Public Works
Manly Hydraulics Laboratory

ACW BLOCK SEAWALL TESTING
2D PHYSICAL MODEL

Report MHL 2276
April 2014

prepared for
Australian Coastal Walls Pty Ltd

A division of the Department of Finance and Services

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ACW Block Seawall Testing Physical Model

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Foreword

This report was prepared by NSW Public Works Manly Hydraulics Laboratory (MHL) for Australian Coastal Walls Pty Ltd. The report presents examples and results of the testing of the proposed design cross-sections carried out by Indra Jayewardene and Oliver Light of NSW Public Works MHL in its 2D wave flume. The report was written by Indra Jayewardene and Oliver Light and was published by Megan Callaghan.

Summary

In response to discussions with Patrick Johnson of Australian Coastal Walls Pty Ltd (ACW), NSW Public Works Manly Hydraulics Laboratory (MHL) conducted 2D physical modelling of stability and overtopping of a proposed generic design **(Appendix A)** for a sea wall utilising the ACW block **(Appendix A).**

The model testing of the proposed design of the sea wall resulted in the following conclusions and recommendations:

- Wave condition and water level. The wave conditions tested were characterised by waves with high surf similarity parameters (exceeding that typical of plunging waves) and resulted in a large number of surging waves. The incident wave conditions were evaluated using reflection analysis (**Appendix B**). Testing was carried out at water levels covering a range of predicted high water levels for the relevant return periods and also took into consideration sea level rise (SLR) due to climate change.

- The structure was tested at extreme high water levels (100-year ARI (Average Recurrence Interval) and greater) resulting in extreme broken wave conditions at the structure on the coastline.

- Stability of sea wall section 1 – placement density 11.9 units/m^2, 3.45 m AHD (Australian Height Datum) crest level (**Figure 3.1**). Tested at water levels 1.5 m AHD (100-year ARI water level), 1.6 m AHD (1-year ARI +0.4 m for SLR water level) and 1.9 m AHD (100-year ARI + 0.4 m for SLR water level). The testing indicated that at this placement density no damage to the structure was observed.

- Stability of sea wall section 2 – placement density 10.8 units/m^2, 3.45 m AHD crest level. Tested at water levels 1.5 m AHD (100-year ARI water level), 1.6 m AHD (1-year ARI +0.4 m for SLR water level) and 1.9 m AHD (100-year ARI + 0.4 m for SLR water level). The testing indicated that at this placement density no damage to the structure was observed at water levels 1.5 m AHD and 1.6 m AHD. At 1.9 m AHD three units were displaced, resulting in less than 1% damage.

- Average wave overtopping values for a seawall crest height of 3.45 m AHD. Wave overtopping estimates at a water level of 1.5 m AHD were found to be acceptable and meet the criteria for a lightly protected promenade. Overtopping at 1.6 m AHD indicated conditions are unsafe for pedestrians, albeit acceptable for a lightly protected promenade. At 1.9 m AHD, conditions would be unacceptable for pedestrians, albeit acceptable for vehicles moving at low speeds. At higher crest heights overtopping would be reduced. A wave deflector could also be used to reduce wave overtopping.

- Testing modes of failure. Although the structure did not fail during testing it is customary to initiate possible modes of failure and test these modes under design conditions. Two possible modes of failure were tested, the first consisting of a single unit being removed from the structure. The subsequent test using 2000 waves did not result in any progressive deterioration to the stability condition of the structure. The second mode of failure consisted of two adjacent units being removed. Similar to the previous result there was no further deterioration to the stability of the structure.

- Other possible modes of failure – loss of underlayer material. Suitable filter materials should be incorporated in the prototype design in order to satisfy filter rules and avoid washout of underlayer materials as well as ensure efficient drainage of overtopping waves. The leaching of sand from behind the model structure highlighted the structural significance of utilising a suitable geofabric filters such as Terrafix 1200R, 900R and Elcomax 600R.

- Other possible modes of failure - toe scour. Since sand cannot be scaled accurately in a Froude model, scour was not modelled. The toe was pinned to the floor to ensure that no toe movement took place during the modelling. Notwithstanding this modelling constraint, toe scour is expected to be a possible mode of failure for this structure. As for any rigid coastal structure, instability at the toe can lead to progressive and/or sudden collapse of the structure. As such, proper toe design by a suitably experienced coastal engineer should be considered mandatory to ensure the stability of this structure.

- Scaling effects. For the average return intervals tested (1 year to 100 years) the criteria for scaling effect used indicate that there would be negligible scaling effects during testing.

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1. Introduction

1.1 Project Background

NSW Public Works Manly Hydraulics Laboratory (MHL) conducted two-dimensional (2D) physical modelling of stability and wave overtopping of a proposed generic design for Australian Coastal Walls Pty Ltd. The shape and measurements of the ACW block are indicated in **Figures A1** and **A2**. The cross-section of a wall made from the block is indicated in **Figures A3** and **A4**. The block is comprised of a compartment wall and two wings which lock the individual blocks together. When these wings and the compartment walls of the blocks are aligned, by using a dry stacking method, a retaining wall is created. The front pockets of the block are designed to provide friction and thereby reduce wave orbital velocities and dissipate wave energy. The dry weight of a prototype concrete block is 75 kg. The tests are designed to replicate a typical wall built to the standard recommended layout.

1.2 Scope of Works and Study Objectives

The specifications and objectives of the proposed study, as set out in a brief prepared in conjunction with ACW are as follows:

- sloping seabed with a block structure, non–moving bed
- 1 cross-section – 1 generic bathymetry
- 6 wave/water level scenarios (**Table 2.1**)
- 2 placement densities for block (standard and approximately 10% lower)
- average wave overtopping discharge in order to assess pedestrian safety (EuroTop 2007) for selected water level and wave height scenarios which resulted in critical wave breaking conditions
- scale at 1:10
- the prototype wall height simulated was 3.45 m and could be built higher according to site specific requirements.

2. Methodology and Test Conditions

2.1 General

The physical model testing was carried out in NSW Public Works MHL's random wave facility (**Figure 2.1**). A schematic of the generic profile of the sea wall design tested is shown in **Appendix A**. The profile details for testing were provided by ACW. Wave generation was accomplished by a sliding wedge wave paddle driven by a servo-hydraulic system (**Figure 2.1**). The paddle is controlled by a digital input signal. The user defines the peak frequency of a Pierson-Moskowitz (PM)/Jonswap spectra to be generated. The computer controls the data acquisition from the wave recording probes as well as the probes recording water level.

Well respected software from the Danish Hydraulic Institute (DHI) is used for wave analysis and presentation. The DHI software uses industry standard methodology (Zelt and Skjelbreia 1992). This is an extension to the three-probe reflection measurement (Funke and Mansard 1980 1987). This methodology uses four or five gauges in order to obtain the largest possible frequency range for which the bands around the singularities do not coincide for all internal gauge distances. Together with the flume transfer function, this method is used during the tests to differentiate between the incoming wave and the reflected wave to a high degree of accuracy. The stability parameters of the design and model were equal. The test program, water levels and design wave conditions have been provided by ACW in the project brief and are presented in **Table 2.1**.

Three water levels were utilised for the tests. The 1-100-year ARI of 1.5 m AHD, the 1-in-1-year ARI water level of 1.2 m AHD and an additional 0.4 m increment (total 1.6 m AHD) for future sea level rise, and also the 1-100 year ARI with the 0.4 m increment (total 1.9 m AHD). These water levels resulted in extremely high broken wave conditions on the shoreline in close proximity to the structure. Two additional stability tests were conducted after the removal of one unit at the water line, with a further test conducted following the removal of a second unit from a position diagonal to the previously moved unit (**Appendix C**).

2.2 Model Scales

A length scale of 10 was chosen on the basis of the dimensions of the structure to be modelled, water levels, wave heights and the need to minimise scale effects (Cornett 1995). The scaling condition for armour specified by Cornett (1995) and Dai and Kamel (1969) is estimated to be satisfied for the proposed primary armour/seawall at a scale of 1:10 or larger.

The model scales selected for the study were:

Length scale	L_r	=	10		
Time scale	T_r	=	$\sqrt{L_r}$	=	3.16
Average Overtopping Rate/Unit Length Scale		=	$(L_r)^{3/2}$	=	31.6

2.3 Construction of the Structure in the Wave Flume

The structure was constructed in the wave flume with a representative from ACW in attendance. Initially fine sand (**Figure E1**) was utilised to represent compacted rubble behind the structure. However, due to problems from slow and gradual leaching, fine rubble and blue metal (**Figure E2** and **E3**) were subsequently utilised in the model to prevent leaching and also act as a suitable filter. Suitable filter materials should be incorporated in the prototype design in order to satisfy filter rules and avoid washout of underlayer materials as well as ensuring efficient drainage of overtopping waves. The leaching of sand from behind the model structure highlighted the structural significance of utilising a suitable geofabric filters such as Terrafix 1200R, 900R and Elcomax 600R.

2.4 Measurement of Wave Overtopping, Wave Forces and Wave Reflection

NSW Public Works MHL has extensive experience in the physical modelling of wave forces on structures, breakwater wave overtopping and stability. Irregular wave testing using PM/Jonswap spectra was used for the testing. Wave grouping effects play a significant role when both wave overtopping and armour stability are design considerations.

Average wave overtopping conditions were simulated by selecting suitable time series with appropriate wave grouping characteristic conditions using the significant wave height (H_s), spectral peak period (T_p) and water levels (including allowance for sea level rise conditions) as provided by ACW. Wave overtopping performance was also qualitatively captured by video footage.

2.5 Testing Procedure

The test schedule is shown in **Table 2.1**. Each stability test was conducted for at least 2000 waves.

Table 2.1 Test Schedule

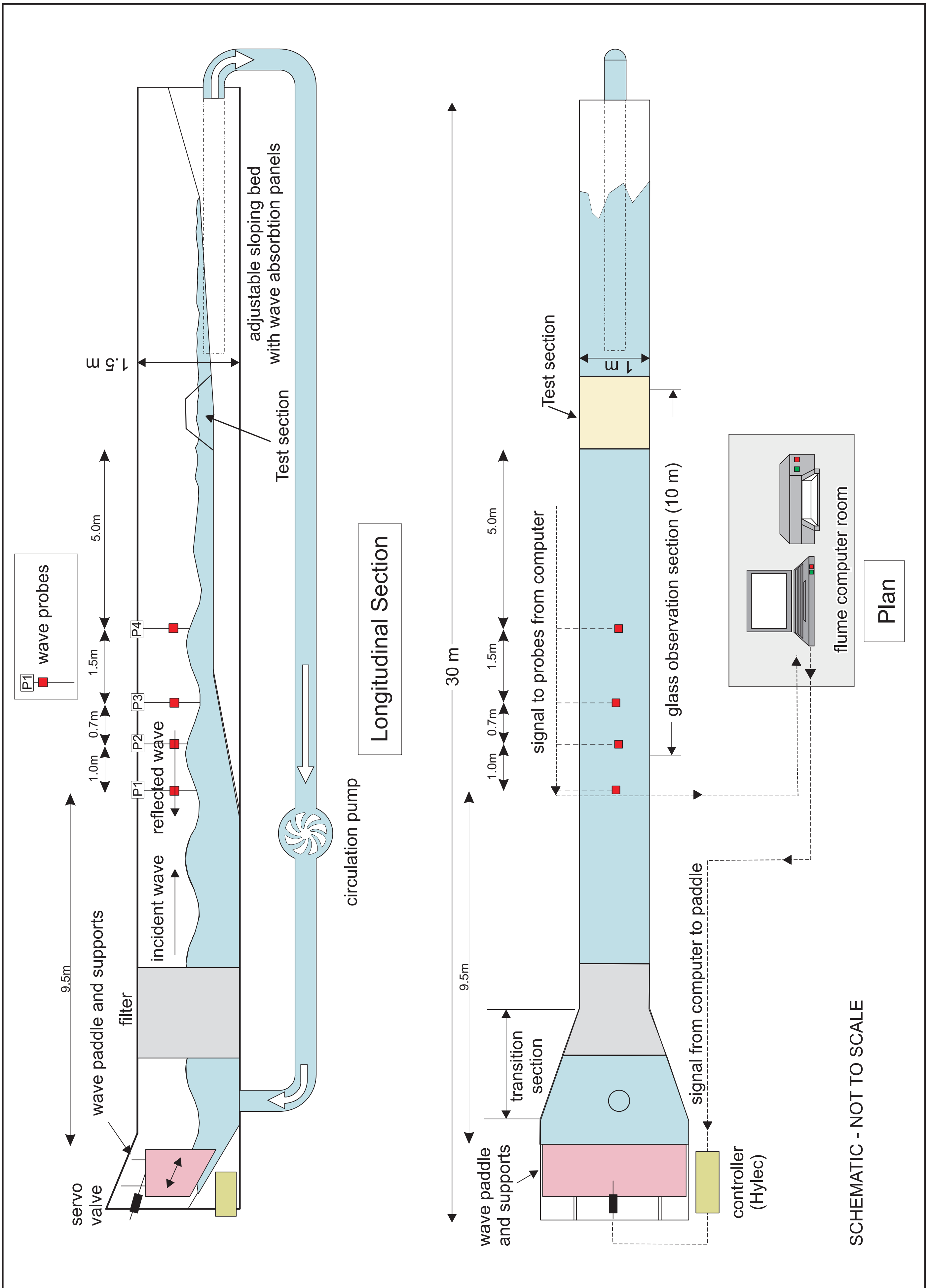
Cross-section Placement Density (blocks/m^2) and Description	Design Conditions/ Number of Waves	Seawall Crest Level	Measurement Regime
1. 11.9 with filter cloth placed between unit and backfill (**Figures 3.1-3.4**)	1. 1-yr ARI water level + SLR* for 2000 waves 2. 100-yr ARI water level + SLR for 2000 waves 3. Any water level agreed by client for 2000 waves	3.45 m	The maximum H_s in proximity to the structure was at least 2.2 m. Wave overtopping and reflection measurements carried out as well
2. 10.8 or any number requested by ACW, with filter cloth placed over the sand (**Figures 3.5-3.10**)	1. 1-yr ARI water level + SLR* for 2000 waves 2. 100-yr ARI water level + SLR for 2000 waves 3. Any water level agreed by client for 2000 waves	3.45 m	The maximum H_s in proximity to the structure was at least 2.2 m. Wave overtopping and reflection measurements carried out as well

*A 50-year SLR of 0.4 m

Outcome

- Structural stability.
- Average wave overtopping estimates (L/m/s) under breaking wave conditions.

All wave overtopping tests were carried out for an adequate length to simulate the effects of wave grouping on the structure (Jayewardene et al. 1993).



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FLUME LAYOUT

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Figure 2.1

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3. Model Test Results

3.1 Test Results

Table 3.1 presents the results for the stability and wave overtopping tests conducted. **Figures 3.1** to **3.10** present a selection of photographs taken during the model tests. These figures provide examples of both the wave overtopping and breaking wave stability tests.

The most extreme wave overtopping occurs when testing the cross-sections with normal (zero obliquity) wave direction (EuroTop 2007). Three-dimensional testing may be carried out to clarify and address any further issues relating to the influence of wave obliquity and influence of structure shapes.

Table 3.1 Stability and Wave Overtopping Results

Test No.	Wave Event	Water Level (m AHD)	Overtopping Measured (L/m/s)	Comment on Overtopping	Hs, Tp (probe 50 m from structure)	Stability
Cross-section – Placement Density 11.9 units/m^2						
1	100-year water level and breaking waves	1.5	8.8-10.8	Acceptable for properly attired, trained staff (see section 3.5). Meets criteria for lightly protected promenade	2.28, 8.10	No damage, stable
2	1-year +SLR, water level and waves breaking offshore	1.6	13.35-15.8	Unsafe for pedestrians and acceptable for lightly protected promenade (see section 3.5)	2.21, 8.10	No damage to wall, capping displaced
3	100-year + SLR and waves breaking offshore	1.9	26.8-30.3	Unacceptable for pedestrians and acceptable for vehicles moving at low speeds	2.15, 8.10	No damage to wall, capping displaced
Cross-section – Placement Density 10.8 units/m^2						
4	100-year water level and breaking waves	1.5	8.8-10.8	Not applicable	2.35, 8.10	No damage, stable
5	1-year +SLR, water level and waves breaking offshore	1.6	13.35-15.8	Not applicable	2.34, 8.10	No damage to wall, capping displaced
6	100-year + SLR and waves breaking offshore	1.9	26.8-30.3	Not applicable	2.26, 8.10	Wall deemed to be stable (<1% damage), capping displaced

Test No.	Wave Event	Water Level (m AHD)	Overtopping Measured (L/m/s)	Comment on Overtopping	Hs, Tp (probe 50 m from structure)	Stability
Testing Possible Failure Modes						
7	1 unit removed from structure	1.9	Not applicable	Not applicable		No additional damage, stable
8	2 units removed from structure	1.9	Not applicable	Not applicable		No additional damage, stable

The measured wave overtopping of 10 L/m/s to 30 L/m/s was less than the theoretical values of 70 L/m/s to 100 L/m/s for all the tests (EuroTop 2007). This may be attributed to waves breaking in front of the structure (**Appendix C**). Also, since a single wave can result in overtopping rates that are 100 times greater than the average (van der Meer 1994) the discrepancy may be attributed to the lack of high waves reaching the structure due to wave breaking occurring further offshore. The influence of a wave deflector shape on reducing wave overtopping has not been estimated.

3.2 Acceptable Overtopping Rates for Pedestrians and Vehicles

The measured overtopping quantities indicate that relatively high overtopping values could occur at the design cross-section (**Appendix A**) for the design wave heights. Criteria for acceptable overtopping rates and spray intensities have been developed by many institutions (PIANC 2003, EAUK 2007). EuroTop (EAUK 2007) indicates that the following mean overtopping rates are allowable for pedestrians:

- 1-10 L/s/m for properly attired trained personnel under conditions of no falling jet, safe walkway and overtopping flows at low levels
- 0.1 L/s/m for aware pedestrians with a clear view of the sea and wider walkway.

A further precautionary limit of 0.03 L/m/s might apply for conditions where pedestrians have no clear view of incoming waves

EuroTop (EAUK 2007) indicates that the following mean overtopping limits are allowable for vehicles:

- 10-50 L/s/m, driving at low speed, overtopping by pulsating flows at low flow depths, no falling jets, vehicles not immersed and location defined at a highway
- 0.01-0.05 L/s/m, driving at moderate or high speed, impulsive overtopping giving falling or high velocity jets, as defined at a sea defence.

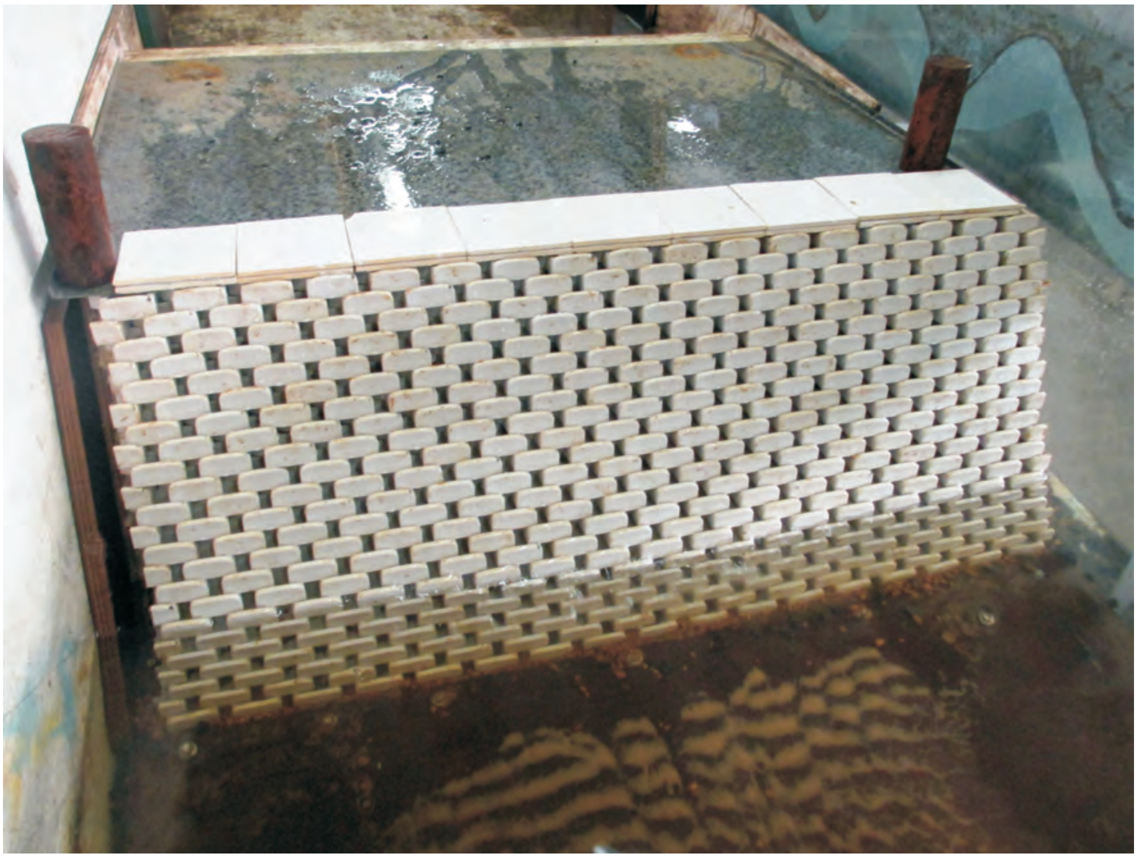
EuroTop (EAUK 2007) indicates the following mean overtopping limits are allowable for property behind the defence:

- 1 L/s/m, building structure elements
- 0.4 L/s/m, damage to equipment set back 5-10 m.

EuroTop (EAUK 2007) indicates that the following mean overtopping limits are allowable for damage to the defence crest or rear slope:

- 200 L/s/m, damage to paved or armoured promenade behind seawall
- 50 L/s/m, damage to grassed or lightly protected promenade or reclamation cover.

Hence, a value of 10 L/s/m could be utilised for trained personnel depending on the shape of the jet and 0.4 L/s/m for equipment 5-10 m behind the crest. It is noted that untrained, general public pedestrians would be considered at risk under such conditions.

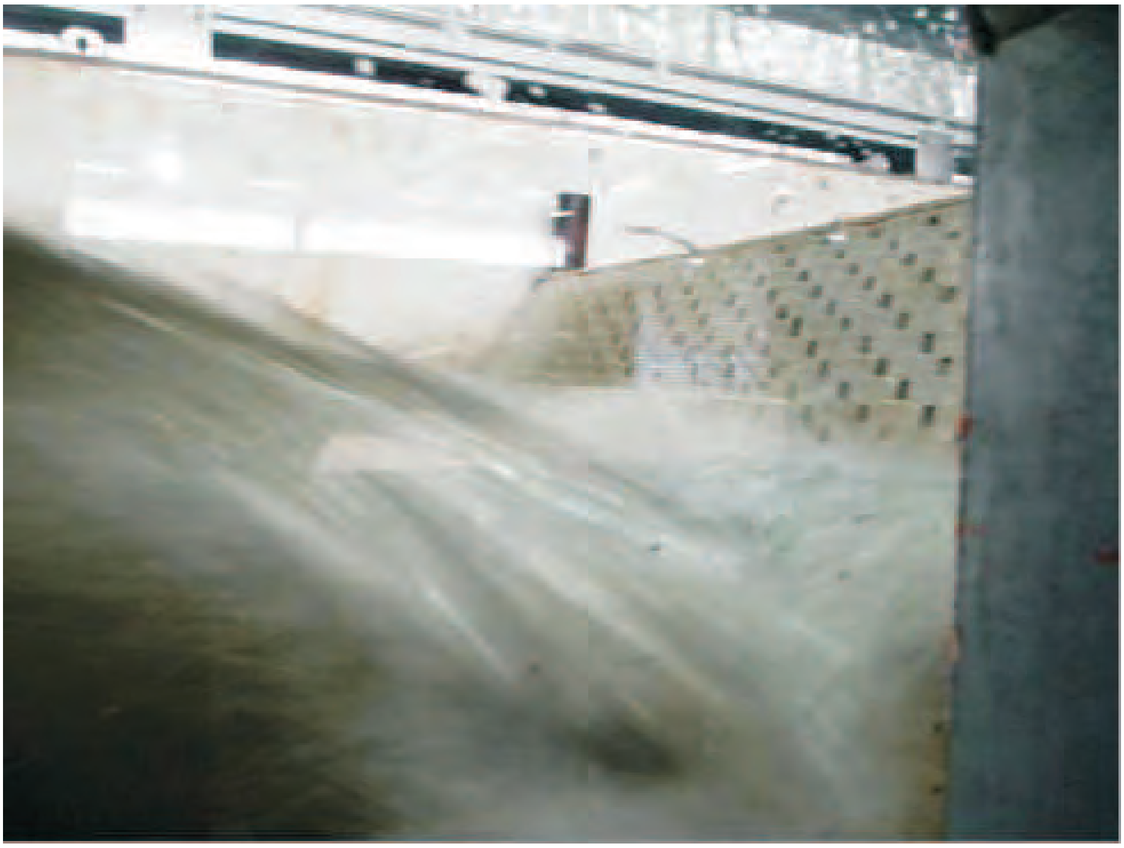


a) Cross-section 1 - placement density 11.9 units/m^2, prior to testing

b) Cross-section 1 - placement density 11.9 units/m^2, WL 1.5m AHD, wave overtopping during testing

a) Cross-section 1 - placement density 11.9 units/m^2, WL 1.5m AHD, after testing with 2000 waves, view from above, no damage

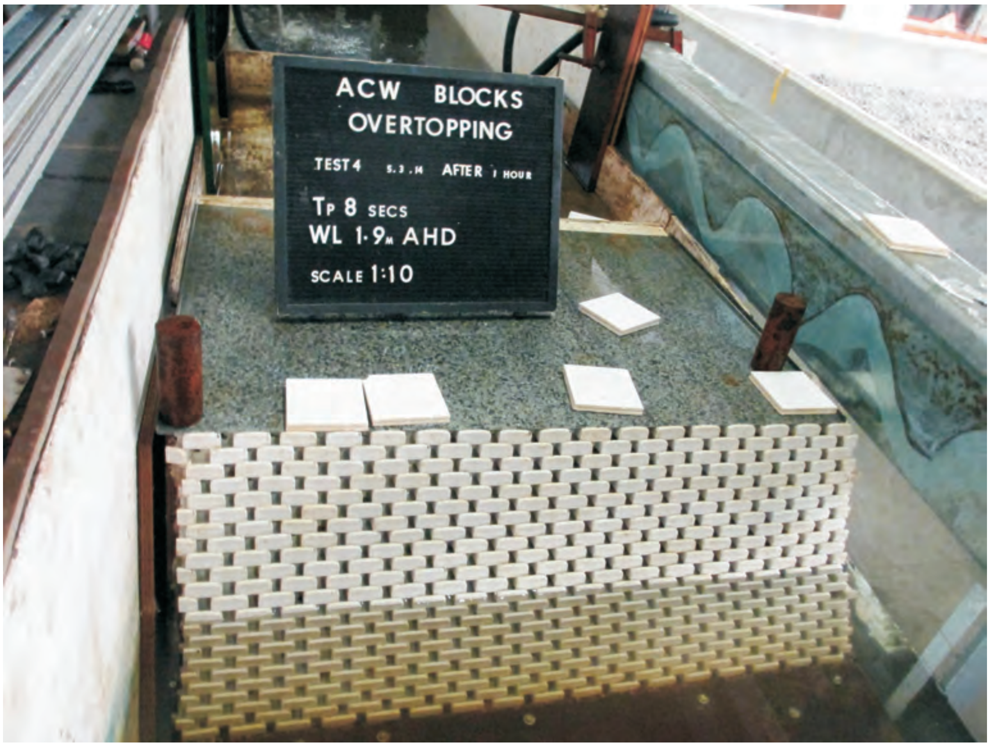
b) Cross-section 1 - placement density 11.9 units/m^2, WL 1.6m AHD, wave overtopping structure during testing



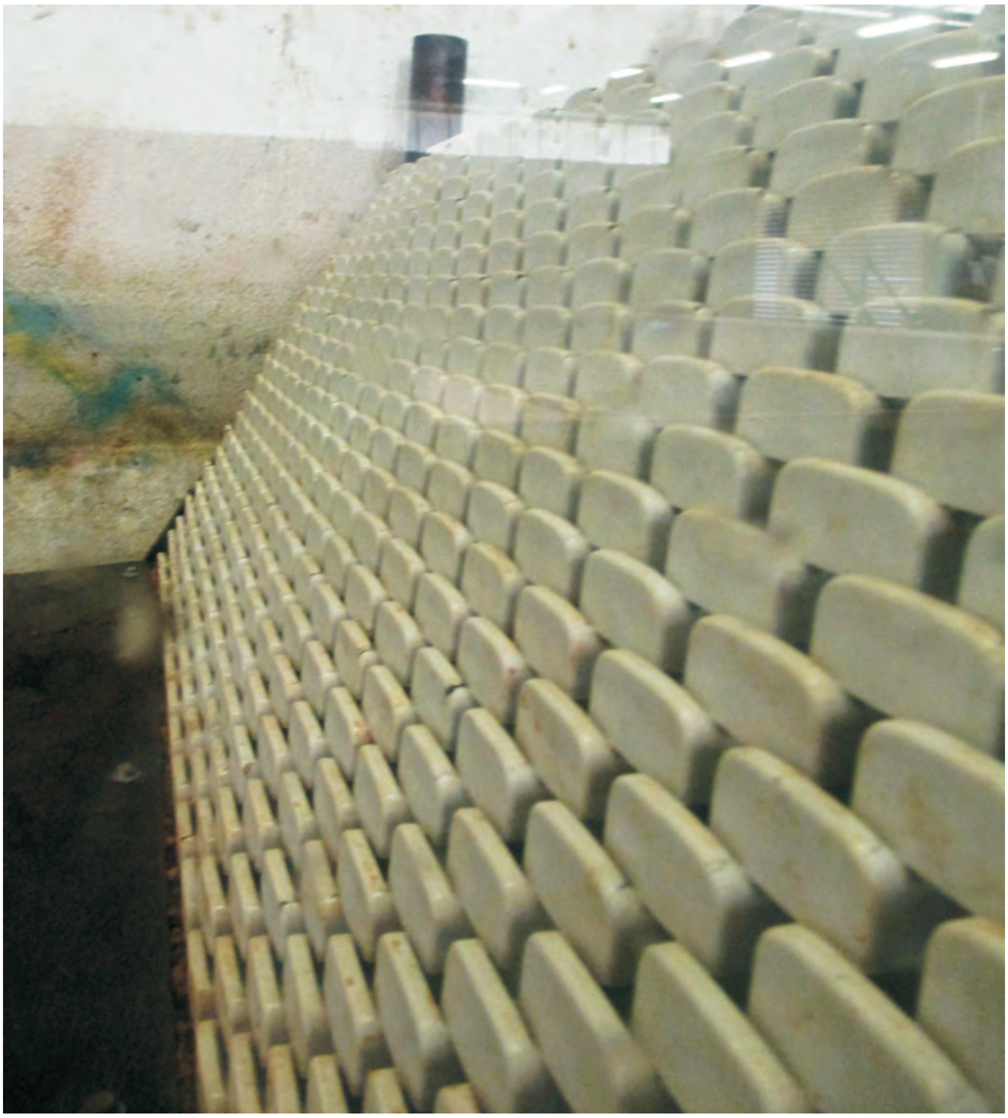
a) Cross-section 1 - placement density 11.9 units/m^2, WL 1.6m AHD, wave breaking on structure during testing

b) Cross-section 1 - placement density 11.9 units/m^2, WL 1.6m AHD, structure after testing indicating no damage

a) Cross-section 1 - placement density 11.9 units/m^2, WL 1.9m AHD, during testing



b) Cross-section 1 - placement density 11.9 units/m^2, WL 1.9m AHD, after 2000 waves, capping dislodged



c) Cross-section 1 - placement density 11.9 units/m^2, WL 1.9m AHD, after 2000 waves, no damage to wall

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Figure
3.4c

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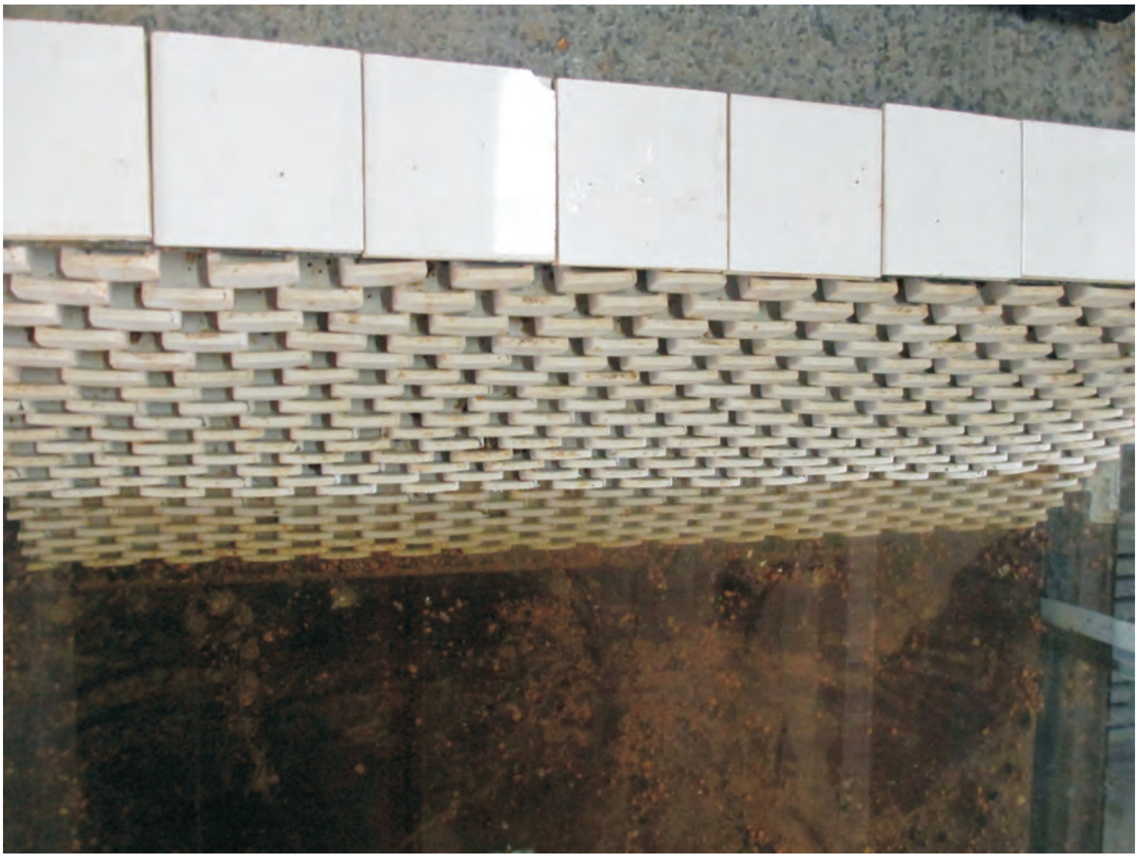
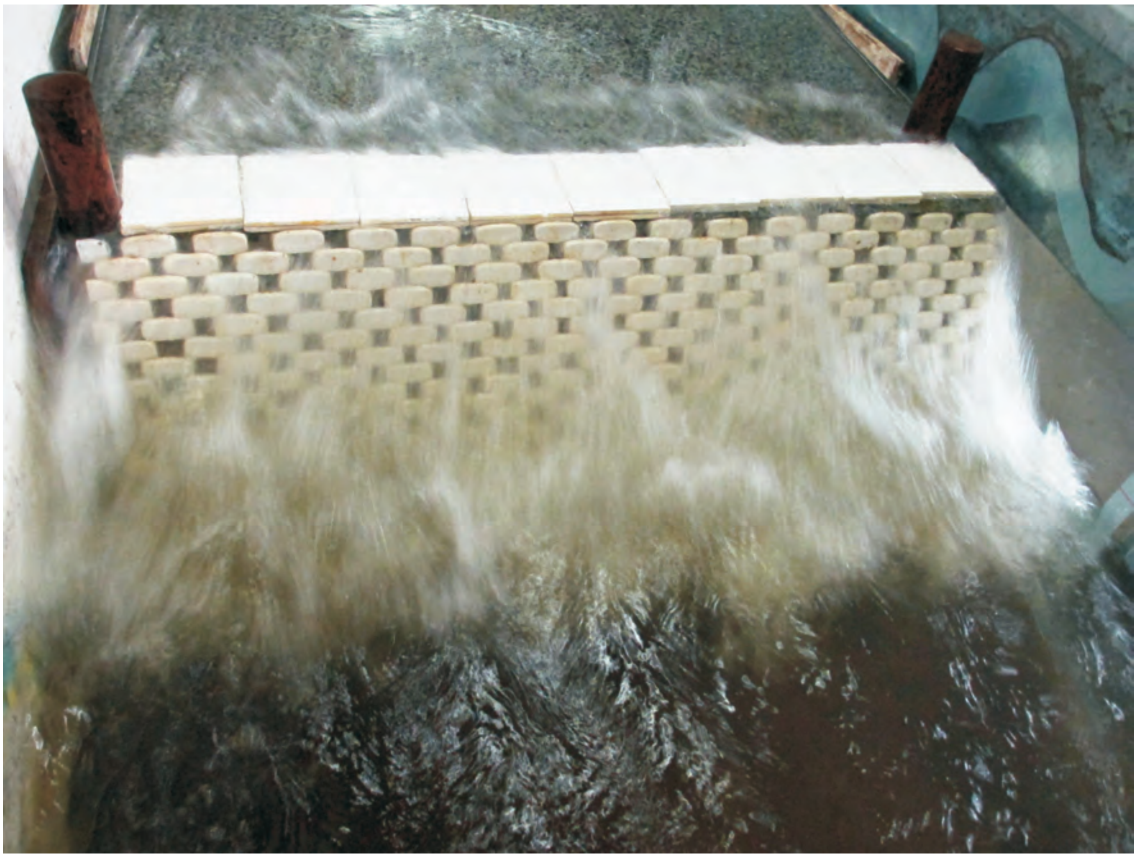


a) Cross-section 2 - placement density 10.8 units/m^2, WL 1.5m AHD, before testing



b) Cross-section 2 - placement density 10.8 units/m^2, WL 1.5m AHD, waves breaking on structure

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Figure 3.5

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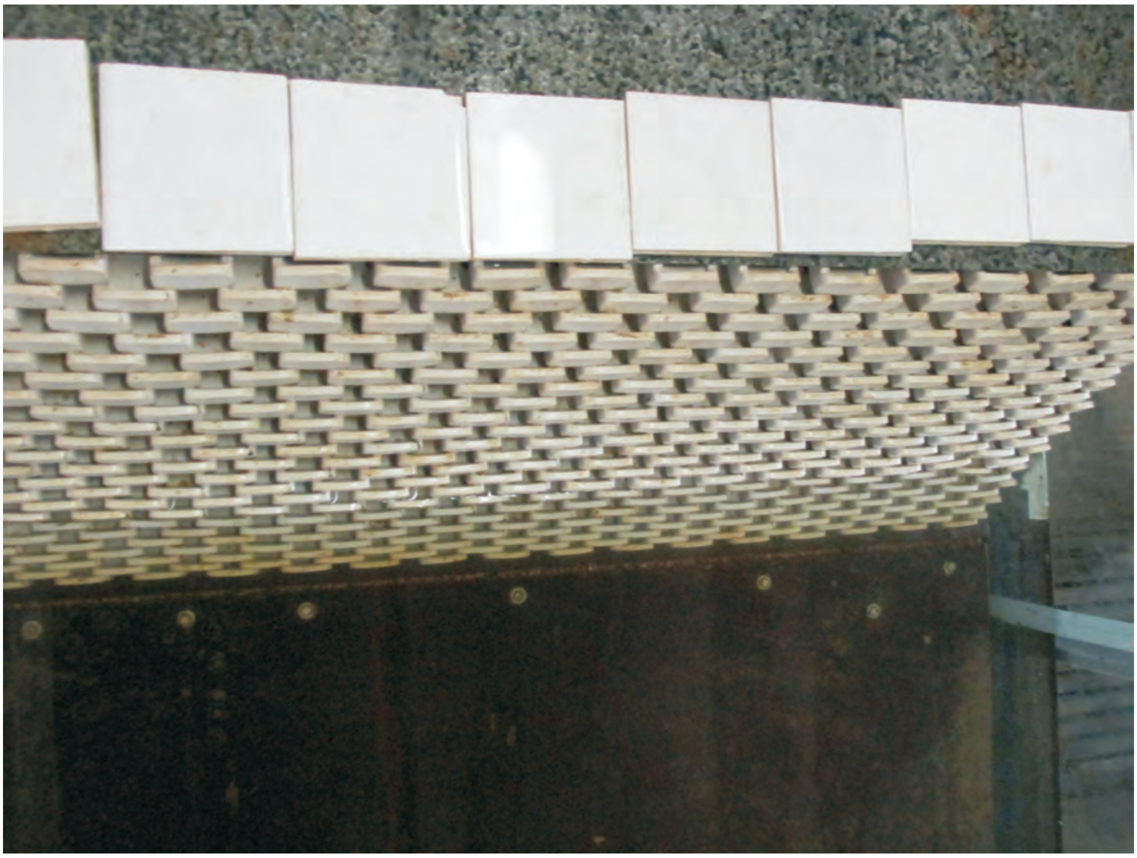
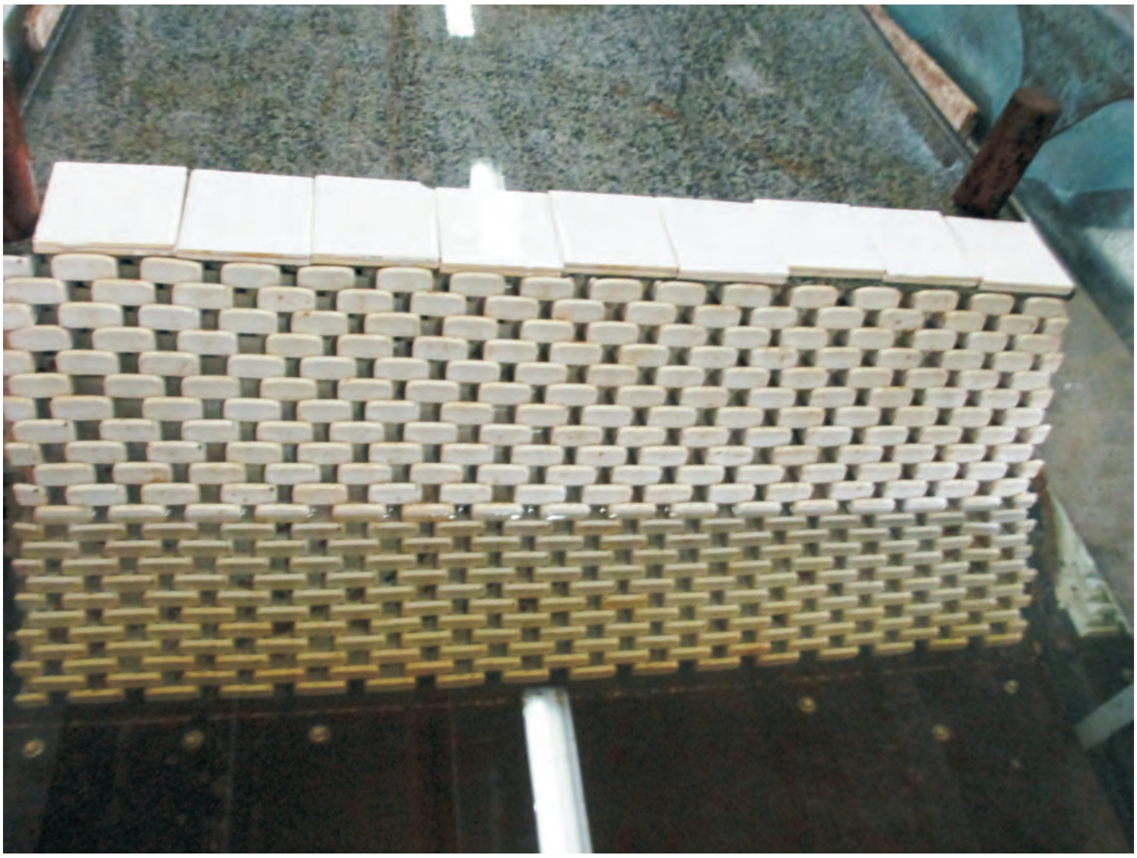


a) Cross-section 2 - placement density 10.8 units/m^2, WL 1.5m AHD, after 2000 waves, no damage to wall

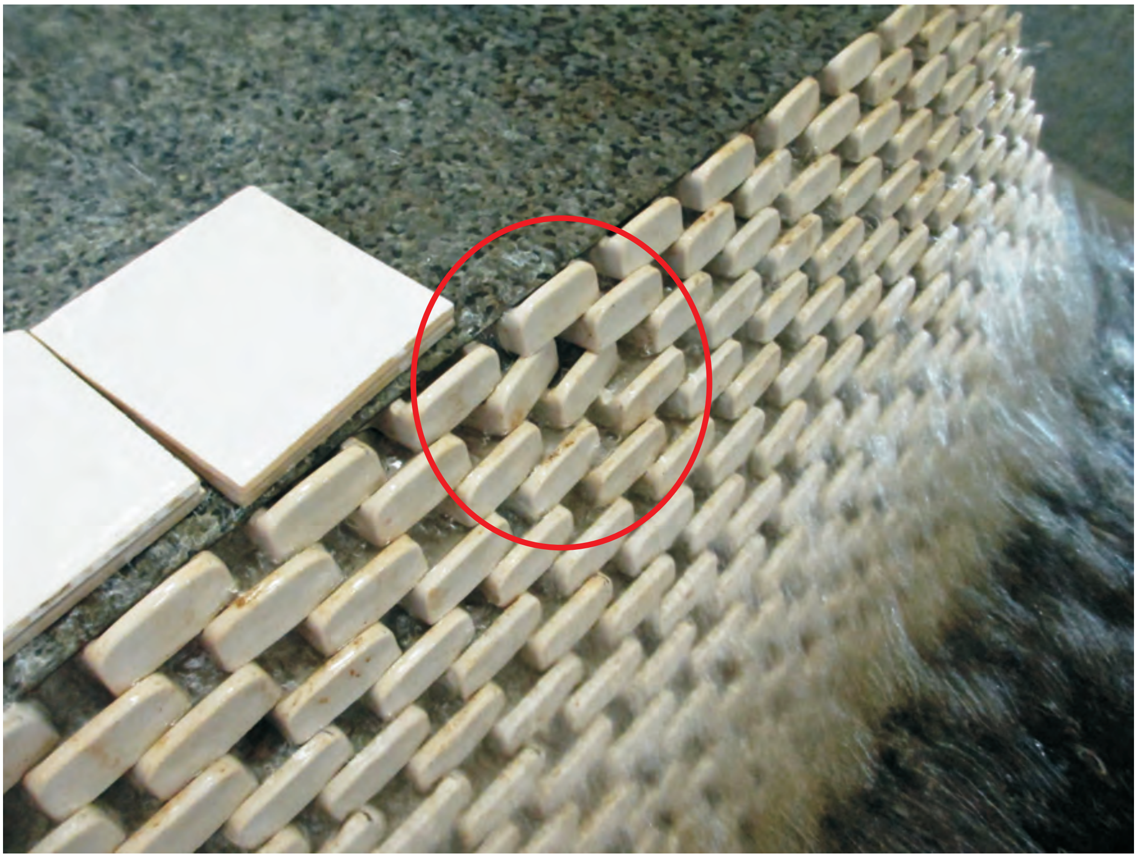
b) Cross-section 2 - placement density 10.8 units/m^2, WL 1.6m AHD, during testing



a) Cross-section 2 - placement density 10.8 units/m^2, WL 1.6m AHD, after 2000 waves, no damage to wall

b) Cross-section 2 - placement density 10.8 units/m^2, WL 1.6m AHD, after 2000 waves, no damage to wall

a) Cross-section 2 - placement density 10.8 units/m^2, WL 1.9m AHD, displacement of capping during testing



b) Cross-section 2 - placement density 10.8 units/m^2, WL 1.9m AHD, after 2000 waves. Note displacement of three units resulting in minor damage (<1%) on the wall in addition to displacement of capping

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Figure 3.8

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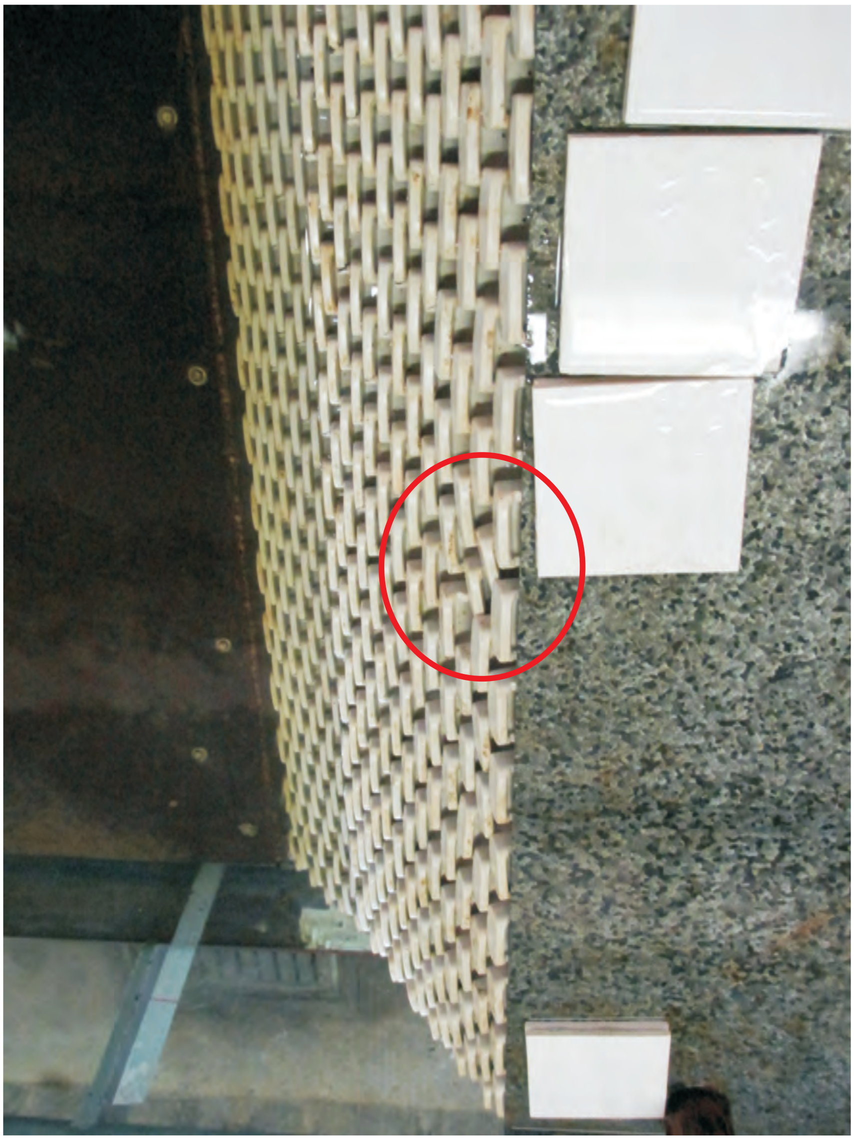
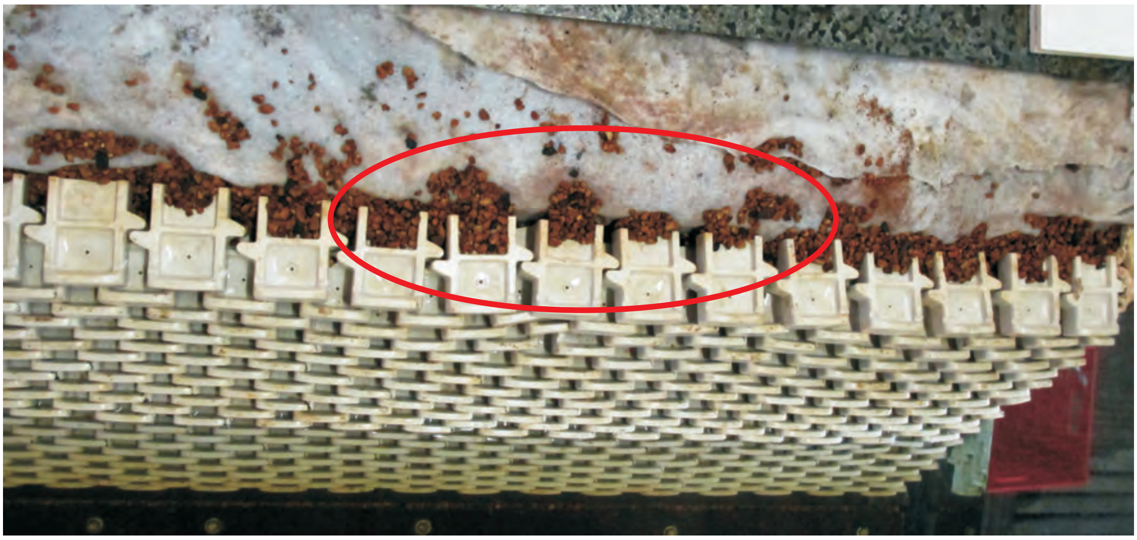


a) Cross-section 2 - placement density 10.8 units/m^2, WL 1.9m AHD, after 2000 waves, note displacement of three units resulting in minor damage (<1%) on the wall in addition to displacement of capping



b) Cross-section 2 - placement density 10.8 units/m^2, WL 1.9m AHD, after 2000 waves, note backfill partially washed away

Cross-section 2 - placement density 10.8 units/m^2, WL 1.9m AHD, after 2000 waves, note displacement of three units resulting in minor damage (<1%), another view

4. Conclusions and Recommendations

The model testing of the proposed design of the sea wall for Australian Coastal Walls resulted in the following conclusions and recommendations:

- Wave condition and water level. The wave conditions tested were characterised by waves with high surf similarity parameters (exceeding that typical of plunging waves) and resulted in a large number of surging waves. The incident wave conditions were evaluated using reflection analysis (**Appendix B**). Testing was carried out at water levels covering a range of predicted high water levels for the relevant return periods and also took into consideration sea level rise (SLR) due to climate change.

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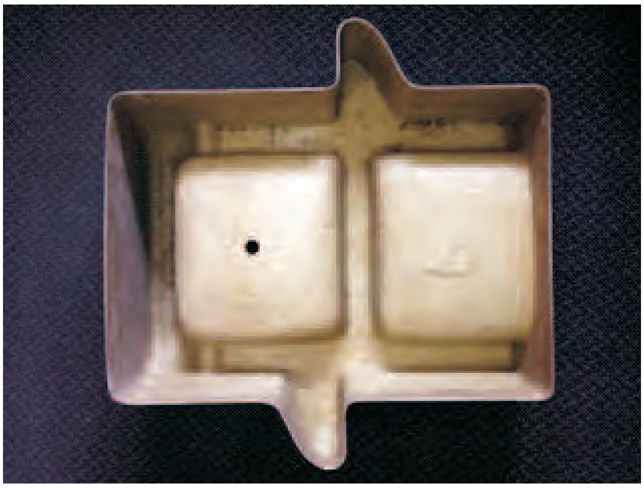
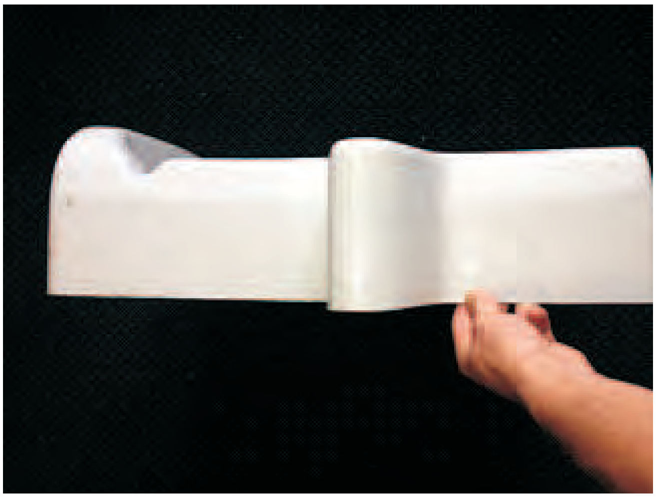
Appendix A

ACW Proposed Generic Design





Source: Australian Coastal Walls

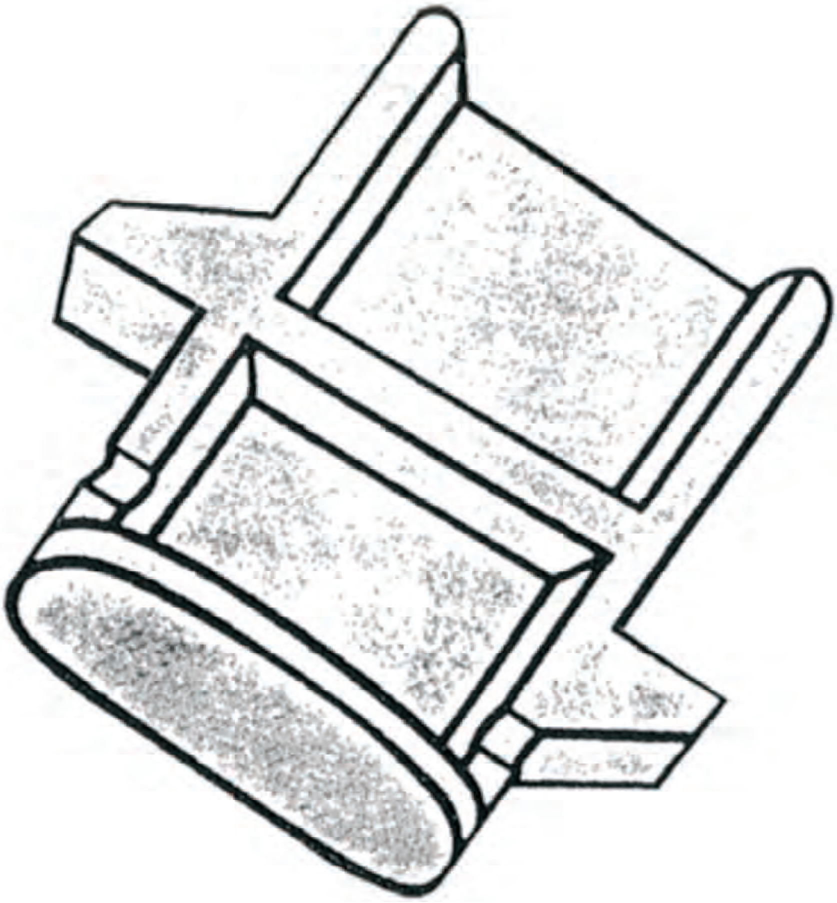
Public Works
Manly Hydraulics Laboratory

PHOTOS INDICATING SHAPE
OF THE ACW BLOCK

MHL
Report 2276

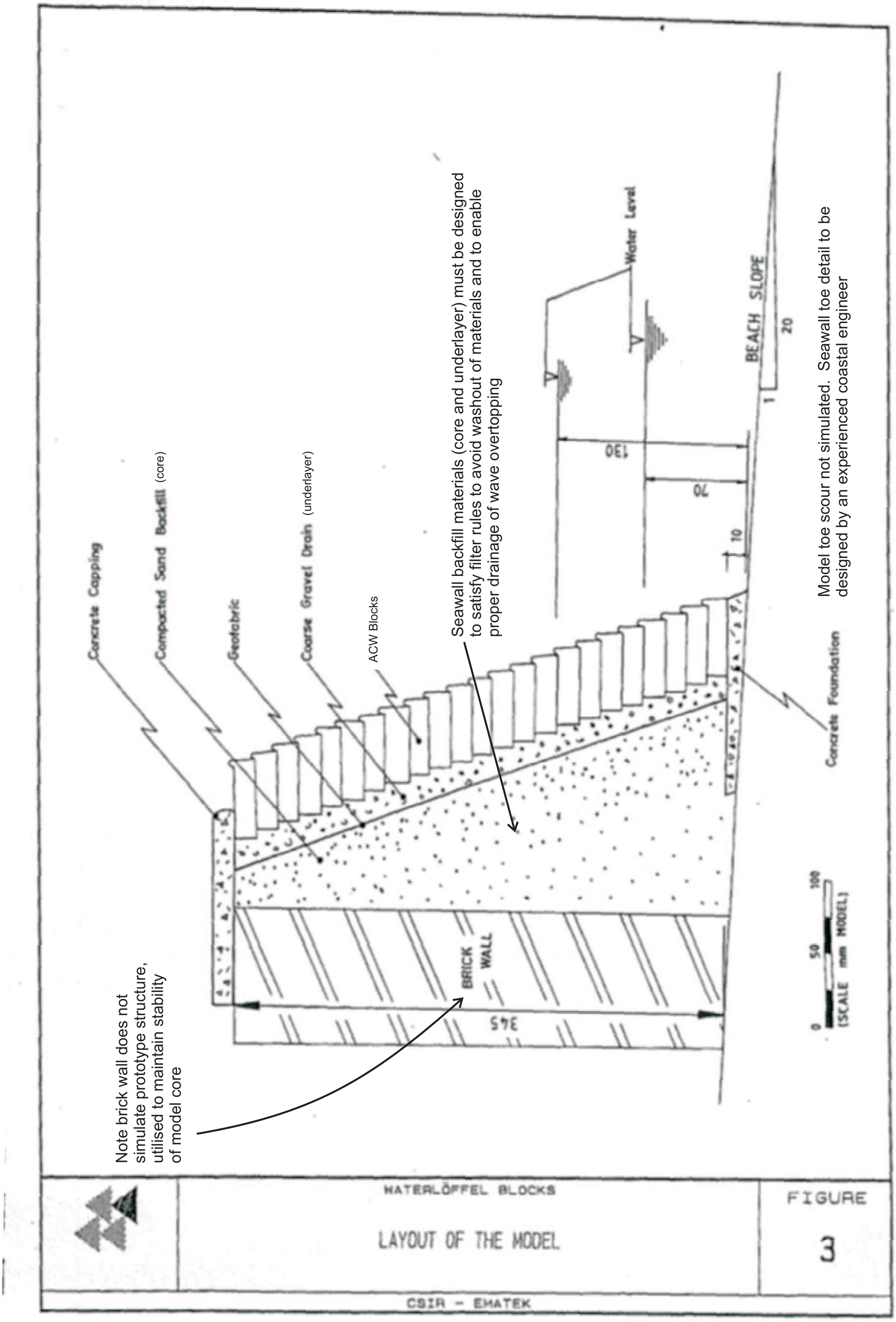
Figure
A1

DRAWING 2276-A1.cdr

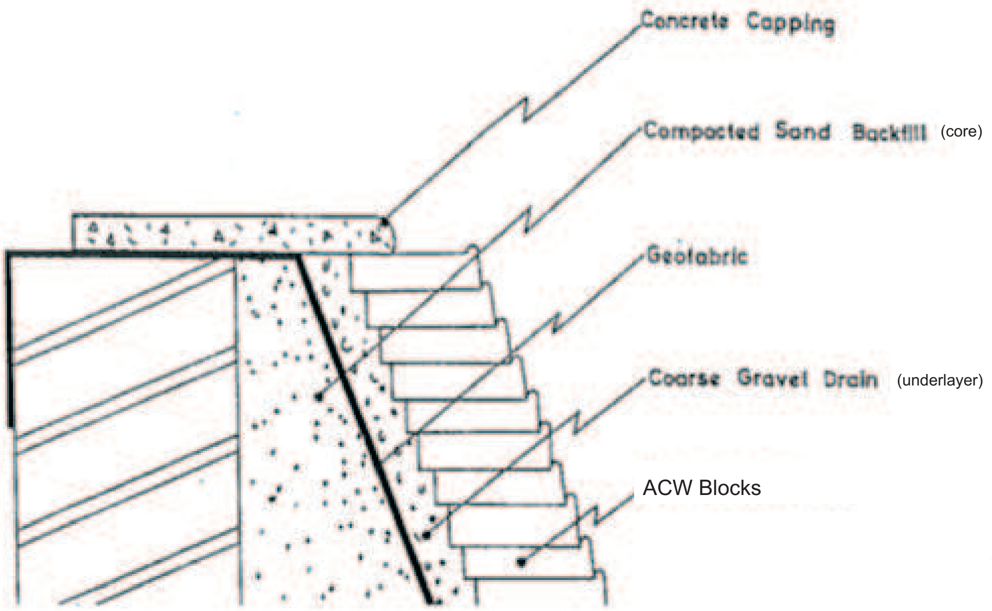


TECHNICAL DATA

Mass per block: 65 kg
Length of block: 650 mm
Width of block: 450 mm (without wings)
Width of block: 680 mm (inclusive of wings)
No. of units/m^2: 10 – 12
Course height: 150 mm.



Source: Australian Coastal Walls



Seawall backfill materials (core and underlayer) must be designed to satisfy filter rules to avoid washout of materials and to enable proper drainage of wave overtopping

Source: CSIR

Public Works
Manly Hydraulics Laboratory

CROSS-SECTION WITH GEOFABRIC DETAIL

MHL Report 2276

Figure A4

DRAWING 2276-A4.cdr

Appendix B

Wave Statistics

Appendix B Wave Statistics

Random Wave Facility and Model Construction

In the MHL random wave flume waves are generated by a sliding wedge wave paddle driven by a servo-hydraulic system. The three probe method (Funke 1980) is used during the tests to differentiate between the incoming wave and the reflected wave.

The floor of the flume was built at the start of the modelling to best approximate the bathymetry offshore of the seawall sections. The floor slope was adjusted to an average offshore slope based on information supplied by ACW's Patrick Johnson. The design profile was built in the 10 m glass section of the flume for model inspection, photography and video filming. The seawall cross-sections are as in profiles indicated in **Appendix A**. The waves were generated at a level of -10.0 m AHD**.**

Wave Conditions

The wave conditions used in the model were generated utilising Jonswap spectra. A gamma of 3.3 was used for all the tests. The wave conditions resulted from high wave conditions offshore and the extreme water levels at the structure resulting in broken waves. Random waves were generated according to a specified Jonswap energy spectrum defined by the wave height and peak spectral wave period. All model tests for overtopping were averaged over period 18.4 minutes (prototype). The probe closest to the structure (P4) was utilised to ensure accuracy of the stipulated wave conditions. The somewhat lower values for the ratio H_{max}/H_s may be attributed to the larger waves breaking. Reflection analysis carried out using DHI software indicated a reflection coefficient for the structure between 60% and 62% for the 8 s wave spectra.

Wave Probes

The wave probes were calibrated at the start and end of testing. The distance from probe P1 to P2 was 1.0 m, from P2 to P3 0.7 m. Probe P4 (probe closest to structure) is 5.0 m from the structure and simulates wave measurement at a distance of approximately 50 m from the toe of the structure.

Wave Statistics and Reflection Coefficients

The reflection coefficients were calculated utilising the DHI software. The total wave heights for Probe 4 are provided in **Table B1** and the incident wave heights for Probe 4 are provided in **Table B2** for the tests that were conducted.

Table B1 Wave Statistics of Measured Time Series for Stability/Overtopping Test

Test No.	Wave Event	Water Level (m AHD)	Measured Wave Height (m) Probe 1	Measured Wave Height (m) Probe 2	Measured Wave Height (m) Probe 3	Measured Wave Height (m) Probe 4
Cross-section – Placement Density 11.9 units/m^2						
1	100-year water level and breaking waves	1.5	2.11	2.19	2.19	2.28
2	1-year +SLR, water level and waves breaking offshore	1.6	2.13	2.14	2.18	2.21
3	100-year + SLR and waves breaking offshore	1.9	2.14	2.18	2.22	2.15
Cross-section – Placement Density 10.8 units/m^2						
4	100-year water level and breaking waves	1.5	2.15	2.17	2.24	2.35
5	1-year +SLR, water level and waves breaking offshore	1.6	2.15	2.21	2.29	2.34
6	100-year + SLR and waves breaking offshore	1.9	2.10	2.20	2.27	2.26

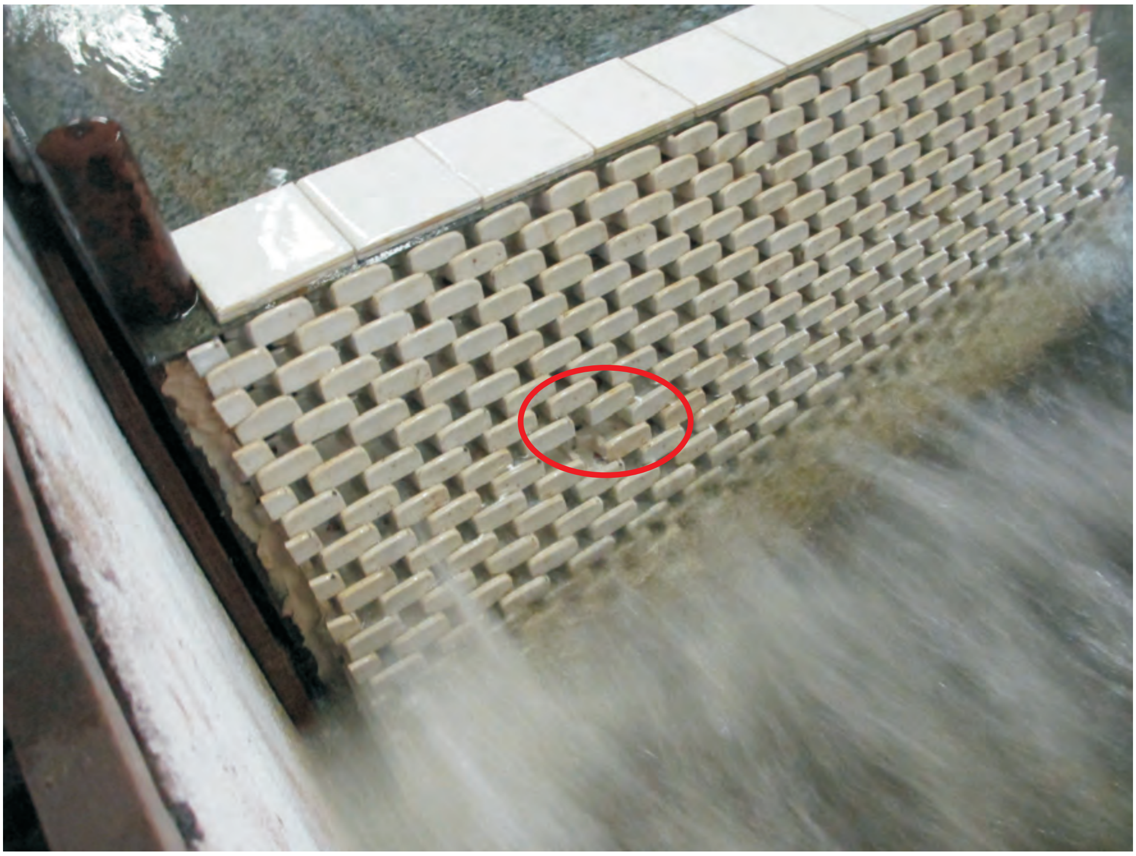
Table B2 Detailed Wave Statistics of Measured Time Series for Stability Tests – Probe 4

Test No.	Wave Event	Water Level (m AHD)	Measured Wave Height (m) Probe 4	Reflection Coefficient	Incident Wave Height (m) Probe 4	Peak Spectral Wave Period (s) Probe 4
Cross-section – Placement Density 11.9 units/m^2						
1	100-year water level and breaking waves	1.5	2.28	0.53	2.05	8.10
2	1-year +SLR, water level and waves breaking offshore	1.6	2.21	0.52	1.93	8.10
3	100-year + SLR and waves breaking offshore	1.9	2.15	0.53	1.89	8.10
Cross-section – Placement Density 10.8 units/m^2						
4	100-year water level and breaking waves	1.5	2.35	0.53	2.08	8.10
5	1-year +SLR, water level and waves breaking offshore	1.6	2.34	0.52	2.09	8.10
6	100-year + SLR and waves breaking offshore	1.9	2.26	0.53	2.0	8.10

Appendix C

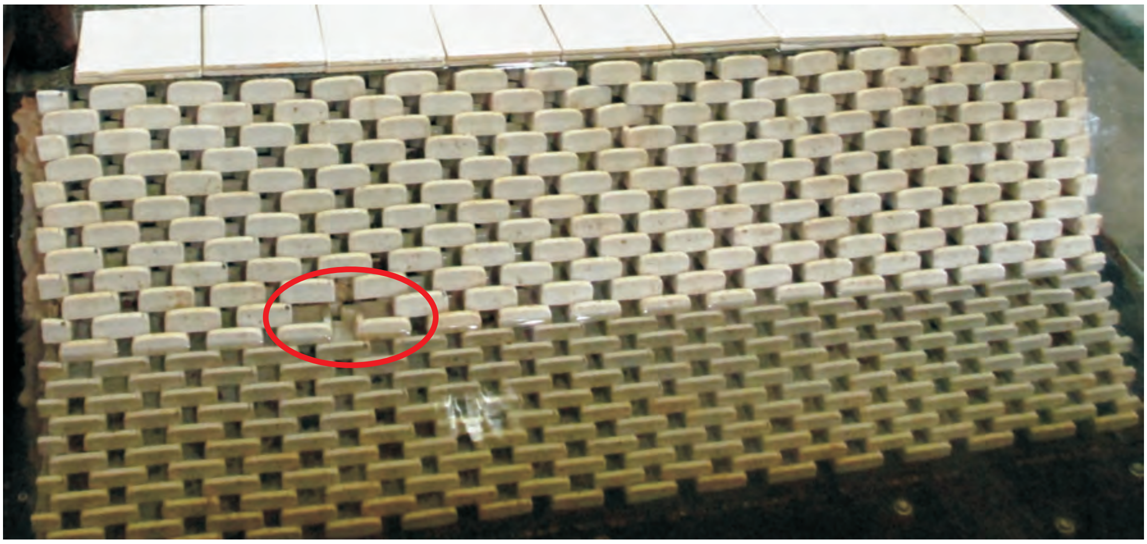
Testing Modes of Failure

a) Testing possible modes of failure. Cross-section 2 - placement density 10.8 units/m^2, WL 1.9m AHD, one unit removed, before testing

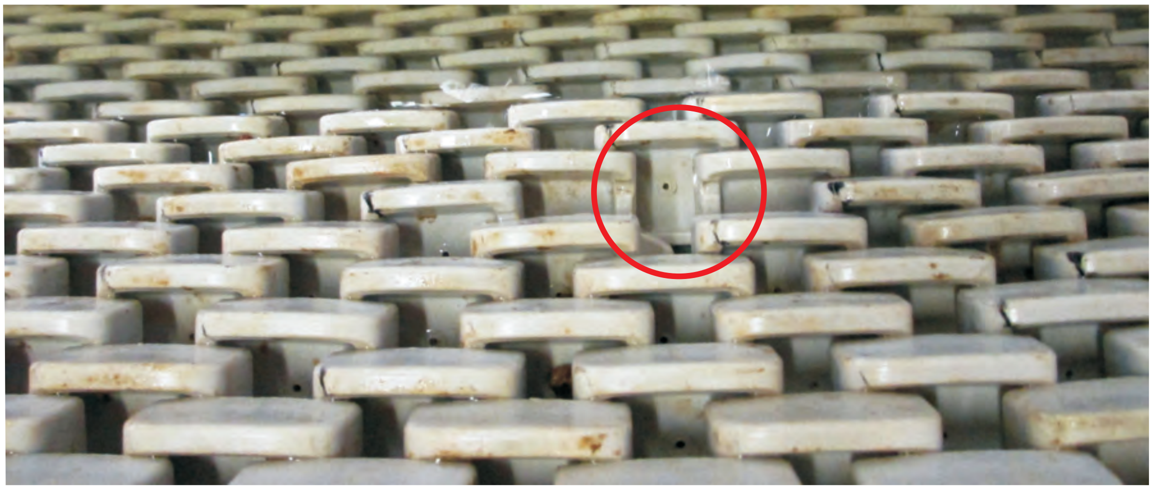


b) Testing possible modes of failure. Cross-section 2 - placement density 10.8 units/m^2, WL 1.9m AHD, one unit removed, during testing

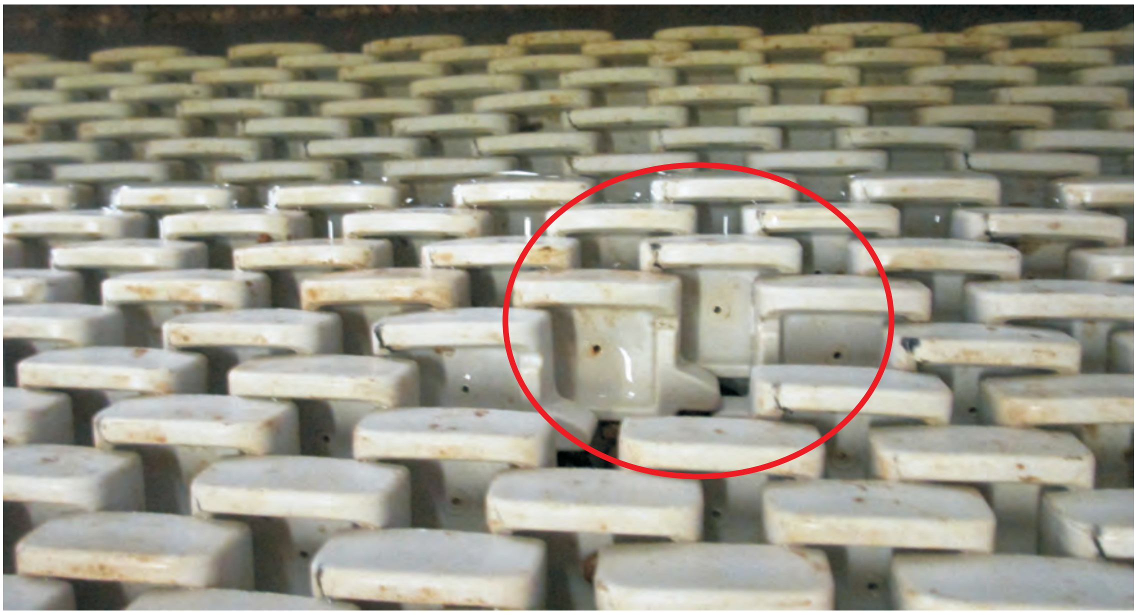
NSW GOVERNMENT Public Works Manly Hydraulics Laboratory	ACW BLOCK SEAWALL 2D PHYSICAL MODEL TESTING POSSIBLE MODES OF FAILURE	MHL Report 2276 Figure C1 DRAWING 2276-C1.cdr



a) Testing possible modes of failure. Cross-section 2 - placement density 10.8 units/m^2, WL 1.9m AHD, one unit removed, after testing, no further damage was recorded



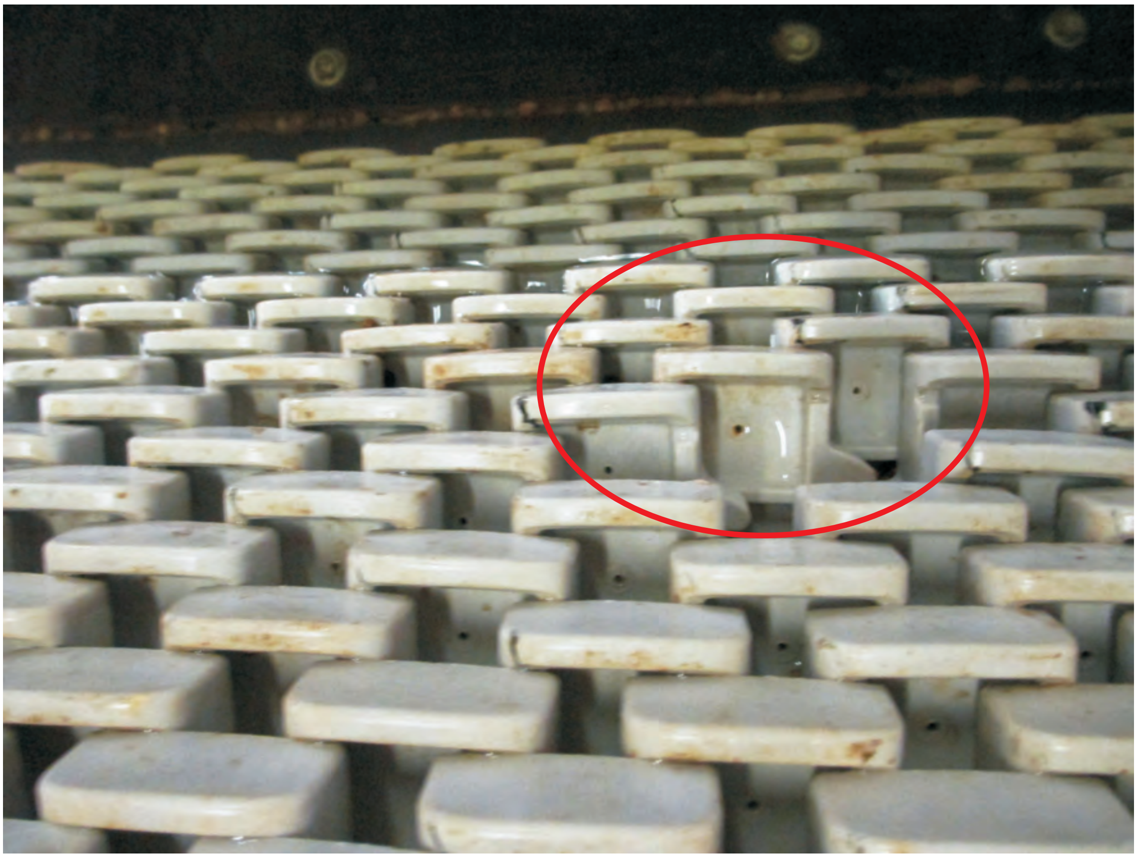
b) Testing possible modes of failure. Cross-section 2 - placement density 10.8 units/m^2, WL 1.9m AHD, one unit removed, after testing, no further damage was recorded



c) Testing possible modes of failure. Cross-section 2 - placement density 10.8 units/m^2, WL 1.9m AHD, one unit removed, after testing, no further damage was recorded, another view

NSW GOVERNMENT Public Works Manly Hydraulics Laboratory	ACW BLOCK SEAWALL 2D PHYSICAL MODEL TESTING POSSIBLE MODES OF FAILURE	MHL Report 2276 Figure C2 DRAWING 2276-C2.cdr

a) Testing possible modes of failure. Cross-section 2 - placement density 10.8 units/m^2, WL 1.9m AHD, two units removed, during testing



b) Testing possible modes of failure. Cross-section 2 - placement density 10.8 units/m^2, WL 1.9m AHD, two units removed, after testing, no additional damage

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Manly Hydraulics Laboratory

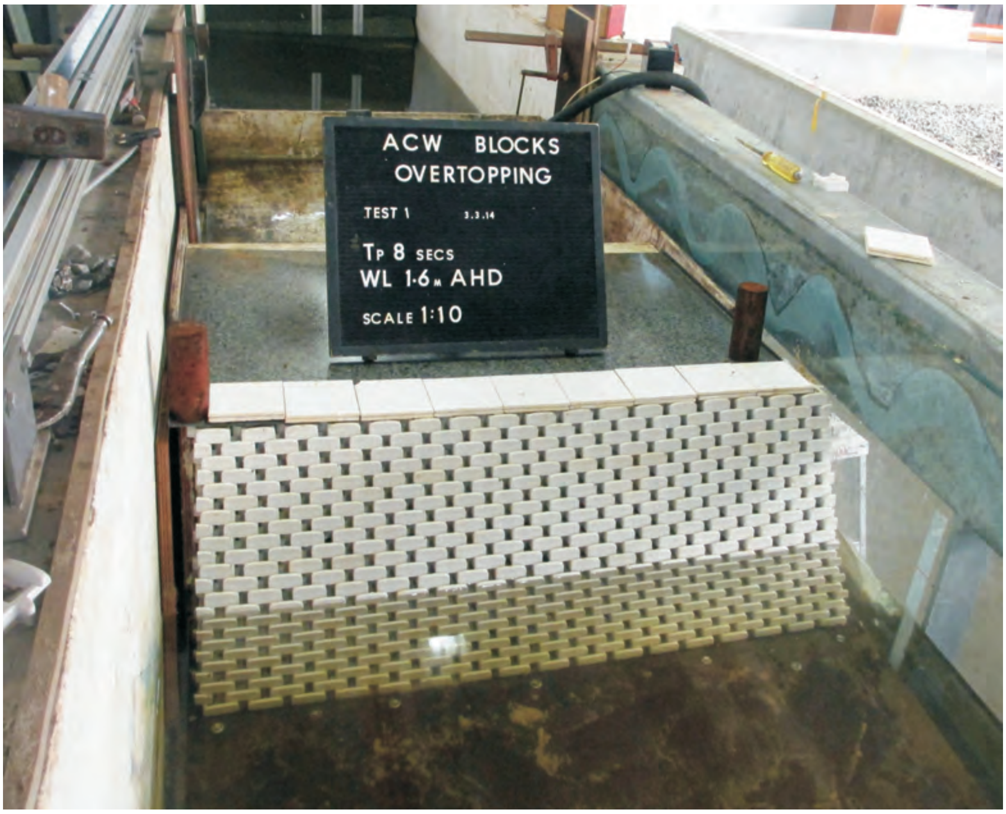
ACW BLOCK SEAWALL
2D PHYSICAL MODEL
TESTING POSSIBLE MODES OF FAILURE

MHL Report 2276

Figure C3

DRAWING 2276-C3.cdr

a) Testing possible modes of failure. Cross-section 1 - placement density 11.9 units/m^2, WL 1.6m AHD, during construction



b) Testing possible modes of failure. Cross-section 1 - placement density 11.9 units/m^2, WL 1.6m AHD, before testing

NSW GOVERNMENT Public Works Manly Hydraulics Laboratory	ACW BLOCK SEAWALL 2D PHYSICAL MODEL TESTING POSSIBLE MODES OF FAILURE	MHL Report 2276 Figure C4 DRAWING 2276-C4.cdr

Appendix D

Modes of Failure for Crib Walls

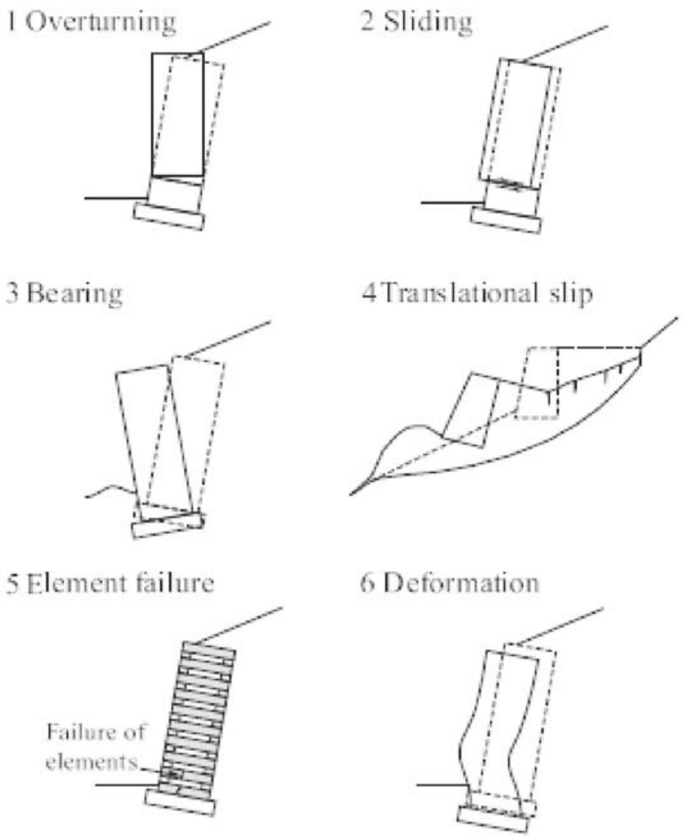


Figure 2.1 Examples of limit modes of failure

2.8 *Limit mode 1: Overturning failure.*

2.9 *Limit mode 2: Sliding failure.*

2.10 *Limit mode 3: Bearing failure of the foundation.*

2.11 *Limit mode 4: Slip failure of the soil.*

2.12 *Limit mode 5: Failure of the headers and stretchers.*

2.13 *Limit mode 6: Deformation of the structure.*

Source: *Design Manual for Roads and Bridges - Crib Wall Design*, Vol. 2 1997, The Scottish, Irish and Welsh Departments of Environment

NSW GOVERNMENT Public Works Manly Hydraulics Laboratory	MODES OF FAILURE FOR CRIB WALLS	MHL Report 2276 Figure D1 DRAWING 2276-D1.cdr

Appendix E

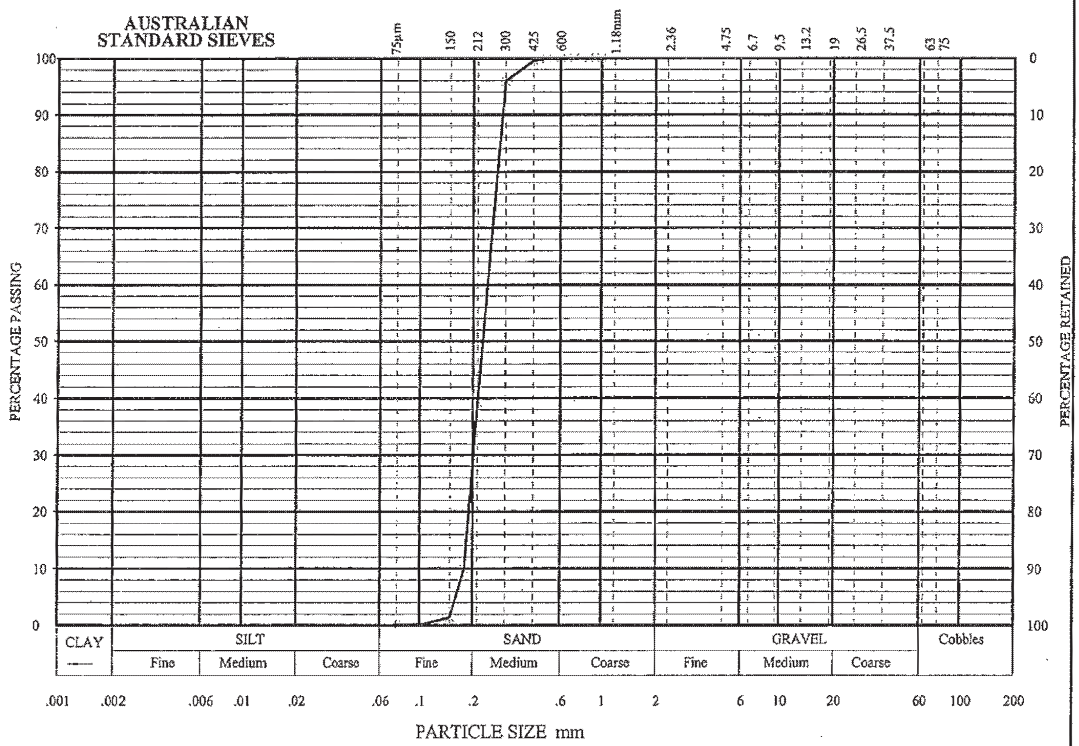
Sieve Analysis of Fine Sand, Rubble and Blue Metal

Geotechnical Centre

110B King Street, Manly Vale NSW 2093
Telephone 02 9949 0200 Facsimile 02 9948 6185
NATA Accreditation Number: 13380

NSW GOVERNMENT | Public Works

CLIENT: GEOTECHNICAL & ENVIRONMENTAL		REPORT No: 14003/4575/R1119
PARTICLE SIZE DISTRIBUTION		
PROJECT: MHL MODELLING SAND		SAMPLE No: 4575
LOCATION: Manly Hydraulics Laboratory	HOLE No: Fine	DEPTH (m): 0.00



SIZE DISTRIBUTION	
COBBLES	0 %
GRAVEL	0 %
SAND RETAINED 75um	100 %
MATERIAL FINER THAN 75um	0 %
EFFECTIVE SIZE D10:	.18 mm
UNIFORMITY COEFFICIENT D60/D10(Cu):	1.3
CURVATURE COEFFICIENT $D30^2/(D60 \times D10)$ (Cc):	0.9

SIZE (mm)	% PASSING	SIZE (mm)	% PASSING
0.600	100.0	0.150	1.4
0.500	99.9	0.106	0.2
0.425	99.6	0.075	0.1
0.300	95.9		
0.212	39.3		
0.180	9.9		

Notes on Test: Tested as received
Loss in pre-treatment: 0 %

Test Methods:
DPWS GM 9: Determination of the Particle Size Distribution of a Soil

PARTICLE SIZE DISTRIBUTION R1119 (ISSUE 4, 2005)

NATA Accredited for compliance with ISO/IEC 17025.

Tested By: MA   Date Tested: 04/03/2014

APPROVED SIGNATORY M. Ashover
Mark Ashover 06/03/2014

Geotechnical Centre

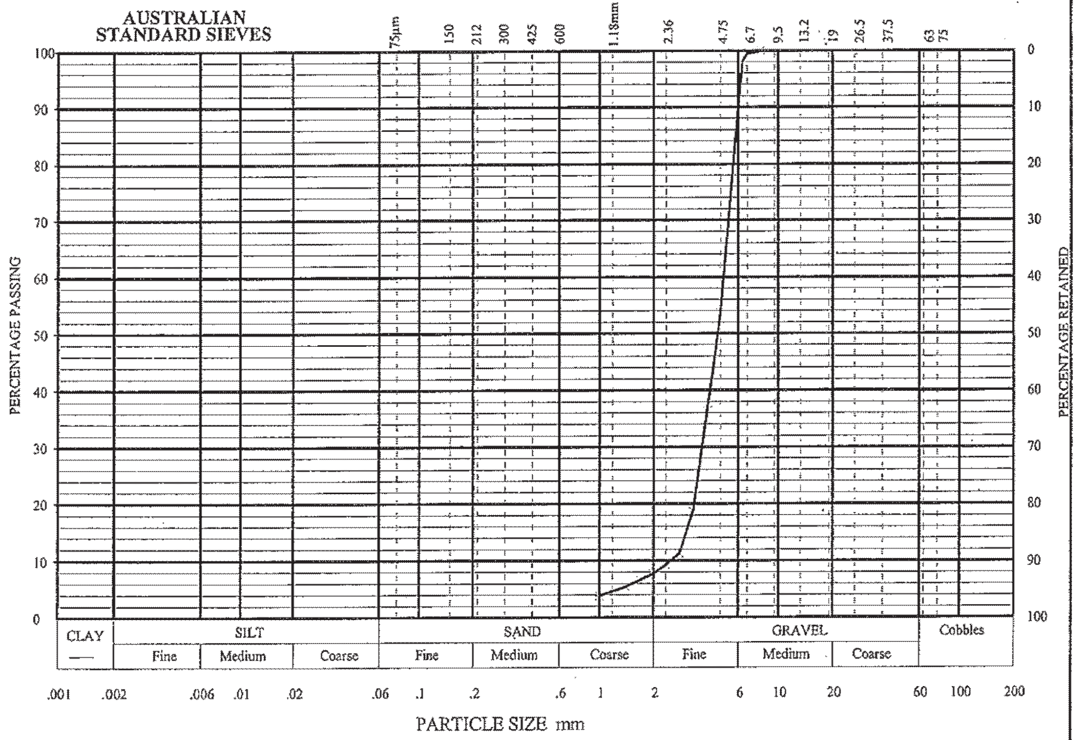
110B King Street, Manly Vale NSW 2093
Telephone 02 9949 0200 Facsimile 02 9948 6185
NATA Accreditation Number: 13380

NSW GOVERNMENT | Public Works

CLIENT: GEOTECHNICAL & ENVIRONMENTAL	REPORT No: 14003/4576/R1119

PARTICLE SIZE DISTRIBUTION

PROJECT: MHL MODELLING SAND		SAMPLE No: 4576
LOCATION: Manly Hydraulics Laboratory	HOLE No: Coarse	DEPTH (m): 0.00



PARTICLE SIZE DISTRIBUTION R1119 (ISSUE 4, 2005)

SIZE DISTRIBUTION	
COBBLES	0 %
GRAVEL	92 %
SAND RETAINED 75um	8 %
MATERIAL FINER THAN 75um	N/A
EFFECTIVE SIZE D10:	2.54 mm
UNIFORMITY COEFFICIENT D60/D10(Cu):	1.9
CURVATURE COEFFICIENT $D30^2/(D60 \times D10)$ (Cc):	1.1

SIZE (mm)	% PASSING	SIZE (mm)	% PASSING
8.00	100.0	2.80	11.2
6.70	99.4	2.36	9.2
6.30	97.9	2.000	7.7
4.75	53.6	1.700	6.6
4.00	36.4	1.400	5.4
3.35	18.6	1.000	3.8

Notes on Test: Tested as received

Loss in pre-treatment: 0 %

Test Methods:
DPWS GM 9: Determination of the Particle Size Distribution of a Soil

NATA Accredited for compliance with ISO/IEC 17025.

Tested By: MA | Date Tested: 04/03/2014

APPROVED SIGNATORY M. Ashover

Mark Ashover 06/03/2014

NSW GOVERNMENT Public Works Manly Hydraulics Laboratory	SIEVE ANALYSIS FOR RUBBLE BACKFILL	MHL Report 2276
		Figure E2
		DRAWING 2276-E.cdr

Geotechnical Centre

110B King Street, Manly Vale NSW 2093
Telephone 02 9949 0200 Facsimile 02 9948 6185
NATA Accreditation Number: 13380

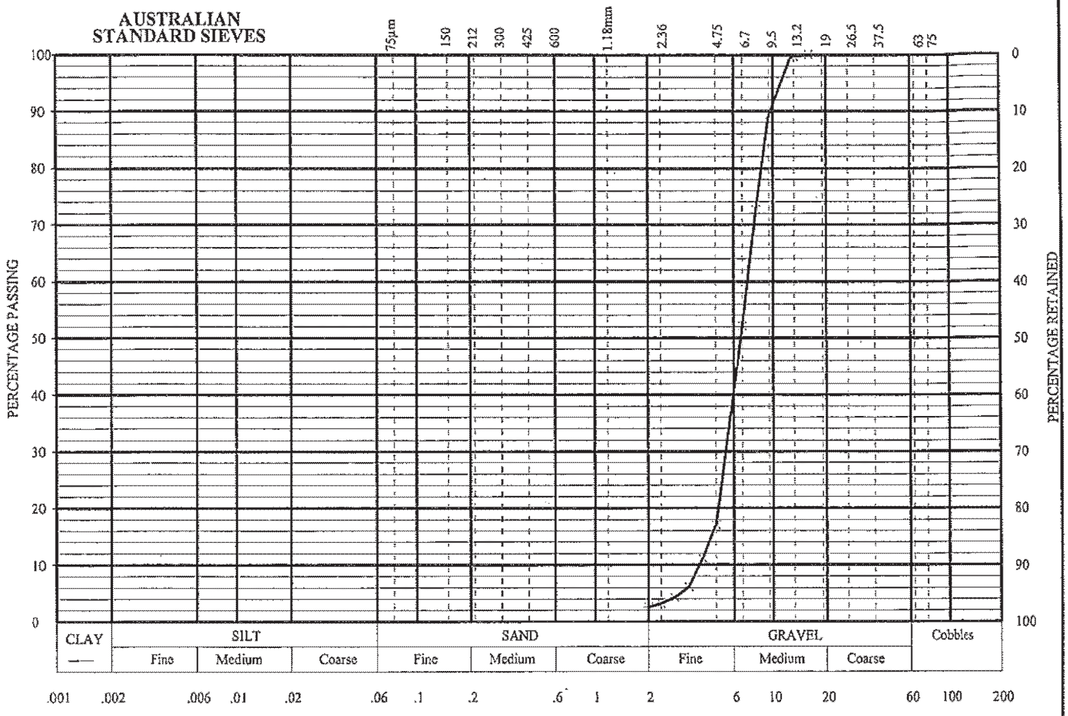
NSW GOVERNMENT | Public Works

CLIENT: GEOTECHNICAL & ENVIRONMENTAL — REPORT No: 14003/4577/R1119

PARTICLE SIZE DISTRIBUTION

PROJECT: MHL MODELLING SAND — SAMPLE No: 4577

LOCATION: Manly Hydraulics Laboratory — HOLE No: Gravel — DEPTH (m): 0.00



SIZE DISTRIBUTION	
COBBLES	0 %
GRAVEL	97 %
SAND AND SILT	3 %
MATERIAL FINER THAN 75um	N/A

EFFECTIVE SIZE D10:	3.86 mm
UNIFORMITY COEFFICIENT D60/D10(Cu):	1.8
CURVATURE COEFFICIENT $D30^2/(D60 \times D10)$ (Cc):	1.0

SIZE (mm)	% PASSING	SIZE (mm)	% PASSING
16.00	100.0	4.75	17.0
13.20	99.7	4.00	11.1
12.70	99.5	3.35	6.1
9.50	89.3	2.80	4.2
8.00	72.7	2.36	3.2
6.70	52.1	2.00	2.6

Notes on Test: Tested as received

Loss in pre-treatment: 0 %

Test Methods:
DPWS GM 9: Determination of the Particle Size Distribution of a Soil

PARTICLE SIZE DISTRIBUTION R1119 (ISSUE 4, 2005)

NATA Accredited for compliance with ISO/IEC 17025.

Tested By: MA — Date Tested: 06/03/2014

APPROVED SIGNATORY M. Ashover

Mark Ashover 06/03/2014

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