recession, sufficiently deep water immediately in front of the structure was achieved to result in structure failure. These conditions could only occur in the prototype if no action were taken to mitigate the complete and long-term recession of Wamberal Beach, effectively leaving the TPS structure stranded in several metres water depth.

The physical modelling investigations detailed in this report have resulted the successful verification and important refinement of the preliminary structure design. In particular, the wave return wall at the crest of the Seabee structure was redesigned to significantly reduce wave over-topping discharge. Physical modelling has proven a cost-effective and indispensable design tool to assist the final detailed design of the proposed Terminal Protection Structure at Wamberal Beach.

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WAMBERAL BEACH - LOCALITY

Figure 1

NOTE: It is proposed, that the Wamberal Beach TPS be mostly buried within the dunal sands.
This photograph is representative of the possible post-storm appearance of the structure.

Unisearch WRL Report No. 97/26	EXAMPLE OF SEABEE ARMOURED SEAWALL - CRONULLA BEACH, NSW	Figure 2

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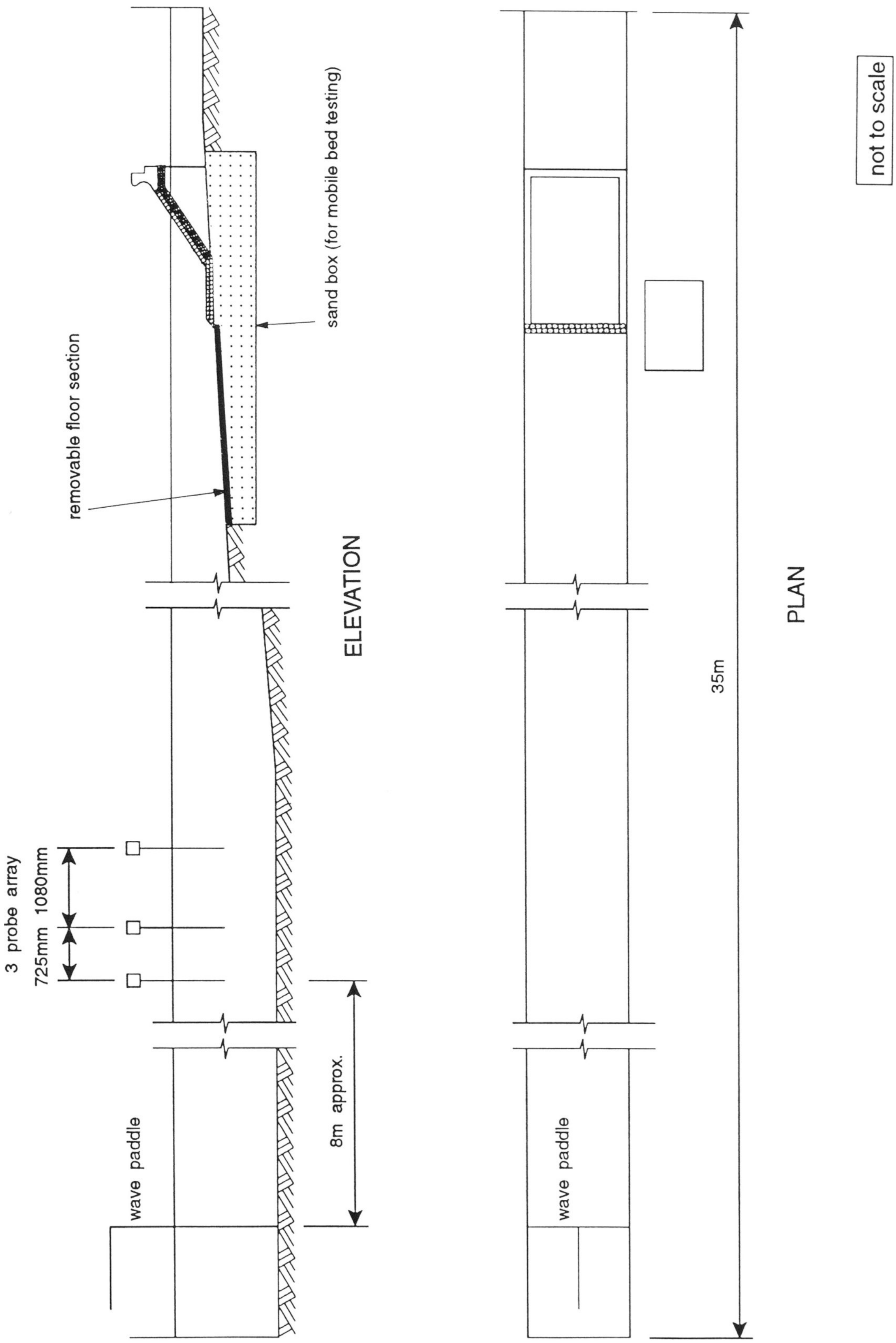
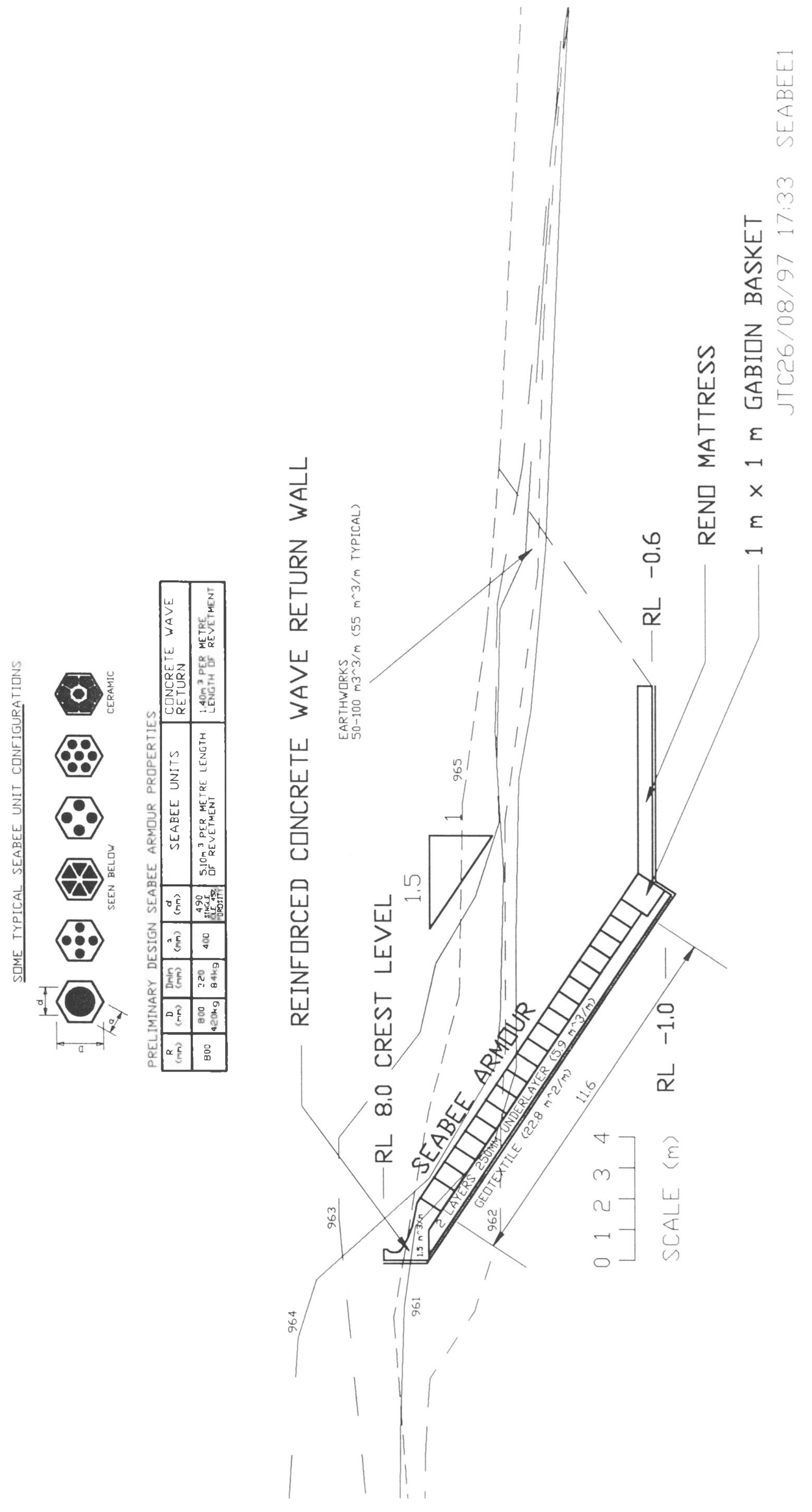
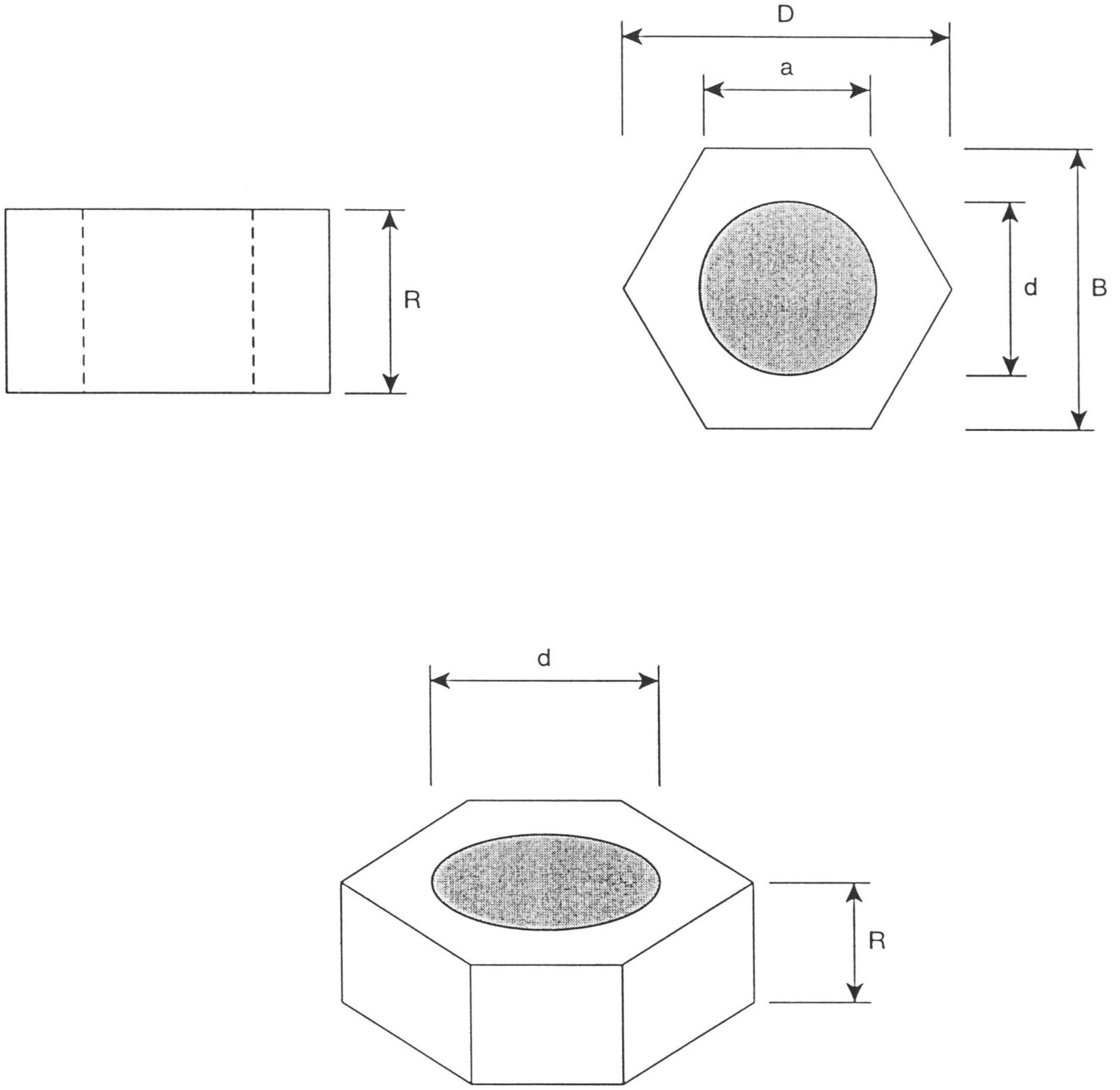


Unisearch WRL Report No. 97/26	WAVE FLUME AND 3D PROBE ARRAY LAYOUT	Figure 3



Unisearch WRL Report No. 97/26	PRELIMINARY DESIGN OF PROPOSED SEABEE STRUCTURE	Figure 4



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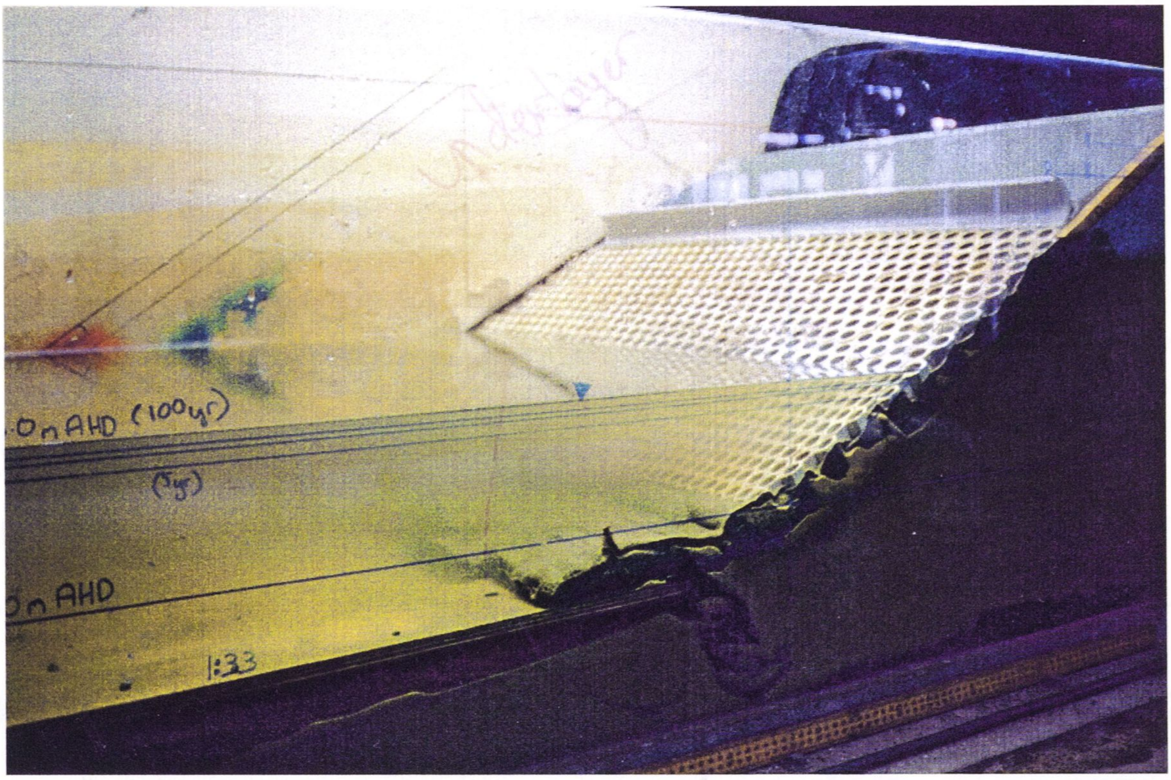
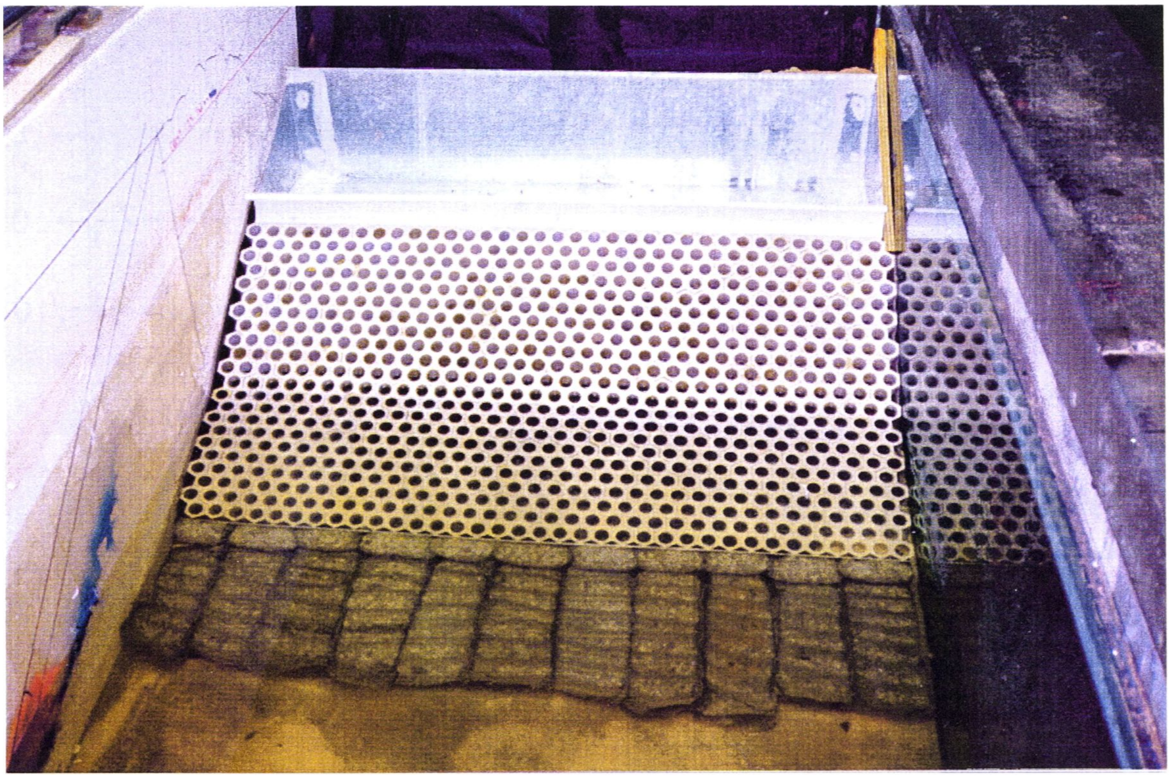
SEABEE DIMENSIONS

Figure
5





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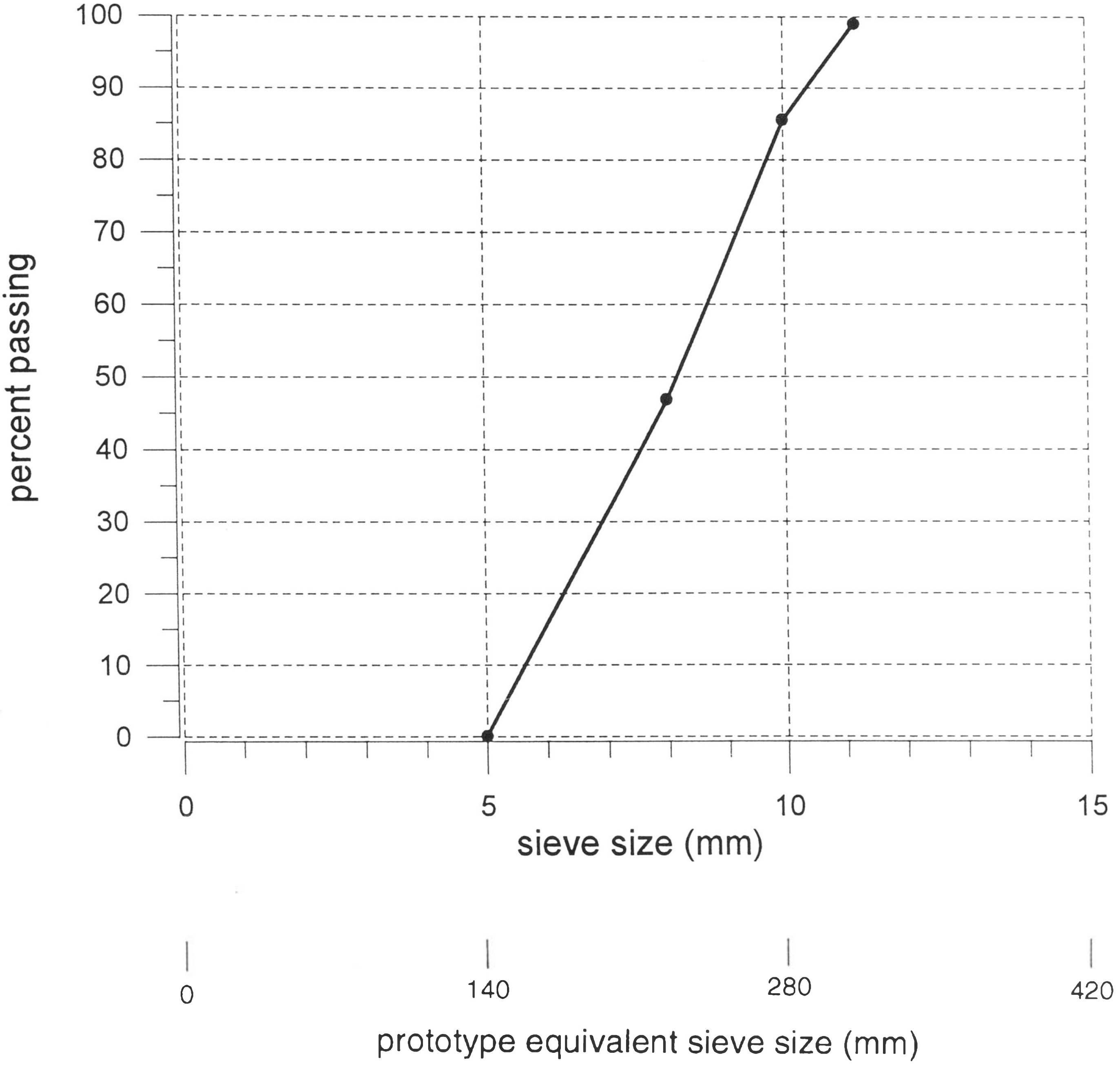
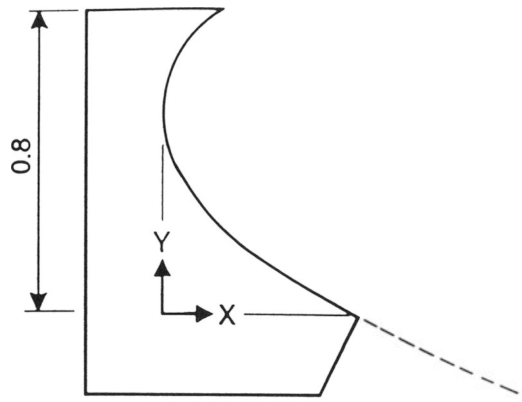
MODEL TEST SECTION

Figure 6

9726-6.cdr



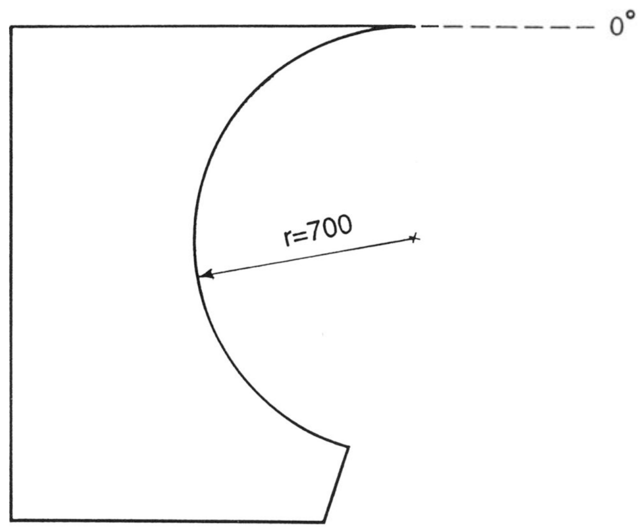
Unisearch WRL Report No. 97/26	GRADING CURVE FOR ROCK UNDERLAYER MATERIAL	Figure 7



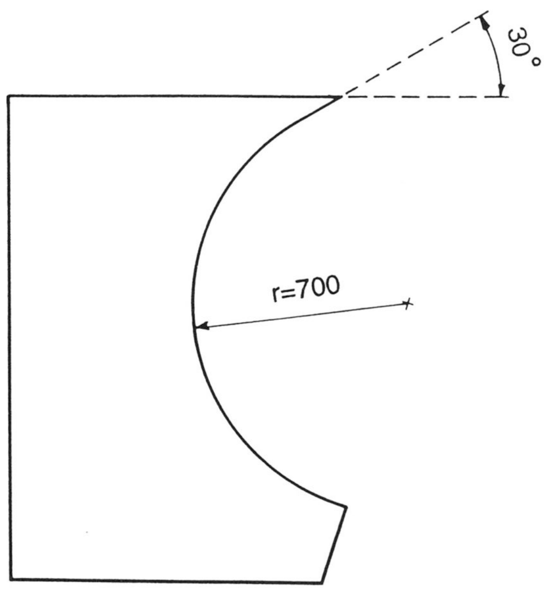
Co-ordinates of curved face

X	Y
0.52	0
0.339	0.1
0.197	0.2
0.093	0.3
0.028	0.4
0.001	0.5
0.012	0.6
0.062	0.7
0.151	0.8

Design A



Design B



Design C

not to scale

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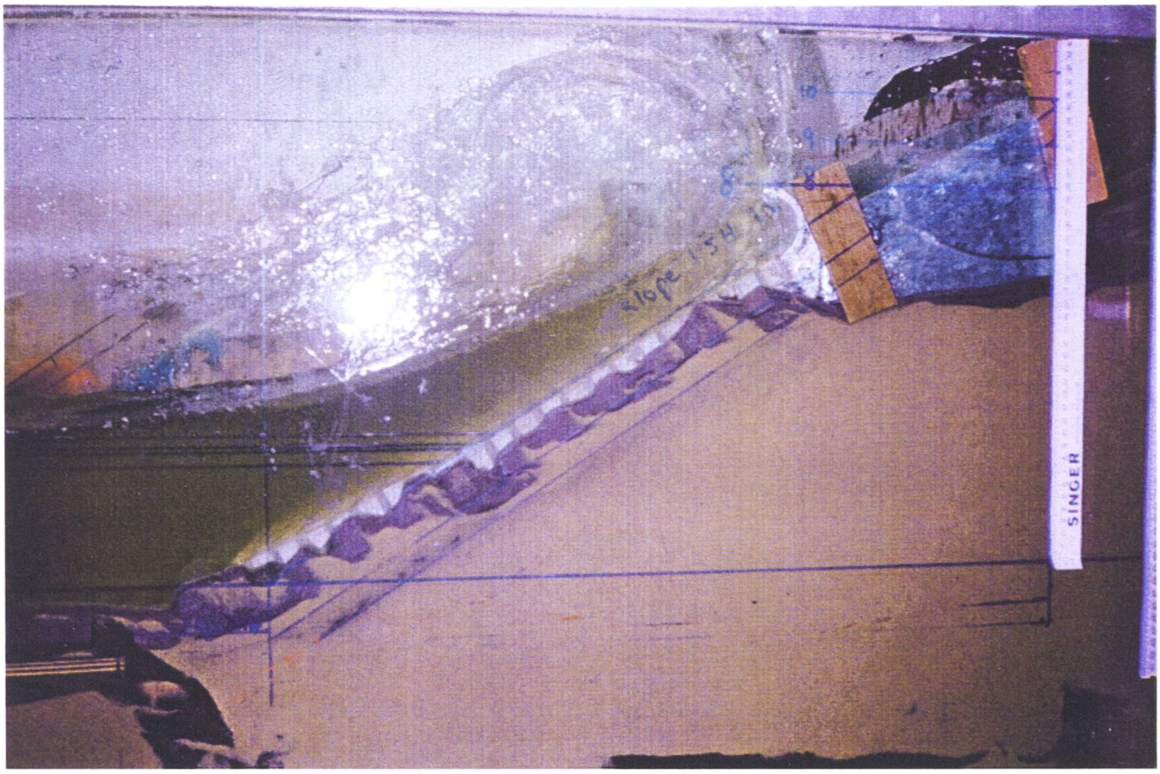
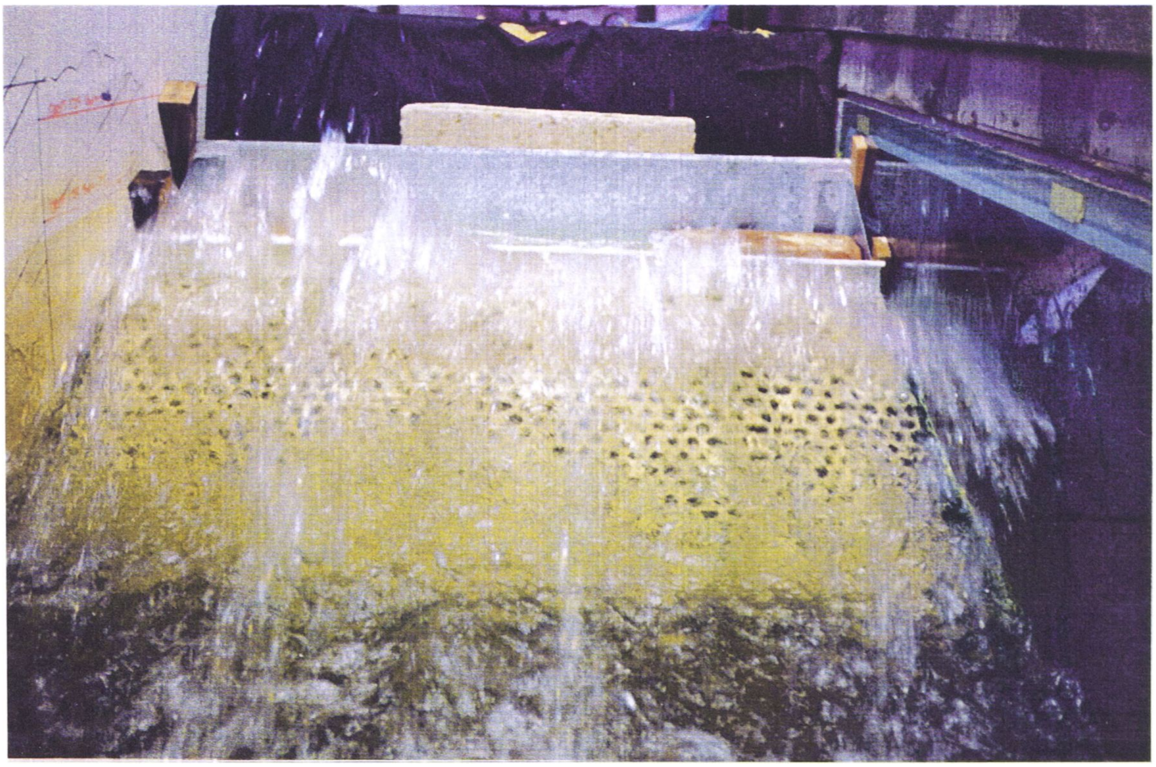
WAVE RETURN WALL DESIGNS A, B AND C

Figure 8





Unisearch WRL
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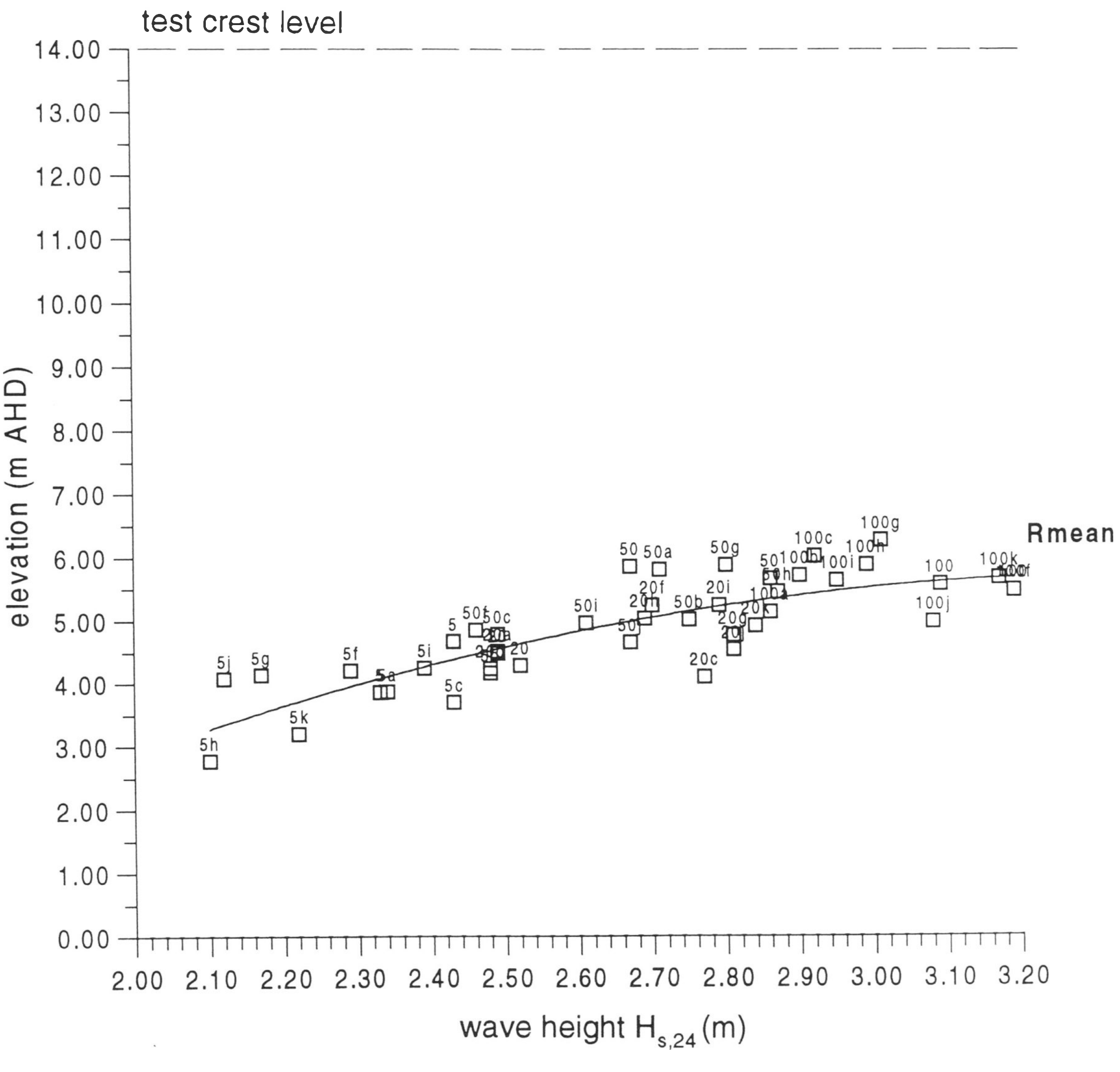
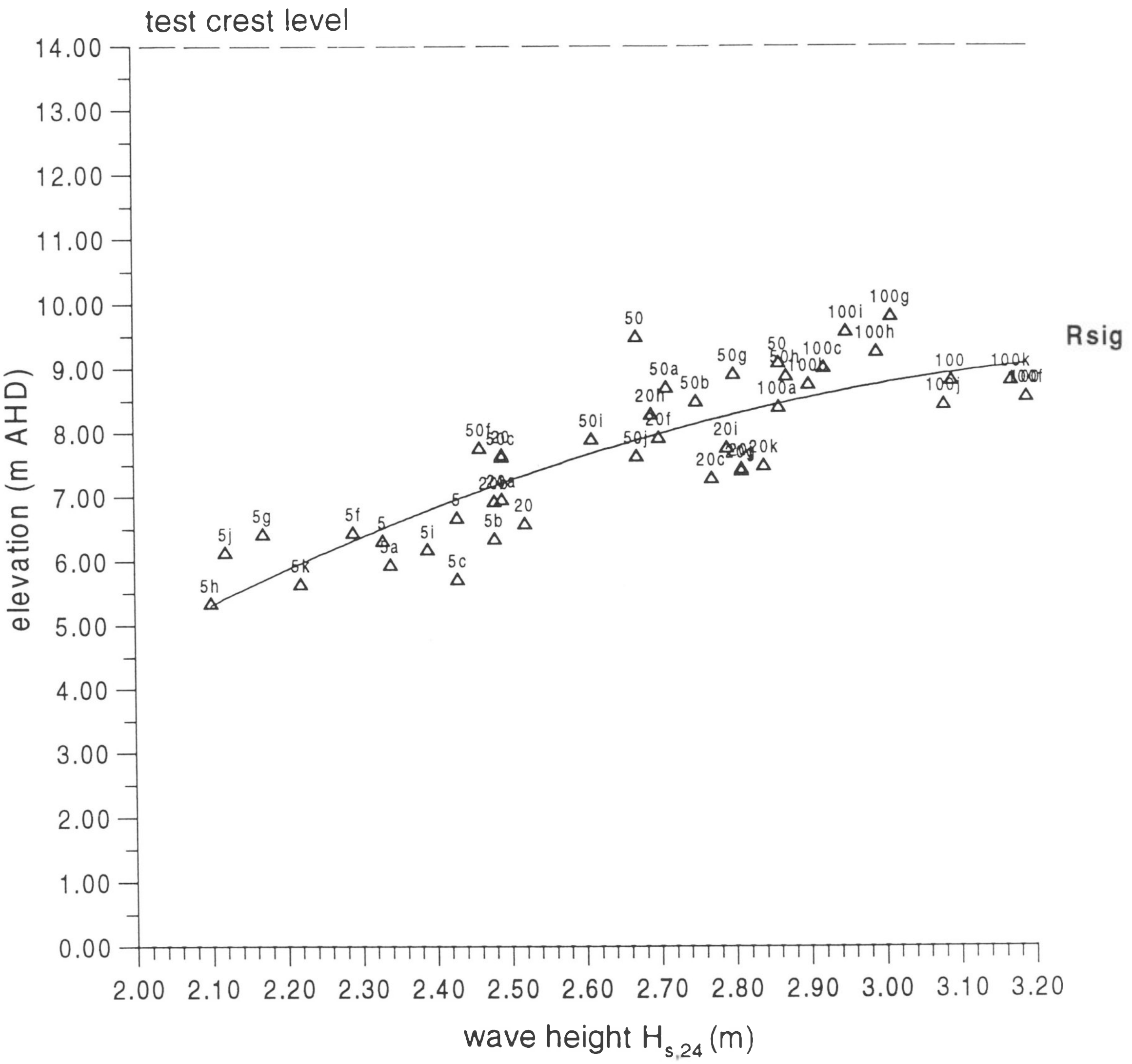
DESIGN C WAVE RETURN WALL
UNDER EXTREME DESIGN CONDITIONS

Figure
9

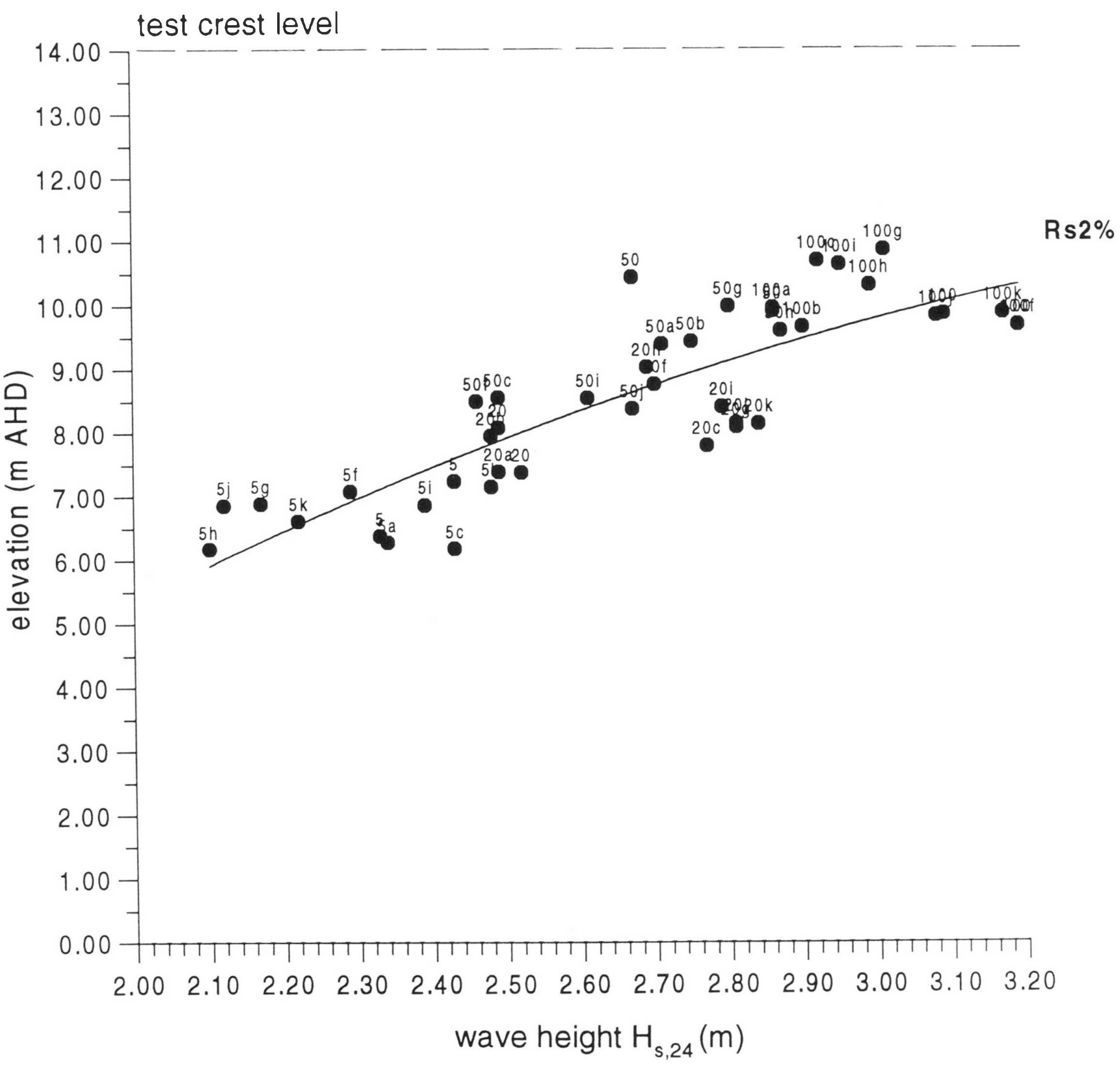
9726-9.cdr



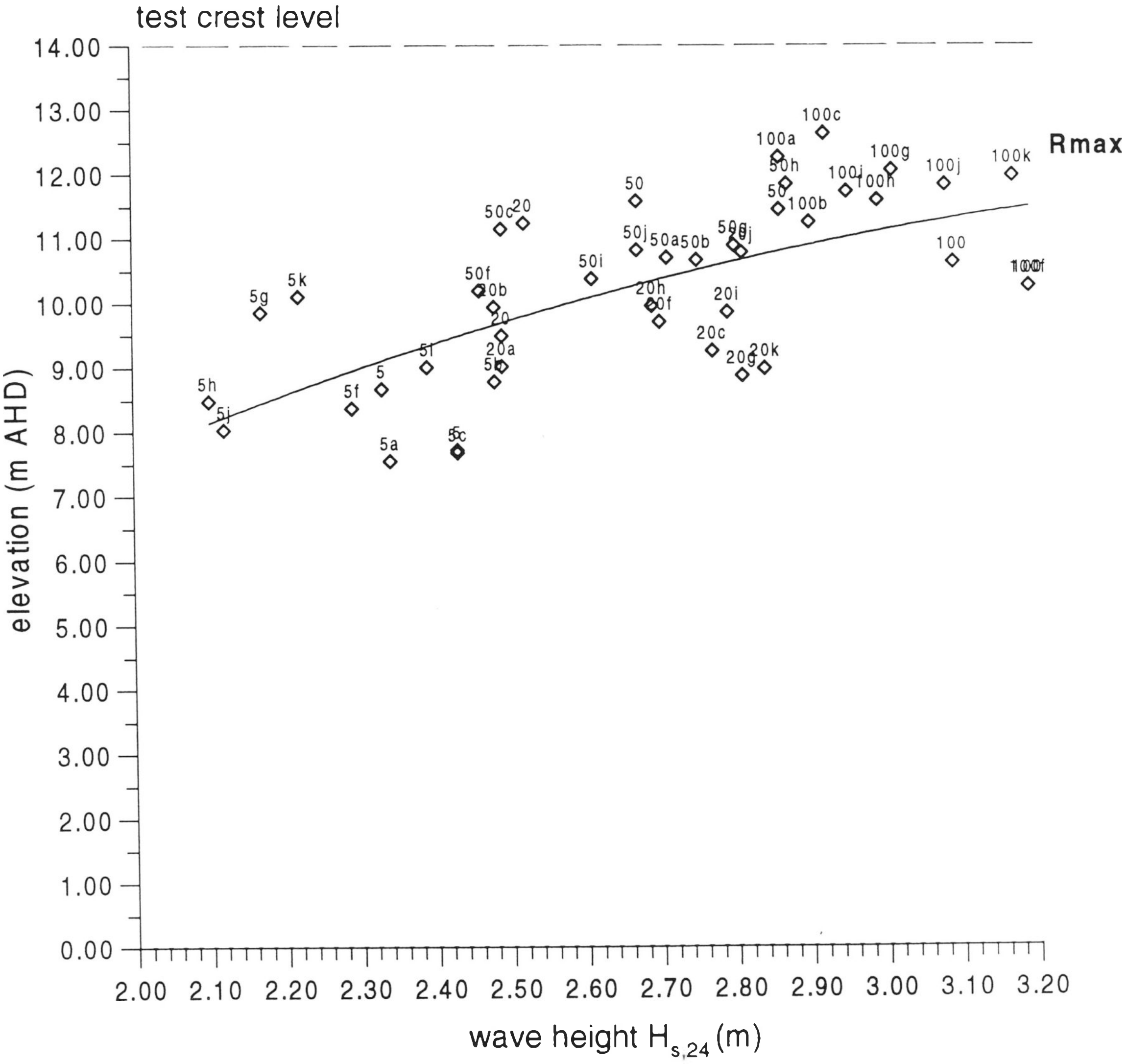
Unisearch WRL Report No. 97/26	RUN-UP TEST RESULTS - Rmean	Figure 10



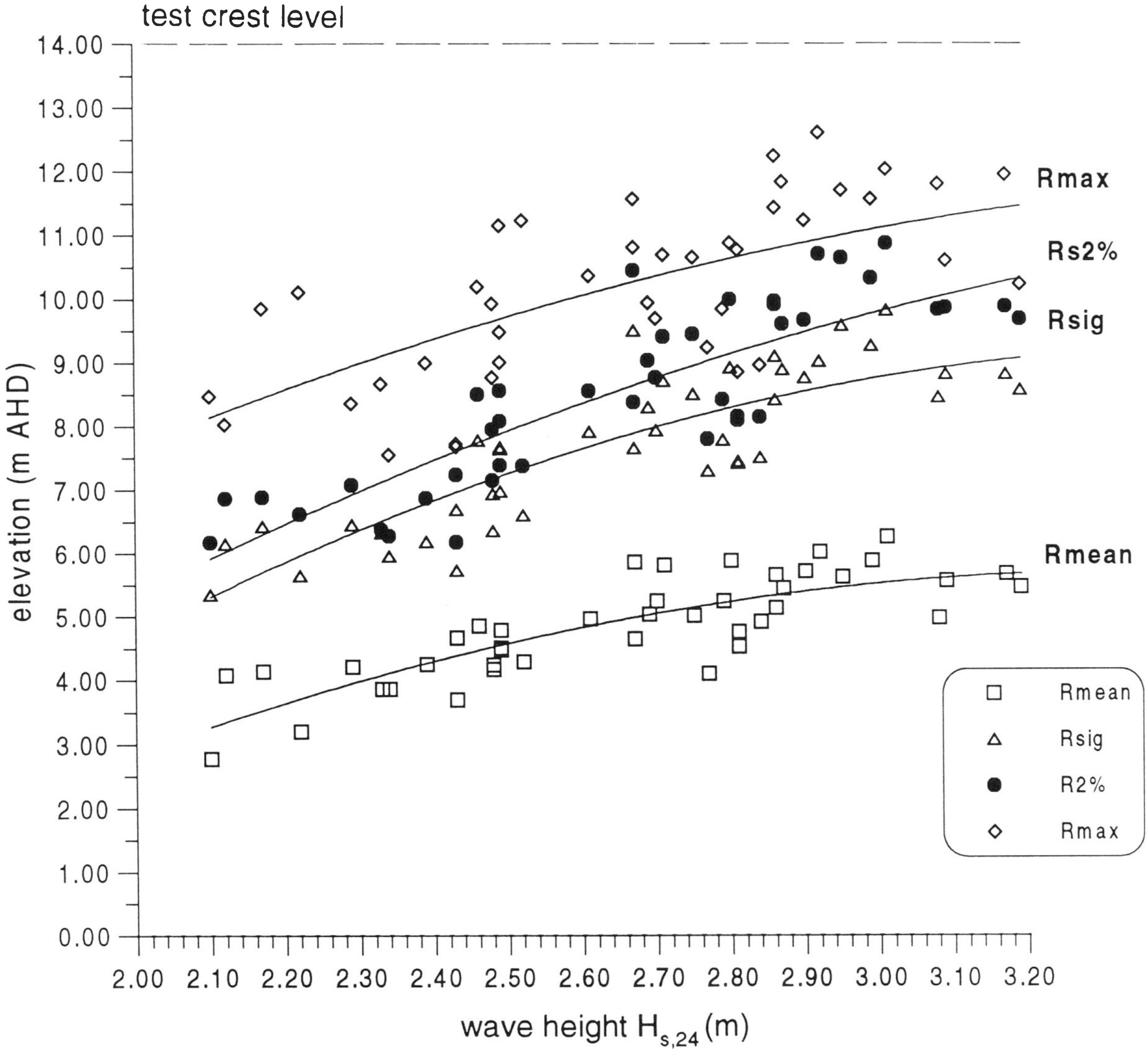
Unisearch WRL Report No. 97/26	RUN-UP TEST RESULTS - Rsig	Figure 11



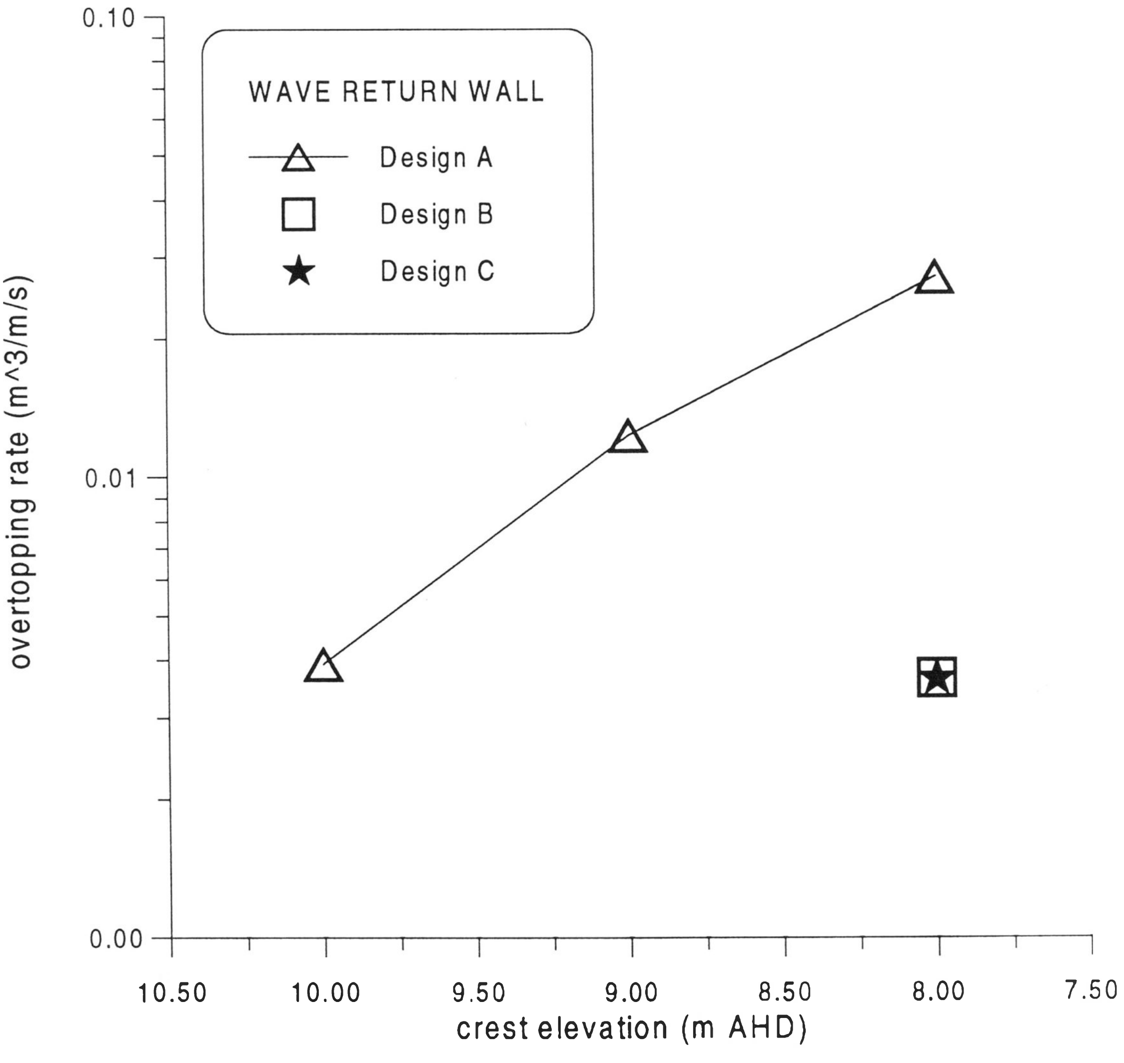
Unisearch WRL Report No. 97/26	RUN-UP TEST RESULTS - Rs2%	Figure 12



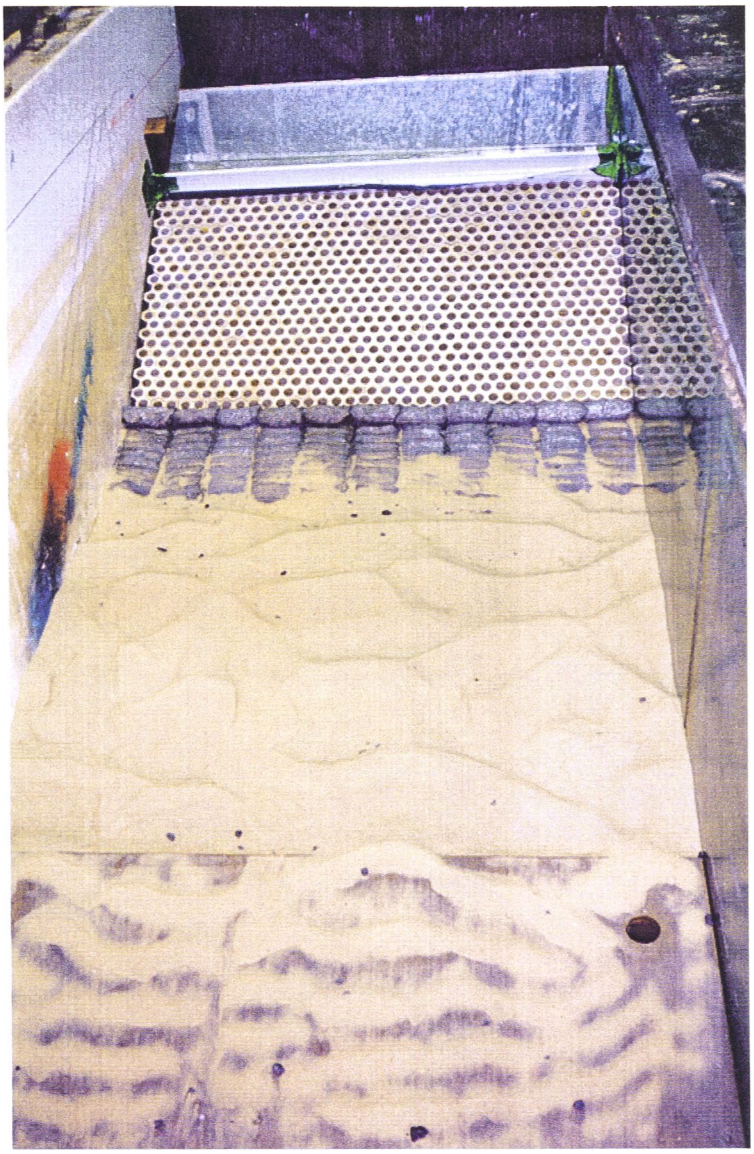
Unisearch WRL Report No. 97/26	RUN-UP TEST RESULTS - Rmax	Figure 13

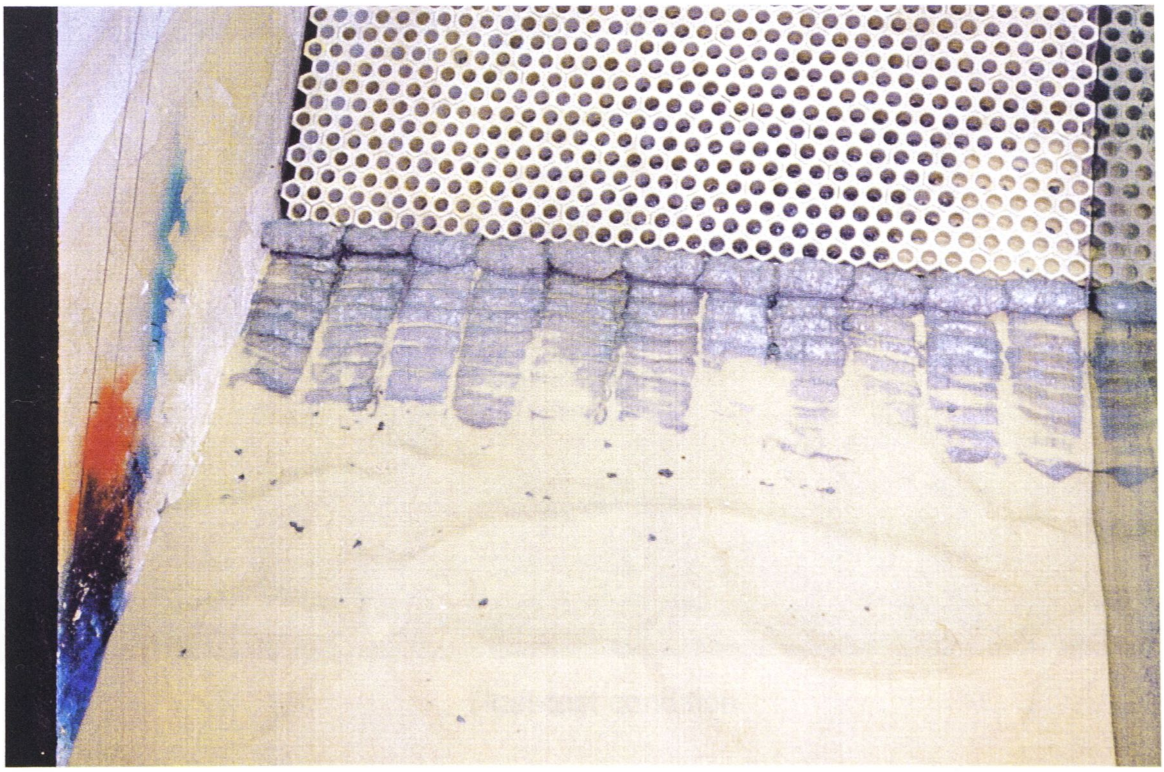


Unisearch WRL Report No. 97/26	SUMMARY OF ALL RUN-UP TEST RESULTS	Figure 14



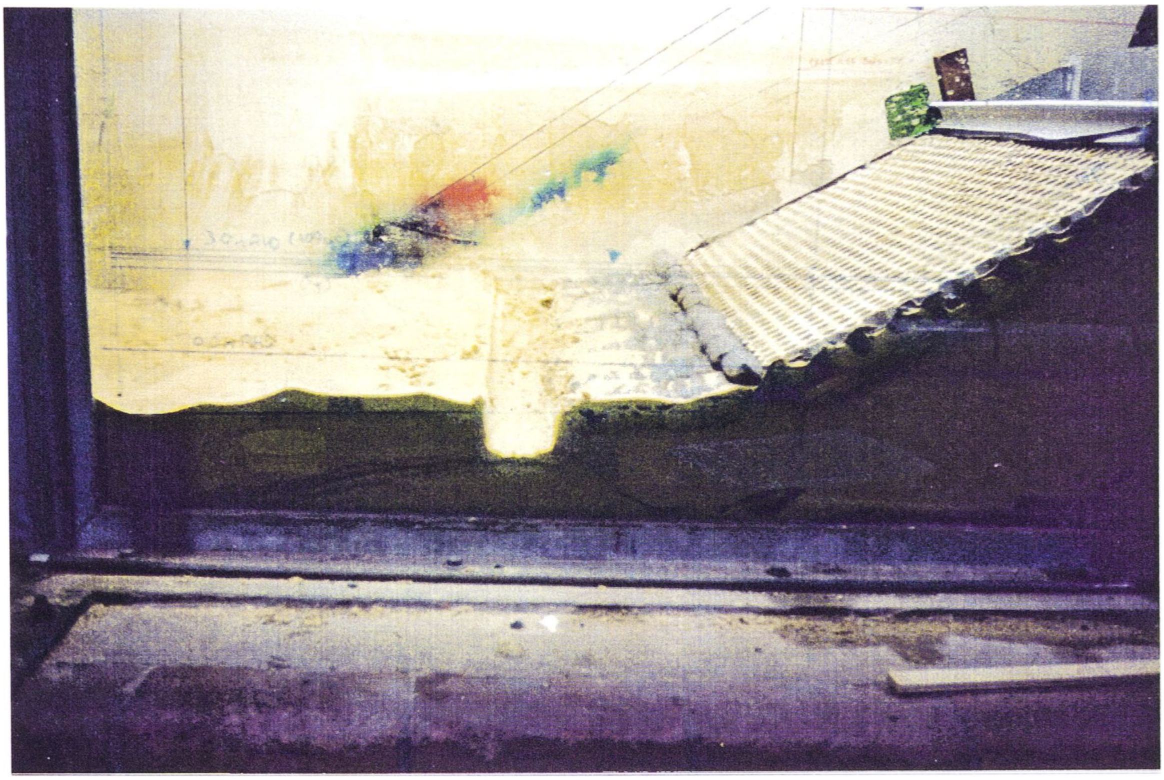
Unisearch WRL Report No. 97/26	OVER-TOPPING MEAN DISCHARGE TEST RESULTS	Figure 15



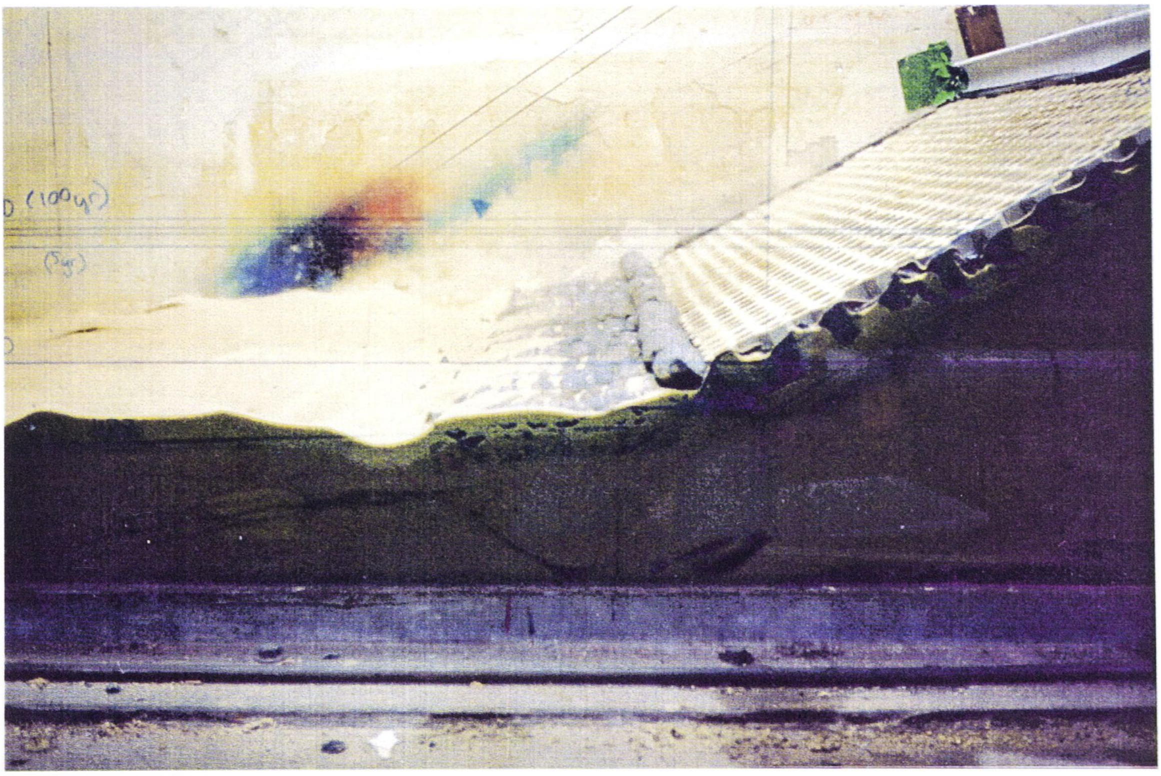


Unisearch WRL Report No. 97/26	MOBILE BED TEST SECTION FOLLOWING RANDOM WAVE TESTS	Figure 16

9726-16.cdr



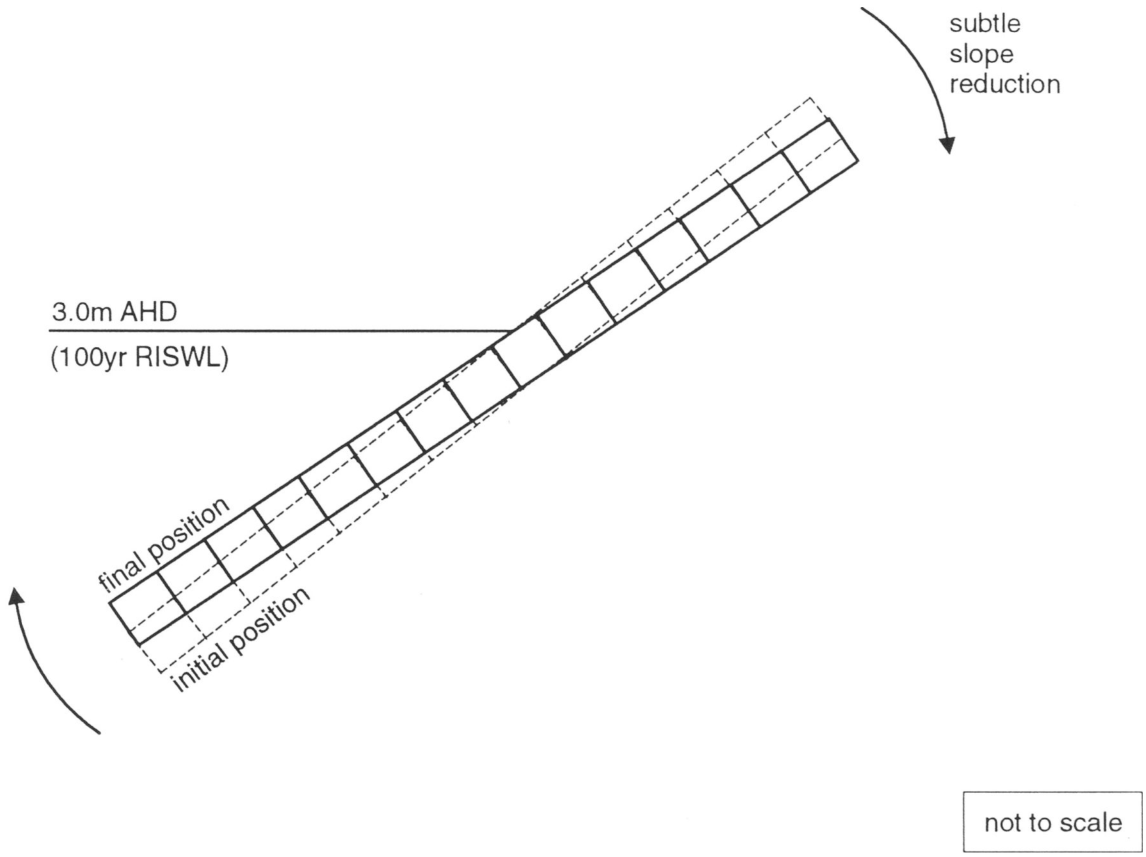
Pre-test condition



Post-test condition

Unisearch WRL Report No. 97/26	ARTIFICIAL SCOUR HOLE TO -2.0m AHD IN FRONT OF TOE	Figure 17

9726-17.cdr



Unisearch WRL Report No. 97/26

SKETCH INDICATING OBSERVED MINOR SLOPE ADJUSTMENT ON FRONT FACE OF STRUCTURE

Figure 18

APPENDIX A – RUN-UP TEST PARAMETERS AND RESULTS

APPENDIX A - RUN-UP TEST PARAMETERS AND RESULTS

Wamberal TPS - WRL97033												
WAVE/WATER-LEVEL CHARACTERISTICS									RUNUP			
FILE NAME	Hs(24) (m)	Tz (s)	SWL (m AHD)	TEST ID	DATA FILE	Hsi(24) (m)	Hsr(24) (m)	Kr	Rmax	Rmean	Rs	Rs2
WAM005a.dat	2.37	9.0	2.4	5a	RAW005a.dat	2.34	1.59	0.68	7.54	3.87	5.95	6.27
WAM005b.dat	2.37	9.0	2.4	5b	RAW005b.dat	2.48	1.62	0.65	8.76	4.17	6.35	7.14
WAM005c.dat	2.37	9.0	2.4	5c	RAW005c.dat	2.43	1.66	0.69	7.67	3.70	5.72	6.17
WAM005d.dat	2.37	9.0	2.4	5d	RAW005d.dat	2.43	1.76	0.72	7.71	4.67	6.68	7.23
WAM005e.dat	2.37	9.0	2.4	5e	RAW005e.dat	2.33	1.52	0.65	8.66	3.86	6.32	6.37
WAM005f.dat	2.37	9.0	2.4	5f	RAW005f.dat	2.29	1.40	0.61	8.36	4.21	6.45	7.07
WAM005g.dat	2.37	9.0	2.4	5g	RAW005g.dat	2.17	1.50	0.69	9.85	4.14	6.43	6.88
WAM005h.dat	2.37	9.0	2.4	5h	RAW005h.dat	2.10	1.40	0.67	8.48	2.78	5.35	6.17
WAM005i.dat	2.37	9.0	2.4	5i	RAW005i.dat	2.39	1.51	0.63	8.99	4.25	6.18	6.86
WAM005j.dat	2.37	9.0	2.4	5j	RAW005j.dat	2.12	1.43	0.67	8.03	4.08	6.15	6.86
WAM005k.dat	2.37	9.0	2.4	5k	RAW005k.dat	2.22	1.46	0.66	10.10	3.20	5.65	6.61
WAM020a.dat	2.53	9.5	2.7	20a	RAW020a.dat	2.49	1.51	0.61	9.00	4.51	6.97	7.38
WAM020b.dat	2.53	9.5	2.7	20b	RAW020b.dat	2.48	1.56	0.63	9.92	4.24	6.93	7.94
WAM020c.dat	2.53	9.5	2.7	20c	RAW020c.dat	2.77	1.81	0.65	9.23	4.10	7.29	7.78
WAM020d.dat	2.53	9.5	2.7	20d	RAW020d.dat	2.52	1.67	0.66	11.22	4.29	6.59	7.37
WAM020e.dat	2.53	9.5	2.7	20e	RAW020e.dat	2.49	1.54	0.62	9.47	4.48	7.66	8.07
WAM020f.dat	2.53	9.5	2.7	20f	RAW020f.dat	2.70	1.74	0.64	9.68	5.24	7.92	8.75
WAM020g.dat	2.53	9.5	2.7	20g	RAW020g.dat	2.81	2.00	0.71	8.84	4.76	7.41	8.08
WAM020h.dat	2.53	9.5	2.7	20h	RAW020h.dat	2.69	1.78	0.66	9.93	5.03	8.29	9.02
WAM020i.dat	2.53	9.5	2.7	20i	RAW020i.dat	2.79	1.81	0.65	9.83	5.24	7.77	8.40
WAM020j.dat	2.53	9.5	2.7	20j	RAW020j.dat	2.81	1.88	0.70	10.76	4.53	7.44	8.14
WAM020k.dat	2.53	9.5	2.7	20k	RAW020k.dat	2.84	1.77	0.62	8.95	4.91	7.49	8.13
WAM050a.dat	2.66	9.8	2.8	50a	RAW050a.dat	2.71	1.71	0.63	10.68	5.80	8.70	9.39
WAM050b.dat	2.66	9.8	2.8	50b	RAW050b.dat	2.75	1.63	0.59	10.64	5.01	8.49	9.43
WAM050c.dat	2.66	9.8	2.8	50c	RAW050c.dat	2.49	1.77	0.71	11.14	4.78	7.63	8.55
WAM050d.dat	2.66	9.8	2.8	50d	RAW050d.dat	2.67	1.77	0.66	11.56	5.85	9.49	10.43
WAM050e.dat	2.66	9.8	2.8	50e	RAW050e.dat	2.86	1.87	0.66	11.42	5.65	9.09	9.90
WAM050f.dat	2.66	9.8	2.8	50f	RAW050f.dat	2.46	1.72	0.70	10.18	4.85	7.77	8.49
WAM050g.dat	2.66	9.8	2.8	50g	RAW050g.dat	2.80	1.88	0.67	10.87	5.87	8.91	9.98
WAM050h.dat	2.66	9.8	2.8	50h	RAW050h.dat	2.87	1.87	0.65	11.82	5.44	8.88	9.59

WAM050i.dat	2.66	9.8	2.8	50i	RAW050i.dat	2.61	1.85	0.71	10.35	4.96	7.90	8.54
WAM050j.dat	2.66	9.8	2.8	50j	RAW050j.dat	2.67	1.87	0.70	10.80	4.65	7.64	8.37
WAM100a.dat	2.82	10.1	3.0	100a	RAW100a.dat	2.86	1.94	0.67	12.23	5.13	8.40	9.95
WAM100b.dat	2.82	10.1	3.0	100b	RAW100b.dat	2.90	2.04	0.70	11.22	5.70	8.75	9.65
WAM100c.dat	2.82	10.1	3.0	100c	RAW100c.dat	2.92	1.97	0.68	12.60	6.01	9.01	10.69
WAM100d.dat	2.82	10.1	3.0	100d	RAW100d.dat	3.09	2.13	0.69	10.59	5.56	8.81	9.85
WAM100e.dat	2.82	10.1	3.0	100e	RAW100e.dat	3.19	2.25	0.70	10.22	5.46	8.56	9.67
WAM100f.dat	2.82	10.1	3.0	100f	RAW100f.dat	3.19	2.25	0.70	10.22	5.46	8.56	9.67
WAM100g.dat	2.82	10.1	3.0	100g	RAW100g.dat	3.01	2.03	0.68	12.02	6.25	9.81	10.86
WAM100h.dat	2.82	10.1	3.0	100h	RAW100h.dat	2.99	1.98	0.66	11.56	5.87	9.26	10.31
WAM100i.dat	2.82	10.1	3.0	100i	RAW100i.dat	2.95	1.90	0.64	11.70	5.62	9.57	10.63
WAM100j.dat	2.82	10.1	3.0	100j	RAW100j.dat	3.08	2.22	0.72	11.80	4.97	8.44	9.82
WAM100k.dat	2.82	10.1	3.0	100k	RAW100k.dat	3.17	2.36	0.74	11.94	5.66	8.81	9.87

APPENDIX B – OVER-TOPPING TEST PARAMETERS AND RESULTS

APPENDIX B - OVER-TOPPING TEST PARAMETERS AND RESULTS

WRL97033 WAMBERAL TPS - overtopping							
RETURN WALL:		**A**	**A**	**A**	**B**	**C**	
CREST ELEVATION:		**10.00**	**9.00**	**8.00**	**8.00**	**8.00**	
wave file	**Hs(24)**		**water removed during testing (ml)**				
WAM100a.dat	2.78	1000	8000	22140	0	0	
WAM100b.dat	2.77	2000	6000	17310	0	0	
WAM100c.dat	2.72	5000	13150	21940	0	0	
WAM100d.dat	3.01	3000	7650	24200	0	0	
WAM100f.dat	3.01	4680	14280	23030	14630	14590	
total volume (model):		15680	49080	108620	14630	14590	**ml**
total volume (proto):		344207360	1.077E+09	2.384E+09	321157760	320279680	**ml**
total volume (proto):		344	1077	2384	321	320	**m^3**
OVERTOPPING RATE		3.93E-03	1.23E-02	2.72E-02	3.66E-03	3.65E-03	**m^3/m/s**