Ravenna offshore LNG terminal

2D physical model study

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Client representative	Pierluigi Russo
Project manager	John Alderson
Project director	Mark McBride

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John Alderson	Thierry Rault	Thierry Rault

Executive summary

This report describes a 2D breakwater physical model study carried out by HR Wallingford in support of the detailed design of an offshore LNG terminal near the port of Ravenna.

This report focuses on the 2D physical model study of the offshore caisson breakwater. The scope of work for the 2D study was as follows:

- Measure overtopping and transmission coefficients:
  - Mean overtopping discharge;
  - Wave by wave overtopping volumes;
  - Wave disturbance behind the breakwater and calculated transmission coefficient.
- Measure wave loading on the caisson breakwater using pressure transducers:
  - Providing pressures at each measurement point;
  - Proving derived forces and moments through integration of pressures to assigned areas.
- Measure the reflection coefficient from the front of the caisson breakwater.
- Measured stability of the rock armour, blockwork and rock scour protection of the breakwater.

In order to meet the study aims and objectives the physical model covered the following series of tests:

- Test Series A – Wave calibration without breakwater in place;
- Test Series B – Overtopping, wave loading, transmission and reflection tests;
- Test Series C – Armour and scour protection tests.

Wave loadings on caisson

Wave loading tests were carried out using the following conditions:

- H_{m0} = 5.5 m, T_p = 9.6 s, SWL = +1.7 m m.s.l, T_r = 100 years;
- H_{m0} = 5.5 m, T_p = 9.6 s, SWL = +0.46 m m.s.l, T_r = 100 years;
- H_{m0} = 6.6 m, T_p = 10.9 s, SWL = +0.46 m m.s.l (overload).

All sea state parameters were defined at the -14.25 m contour, close to the toe of the foundation mound. The wave loadings were measured by 24 pressure transducers embedded in the surface of the caisson. The output of the transducers was integrated to derive whole body forces and moments, allowing statistical parameters to be determined for the following loading types:

- The landwards and seawards horizontal forces;
- The upwards and downwards vertical forces;
- The landwards and seawards overturning moments.

All loading results were post-processed by applying filters that uses the estimated dynamic response of the prototype structure, so that they represented static-equivalent global forces and moments. The unfiltered pressure signals have also been presented, to provide the client with information on the highest pressures acting on the caisson surface and recurve, including the overtopping-induced downfall pressures on the caisson deck slab.

Armour damage

Damage to the rock armour was tested in the two extreme conditions:

- H_{m0} = 5.5 m, T_p = 9.6 s, SWL = +0.46 m m.s.l, T_r = 100 years;
- H_{m0} = 6.6 m, T_p = 10.9 s, SWL = +0.46 m m.s.l (overload).

The damage was recorded by photographing the armour layers before and after each test, and counting the cumulative number of rocks extracted to provide a continuous record of damage progression through the test sequence. The damage parameter, N_{od}, was then calculated and compared with tolerable damage levels given in the Rock Manual for caisson foundation mounds. Even after the overload test, a maximum damage of N_{od} = 1.09 was recorded after the overload condition, which approximates to 'almost no damage' to 'acceptable damage' according to the Rock Manual descriptions.

The concrete toe blocks were also monitored photographically, showing that none were displaced during the test sequence.

Overtopping and transmission

The overtopping tests recorded the discharge over the caisson for 6 different design using the flowing sea state:

- H_{m0} = 5.5 m, T_p = 9.6 s, SWL = +1.7 m m.s.l, T_r = 100 years.

The largest volumes of overtopping was with a vertical wall set at +6.5 m MSL and the smallest volume for the recurve wall set at +10.5 m MLS.

The caisson wall with a vertical wall set at +6.5 m allowed the highest wave transmission, as registered by a wave gauge placed 50 m, 100 m and 150 m behind the caisson's harbour-side face.

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1 Introduction

The Italian company Snam Rete e Gas (SRG) is developing an offshore LNG terminal in the Adriatic Sea, off the coast of Ravenna. The terminal is located approximately 8.5 km offshore and will accommodate one FSRU (Floating Storage and Regasification Unit).

1.1 Project background

The Ravenna LNG terminal is to be located approximately 8.5 km offshore, see Figure 1.1. The pre-FEED design is progressing on the basis of using a location with existing infrastructure (an offshore jetty) and providing berthing with protection from the wave climate using a vertical caisson breakwater. The current design provides an approximately 1,030 m long breakwater structure, with integrated anti-reflection caissons (see Figure 1.2) placed in a water depth of around 14 m to 15 m. New mooring dolphins and a jetty will be constructed at the site to accommodate the FSRU and associated LNGC vessels.

Technip Energies have contracted HR Wallingford to provide a 2D physical model study, which will be used to confirm feasibility study results.

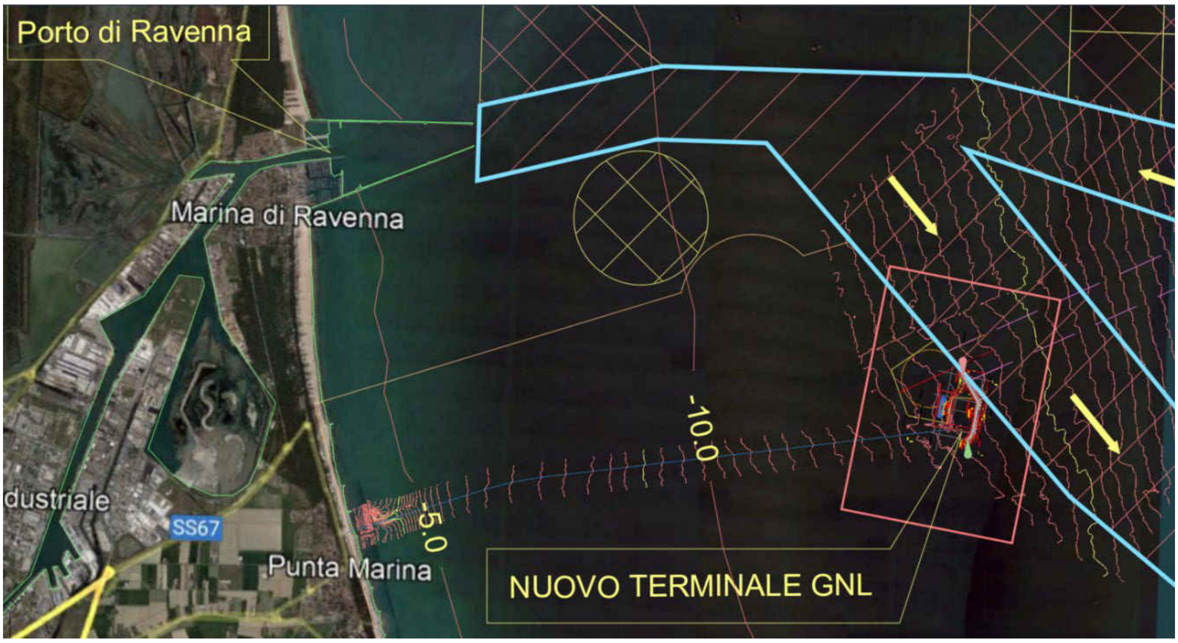

Figure 1.1: Proposed site

Source: GoogleEarth and DW-3320-003 - Layout Breakwater - Alternativa A2.1.dwg

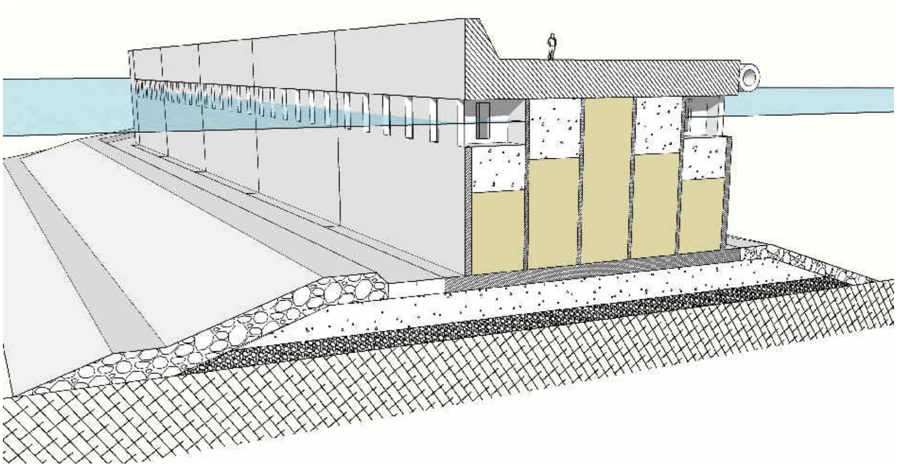

Figure 1.2: Caisson breakwater with anti-reflection chambers

Source: DW-3320-003 - Layout Breakwater - Alternativa A2.1.dwg

In April 2023 the breakwater went through a design revision, bringing the vessel berths closer to the breakwater and revising the possibility of a second berth behind an extended caisson breakwater. The revised layout which was investigated is given in Figure 1.3.

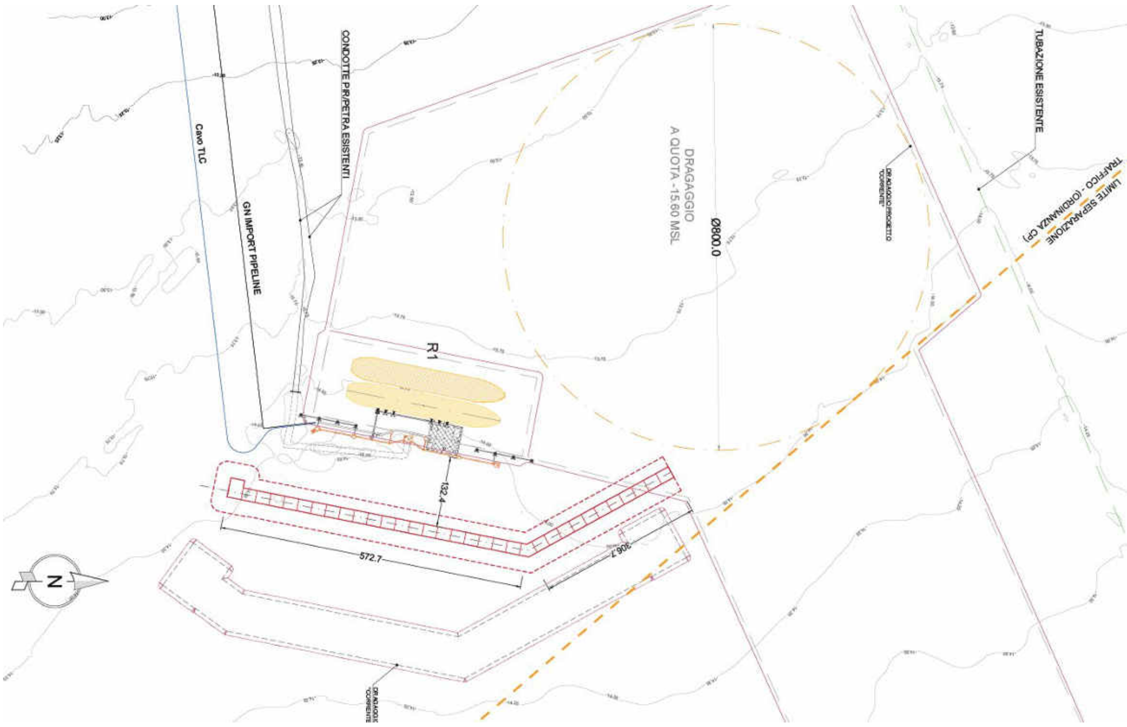

Figure 1.3: Revised terminal layout

Source: Planimetria 2 FSRU allineate rev.3-TAV 3 Ali di gabbiano.pdf

1.2 Study scope of work

The physical model study will be completed in two stages:

- 2D physical model study investigating the performance of caisson breakwater;
- 3D model study investigating the vessel response and three-dimensional aspects of the breakwater.

This report focuses on the 2D physical model study of the offshore caisson breakwater. The scope of work for the 2D study was as follows:

- Measure overtopping and transmission coefficients:
  - Mean overtopping discharge;
  - Wave by wave overtopping volumes;
  - Observations of overtopping characteristics, such as distance reached;
  - Wave disturbance behind the breakwater and calculated transmission coefficient.
- Measure wave loading on the caisson breakwater using pressure transducers:
  - Providing pressures at each measurement point;
  - Proving derived forces and moments through integration of pressures to assigned areas.
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In order to meet the study aims and objectives the physical model covered the following series of tests:

- Test Series A – Wave calibration without breakwater in place;
- Test Series B – Overtopping, wave loading, transmission and reflection tests;
- Test Series C – Armour and scour protection tests.

1.3 Outline of this report

Following this introduction this report is broken down as follows:

- Chapter 2 describes the modelling facilities, adopted scale, instrumentation and Flume layout used to complete the study.
- The wave and water level conditions used for the study are given in Chapter 3, along with details of the model structure setup, the planning, post-processing methods and test programme.
- Chapter 4 presents details of the wave calibration.
- The results for the caisson wave loading, transmission, reflection and overtopping performance are given in Chapter 5.
- Chapter 6 provides results of the caisson rubble mound and toe block performance for design and overload wave conditions.
- Finally summary and conclusion are in given Chapter 7.

2 Physical model facilities

2.1 Scaling

For this study a scale of 1:35 was chosen to allow enough space behind the breakwater for transmission measurements to be made, whilst maintaining the caisson position at the flume observation window. With Froude scaling and a linear scale of 1:35, the time and velocity scales are $35^{½}$ or 1:5.92. This means events occurred about 5.9 times faster at model scale than at full (prototype) scale.

Froude's scaling law is applied to physical models where gravity and momentum are predominant factors in the fluid motion and where free surfaces occur, including all wave and current models, since wave motion is essentially a gravitational phenomenon. Froude's law states that the Froude number, Fr, should be the same in model and nature, where Fr is defined as:

$$Fr = \frac{u}{\sqrt{gD}}$$

Where:	u	is a characteristic velocity;
	g	is acceleration due to gravity; and
	D	is a characteristic length.

Such models should not be distorted. The linear scale of the model, to which the model was constructed, is known as the geometric scale, λ. The scaling relationships between measurements in the model and in nature can be derived from Froude's law and some important scaling relationships are:

Length	=	λ	
Time	=	$\lambda^{1/2}$	
Velocity	=	$\lambda^{1/2}$	
Discharge	=	$\lambda^{5/2}$	
Mass	=	$\lambda^3\rho$	(where ρ is the fluid density ratio; 1.025, assuming sea water to be normally 1025 kg/m^3)
Force	=	$\lambda^3\gamma$	(where γ is the fluid specific weight ratio; 1.025)
Moments	=	$\lambda^4\gamma$	

In this report all values are quoted in prototype (full scale), unless stated differently. The specific gravity of water was assumed to be unity (1025 kg/m^3).

2.2 Flume

The study was carried out in HR Wallingford's Flume 2, see Figure 2.1, which has a usable area 40 m long, 1.2 m wide and has a maximum working depth of 1.5 m. The wave paddle is of piston-type and is controlled by HR Wallingford's Merlin software. It is equipped with an active wave-absorbing system to reduce the effect of waves reflected from the test section. The paddle can generate non-repeating random sea-states to any required spectral form, e.g. JONSWAP, Pierson Moskowitz, or user-defined forms - including bi-modal spectra. Second order wave effects, including long waves and set-down, are realistically represented.

A parabolic spending beach was designed and placed at the downstream end of the flume to absorb the waves, see Figure 2.1. This provided a consistent reflection coefficient between 10% and 17% of wave height and reduced corruption of waves during wave calibration and good dissipation of overtopping generated waves during testing.

The flume has a window on one side through which the test section can be viewed. The window is 3.3 m in length (including a narrow central pillar) and covers the majority of the flume height, allowing video of the wave/structure interactions to be recorded.

An array of four wave gauges were placed over the model calibration location at the toe of the caisson breakwater mount and used to define the incident waves. These wave probes were moved during testing of the structure to a distance of 1 wavelength from the caisson.

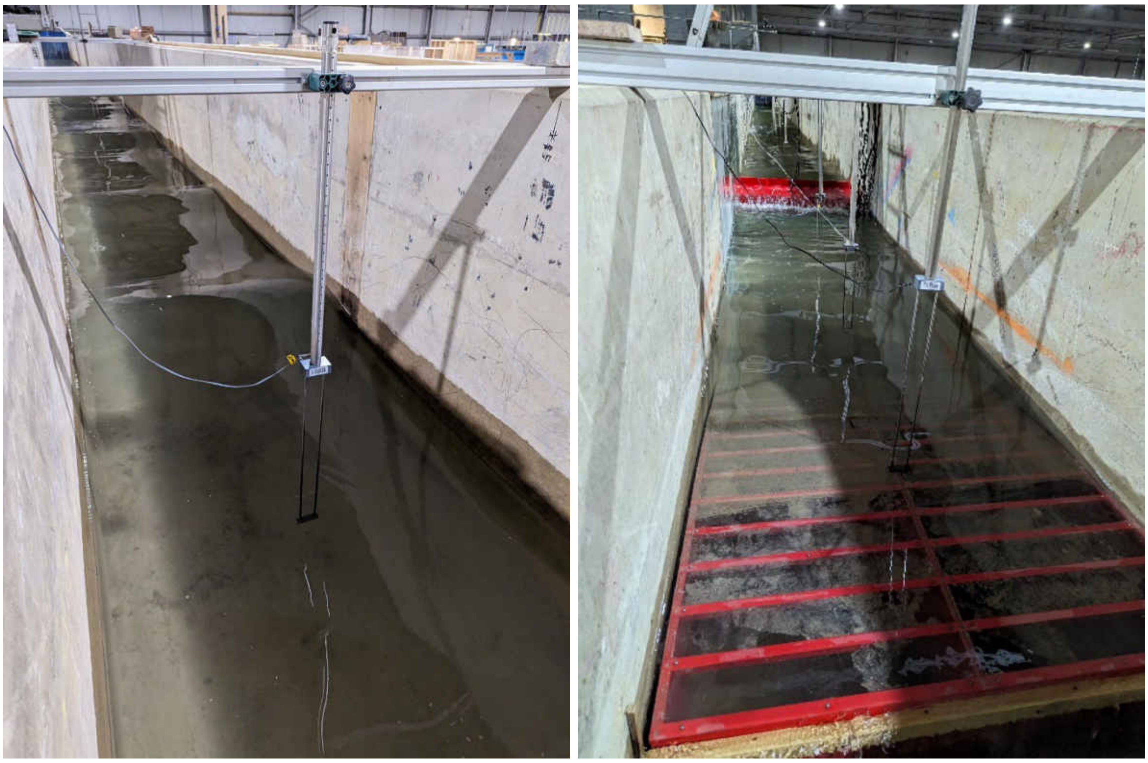


Figure 2.1: HR Wallingford's Flume 2; during wave calibration (left) and testing (right)

2.3 Bathymetry

A model bathymetry representing the near-shore approach slope at the test section was constructed in the flume. It was moulded from cement mortar placed over compacted sand fill, and extended sufficiently far offshore to accurately reproduce the near-shore processes of shoaling and breaking. The HYDRALAB III guidelines on the physical modelling of breakwaters (Ref. 1) were consulted in setting the length of the model bathymetry.

A review of the bathymetry over 3 to 5 wavelengths in front of the breakwater provided varying slope in the range of 1:1100 and 1:1300. A slope of 1:1200 was therefore placed running 490 m from the front face of the caisson. This section of bathymetry represented 3.3 to 4 wavelengths. At the end of the bathymetry an approach slope with a 1:35 slope was installed which continued to the flume floor set at -24.5 m, see Figure 2.2.

The caisson breakwater was constructed using a toe depth of -14.25 mMSL to simulate the breakwater section at its greatest depth. The details used for construction are given in Figure 2.3.

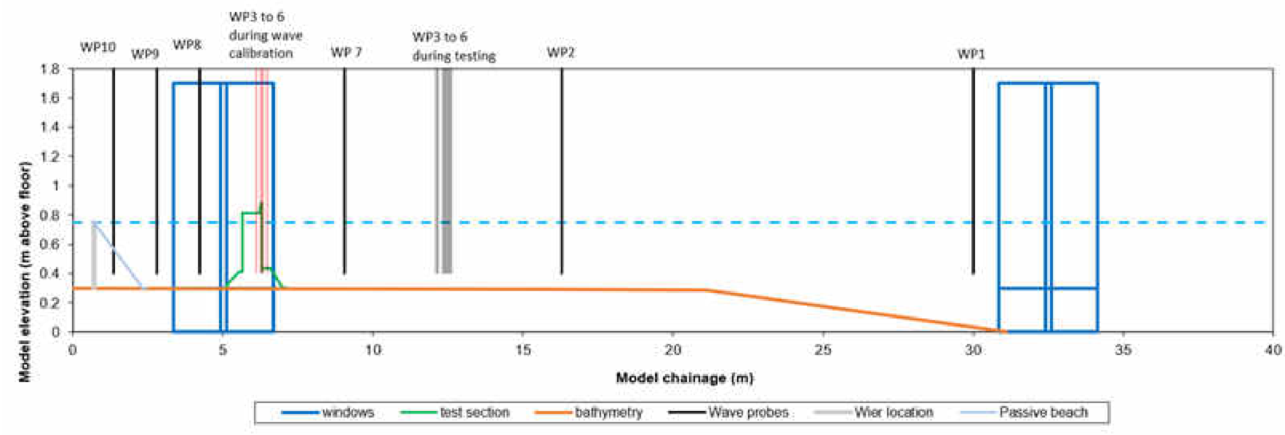


Figure 2.2: Flume bathymetry layout

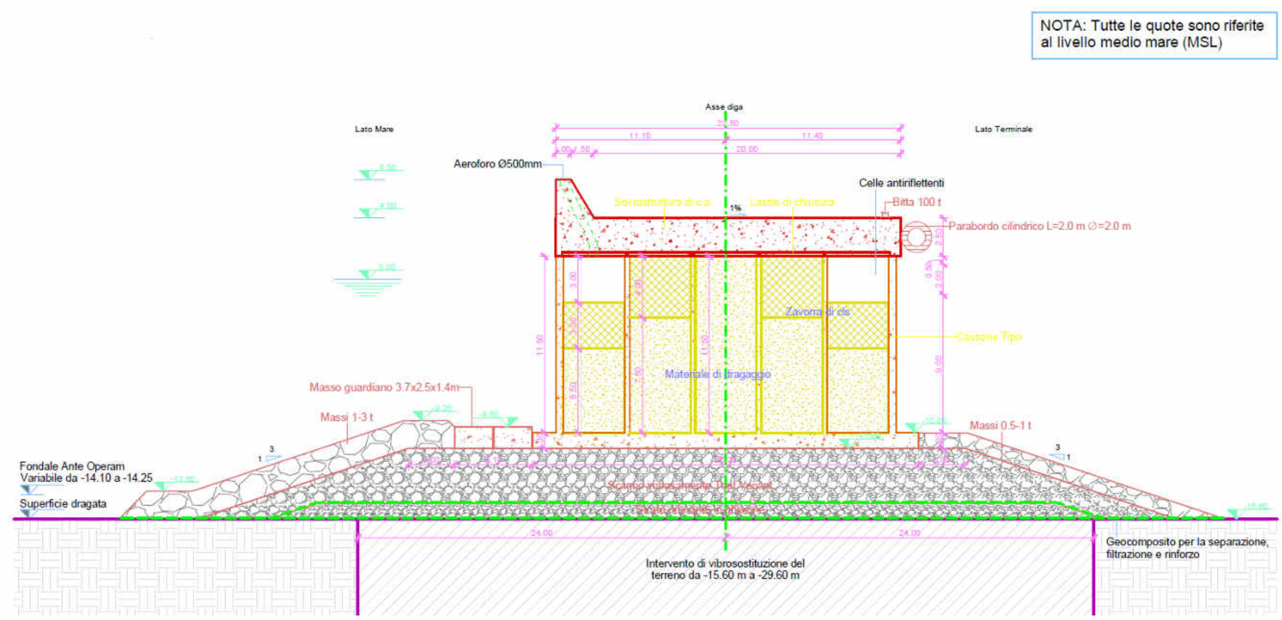


Figure 2.3: Caisson test cross-section (ref: 201064C-055-DW-3320-009 - Sezioni tipologiche)

2.4 Model caisson structure

The model caisson was designed using drawings provided by the client. It was constructed from marine plywood using a layout that provided a stiff structure under wave action, see Figure 2.4. Whilst the absorption chambers are not expected to provide much wave absorption during design conditions, they were included in the front face on the caisson to provide an insight into wave interaction behaviour during extreme conditions.

Pressure transducers were fitted to the centreline of the caisson (replacing an absorption chamber opening on the front face) to provide wave load distribution around the caisson perimeter. This layout provide wave loading for sections of the caisson without open absorption chambers and is expected to provide a worse case for horizontal sliding loads.

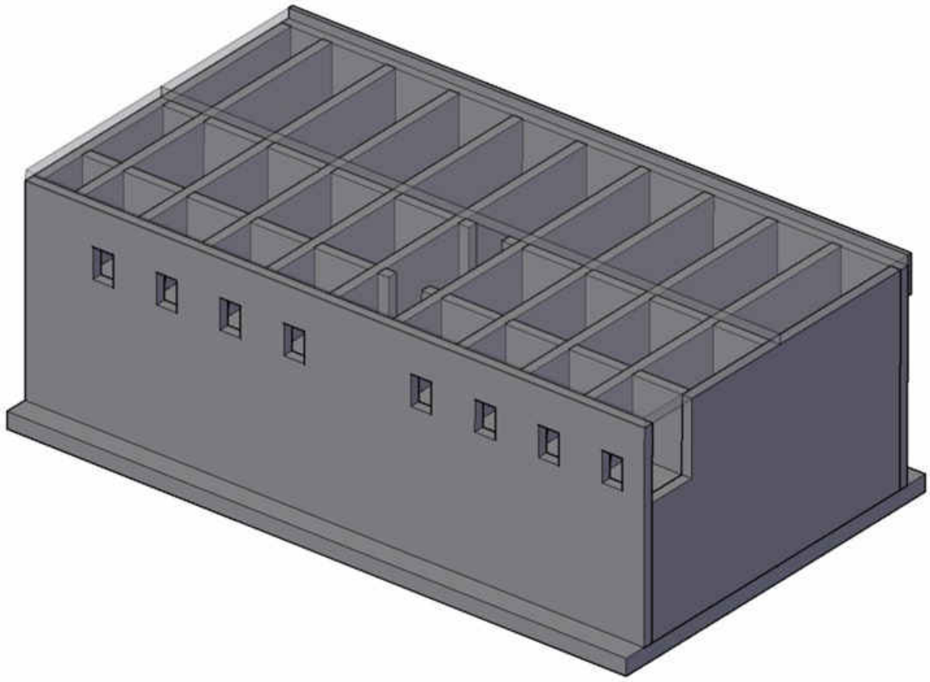


Figure 2.4: Model caisson design

Six wave wall designs were tested during the study, these were designed to form part of the model caissons lid, so they could be swapped out during testing, see Figure 2.5. The six designs are summarised as follows:

- Design 1 – Vertical wall with crest height +6.5 mMSL;
- Design 2 – Vertical wall with crest height +8.5 mMSL;
- Design 3 – Vertical wall with crest height +10.5 mMSL;
- Design 4 – recurved wall with crest height +6.5 mMSL;
- Design 5 – recurved wall with crest height +8.5 mMSL;
- Design 6 – recurved wall with crest height +10.5 mMSL.

Figure 2.5: Model caisson; wall design 1 (left) and 5 alternatives (right)

2.5 Instrumentation

During the 2D physical model study wave disturbance, overtopping and loads were measured.

2.5.1 Wave probes

In order to measure waves/water surface elevations in the model, twin wire resistance wave probes were used in conjunction with HR Wallingford's wave probe measurement units. The location of the wave probes during wave calibration and testing is given in Table 2.1. Wave probes were calibrated to check their linear response on a daily basis to ensure accurate results. Water surface elevations were acquired at 16.9 Hz prototype. The resolution of a wave probe is in the order of ±3.5 mm (prototype) and after accounting for instrumentation and data acquisition noise provides an accuracy of ±70 mm (prototype).

Table 2.1: Wave probe locations

Wave probe	Location	Chainage from caisson front face during testing (m)	Chainage from caisson front face during wave calibration (m)
WP 1	offshore	829.5	829.5
WP 2	offshore	349.3	349.3
WP 3 - Array1	1 wave length from caisson or at face for wave calibration	212.3	0.0
WP 4 - Array2		210.5	-1.7
WP 5 - Array3		203.5	-8.8
WP 6 - Array4		184.3	-28.0
WP 7	in front of test section	96.1	96.1
WP 8 - Ct 1	50 m behind Caisson	-72.5	-72.5
WP 9 - Ct 2	100 m behind caisson	-122.5	-122.5
WP 10 - Ct 3	150 m behind caisson	-172.5	-172.5

2.5.2 Overtopping tanks

Overtopping water was collected in a calibrated tank placed at the landward edge of the caisson breakwater in the centre of the flume. The volume of water collected during each test (approximately 1000 waves) was used to determine the mean overtopping discharge, q, per unit length of the structure (l/s/m).

A water level gauge installed in the tank recorded wave-by-wave overtopping. The output of this gauge was used to determine the maximum volumes associated with large individual waves (m^3/m). Tables of the largest overtopping volumes recorded during each test condition were compiled. For a visual impression of the overtopping, the video record of the tests can be viewed.

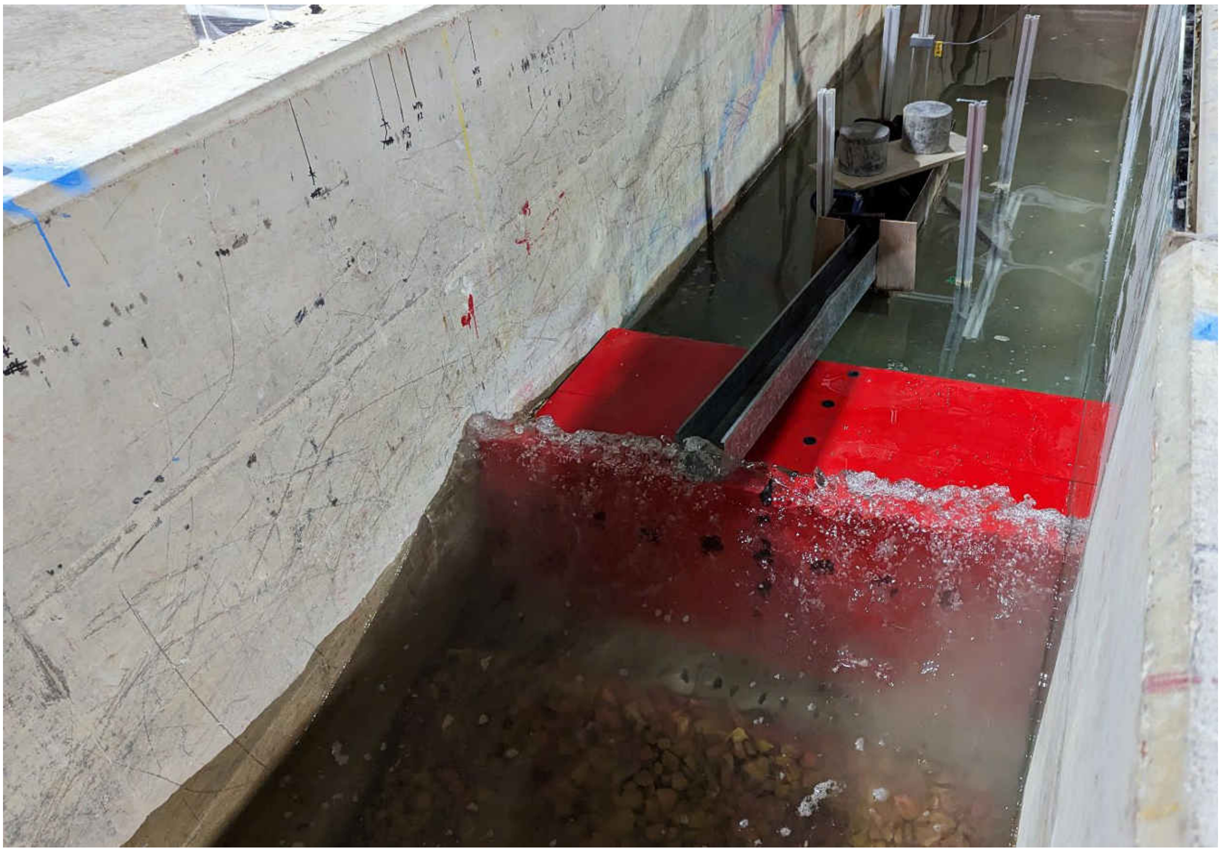


Figure 2.6: Overtopping shoot and tank behind caisson breakwater

2.5.3 Pressure transducers

A total of 24 pressure transducers were used for the study. These were mounted into the wooden caisson using 3D printed fittings to ensure a tight fit and a flush mounting to the caisson face, see Figure 2.7. The pressure transducers had stainless steel membrane to avoid damage and are designed to have a response rate up 10 kHz (model).

The pressure transducers were logged at 422.6 Hz prototype to ensure that any impulsive wave response would be captured.

All loading results were post-processed raw and by applying filters that are based on estimated dynamic response of the prototype structure, so that they represented static-equivalent global forces and moments.

It was suggested by the clients engineers, that the fundamental natural period of the stabilized caisson in situ is estimated at approximately 0.5-1.0 s. For precautionary approach, the filter was designed such that it remove loads with a response shorter than 0.25 to 0.5 s. The raw pressure data has been provided digitally with this report to allow future post processing to be completed, should the natural periods of the structure change or differ from the values given here.

The method used for post-processing of the pressure transducer is given in Section 3.4.

The unfiltered pressure signals have also been presented, to provide the client with information on the highest pressures acting on the caisson surface, including the recurve crown wall and overtopping-induced downfall pressures on the caisson deck slab.

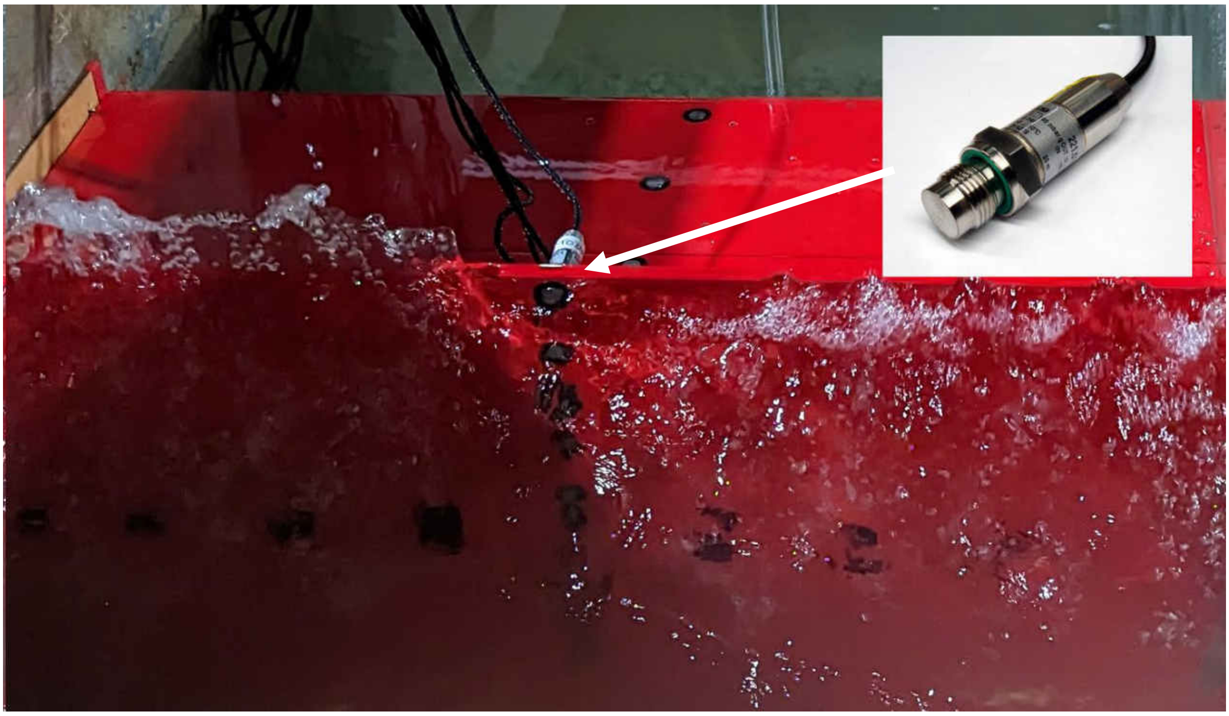


Figure 2.7: Pressure transducers

The positions of the pressure transducers are given below for the vertical wall, in Figure 2.8 and for the recurve wall in Figure 2.9.

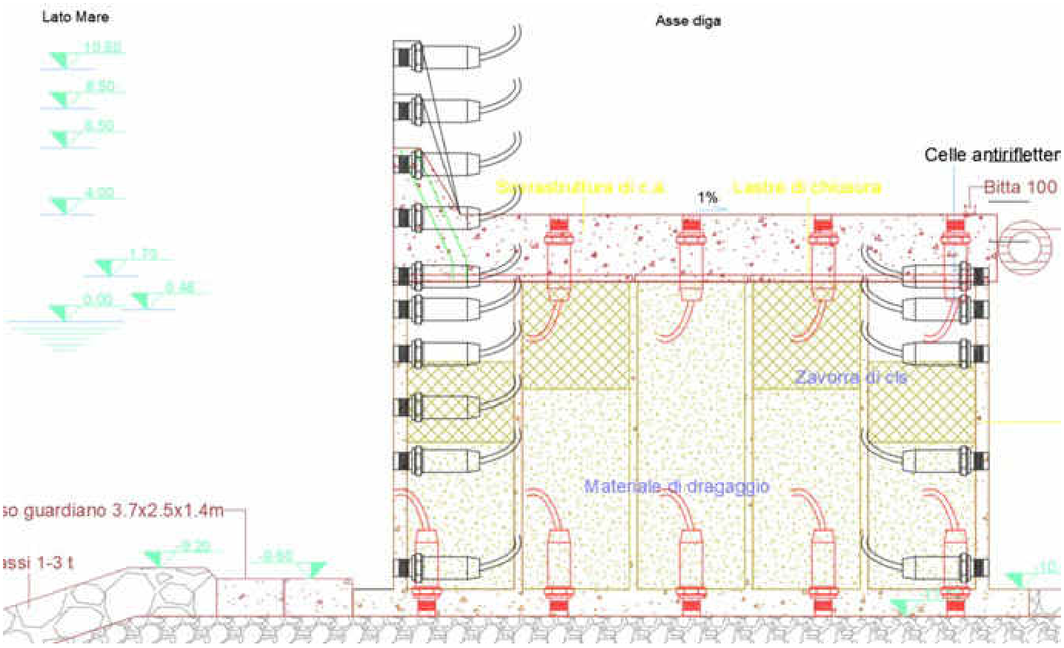


Figure 2.8: Vertical wall pressure transducer layout

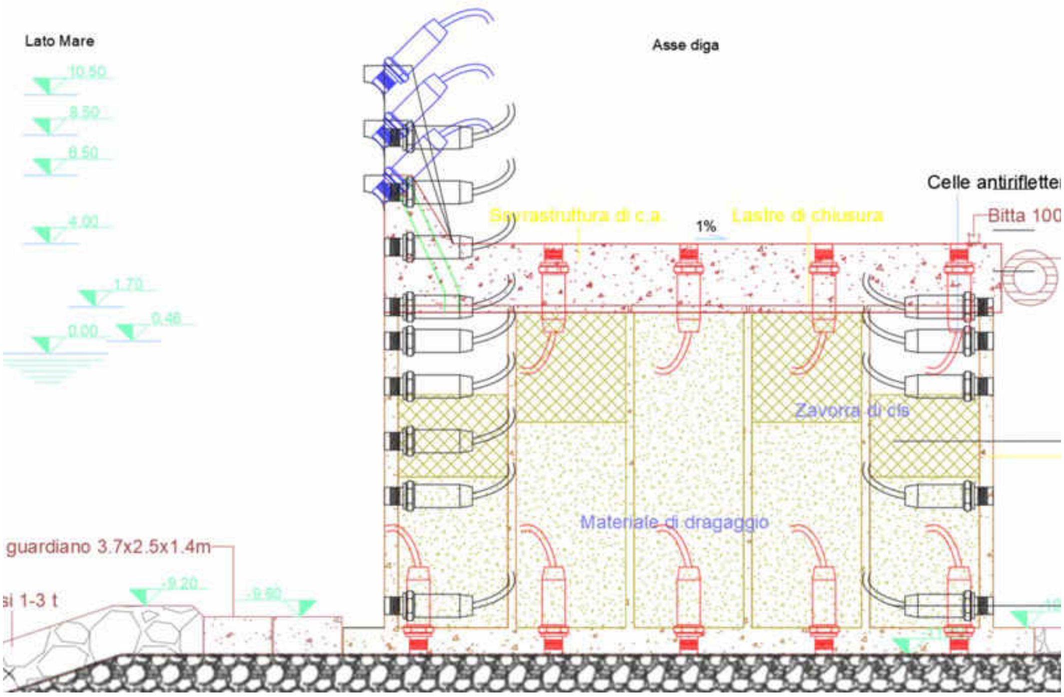


Figure 2.9: Recurve wall pressure transducer layout

2.5.4 Videos

Video of the tests were recorded with a camera located on a tripod viewing through the flume observation window. This video record has been provided digitally with this report.

3 Planning and design

This chapter of the report provided the reader with information on the methodology and test programme used during the 2D physical model study.

3.1 Wave and water level conditions

A total of 5 wave and water level conditions were used during the study.

Two design wave conditions defined as a 100 year event, one with a low water condition and the second with a high water level condition.

An overload condition was used to confirm the rock armour stability with a 120% increase in the design wave height, whilst maintaining wave steepness to enhance the peak wave period and two more frequently occurring conditions.

The design and overload wave conditions were calibrated before the breakwater was installed. The more frequency storm conditions used to investigate the transmission over the breakwater during its construction stage, before the wave wall is installed, were requested at a later stage in the study after wave calibration was complete.

These two wave and water level conditions were therefore not calibrated and the wave generation requirements were estimated (using knowledge gained from the completed calibrations) and the incident wave condition extracted using the reflection analysis method.

A summary of the wave and water level conditions used during the study are given in Table 3.1.

Table 3.1: Wave conditions

Incident wave condition	Return Period (years)	SWL (m MSL)	Significant wave heigh Hs (m)	Peak water period Tp (s)	JONSWAP γ factor
Design 1	100	0.46	5.50	9.60	3.3
Design 2	100	1.70	5.50	9.60	3.3
Overload	120%	0.46	6.60	10.90	3.3
Transmission 1	2	1.0	3.70	8.00	3.3
Transmission 2	10	1.20	4.50	8.70	3.3

3.2 Cassion mound

The caisson mound was constructed in the flume in stages using templates to guide position and depth of each rock layer. A cross section showing the mounds details is given in Figure 3.1.

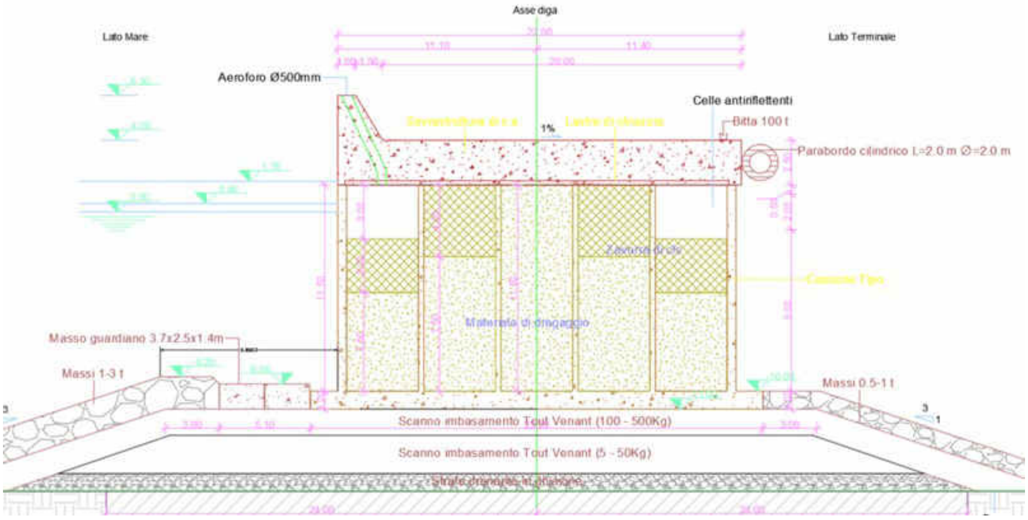


Figure 3.1: Caisson mound cross section

Following wave calibration an area of bathymetry was removed to represent the dredged pocket which holds the caisson breakwater, see Figure 3.2. Templates were then located in the dredged pocket and core material, made up of two sizes, placed. Following the core installation, underlayer and then rock armour was placed, leaving space for the caisson to be installed. The caisson was placed on the prepared mound and ballasted with lead to hold it in position.

After installation of the model caisson, toe blocks were added and the gaps in the rock armour closed, see Figure 3.3.

Figure 3.2: Breakwater mound construction sequence

Figure 3.3: Caisson toe blocks

3.2.1 Rock classes

The 1.0-3.0 t and 0.5-1.0 t rock armour classes were scaled using the Froude scaling laws to correctly reproduce its stability under wave attack, taking account of the differences between model and prototype rock densities, as well as model and prototype fluid densities. Details of the method are given in Appendix A.

The core mound/underlayer class (5-50 kg and 100-500 kg) were both scaled to reproduce the correct degree of permeability to water flow, thereby reducing scaling errors on armour stability. This scaling method involves making the materials slightly larger than they would be if only geometric scaling were to be applied. Details are given in Appendix B.

The following density values were provided and used in all scaling calculations:

- Armour rock density = 2,600 kg/m^3;
- Core rock density = 2,600 kg/m^3;
- Seawater density = 1,025 kg/m^3.

The model rock was crushed limestone with a density of 2,650 kg/m^3, and the model water density was assumed to be 1,000 kg/m^3.

The model rock classes were prepared by sieving and mixing standard crusher grades. Grade curves of the resulting classes were obtained: by weighing individual rocks in the case of the of the armour classes. The results are plotted in Figure 3.4 and Figure 3.5, along with the upper and lower limits of the specifications.

Collectively, the grade curves show that all rocks were adequately scaled for physical modelling purposes. The grading curve for the 0.5 t to 1.0 t rock provides a good distribution, with a slightly heavy M_{50}. As overtopping driven waves in the 2D tests are unlikely to be the driving factor for the design case the grading curve has been accepted for these tests.

The rocks were placed using methods that replicated the prototype construction techniques as closely as practicable. The core and filter layers were dumped from hand-held scoops, to profiles controlled by timber templates that had been levelled in position on the bathymetry with an optical level. The 0.5-1.0 t and 1.0-3.0 t armour rocks were placed in two distinct layers, to ensure that they were properly interlocked. The finished layers were spray painted and colour coded to help identify areas of damage.

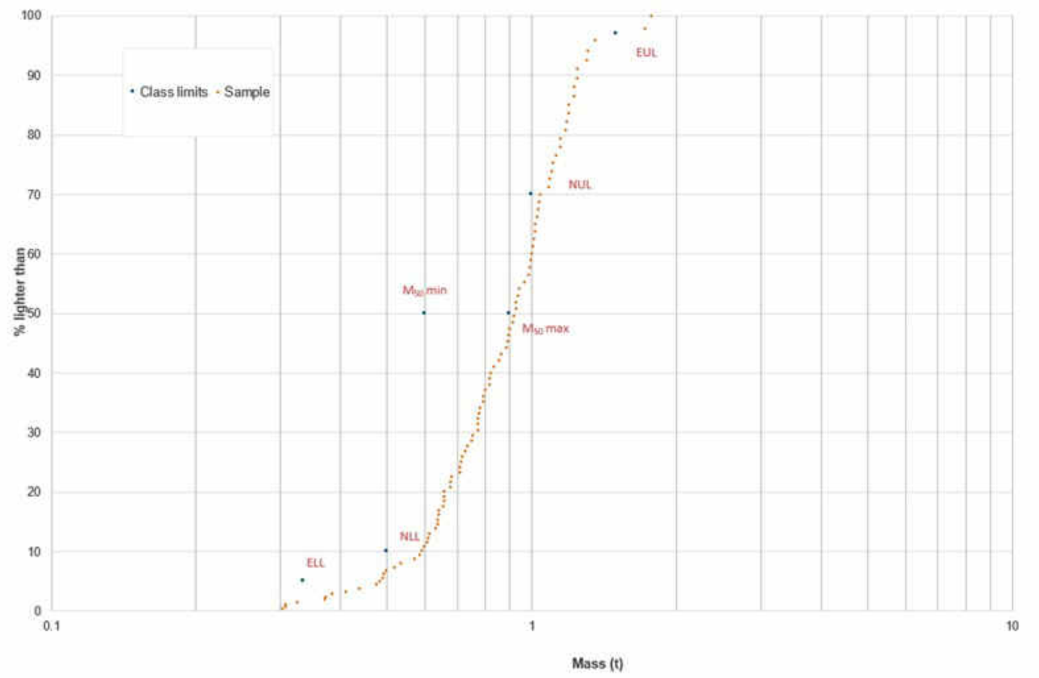


Figure 3.4: Rock grading curve for 0.5 t to 1 t rear armour

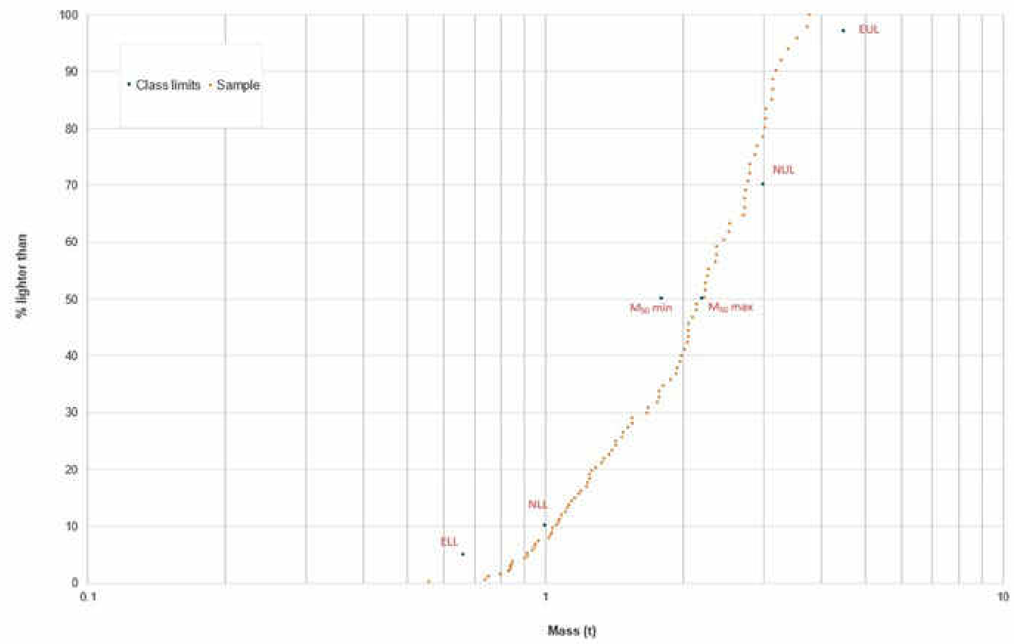


Figure 3.5: Rock grading curve for 1 t to 3 t seaward rock armour

Table 3.2: Armour rock class parameters

Class designation	y<5% (ELL)	y<10% (NLL)	y>70% (NUL)	y>97% (EUL)
0.5-1.0 t	335 kg	500 kg	1,000 kg	1,500 kg
1.0-3.0 t	665 kg	1,000 kg	3,000 kg	1,429 kg

3.2.2 Concrete toe blocks

The toe blocks were moulded from cement mortar with a barium additive that helped to control their density. They were scaled using the same Froude method as applied to the rock armour (described above). The following density value was provided for the scaling calculation:

- Concrete density = 2,300 kg/m³.

Allowing for water density correction, Froude scaling required a model block density of 2,244 kg/m³. All of the moulded blocks were sampled and found that the mass of the blocks fell within +2.6% (0.68 t) and -1.6% (0.42 t) of the target value of 26.6 t.

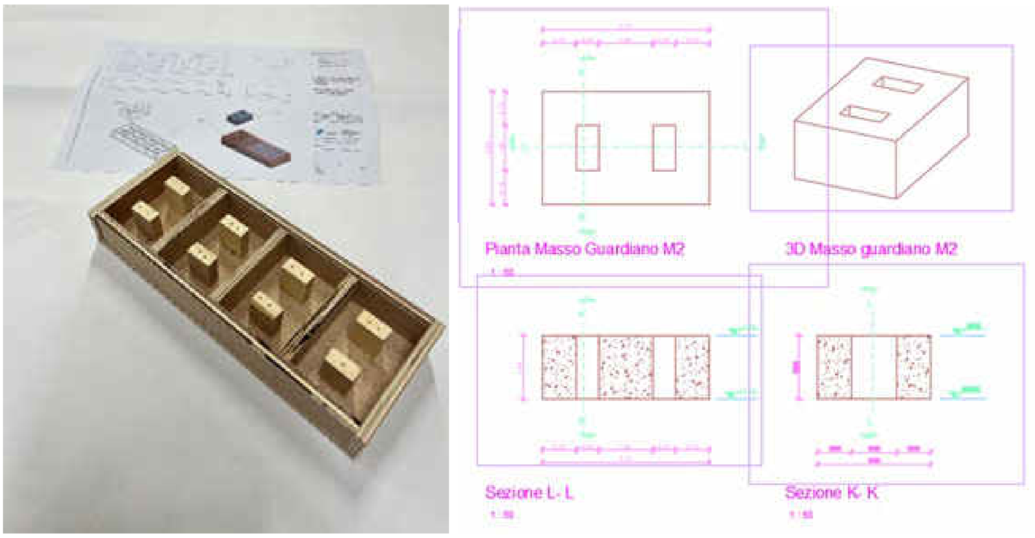


Figure 3.6: Toe block mounds

3.3 Test programme

Following construction of the bathymetry, the study was completed in 3 stages, with each stage consisting of a number of Test Series:

- Test Series A – Wave calibration without breakwater in place was completed in Test Series A1.
- Test Series B – Overtopping, wave loading, transmission and reflection tests investigating crest levels between +6.5 m and +10.5 m:
  - B1 - Wave loading, transmission & reflection;
  - B2 - Wave overtopping tests;
  - B3 - Transmission tests for phased caisson construction with flat crest levels at +3 m and +5 m.
- Test Series C – Armour and scour protection tests were completed in Test Series C1 us a vertical wall with a +10.5 m MSL crest level.

A table given details of all the tests completed in provided in Table 3.3. The armour/scour protection test were completed with a vertical wall with maximum crest level tested. The +10.5 mMSL vertical wall layout presented the worse scenario for the armour stability; being the most reflective, whilst allowing wave run up/draw down on the caisson breakwater face to act directly on the toe armour rather than being redirected seaward by a recurve.

Table 3.3: 2D model test programme

Test Series	Test	Breakwater configuration		Wave conditions					Description
		Crest elevation (m MSL)	With or without recurve	Return period Tr (yrs)	Hs (m)	Tp (m)	Wave spectra	Still water level (m MSL)	
A1 - Design wave calibration (without breakwater)	A1-1	-	-	100	5.5	9.6	JONSWAP γ = 3.3.	+1.7	Design & survival sea state wave calibration
	A1-2			100	5.5	9.6		+0.46	
	A1-3			overload (120% design)	6.6	TBC		+0.46	
B1 - Wave loading, transmission & reflection Tests	B1-1	+6.5	without	100	5.5	9.6	JONSWAP γ = 3.3.	+1.7	Using the design wave condition to investigate wave loading, transmission & reflections for varying breakwater crest elevations and recurve wall options
	B1-2	+8.5		100	5.5	9.6		+1.7	
	B1-3	+10.5		100	5.5	9.6		+1.7	
	B1-4	+6.5	with	100	5.5	9.6		+1.7	
	B1-5	+8.5		100	5.5	9.6		+1.7	
	B1-6	+10.5		100	5.5	9.6		+1.7	
B2 - Wave overtopping tests	B2-1	+6.5	without	100	5.5	9.6	JONSWAP γ = 3.3.	+1.7	Investigate overtopping for varying breakwater crest elevations and recurve wall options
	B2-2	+8.5		100	5.5	9.6		+1.7	
	B2-3	+10.5		100	5.5	9.6		+1.7	
	B2-4	+6.5	with	100	5.5	9.6		+1.7	
	B2-5	+8.5		100	5.5	9.6		+1.7	
	B2-6	+10.5		100	5.5	9.6		+1.7	
B3 - Transmission tests	B3 -1	+5.0	without	2	3.7	8	JONSWAP γ = 3.3.	+1.0	Evaluation of the transmission coefficient during the construction phases of the caisson's superstructure
	B3 -2	+5.0		10	4.5	8.7		+1.2	
	B3 -3	+3.0		2	3.7	8		+1.0	
	B3 -4	+3.0		10	4.5	8.7		+1.2	
C1 - armour and scour protection stability tests	C1-1	+10.5	without	100	5.5	9.6	JONSWAP γ = 3.3.	+0.46	Stability of the design scour protection rock and caisson toe blocks
	C1-2			overload (120% design)	6.6	10.9		+0.46	

3.4 Post processing pressures

In the design of the breakwater, slowly-varying wave loads (termed here as 'pulsating' loads) are treated as static loads without the need to consider dynamic effects. Short duration or 'impulsive' wave loads may be of greater intensity, but will also be of shorter duration. Such loads should not be treated as static, and their effects should be analysed taking account of the dynamic characteristics of the structure, as well as the magnitude and duration of the 'impulsive' load. Loads of durations shorter than the natural response period of the structure or structure element in question will be damped and this should therefore be taken into consideration when selecting design loads.

The wave forces derived in these tests will be used for global design of the breakwater structure, and also for the design of smaller discrete elements (e.g. panels, recurve wall) within the structure. Information on the dynamic characteristics of the structure and elements is required to process the pressure measurements.

The pressure transducers were logged at 422.6 Hz prototype to ensure that any impulsive wave response would be captured. This logging rate was selected based on information provided by the clients engineers.

It was suggested by the clients engineers, that the fundamental natural period of the stabilized caisson in situ be estimated at about 0.5-1.0 s.

All loading results were post-processed by applying filters that are based on the dynamic response of the prototype structure, so that they represented static-equivalent global forces and moments. This was based around a filter designed that remove loads with a response shorter than 0.25 to 0.5 s. The raw pressure data has been provided with this report to allow future post processing to be completed, should the natural periods of the structure change or diff from that given here.

For the analysis of impulsive loadings, the recorded wave pressure signals are analysed directly (after minor filtering to remove noise and transient signals). For pulsating loads, a more severe filter (using a Chebyshev type II digital filter with a cut-off frequency, f_c = 2 to 4 Hz at full scale, equivalent to an estimated natural response of the caisson, T_n=0.5 s) was applied to remove the short duration events whilst retaining information on pulsating loads. The characteristics of the filter are shown in Figure 3.7.

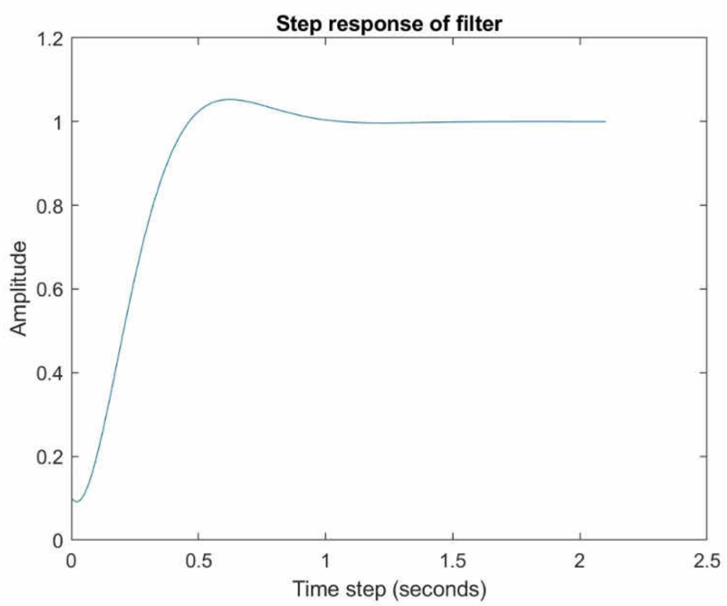


Figure 3.7: Filter characteristics for deriving pulsating wave loads

The signal filtering process means that the static-equivalent loading results are structure-specific. If the loads are to be applied to other structural designs, with different dynamic properties, the effective loadings should be re-calculated. For this reason the unfiltered data sets have been retained. The unfiltered data have been submitted digitally with this report and should be used for a full dynamic analysis of the caisson.

The data from the pressure transducers are pressures relative to a zero pressure defined by “no waves”, so net of any hydrostatic component. These pressures are used to compute a set of forces and moments, by integrating the pressures over the areas they represent. The naming convention used for the pressure transducers is given in Figure 3.8. A figure showing the representative areas for force calculation and lever arms for moment calculation are given in Figure 3.9. The values used for each test/wall configuration is given in Table 3.4.

Forces and moments where calculated as follows:

- F_x is the instantaneous horizontal force on the caisson, given by:

$$F_x = \sum_{n=15}^{n=24} P_n A_n - \sum_{n=5}^{n=9} P_n A_n$$

Or for recurve walls:

$$F_x = \sum_{n=15}^{n=22} P_n A_n + \sum_{n=22}^{n=24} \frac{P_n}{2^{0.5}} A_n - \sum_{n=5}^{n=9} P_n A_n$$

A positive value of F_x indicates that the force acts in a landward direction.

Pressures measured on the recurve was resolved into two force components, a vertical force and a horizontal force. Assuming that:

$$Fx = \cos A \times F_1$$
$$Fy = \sin A \times F_1$$

Where:

A = 45 degrees and therefore:

$$Fx = Fy = \sqrt{2}/2 \times F_1$$

Therefore the following was used to calculate the force acting positively vertically and positively horizontally:

$$F_h or F_v = \frac{pressure_n}{2^{0.5}} \times Area_n$$

- F_z is the instantaneous uplift force on the caisson, given by:

$$F_z = \sum_{n=10}^{n=14} P_n A_n - \sum_{n=1}^{n=4} P_n A_n$$

Or for recurve walls:

$$F_z = \sum_{n=10}^{n=14} P_n A_n - \sum_{n=1}^{n=4} P_n A_n - \sum_{n=22}^{n=24} \frac{P_n}{2^{0.5}} A_n$$

A positive value of F_z indicates that the force acts in an upward direction:

- M_y is the overturning moment calculated about the front toe (Mf) or rear heel (Mr) of the caisson, given by:

$$M_y = \sum_{n=1}^{n=24} \pm P_n A_n L_n$$

A positive value of M_y indicates that the moment acts in a clockwise direction. The moments were calculated separately for overturning about the front toe and the rear heel.

The input parameters to the above expressions were as follows:

- P_n = instantaneous pressure recorded by transducer n;
- A_n = area of the caisson surface associated with transducer n;
- L_n = lever arm, relative to the landward heel (Mr) or seaward toe (Mf), of transducer n;
- α = angle (to vertical) of the landward of the crown wall.

In the calculation of the above parameters, the following assumptions were made:

- All loading parameters were defined in terms of their value per metre run of the prototype structure;
- The pressure transducers were 'zeroed' before testing, so all pressures were measured relative to the pressure recorded at still water;
- The lever arm associated with each transducer was taken to be the distance from the landward or seaward toe to the centre of the area associated with each transducer. Pressure signals were inverted where required to ensure that each measurement made the correct contribution to the direction of the moments.
- The measured pressure is corrected assuming a prototype water density of 1025 kg/m^3.

The pressure transducer signals were processed as described above to give continuous time histories of the force and moment parameters for the full test duration.

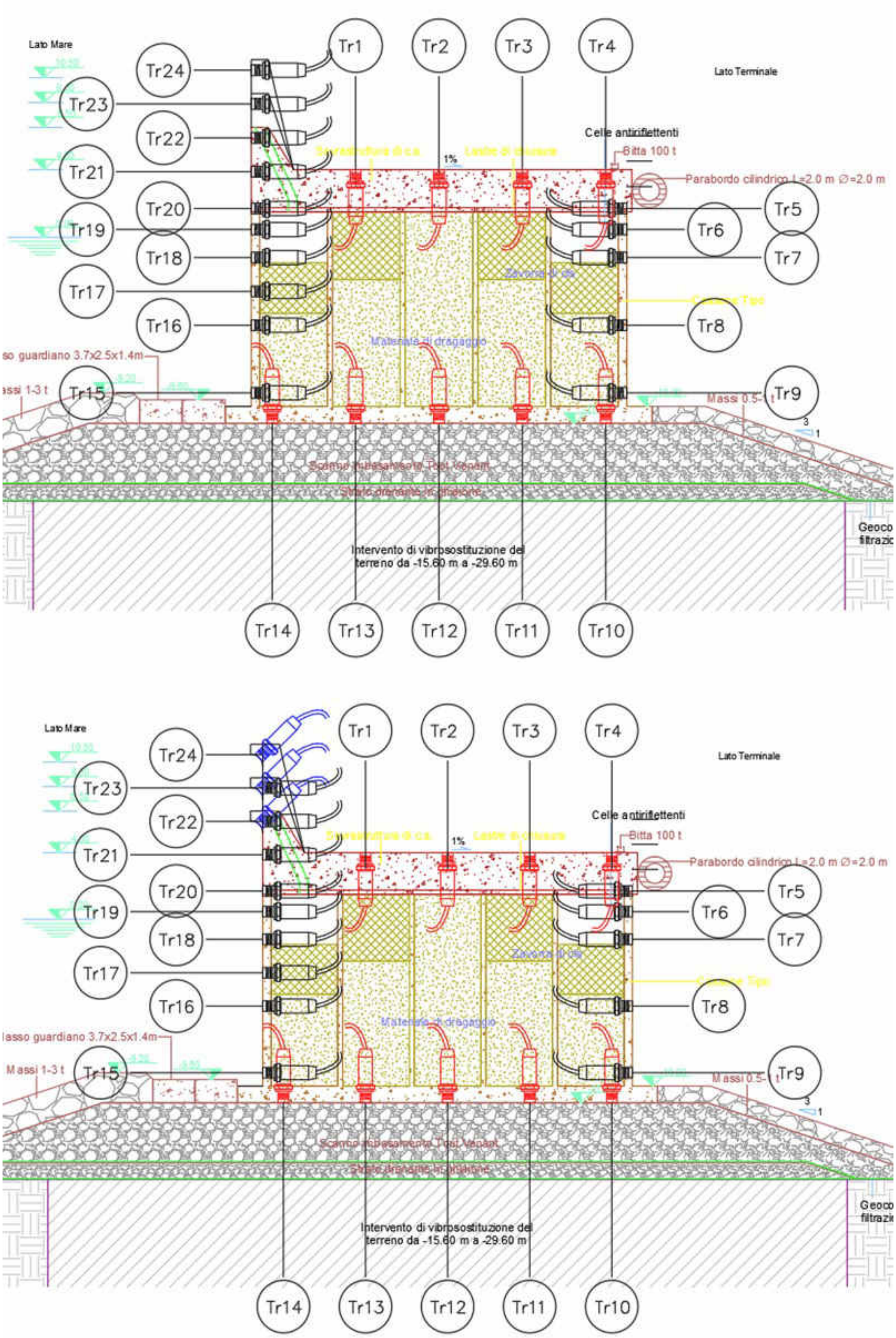


Figure 3.8: Pressure transducer naming convention; vertical wall (top) and recurve (bottom)

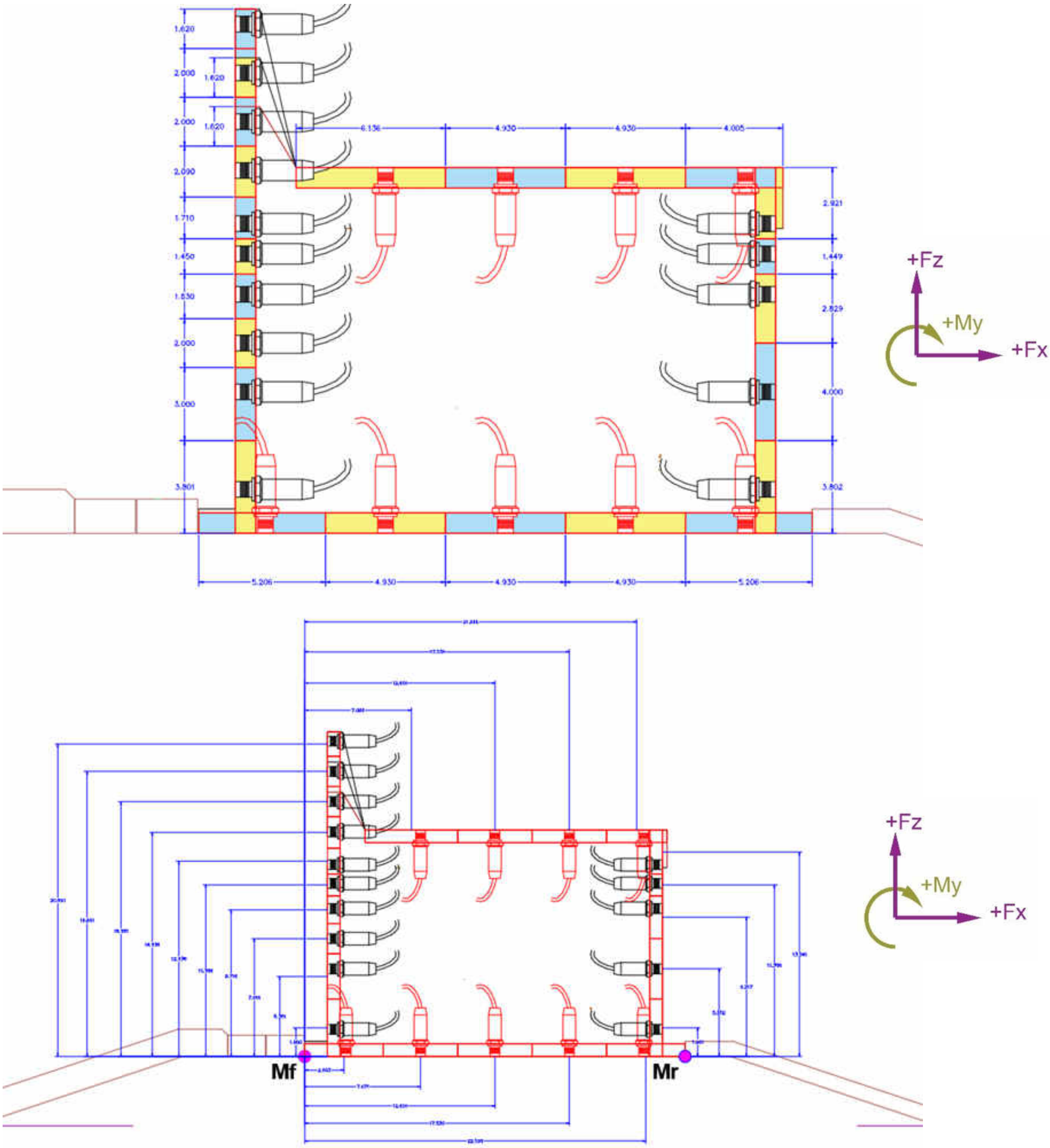


Figure 3.9: Pressure applied area (top) and lever arms to caisson from front toe (bottom)

Table 3.4: Areas and lever arms applied to measured pressures to derive forces and moments

Tr	Representative Areas (m)						Lever arms to heel for Mr (m)						Lever arms to toe for Mf (m)					
	B1-1	B1-2	B1-3	B1-4	B1-5	B1-6	B1-1	B1-2	B1-3	B1-4	B1-5	B1-6	B1-1	B1-2	B1-3	B1-4	B1-5	B1-6
1	6.14	6.14	6.14	6.14	6.14	6.14	18.14	18.14	18.14	18.14	18.14	18.14	7.07	7.07	7.07	7.07	7.07	7.07
2	4.93	4.93	4.93	4.93	4.93	4.93	12.60	12.60	12.60	12.60	12.60	12.60	12.60	12.60	12.60	12.60	12.60	12.60
3	4.93	4.93	4.93	4.93	4.93	4.93	7.67	7.67	7.67	7.67	7.67	7.67	17.53	17.53	17.53	17.53	17.53	17.53
4	4.01	4.01	4.01	4.01	4.01	4.01	3.20	3.20	3.20	3.20	3.20	3.20	22.00	22.00	22.00	22.00	22.00	22.00
5	2.92	2.92	2.92	2.92	2.92	2.92	13.54	13.54	13.54	13.54	13.54	13.54	13.54	13.54	13.54	13.54	13.54	13.54
6	1.45	1.45	1.45	1.45	1.45	1.45	11.36	11.36	11.36	11.36	11.36	11.36	11.36	11.36	11.36	11.36	11.36	11.36
7	2.83	2.83	2.83	2.83	2.83	2.83	9.22	9.22	9.22	9.22	9.22	9.22	9.22	9.22	9.22	9.22	9.22	9.22
8	4.00	4.00	4.00	4.00	4.00	4.00	5.80	5.80	5.80	5.80	5.80	5.80	5.80	5.80	5.80	5.80	5.80	5.80
9	3.80	3.80	3.80	3.80	3.80	3.80	1.90	1.90	1.90	1.90	1.90	1.90	1.90	1.90	1.90	1.90	1.90	1.90
10	5.21	5.21	5.21	5.21	5.21	5.21	2.61	2.61	2.61	2.61	2.61	2.61	22.60	22.60	22.60	22.60	22.60	22.60
11	4.93	4.93	4.93	4.93	4.93	4.93	7.67	7.67	7.67	7.67	7.67	7.67	17.53	17.53	17.53	17.53	17.53	17.53
12	4.93	4.93	4.93	4.93	4.93	4.93	12.60	12.60	12.60	12.60	12.60	12.60	12.60	12.60	12.60	12.60	12.60	12.60
13	4.93	4.93	4.93	4.93	4.93	4.93	17.54	17.54	17.54	17.54	17.54	17.54	7.67	7.67	7.67	7.67	7.67	7.67
14	5.21	5.21	5.21	5.21	5.21	5.21	22.60	22.60	22.60	22.60	22.60	22.60	2.60	2.60	2.60	2.60	2.60	2.60
15	3.80	3.80	3.80	3.80	3.80	3.80	1.90	1.90	1.90	1.90	1.90	1.90	1.90	1.90	1.90	1.90	1.90	1.90
16	3.00	3.00	3.00	3.00	3.00	3.00	5.30	5.30	5.30	5.30	5.30	5.30	5.30	5.30	5.30	5.30	5.30	5.30
17	2.00	2.00	2.00	2.00	2.00	2.00	7.80	7.80	7.80	7.80	7.80	7.80	7.80	7.80	7.80	7.80	7.80	7.80
18	1.83	1.83	1.83	1.83	1.83	1.83	9.72	9.72	9.72	9.72	9.72	9.72	9.72	9.72	9.72	9.72	9.72	9.72
19	1.45	1.45	1.45	1.45	1.45	1.45	11.36	11.36	11.36	11.36	11.36	11.36	11.36	11.36	11.36	11.36	11.36	11.36
20	1.71	1.71	1.71	1.71	1.71	1.71	12.94	12.94	12.94	12.94	12.94	12.94	12.94	12.94	12.94	12.94	12.94	12.94
21	2.09	2.09	2.09	2.09	2.09	2.09	14.88	14.88	14.88	14.88	14.88	14.88	14.88	14.88	14.88	14.88	14.88	14.88
22	1.62	2.00	2.00	2.04	2.00	2.00	16.78	16.78	16.78	16.78	16.78	16.78	16.78	16.78	16.78	16.78	16.78	16.78
23	-	1.62	2.00	-	2.04	2.00	-	18.87	18.87	-	18.87	18.87	-	18.87	18.87	-	18.87	18.87
24	-		1.62	-	-	2.04	-	-	20.69	-	-	20.69	-	-	20.69	-	-	20.69

3.5 Rock stability performance criteria

Damage was recorded by photographing the armour layer before and after each test and counting the cumulative number of rocks extracted, N. The damage was quantified in terms of the damage parameter, N_{od}, which is the average number of rocks extracted in each strip of width D_{n50}, where D_{n50} is the nominal diameter of the rock class:

- N_{od} = N / (W/D_{n50}):
  Where W is the width of the flume or length being analysed.

Damage was determined on the 1.5-3.0 t seaward side rock only as no observed damage was seen to be caused by wave overtopping. The damage criteria used in the study were based on recommendations for vertical structures in Section 5.2.2.9 of the Rock Manual (Ref. 2), which are as follows:

- N_{od} = 0.5 'almost no damage';
- N_{od} = 2.0 'acceptable damage';
- N_{od} = 5.0 'failure'.

As no formal damage criteria were specified by the client, the N_{od} values measured on the model were interpreted in the context of these definitions. All stability test were completed for a 3 hour storm duration.

3.6 Reflection and transmission coefficients

The calibration wave probe array (as described in Section 2.5.1) was placed approximately one wavelength from the caisson breakwater during testing and used to measure the reflection coefficient of the structure, i.e. the ratio between the reflected spectral significant wave height, H_{m0r}, and the incident spectral significant wave height, H_{m0i}.

The transmitted wave was recorded by wave probes placed at 3 locations on the berth side of the breakwater. These were approximately 50 m, 100 m and 150 m from the berth-side vertical face. The signal recorded by these gauges was first conditioned using a 100s high-pass filter to eliminate signal drift and other potential model effects caused by the pump and weir controlling the landward side water level.

To determine wave transmission coefficients, C_t the incident wave height at the seaward side of the breakwater was use:

$$C_t = \frac{H_{s\ transmitted}}{H_{s\ incient}}$$

No formal reflection or transmission criteria were specified, so these results are presented for information only.

4 Test Series A – wave calibration

A total of 3 wave conditions were calibrated at wave probe 3 (at the caisson face) for this study, see Figure 2.2 and Table 2.1.

Twin wire resistance wave probes were used to measure wave heights and periods. A four-probe linear array normal to the wave direction was used to measure reflected and incident wave conditions representative of those at the calibration position. Wave probes 3, 4, 5 and 6 where used in the array and moved one wavelength offshore during testing to limit the effects of node and antinode behaviour on the reflection analysis.

For each test, the measured spectra at the calibration point was compared to the theoretical demand and the significant wave height was compared to the target, see example plot in Figure 4.1.

A summary table of the calibrated wave conditions is given in this section of the report. In addition, the spectral distribution and peak period plots for each test, along with tabled values of wave conditions and reflection coefficients can be found in Appendix C.

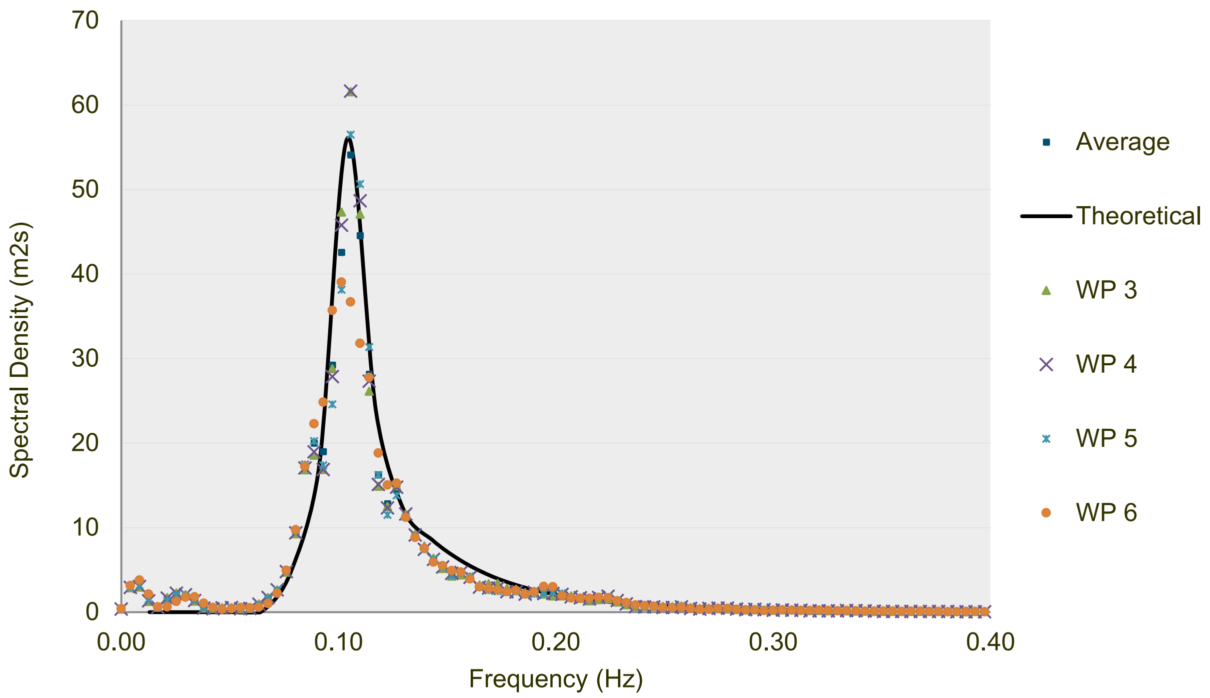


Figure 4.1: Example spectra plot for Wave condition A1-1 (Design condition 2 with HW)

4.1 Wave calibration method

The wave conditions were calibrated in the wave flume before installing the breakwater. The objective of the calibration tests was to produce the specified wave conditions at the calibration point, in this case, the location of the seaward face of caisson at wave probe 3. During wave calibration a parabolic beach was installed at the back of the flume, to reduce reflections and minimise uncertainty and ensure the incident wave height was correctly measured.

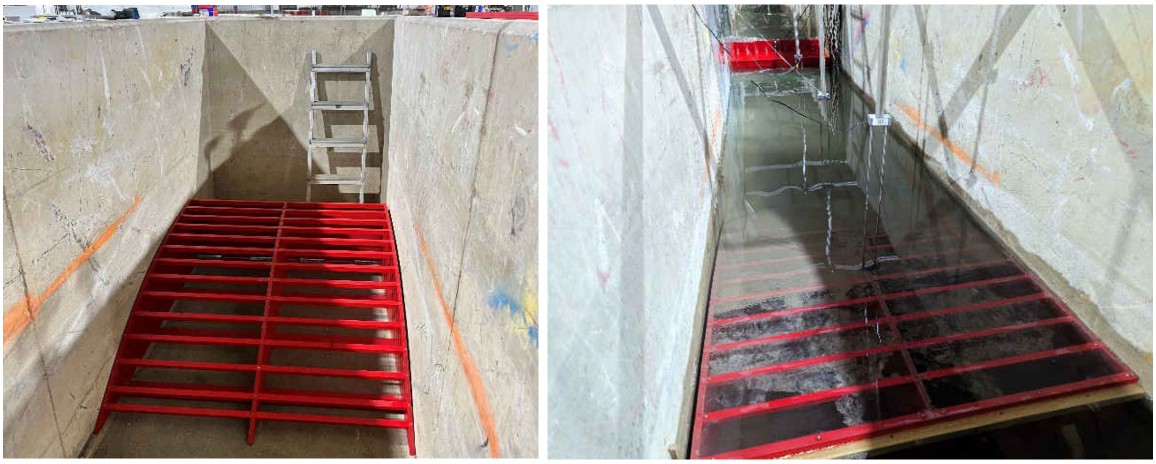


Figure 4.2: Absorption parabolic beach at end of flume to limit reflections and control SWL

Calibration was an iterative process, with the amplitude of the signal driving the wave generator being adjusted until the spectral significant wave height measured at the calibration point was within a tolerance of ±5% of the target significant wave height. An absorption beach was used to limit reflected waves and reduce multi-directional reflections at the calibration point.

For each wave condition, an in-line array of four wave probes were used to measure the incident wave. Time histories recorded by each gauge in the array were analysed spectrally by HR Wallingford's HR DAQ software to provide the following parameters:

- Significant incident spectral wave height, H_{m0};
- Peak spectral wave period, T_p of the incident wave spectrum;
- The mean spectral wave period, $T_{m0,2}$, defined using the 0th and 2nd moments of the incident spectrum;
- The spectral energy wave period, $T_{m-1,0}$, defined using the -1st and 0th moments of the incident spectrum.

Data recorded by the array was analysed to determine the reflection coefficient and adjust the measured total significant wave height to the incident spectral parameters defined above. For each condition, the wave generation produced non-repeating wave sequences, with durations of at least 1000 times the mean wave period, $T_{m0,2}$, of the target spectrum in order to gain a good description of the wave spectra. This gave a statistically significant sample for the wave calibration analysis.

The reflection analysis was based on measuring the wave height at the four wave-probe array (at differing and known gauge spacings in constant water depth). The method calculates the incident and reflected wave spectral energy and the reflection coefficient using the method of Isaacson (1991) at frequencies spread over the relevant frequency range. The reflection coefficient (C_r) was calculated using the following equation:

$$C_r = \sqrt{\frac{m_{0r}}{m_{0i}}} \qquad 1$$

Where m_{0i} and m_{0r} are the incident and reflected wave spectral energy respectively.

The total wave height (H_{Tot}) is therefore expressed as:

$$H_{Tot} = \sqrt{H_{m0i}^2 + H_{m0r}^2} \qquad 2$$

This can be expressed in terms of the incident wave height (H_{m0i}) as:

$$H_{Tot} = \sqrt{H_{m0i}^2 + (C_r H_{m0i})^2} \quad 3$$

or:

$$H_{m0i} = \frac{H_{Tot}}{\sqrt{1 + C_r^2}} \quad 4$$

This method was also used to estimate the reflection coefficient from the beach and later the caisson, during a number of tests completed during test series B (see Section 5).

4.2 Wave calibration results

A summary of the wave conditions calibrated during Test Series A is presented in Table 4.1. The measured refection coefficient from absorption beach placed at the end of the flume is given in addition to the measured total and incident significant waves heights. Further measured wave parameters and plotted wave spectra are given in Appendix C.

Table 4.1: Results of wave calibration

Test	Wave condition	Target			Measured				H_s incident Error (%)
		Spectral type	H_{m0} (m)	T_p (s)	Total H_s (m)	Incident H_s (m)	Cr	T_p (s)	
A1-1	Design 1	JONSWAP (γ = 3.3)	5.5	9.6	5.41	5.33	0.17	9.6	-3.1
A1-2	Design 2		5.5	9.6	5.44	5.41	0.10	9.6	-1.6
A1-3	Overload		6.6	10.9	6.58	6.51	0.16	10.9	-1.4

The wave calibrations was completed to the required ±5% of the target value. Reflections were kept to a minimum to provide confident in the incident condition being generated from the wave paddles and spectral shape and peaks where found to match well that of the theoretical target conditions (see Appendix C).

4.3 Additional wave conditions

During testing additional wave conditions were requested to investigate the transmission coefficient of the caisson breakwater without the crown wall installed. Two flat crest levels were investigated +3 mMSL and +5 mMSL, both for two waves conditions:

- 2 year return period Hs = 3.7 m and Tp = 8 s;
- 10 year return period Hs = 4.5 m and Tp = 8.7 s.

The wave generation system settings for these wave conditions were estimated and the measured values recorded during testing given in Table 4.2.

Table 4.2: Measured wave conditions during testing

Test	Wave condition	Target			Measured				H_s inciden t Error (%)
		Spectral type	H_{m0} (m)	T_p (s)	Total H_s (m)	Incident H_s (m)	Cr	T_p (s)	
B3-1	Transmission 1	JONSWAP (γ = 3.3)	3.7	8.0	4.73	3.71	0.79	8.0	0.1
B3-2	Transmission 2		4.5	8.7	5.75	4.53	0.79	8.7	0.6
B3-3	Transmission 1		3.7	8.0	4.41	3.62	0.70	8.0	-2.1
B3-4	Transmission 2		4.5	8.7	5.36	4.44	0.68	8.7	-1.4

5 Test Series B – caisson performance

Test Series B was used to investigate the caisson breakwater performance. The test completed during Test Series B are given in Table 5.1.

Table 5.1: Test Series B test matrix

Test Series	Test	Breakwater configuration		Wave conditions				
		Crest elevation (m MSL)	With or without recurve	Return period Tr (yrs)	Hs (m)	Tp (m)	Wave spectra	SWL (m MSL)
B1 **Wave loading, transmission & reflection Tests**	B1-1	+6.5	without	100	5.5	9.6	JONSWAP γ = 3.3	+1.7
	B1-2	+8.5		100	5.5	9.6		+1.7
	B1-3	+10.5		100	5.5	9.6		+1.7
	B1-4	+6.5	with	100	5.5	9.6		+1.7
	B1-5	+8.5		100	5.5	9.6		+1.7
	B1-6	+10.5		100	5.5	9.6		+1.7
B2 **Wave overtopping tests**	B2-1	+6.5	without	100	5.5	9.6	JONSWAP γ = 3.3	+1.7
	B2-2	+8.5		100	5.5	9.6		+1.7
	B2-3	+10.5		100	5.5	9.6		+1.7
	B2-4	+6.5	with	100	5.5	9.6		+1.7
	B2-5	+8.5		100	5.5	9.6		+1.7
	B2-6	+10.5		100	5.5	9.6		+1.7
B3 **Transmission tests**	B3 -1	+5.0	without	2	3.7	8	JONSWAP γ = 3.3	+1.0
	B3 -2	+5.0		10	4.5	8.7		+1.2
	B3 -3	+3.0		2	3.7	8		+1.0
	B3 -4	+3.0		10	4.5	8.7		+1.2

5.1 Test Series B1 – Wave loading, transmission and reflections

Wave load results here are presented as pulsating (quasi static) and impulsive. The pulsating loads have been extracted using a filtered pressure signal, whilst the impulsive are those extracted from the unfiltered time series signals.

5.1.1 Pulsating wave loads

The static-equivalent time histories were analysed to determine the highest individual loading events, an event being a local (temporal) maximum or minimum of force or moment that can be associated with an individual wave, which was defined using zero-crossing analysis of the lower front pressure transducer Tr15. Events were identified for the following loading types:

- *Fh_Max* is the horizontal landwards force;
- *Fh_min* is the horizontal seawards force;
- *Fv_max* is the vertical upwards force;
- *Fv_min* is the vertical downwards force;
- *Mr_max* is the landward overturning moment, calculated about the landward toe;
- *Mf_min* is the seaward overturning moment, calculated about the seaward toe.

For each force, moment and pressure , the individual events were ranked in order to allow the following statistical parameters to be calculated:

- The maximum event;
- The 1/250 event, i.e. the average of the highest 1/250th of the events.

The maximum horizontal and vertical loadings may not occur simultaneously; this is an important consideration to make when determining the caisson's resistance to sliding. For each force and moment events in the time histories, the instantaneous values of the other forces and pressures that occurred simultaneously with the peaks of the events are also extracted.

The 1/250th forces, moments and pressures are plotted in Figure 5.1 and Figure 5.2. The results are also presented in Table 5.2. The top for events, ranked by horizontal force, are given in Table 5.3.

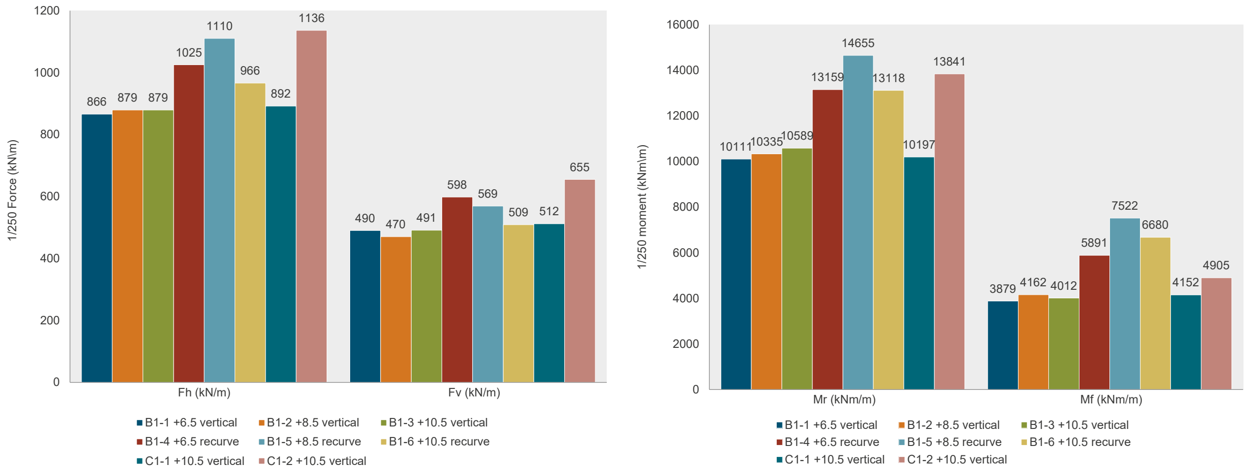


Figure 5.1: pulsating 1/250th forces (left) and moments (right)

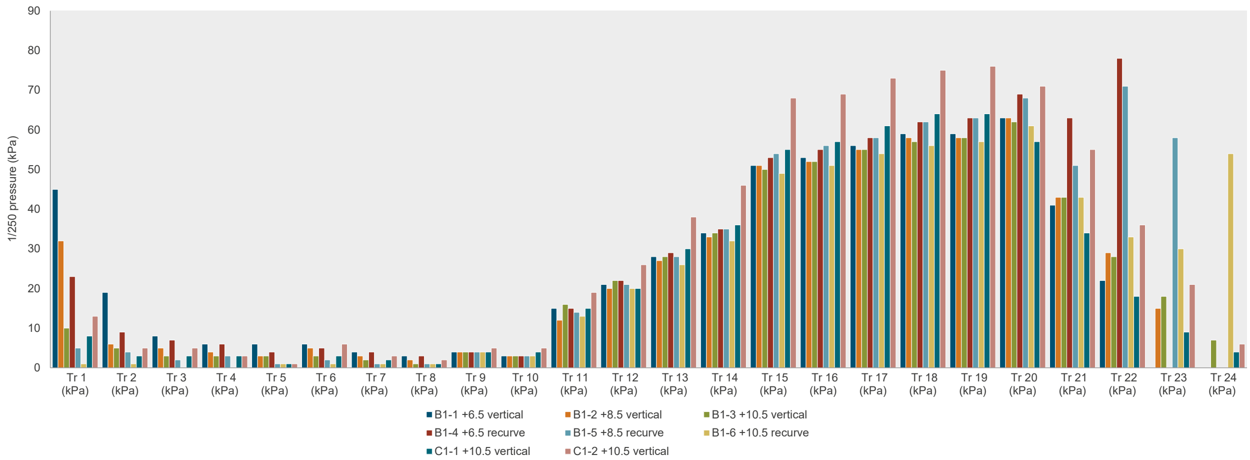


Figure 5.2: pulsating 1/250th pressures

The statistical and time history data from which these results are plotted has been provide digitally in comma separated value (csv) format with this report.

Negative horizontal forces generated by the wave trough are given in Table 5.4 along with the values of uplift force, moments and pressures are time coincident within the top 4 negative horizontal force events.

Table 5.2: Pulsating 1/250th forces, moments and pressures

Test	Wall	Fh (kN/m)	Fv (kN/m)	Mr (kNm/m)	Mf (kNm/m)	Tr 1 (kPa)	Tr 2 (kPa)	Tr 3 (kPa)	Tr 4 (kPa)	Tr 5 (kPa)	Tr 6 (kPa)	Tr 7 (kPa)	Tr 8 (kPa)	Tr 9 (kPa)	Tr 10 (kPa)	Tr 11 (kPa)	Tr 12 (kPa)	Tr 13 (kPa)	Tr 14 (kPa)	Tr 15 (kPa)	Tr 16 (kPa)	Tr 17 (kPa)	Tr 18 (kPa)	Tr 19 (kPa)	Tr 20 (kPa)	Tr 21 (kPa)	Tr 22 (kPa)	Tr 23 (kPa)	Tr 24 (kPa)
B1-1	+6.5 vertical	866	490	10111	3879	45	19	8	6	6	6	4	3	4	3	15	21	28	34	51	53	56	59	59	63	41	22	n/a	n/a
B1-2	+8.5 vertical	879	470	10335	4162	32	6	5	4	3	5	3	2	4	3	12	20	27	33	51	52	55	58	58	63	43	29	15	n/a
B1-3	+10.5 vertical	879	491	10589	4012	10	5	3	3	3	3	2	1	4	3	16	22	28	34	50	52	55	57	58	62	43	28	18	7
B1-4	+6.5 recurve	1025	598	13159	5891	23	9	7	6	4	5	4	3	4	3	15	22	29	35	53	55	58	62	63	69	63	78	n/a	n/a
B1-5	+8.5 recurve	1110	569	14655	7522	5	4	2	3	1	2	1	1	4	3	14	21	28	35	54	56	58	62	63	68	51	71	58	n/a
B1-6	+10.5 recurve	966	509	13118	6680	1	1	0	0	1	1	1	1	4	3	13	20	26	32	49	51	54	56	57	61	43	33	30	54
C1-1	+10.5 vertical	892	512	10197	4152	8	3	3	3	1	3	2	1	4	4	15	20	30	36	55	57	61	64	64	57	34	18	9	4
C1-2	+10.5 vertical	1136	655	13841	4905	13	5	5	3	1	6	3	2	5	5	19	26	38	46	68	69	73	75	76	71	55	36	21	6

Table 5.3: Top 4 events ranked by positive horizontal force (Fh), with time coincident values of uplift, moments and pressures

Test	Wall	Event	Fh (kN/m)	Fv (kN/m)	Mr (kNm/m)	Mf (kNm/m)	Tr 1 (kPa)	Tr 2 (kPa)	Tr 3 (kPa)	Tr 4 (kPa)	Tr 5 (kPa)	Tr 6 (kPa)	Tr 7 (kPa)	Tr 8 (kPa)	Tr 9 (kPa)	Tr 10 (kPa)	Tr 11 (kPa)	Tr 12 (kPa)	Tr 13 (kPa)	Tr 14 (kPa)	Tr 15 (kPa)	Tr 16 (kPa)	Tr 17 (kPa)	Tr 18 (kPa)	Tr 19 (kPa)	Tr 20 (kPa)	Tr 21 (kPa)	Tr 22 (kPa)	Tr 23 (kPa)	Tr 24 (kPa)
B1-1	+6.5 vertical	1	925	505	10885	2797	2	1	1	1	1	2	1	1	-3	3	16	23	29	35	53	55	59	62	64	68	43	14	n/a	n/a
		2	869	482	10170	2443	1	1	1	0	0	0	0	0	-3	3	14	21	27	34	50	53	56	59	60	63	37	11	n/a	n/a
		3	860	488	10031	2234	1	1	1	1	0	0	0	0	-3	3	14	21	28	34	52	53	55	57	57	60	35	9	n/a	n/a
		4	808	427	9343	2443	1	1	1	0	0	-1	-1	0	-2	3	13	19	24	30	44	48	52	56	57	60	34	9	n/a	n/a
B1-2	+8.5 vertical	1	926	451	10869	3799	0	1	1	1	0	3	1	0	-3	3	11	19	26	33	50	52	56	60	62	66	43	25	8	n/a
		2	890	424	10601	3883	0	1	0	0	0	0	0	0	-2	3	11	18	24	30	45	48	52	56	58	63	43	27	12	n/a
		3	886	474	10395	2841	0	1	0	0	0	1	0	0	-3	3	12	20	27	34	51	53	56	58	58	62	36	18	5	n/a
		4	814	407	9385	2988	1	1	0	0	0	-1	-1	0	-2	3	10	17	23	29	45	47	50	53	54	58	34	17	5	n/a
B1-3	+10.5 vertical	1	957	482	11355	3716	2	1	0	0	0	0	0	0	-3	3	14	21	28	34	52	54	58	61	62	65	44	24	10	1
		2	952	488	11510	3617	3	1	0	1	0	-1	0	0	-3	3	16	22	28	34	49	52	57	61	63	67	45	25	10	0
		3	822	445	10132	2897	2	1	0	0	0	0	0	0	-2	2	15	20	25	30	43	46	49	52	54	58	38	22	10	1
		4	784	451	9183	1871	2	1	0	0	0	0	0	0	-3	2	15	20	26	31	47	48	50	52	52	54	29	10	3	-1
B1-4	+6.5 recurve	1	1099	622	14208	6188	2	2	1	1	0	2	1	0	-3	3	15	23	30	36	54	57	59	66	68	74	65	81	n/a	n/a
		2	1042	629	13407	4639	1	1	0	0	-1	0	0	0	-3	3	16	23	31	37	55	57	59	63	64	68	56	61	n/a	n/a
		3	983	572	12506	4899	2	1	0	0	0	0	-1	0	-3	3	14	21	28	33	51	53	55	58	58	62	54	67	n/a	n/a
		4	975	555	12277	4671	2	1	0	0	0	0	0	0	-3	3	14	21	27	33	49	53	56	61	63	68	53	55	n/a	n/a
B1-5	+8.5 recurve	1	1195	574	15916	8236	1	1	0	0	0	0	0	0	-3	3	14	21	28	34	52	55	59	65	68	74	55	77	64	n/a
		2	1159	628	15569	6793	0	0	0	0	0	0	0	0	-3	3	15	23	30	37	55	58	61	65	67	72	48	60	61	n/a
		3	1049	508	13799	6902	0	1	0	0	0	0	0	0	-2	3	11	18	25	31	49	50	53	57	59	63	43	69	52	n/a
		4	1037	522	13320	6222	1	1	0	0	0	0	0	0	-3	3	12	19	26	33	52	53	55	58	58	62	41	56	49	n/a
B1-6	+10.5 recurve	1	1105	552	15041	7316	0	0	0	0	0	0	0	0	-3	3	13	20	27	33	50	53	56	60	62	67	50	38	32	52
		2	930	473	12360	5587	1	1	0	0	0	0	0	0	-3	3	12	18	24	29	44	46	49	52	54	59	43	30	21	36
		3	922	500	12836	6286	0	0	0	0	0	0	0	0	-2	2	11	17	23	28	44	45	47	51	53	56	35	22	19	68
		4	906	437	11823	5561	1	1	0	0	0	0	0	0	-2	2	11	17	22	28	42	45	48	52	54	59	41	29	22	31
C1-1	+10.5 vertical	1	967	534	11263	2660	1	1	0	0	0	0	0	0	-3	4	16	21	32	37	55	59	64	68	68	60	35	17	3	0
		2	897	516	10340	1992	1	1	0	0	0	0	0	0	-3	4	15	19	31	35	54	56	61	64	65	57	29	8	1	0
		3	864	472	9713	2176	1	1	1	1	0	-1	-1	-1	-3	3	14	19	28	33	52	54	58	62	62	55	28	7	1	-1
		4	839	447	9397	2334	0	0	0	0	0	0	0	0	-3	3	11	16	27	32	52	53	56	58	58	51	27	11	3	0
C1-2	+10.5 vertical	1	1199	591	13969	4696	1	1	0	0	0	0	0	0	-3	4	16	21	35	43	65	68	74	79	81	77	53	30	11	0
		2	1150	663	14326	3597	1	0	0	0	0	1	1	0	-4	4	20	25	38	46	63	66	70	74	76	72	52	32	10	1
		3	1101	569	13562	4442	1	1	0	0	0	0	0	0	-4	5	15	22	34	40	57	60	65	68	70	66	49	34	21	3
		4	1093	587	13318	3945	1	0	0	0	0	0	0	0	-3	4	16	22	35	41	59	62	66	70	72	67	45	32	17	-1

Table 5.4: Top 4 events ranked by negative horizontal force (Fh), with time coincident values of uplift, moments and pressures

Test	Wall	Event	Fh (kN/m)	Fv (kN/m)	Mr (kNm/m)	Mf (kNm/m)	Tr 1 (kPa)	Tr 2 (kPa)	Tr 3 (kPa)	Tr 4 (kPa)	Tr 5 (kPa)	Tr 6 (kPa)	Tr 7 (kPa)	Tr 8 (kPa)	Tr 9 (kPa)	Tr 10 (kPa)	Tr 11 (kPa)	Tr 12 (kPa)	Tr 13 (kPa)	Tr 14 (kPa)	Tr 15 (kPa)	Tr 16 (kPa)	Tr 17 (kPa)	Tr 18 (kPa)	Tr 19 (kPa)	Tr 20 (kPa)	Tr 21 (kPa)	Tr 22 (kPa)	Tr 23 (kPa)	Tr 24 (kPa)
B1-1	+6.5 vertical	1	-607	-662	-7887	3177	3	3	3	2	0	2	1	1	4	-4	-15	-24	-34	-43	-60	-64	-50	-28	-10	1	1	1	n/a	n/a
		2	-597	-759	-8612	4350	4	4	3	3	0	-2	-3	-2	4	-5	-19	-28	-37	-47	-65	-65	-50	-28	-10	0	1	1	n/a	n/a
		3	-595	-657	-7706	3181	1	1	1	1	0	1	1	0	4	-5	-17	-26	-35	-43	-61	-61	-50	-28	-10	1	1	1	n/a	n/a
		4	-588	-599	-7384	2543	3	3	2	1	3	4	2	1	3	-4	-13	-22	-31	-39	-57	-58	-50	-28	-10	1	1	0	n/a	n/a
B1-2	+8.5 vertical	1	-627	-765	-8695	4064	1	1	1	1	0	-2	-2	-1	4	-5	-22	-31	-41	-50	-67	-69	-49	-27	-10	0	0	0	0	n/a
		2	-601	-680	-7935	3388	1	1	0	0	0	0	-1	-1	4	-4	-20	-28	-36	-44	-62	-64	-50	-27	-10	0	0	0	0	n/a
		3	-598	-671	-7876	3318	1	1	0	1	0	3	1	0	4	-4	-19	-27	-35	-44	-60	-62	-50	-27	-10	1	0	0	0	n/a
		4	-596	-696	-8231	3539	3	2	2	2	0	2	2	1	4	-4	-18	-27	-36	-44	-59	-60	-50	-27	-10	1	0	0	0	n/a
B1-3	+10.5 vertical	1	-600	-684	-7895	3438	3	2	1	1	0	-1	0	0	4	-5	-17	-26	-36	-45	-64	-65	-49	-27	-10	1	1	1	1	0
		2	-590	-623	-7307	2875	2	1	0	0	0	0	0	0	4	-5	-15	-24	-33	-42	-62	-65	-49	-27	-10	1	1	1	2	1
		3	-586	-613	-7203	2779	2	1	0	0	0	-1	-1	0	3	-5	-15	-24	-33	-42	-61	-65	-49	-27	-10	1	1	1	1	0
		4	-586	-654	-7551	3144	2	1	0	0	0	0	0	0	4	-5	-16	-26	-35	-44	-62	-62	-49	-27	-10	1	1	1	1	0
B1-4	+6.5 recurve	1	-614	-751	-8570	4031	3	2	1	1	0	-2	-1	-1	4	-5	-20	-30	-39	-48	-65	-68	-49	-27	-10	0	2	1	n/a	n/a
		2	-587	-656	-7719	3232	2	1	1	1	0	1	1	1	4	-5	-17	-26	-34	-43	-59	-61	-49	-27	-10	1	1	1	n/a	n/a
		3	-587	-640	-7572	3027	2	1	1	1	0	2	1	1	4	-5	-16	-25	-33	-42	-59	-61	-49	-27	-10	0	1	1	n/a	n/a
		4	-581	-652	-7649	3176	3	2	1	2	0	2	1	1	4	-5	-15	-25	-34	-43	-58	-59	-49	-27	-10	1	2	1	n/a	n/a
B1-5	+8.5 recurve	1	-622	-751	-8497	4156	1	2	0	0	0	0	0	0	4	-5	-22	-31	-40	-49	-68	-67	-49	-27	-10	0	1	1	5	n/a
		2	-600	-689	-7843	3708	0	1	0	0	0	-1	-1	-1	4	-5	-20	-29	-37	-46	-63	-67	-49	-27	-9	1	1	1	5	n/a
		3	-574	-651	-7461	3414	2	1	0	0	0	0	0	0	4	-5	-18	-26	-35	-44	-61	-61	-49	-27	-9	1	1	1	5	n/a
		4	-573	-658	-7502	3561	1	1	0	0	0	0	0	0	4	-4	-19	-28	-35	-43	-60	-61	-49	-27	-10	1	1	1	5	n/a
B1-6	+10.5 recurve	1	-629	-749	-8490	3876	1	1	0	0	0	0	0	0	4	-5	-21	-30	-40	-50	-68	-67	-50	-27	-10	0	1	1	1	0
		2	-596	-653	-7564	3164	1	0	0	0	0	-1	-1	0	4	-5	-18	-27	-35	-44	-62	-65	-50	-27	-10	0	1	0	1	0
		3	-591	-665	-7659	3321	1	1	0	0	0	0	0	0	4	-5	-18	-27	-36	-44	-61	-63	-50	-27	-10	0	1	1	1	0
		4	-590	-644	-7434	3115	0	0	0	0	0	-1	-1	-1	4	-5	-18	-26	-35	-43	-61	-64	-50	-27	-10	1	1	1	1	0
C1-1	+10.5 vertical	1	-553	-756	-8065	4311	1	1	0	0	0	0	0	0	5	-6	-22	-27	-42	-51	-69	-58	-39	-17	0	0	1	0	1	0
		2	-512	-651	-7103	3550	1	1	0	0	0	0	0	0	4	-5	-19	-23	-36	-44	-59	-58	-39	-17	0	0	1	0	1	0
		3	-509	-705	-7580	4182	0	1	0	1	0	0	0	0	4	-4	-23	-26	-39	-46	-57	-58	-39	-17	0	0	0	0	0	0
		4	-508	-669	-7307	3811	1	1	0	0	0	0	0	0	4	-4	-22	-24	-37	-44	-57	-58	-39	-17	0	0	1	0	0	0
C1-2	+10.5 vertical	1	-527	-701	-7642	4008	1	1	0	1	0	0	0	0	5	-5	-20	-26	-39	-46	-60	-58	-39	-17	0	0	1	0	0	0
		2	-525	-675	-7366	3768	1	1	1	0	0	0	0	0	4	-5	-19	-24	-37	-45	-60	-58	-39	-17	0	0	0	0	1	0
		3	-525	-701	-7618	4015	1	1	1	0	0	0	0	0	4	-5	-21	-26	-38	-46	-61	-58	-39	-17	0	0	0	0	1	0
		4	-523	-698	-7559	4077	1	1	1	1	0	0	-2	-1	4	-5	-21	-25	-38	-45	-61	-58	-39	-17	0	0	0	0	1	0

5.1.2 Impulsive waves loads

The impulsive time histories were analysed to determine the highest individual loading events, in the same way that the pulsating loads were analysed. Events were identified for the following loading types:

- *Unfiltered_Fh_Max* is the horizontal landwards force;
- *Unfiltered_Fh_min* is the horizontal seawards force;
- *Unfiltered_Fv_max* is the vertical upwards force;
- *Unfiltered_Fv_min* is the vertical downwards force;
- *Unfiltered_Mr_max* is the landward overturning moment, calculated about the landward toe;
- *Unfiltered_Mf_min* is the seaward overturning moment, calculated about the seaward toe.

For each force, moment and pressure , the individual events were ranked in order to allow the following statistical parameters to be calculated:

- The maximum event;
- The 1/250 event, i.e. the average of the highest 1/250th of the events.

The 1/250th forces, moments and pressures are plotted in Figure 5.1 and Figure 5.2. he results are also presented in Table 5.5 and Table 5.6.

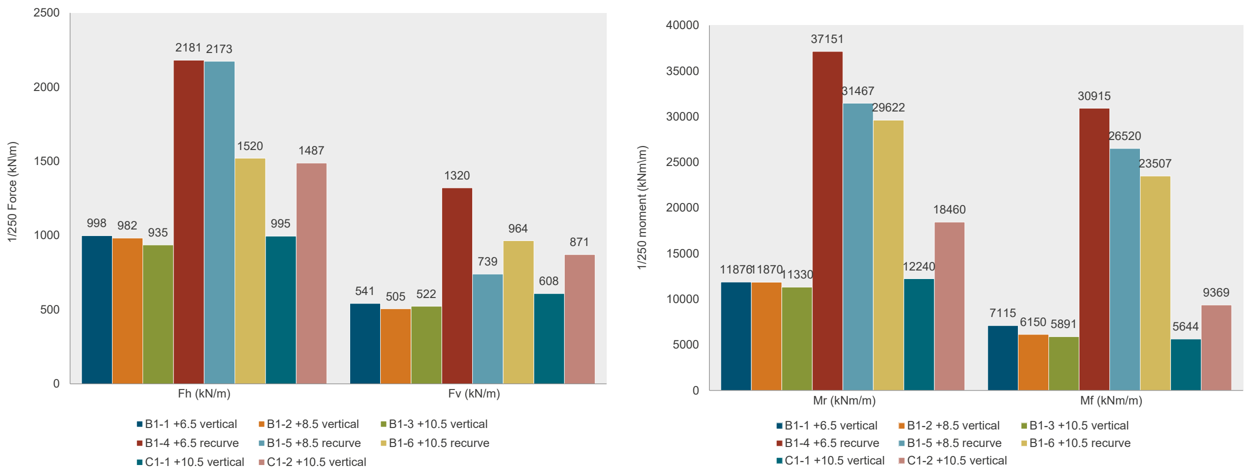


Figure 5.3: impulsive (Unfiltered) 1/250th forces (left) and moments (right)

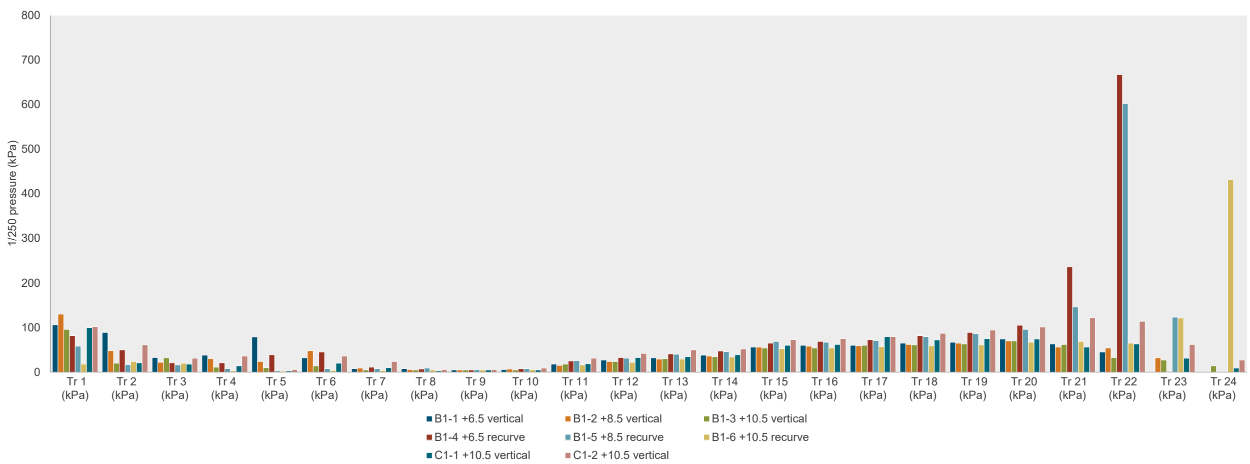


Figure 5.4: pulsating 1/250th pressures

Impulsive negative horizontal forces generated by the wave trough are given in Table 5.7 along with the values of uplift force, moments and pressures are time coincident within the top 4 negative horizontal force events.

From the pressure transducers results, it can be seen that the recurve wall exhibited high impulsive loads on the recurve. The impulse associated with the impact wave loads are plotted below, showing the vertical wall vs the recurve, see Figure 5.5. This clearly shows the recurve wall attracts impulsive wave loads.

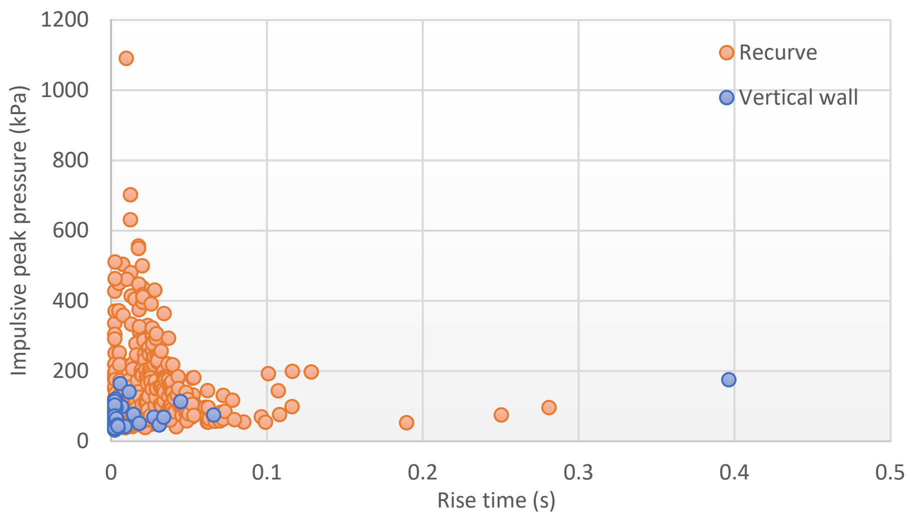


Figure 5.5: Vertical wall vs recurve wall event rise time and peak pressure

The distribution of rise time and impulsive load for each of the recurve designs is given in Figure 5.6.

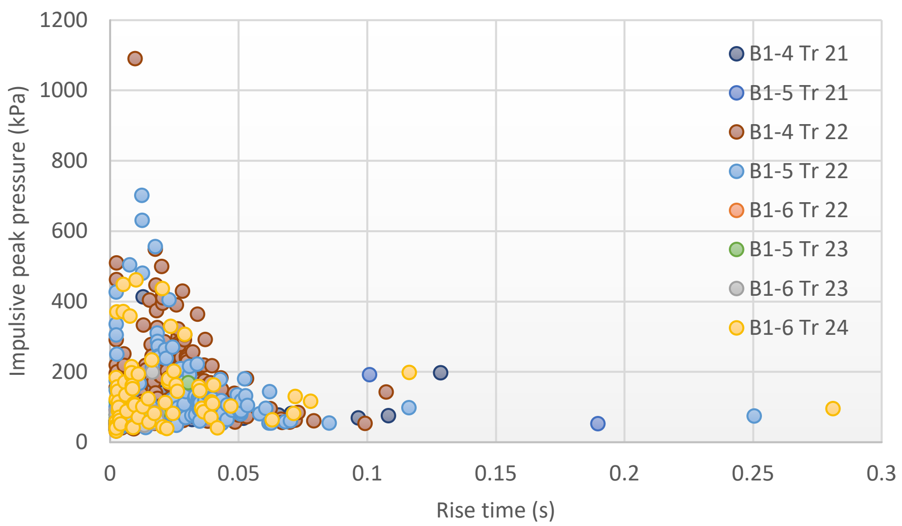


Figure 5.6: Recurve wall impulses

Table 5.5: Unfiltered 1/250th forces, moments and pressures

Test	Wall	Fh (kN/m)	Fv (kN/m)	Mr (kNm/m)	Mf (kNm/m)	Tr 1 (kPa)	Tr 2 (kPa)	Tr 3 (kPa)	Tr 4 (kPa)	Tr 5 (kPa)	Tr 6 (kPa)	Tr 7 (kPa)	Tr 8 (kPa)	Tr 9 (kPa)	Tr 10 (kPa)	Tr 11 (kPa)	Tr 12 (kPa)	Tr 13 (kPa)	Tr 14 (kPa)	Tr 15 (kPa)	Tr 16 (kPa)	Tr 17 (kPa)	Tr 18 (kPa)	Tr 19 (kPa)	Tr 20 (kPa)	Tr 21 (kPa)	Tr 22 (kPa)	Tr 23 (kPa)	Tr 24 (kPa)
B1-1	+6.5 vertical	998	541	11876	7115	106	89	33	38	79	32	8	8	5	6	18	27	32	38	56	60	60	65	67	74	63	45	n/a	n/a
B1-2	+8.5 vertical	982	505	11870	6150	130	48	22	30	24	48	9	6	5	7	15	24	29	36	56	58	59	62	65	70	56	54	32	n/a
B1-3	+10.5 vertical	935	522	11330	5891	96	20	32	11	10	14	5	5	5	5	18	24	30	35	54	54	60	61	63	70	62	33	27	14
B1-4	+6.5 recurve	2181	1320	37151	30915	82	50	21	21	39	45	11	7	5	8	25	33	41	47	65	69	73	82	89	105	236	667	n/a	n/a
B1-5	+8.5 recurve	2173	739	31467	26520	58	17	16	8	3	8	8	9	6	8	26	31	40	46	69	67	71	80	86	96	146	602	123	n/a
B1-6	+10.5 recurve	1520	964	29622	23507	18	24	20	3	2	4	4	5	5	6	16	22	29	34	53	54	57	59	61	67	69	65	121	432
C1-1	+10.5 vertical	995	608	12240	5644	100	21	18	14	3	20	10	3	5	5	19	33	35	39	60	62	80	72	75	74	56	63	31	9
C1-2	+10.5 vertical	1487	871	18460	9369	102	61	31	36	6	36	24	6	6	9	31	42	50	52	73	75	80	87	94	101	122	114	62	27

Table 5.6: Unfiltered Top 4 events ranked by positive horizontal force (Fh), with time coincident values of uplift, moments and pressures

Test	Wall	Event	Fh (kN/m)	Fv (kN/m)	Mr (kNm/m)	Mf (kNm/m)	Tr 1 (kPa)	Tr 2 (kPa)	Tr 3 (kPa)	Tr 4 (kPa)	Tr 5 (kPa)	Tr 6 (kPa)	Tr 7 (kPa)	Tr 8 (kPa)	Tr 9 (kPa)	Tr 10 (kPa)	Tr 11 (kPa)	Tr 12 (kPa)	Tr 13 (kPa)	Tr 14 (kPa)	Tr 15 (kPa)	Tr 16 (kPa)	Tr 17 (kPa)	Tr 18 (kPa)	Tr 19 (kPa)	Tr 20 (kPa)	Tr 21 (kPa)	Tr 22 (kPa)	Tr 23 (kPa)	Tr 24 (kPa)
B1-1	+6.5 vertical	1	1093	566	13067	4050	2	1	1	0	0	1	1	0	-2	2	17	24	34	40	57	64	68	72	71	79	57	29	n/a	n/a
		2	1015	426	12348	5452	1	1	0	0	0	0	-1	-1	-2	3	13	18	24	29	45	50	53	58	64	72	63	59	n/a	n/a
		3	946	469	11125	3478	1	1	1	0	0	-1	-1	-1	-3	3	14	21	27	32	48	53	58	62	64	71	53	16	n/a	n/a
		4	936	467	10580	3279	0	0	0	1	-1	2	3	1	-2	3	13	18	26	34	54	59	58	60	62	69	45	15	n/a	n/a
B1-2	+8.5 vertical	1	1005	378	12279	6361	1	1	0	1	0	0	-1	0	-2	3	10	16	22	28	42	45	50	56	60	69	70	78	-5	n/a
		2	994	477	11927	4390	0	1	1	2	0	4	1	-1	-3	3	11	21	28	34	52	55	59	62	63	69	48	33	11	n/a
		3	984	441	12061	4850	1	1	0	0	0	-1	-1	-2	-4	5	11	19	25	30	46	48	52	57	60	65	50	40	26	n/a
		4	946	494	11003	3014	1	1	0	0	0	-1	-1	-1	-4	4	12	21	28	34	55	55	58	60	59	63	40	22	3	n/a
B1-3	+10.5 vertical	1	1016	493	12110	4311	1	1	0	0	0	0	-1	0	-2	2	13	21	28	35	54	56	60	62	63	68	48	30	13	1
		2	1001	541	12293	3540	2	1	1	2	0	-2	0	0	-3	3	17	24	31	37	53	56	60	63	64	68	45	38	1	0
		3	865	420	10287	3401	2	1	0	0	-1	-2	-1	-2	-3	3	13	19	25	28	41	47	52	55	59	66	49	16	1	1
		4	858	446	10018	2809	3	1	1	1	0	0	-1	0	-3	3	16	19	25	31	47	48	53	55	58	63	38	21	1	0
B1-4	+6.5 recurve	1	3349	2034	61486	54388	1	1	1	2	0	4	5	4	-2	2	14	22	25	33	63	65	-17	86	100	134	415	1096	n/a	n/a
		2	1840	1084	28594	21101	3	2	0	-3	-1	-2	-2	-1	-2	2	12	19	29	37	56	60	61	73	78	89	147	412	n/a	n/a
		3	1794	1169	31391	25289	2	1	0	-1	-1	1	2	2	-2	3	12	15	21	25	45	51	20	55	61	76	154	552	n/a	n/a
		4	1743	878	27131	21566	2	2	0	1	0	0	0	1	-1	1	11	17	20	25	44	48	46	70	80	103	189	365	n/a	n/a
B1-5	+8.5 recurve	1	2663	570	39537	30190	-1	0	0	0	0	0	1	1	-3	4	17	24	29	35	58	59	63	69	73	79	166	704	12	n/a
		2	2112	308	29311	29758	0	-1	0	1	1	10	11	19	9	-9	0	5	0	22	32	52	77	110	121	138	190	308	148	n/a
		3	2053	386	29726	23636	1	1	0	0	0	0	0	1	-3	3	10	16	22	27	45	47	49	53	55	58	112	560	3	n/a
		4	1862	236	27296	22495	2	2	1	1	0	0	0	1	-2	2	10	14	18	22	34	35	36	39	39	42	69	636	-46	n/a
B1-6	+10.5 recurve	1	1598	969	31294	25703	0	0	0	0	0	1	1	2	-1	2	11	14	19	24	41	44	31	52	55	60	48	43	86	439
		2	1540	923	27870	19775	0	0	0	0	0	2	1	3	-2	3	17	21	26	32	46	50	51	58	61	68	56	54	77	308
		3	1530	864	30488	25080	0	1	0	0	0	0	-1	-1	-2	2	9	14	19	23	35	36	26	40	40	43	24	22	199	373
		4	1413	1010	28838	22672	0	-1	1	1	1	-1	-1	-1	-2	2	11	16	21	25	39	39	34	44	46	52	32	19	44	451
C1-1	+10.5 vertical	1	1175	571	14080	4880	0	0	1	1	0	0	0	-1	-4	4	16	22	33	39	60	64	69	76	78	75	66	43	0	0
		2	1017	640	12343	1594	1	1	0	1	0	0	-1	0	-5	5	21	26	37	41	59	63	68	73	75	72	42	-1	0	0
		3	922	472	10087	2836	1	1	1	1	0	-1	-1	-1	-3	4	14	16	29	35	55	56	61	63	67	63	37	1	1	0
		4	866	325	10850	5848	0	0	0	0	0	0	0	0	-2	3	9	7	21	24	37	39	42	45	49	44	29	119	-4	0
C1-2	+10.5 vertical	1	1626	662	20233	10174	1	1	0	0	0	1	-2	0	-3	3	17	24	38	50	71	75	82	89	93	94	92	156	-6	2
		2	1496	446	17523	11072	1	0	0	0	0	0	-2	-2	-2	3	7	16	28	37	65	68	73	78	84	84	88	91	52	-2
		3	1445	759	18916	6395	2	1	0	0	0	-1	0	0	-5	6	23	29	45	50	68	72	78	85	91	91	81	56	38	-1
		4	1381	579	16857	7751	1	1	0	1	0	-1	1	3	-3	3	16	22	34	42	60	64	71	81	92	106	180	-6	0	0

Table 5.7: Unfiltered Top 4 events ranked by negative horizontal force (Fh), with time coincident values of uplift, moments and pressures

Test	Wall	Event	Fh (kN/m)	Fv (kN/m)	Mr (kNm/m)	Mf (kNm/m)	Tr 1 (kPa)	Tr 2 (kPa)	Tr 3 (kPa)	Tr 4 (kPa)	Tr 5 (kPa)	Tr 6 (kPa)	Tr 7 (kPa)	Tr 8 (kPa)	Tr 9 (kPa)	Tr 10 (kPa)	Tr 11 (kPa)	Tr 12 (kPa)	Tr 13 (kPa)	Tr 14 (kPa)	Tr 15 (kPa)	Tr 16 (kPa)	Tr 17 (kPa)	Tr 18 (kPa)	Tr 19 (kPa)	Tr 20 (kPa)	Tr 21 (kPa)	Tr 22 (kPa)	Tr 23 (kPa)	Tr 24 (kPa)
B1-1	+6.5 vertical	1	-637	-665	-8108	2962	3	3	3	3	1	1	1	0	4	-4	-15	-23	-34	-43	-62	-69	-51	-27	-9	1	-2	-4	n/a	n/a
		2	-627	-666	-8089	2965	1	2	1	1	5	5	1	1	4	-4	-18	-26	-35	-44	-62	-63	-49	-27	-11	0	0	1	n/a	n/a
		3	-625	-753	-8758	4041	4	4	3	3	0	-2	-3	-3	4	-5	-19	-27	-37	-47	-66	-69	-51	-28	-9	0	-1	-4	n/a	n/a
		4	-624	-626	-7735	2795	3	5	3	3	0	3	4	5	5	-5	-13	-21	-31	-40	-58	-59	-51	-27	-10	0	-1	-2	n/a	n/a
B1-2	+8.5 vertical	1	-651	-732	-8542	3475	1	2	1	1	0	-3	-4	-2	4	-5	-20	-28	-38	-49	-70	-74	-49	-26	-10	-1	-4	-3	1	n/a
		2	-628	-685	-8430	3114	3	4	2	2	0	2	0	0	4	-4	-17	-25	-35	-44	-61	-65	-51	-27	-9	0	-3	-6	-2	n/a
		3	-628	-647	-7932	2807	1	1	1	2	0	2	0	-1	3	-4	-18	-25	-34	-43	-62	-66	-51	-27	-9	0	-2	-4	-2	n/a
		4	-627	-661	-7978	2898	0	1	1	1	0	-1	-2	-1	4	-4	-19	-26	-35	-44	-63	-68	-49	-27	-10	0	-3	-4	-1	n/a
B1-3	+10.5 vertical	1	-610	-656	-7695	3075	2	1	0	0	0	0	1	1	4	-5	-17	-26	-35	-44	-63	-65	-50	-26	-10	1	1	1	1	0
		2	-599	-617	-7257	2824	2	1	0	0	0	0	0	-1	3	-5	-15	-24	-33	-42	-62	-66	-50	-27	-10	1	1	1	1	1
		3	-597	-613	-7179	2783	2	1	1	1	0	-1	0	0	3	-5	-14	-24	-32	-42	-62	-66	-50	-27	-10	1	2	1	1	0
		4	-596	-658	-7670	3143	2	0	0	0	-1	-1	-1	0	4	-5	-17	-26	-36	-44	-62	-63	-50	-27	-10	0	1	0	1	0
B1-4	+6.5 recurve	1	-636	-738	-8549	3648	3	3	2	3	1	-2	-1	-1	4	-5	-19	-27	-37	-48	-66	-71	-50	-27	-9	0	0	-2	n/a	n/a
		2	-629	-633	-7724	2694	3	2	1	2	0	3	3	3	4	-5	-15	-23	-33	-42	-60	-64	-50	-26	-10	-1	0	0	n/a	n/a
		3	-613	-651	-7841	2883	2	1	1	2	1	1	0	0	4	-4	-17	-24	-34	-42	-61	-64	-50	-27	-10	0	-1	-4	n/a	n/a
		4	-612	-643	-7665	2810	3	3	2	2	1	-1	-2	-1	4	-5	-14	-23	-33	-43	-62	-65	-50	-26	-9	-1	-2	-3	n/a	n/a
B1-5	+8.5 recurve	1	-634	-748	-8438	4188	1	2	0	1	0	0	-1	-1	5	-5	-22	-31	-39	-49	-69	-69	-50	-28	-10	1	1	1	5	n/a
		2	-607	-681	-7748	3669	0	1	0	1	0	-2	-1	-1	4	-4	-20	-28	-36	-45	-64	-68	-50	-27	-10	1	1	1	4	n/a
		3	-592	-644	-7527	3210	2	3	2	3	1	1	1	1	4	-4	-17	-22	-33	-43	-58	-63	-50	-26	-8	0	0	-2	3	n/a
		4	-587	-655	-7495	3411	2	1	0	0	0	0	0	0	4	-5	-18	-26	-35	-44	-62	-63	-50	-27	-9	1	1	1	5	n/a
B1-6	+10.5 recurve	1	-639	-748	-8494	3717	1	0	0	0	0	-1	0	0	4	-5	-20	-30	-40	-51	-68	-68	-50	-27	-10	1	0	0	1	0
		2	-608	-642	-7529	3018	0	0	-1	0	0	0	1	0	4	-5	-18	-27	-35	-43	-62	-65	-51	-28	-11	0	1	1	1	0
		3	-604	-662	-7669	3236	0	1	0	-1	-1	-1	0	0	4	-5	-18	-27	-36	-44	-63	-64	-51	-28	-10	1	1	1	1	0
		4	-603	-644	-7421	3103	0	0	0	0	-1	-1	-1	-1	4	-5	-18	-26	-35	-43	-63	-66	-51	-28	-10	1	1	1	1	0
C1-1	+10.5 vertical	1	-564	-739	-7912	4005	1	-1	0	1	1	0	1	0	5	-6	-21	-24	-43	-51	-69	-58	-38	-17	0	0	0	0	1	0
		2	-525	-676	-7296	3784	0	0	0	1	0	1	1	1	5	-4	-24	-21	-38	-45	-57	-58	-39	-17	-1	-1	0	1	1	1
		3	-522	-650	-7144	3471	0	0	0	0	0	0	0	0	5	-4	-22	-22	-37	-44	-58	-58	-39	-17	0	0	1	0	0	0
		4	-522	-635	-6946	3338	1	0	0	0	0	0	1	0	4	-5	-19	-21	-36	-44	-60	-59	-38	-17	0	0	1	0	1	0
C1-2	+10.5 vertical	1	-537	-674	-7382	3671	1	0	0	0	0	0	0	0	4	-5	-21	-24	-37	-45	-62	-59	-40	-18	0	0	0	0	0	0
		2	-536	-675	-7359	3625	0	0	0	0	0	0	1	1	5	-6	-20	-21	-39	-46	-60	-58	-40	-17	0	0	1	0	1	0
		3	-536	-698	-7648	4003	2	2	1	1	0	0	-1	0	4	-5	-21	-25	-37	-45	-61	-59	-40	-17	0	0	0	0	0	0
		4	-535	-651	-7150	3353	1	1	0	0	0	0	-1	0	4	-6	-20	-19	-38	-45	-61	-59	-40	-17	-1	0	0	0	0	0

Results files have been provide in CSV format along with time histories of the filtered and unfiltered data. An example of the output data for Test B1-4 is given below:

- B1-4_250.csv;
- B1-4_evet_time.csv;
- B1-4_Fh_max.csv;
- B1-4_Fh_min.csv;
- B1-4_Fv_max.csv;
- B1-4_Fv_min.csv;
- B1-4_maxima.csv;
- B1-4_Mf_max.csv;
- B1-4_Mf_min.csv;
- B1-4_minima.csv;
- B1-4_Mr_max.csv;
- B1-4_Mr_min.csv;
- B1-4_tms.zip;
- B1-4_unfiltered_250.csv;
- B1-4_unfiltered_evet_time.csv;
- B1-4_unfiltered_Fh_max.csv;
- B1-4_unfiltered_Fh_min.csv;
- B1-4_unfiltered_Fv_max.csv;
- B1-4_unfiltered_Fv_min.csv;
- B1-4_unfiltered_maxima.csv;
- B1-4_unfiltered_Mf_max.csv;
- B1-4_unfiltered_Mf_min.csv;
- B1-4_unfiltered_minima.csv;
- B1-4_unfiltered_Mr_max.csv;
- B1-4_unfiltered_Mr_min.csv;
- B1-4_unfiltered_Pimp_risetime_Tr01_kPa.csv;
- B1-4_unfiltered_Pimp_risetime_Tr02_kPa.csv;
- B1-4_unfiltered_Pimp_risetime_Tr06_kPa.csv;
- B1-4_unfiltered_Pimp_risetime_Tr19_kPa.csv;
- B1-4_unfiltered_Pimp_risetime_Tr21_kPa.csv;
- B1-4_unfiltered_Pimp_risetime_Tr22_kPa.csv;
- B1-4_unfiltered_tms.zip.

5.1.3 Wave transmission

The transmitted wave was recorded by wave probes placed on the berthing side of the test section, approximately 50 m, 100 m and 150 m from the caisson harbour-side vertical face. The signal recorded by this probe was first conditioned using a 1,000 s high-pass filter to eliminate signal drift and other potential model effects caused by the pump controlling water levels behind the breakwater. Results for each of the six crown wall designs tested are given in Table 5.8 and provides the transmission coefficient Ct, measured significant wave height H_s and the maximum wave H_{max}.

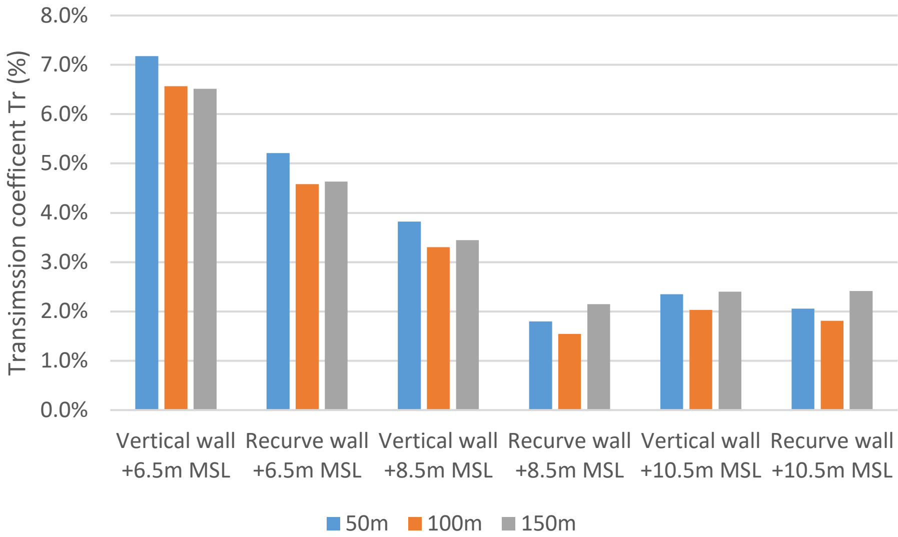
These value are plotted in Figure 5.7 and show that the recurve wall provides the greatest protection against wave transmission. The recurve walls are seen to notable reduce wave transmission for +6.5 mMSL and +8.5 mMSL wall heights, however at +10.5 mMSL the transmission is similar between with and without recurve scenarios.

It can be seen that the recurve wall, set a+8.5 mMSL provides the lowest transmission value, but is similar to the vertical wall set at +10.5 mMSL. The recurve walls at +6.5 mMSL and +8.5 mMSL appears to achieve optimal performance, redirecting waves most efficiently. This is also indicated in the measured wave loads, with the higher loads measured for these configuration.

Results of the measured wave conditions for these test are given in Appendix D.

Table 5.8: Transmission coefficient during design wave conditions

	Transmission coefficient Tr (%)			Significant wave height behind breakwater H_s (m)			Maximum wave height behind breakwater H_{max} (m)		
Distance back	50 m	100 m	150 m	50 m	100 m	150 m	50 m	100 m	150 m
Vertical wall +6.5 mMSL	7.2%	6.6%	6.5%	0.40	0.37	0.36	1.20	1.19	1.08
Recurve wall +6.5 mMSL	5.2%	4.6%	4.6%	0.29	0.25	0.26	1.46	1.48	0.88
Vertical wall +8.5 mMSL	3.8%	3.3%	3.4%	0.21	0.19	0.19	0.91	0.87	0.66
Recurve wall +8.5 mMSL	1.8%	1.5%	2.2%	0.10	0.09	0.12	0.30	0.30	0.19
Vertical wall +10.5 mMSL	2.3%	2.0%	2.4%	0.13	0.11	0.14	0.55	0.39	0.35
Recurve wall +10.5 mMSL	2.1%	1.8%	2.4%	0.12	0.10	0.14	0.20	0.18	0.24



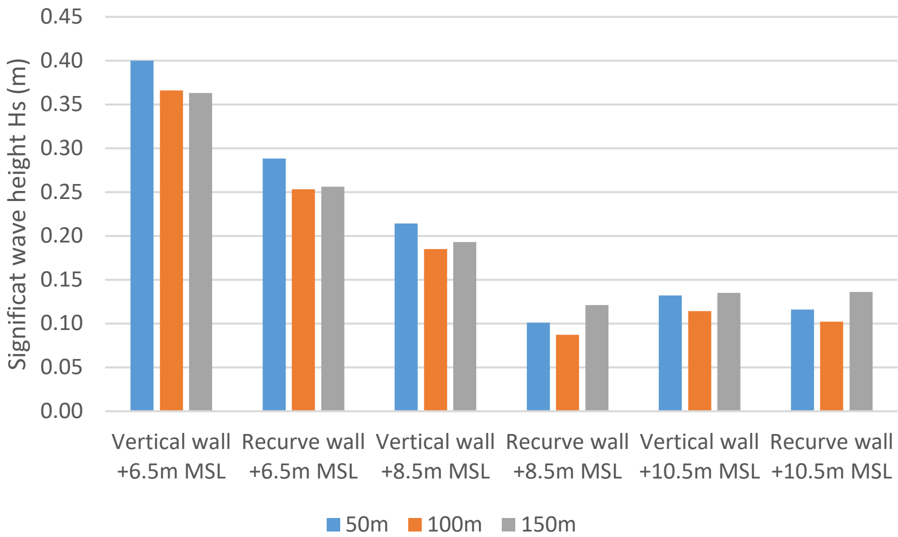


Figure 5.7: Transmission coefficient Ct (top) & transmitted significant wave height Hs (bottom)

5.1.4 Wave reflections

The reflection coefficient, C_r, was defined as the ratio of the reflected height, H_{m0r}, to the incident wave height, H_{m0i}, using the spectra resolved by the wave gauge array in front of the caisson breakwater. The resulting coefficients are presented in Table 5.9, which shows that they varied between 75.0% and 78.4%. The results are plotted in Figure 5.8 and show a small variation seen for the difference wall configurations. The plot shows there is a slight increase in reflection coefficient for the recurve wall configurations.

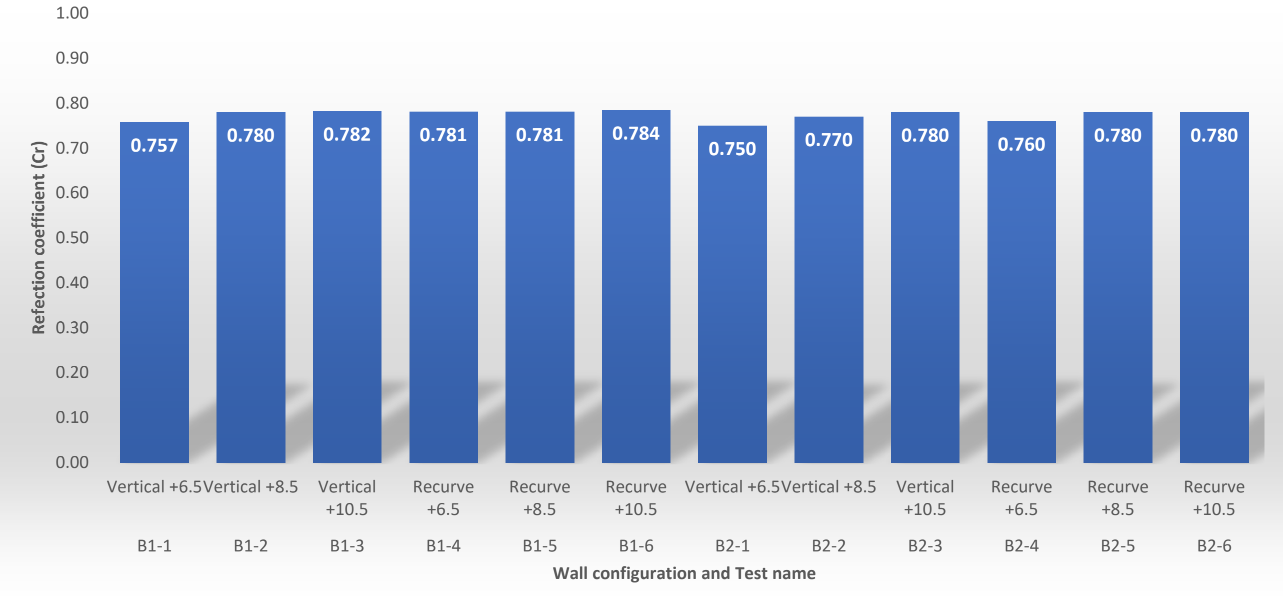


Figure 5.8: Refection coefficient

Table 5.9: Measured caisson reflection coefficient

Test	Wall configuration	Cr
B1-1	Vertical +6.5	0.757
B1-2	Vertical +8.5	0.780
B1-3	Vertical +10.5	0.782
B1-4	Recurve +6.5	0.781
B1-5	Recurve +8.5	0.781
B1-6	Recurve +10.5	0.784
B2-1	Vertical +6.5	0.750
B2-2	Vertical +8.5	0.770
B2-3	Vertical +10.5	0.780
B2-4	Recurve +6.5	0.760
B2-5	Recurve +8.5	0.780
B2-6	Recurve +10.5	0.780

5.2 Test series B2 – Wave overtopping

Wave overtopping passing the crown wall crest elevation was measured for 6 wall configurations. As well as measuring the mean overtopping discharge a wave probe was placed in the overtopping collection tank to measure the wave by wave volumes.

5.2.1 Mean overtopping discharge

The mean overtopping discharge was seen to fall between 167 l/s/m and 4.9 l/s/m. With largest volumes of overtopping for the vertical wall set at +6.5 mMSL and the smallest volume for the recurve wall set at +10.5 mMLS, see Table 5.10.

Table 5.10: Measured and predicted mean overtopping discharge

Test Series	Test	Breakwater configuration		Wave conditions					Mean overtopping discharge (l/s/m)	Predicted mean overtopping discharge (l/s/m)
		Crest elevation (mMSL)	With or without recurve	Return period Tr (yrs)	H_s (m)	T_p (m)	Wave spectra	SWL (mMSL)		
B2 Wave overtopping tests	B2-1	+6.5	without	100	5.5	9.6	Jonswap	+1.7	100	179
	B2-2	+8.5		100	5.5	9.6		+1.7	27	65
	B2-3	+10.5		100	5.5	9.6		+1.7	9.4	24
	B2-4	+6.5	with	100	5.5	9.6		+1.7	53	99
	B2-5	+8.5		100	5.5	9.6		+1.7	16	20
	B2-6	+10.5		100	5.5	9.6		+1.7	4.6	4.7

The results are plotted against prediction methods given by the Wave overtopping manual (Ref. 3). A copy of the calculation can be found in Appendix F. The prediction method provided the corrector order of magnitude for overtopping, however in all case was seen to overpredict the volumes.

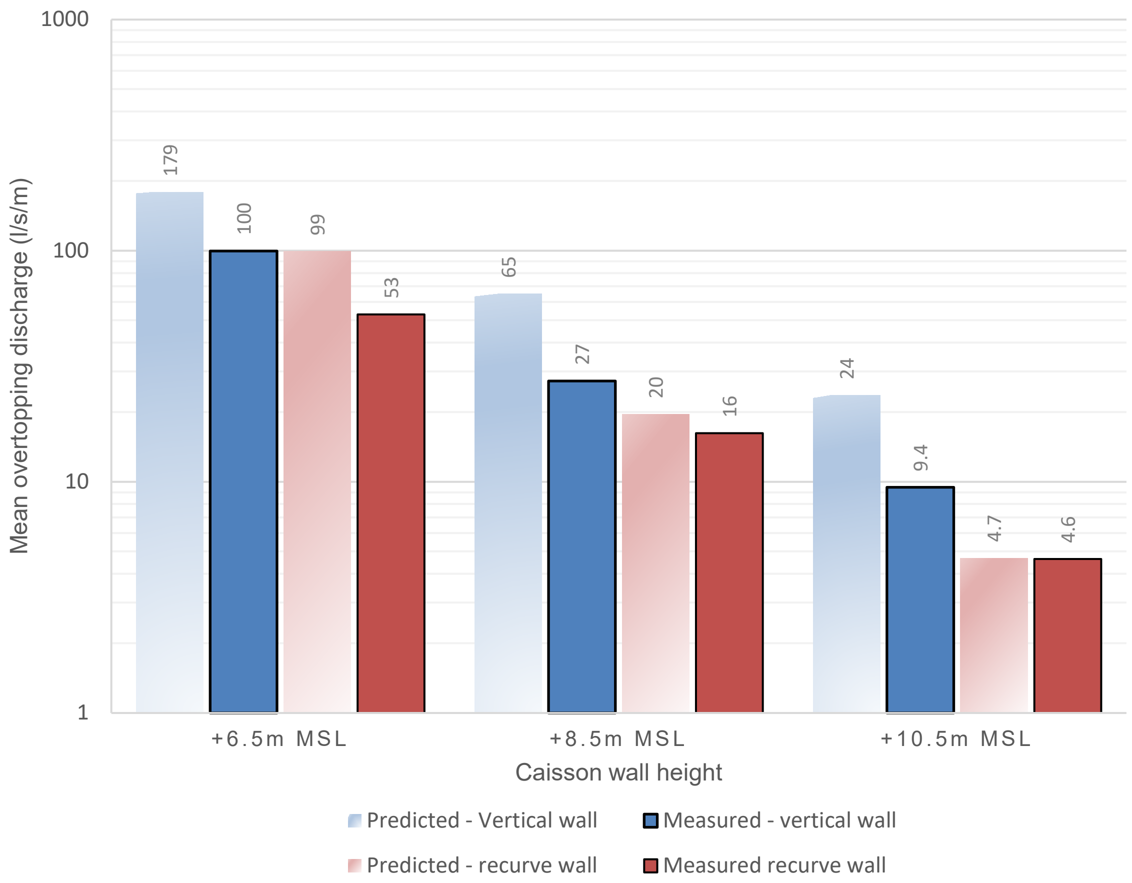


Figure 5.9: Measured and predicted mean overtopping discharge

5.2.2 Wave by wave overtopping discharge

In the lower crested structure tests, the overtopping events were frequent and heavy, and in the first (test B2-1), the collection tank became full before the end of the test. Although this was sufficient to give a reliable measurement of the mean discharge, there may have been individual volumes in the 1,000 wave sequence larger than those recorded in Table 5.11 for that tests.

The overtopping volumes were so small during the higher two recurve crest level tests (B2-5 and B2-6), that no measurable wave by wave measurable discharges could be acquired. The wave by wave overtopping volumes are also plotted in Figure 5.10 and show that on average the overtopping is halved for the recurve wall.

Table 5.11: Top 11 overtopping events

Test	Top events (m^3/m)										
	1	2	3	4	5	6	7	8	9	10	11
B2-1 - Vertical wall +6.5	21.5	20.2	18.3	15.4	14.8	14.6	14.4	12.7	12.5	12.4	12.0
B2-2 - Vertical wall +8.5	10.0	9.1	8.9	8.0	7.9	7.8	7.0	6.4	6.1	5.8	5.5
B2-3 - Vertical wall +10.5	4.3	3.8	3.2	3.0	2.7	2.4	2.2	2.0	1.9	1.9	1.9
B2-4 - Recurve wall +6.5	13.5	9.2	9.0	7.6	7.4	7.3	7.1	7.0	7.0	6.8	6.6
B2-5 - Recurve wall +8.5	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
B2-6 - Recurve wall +10.5	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a

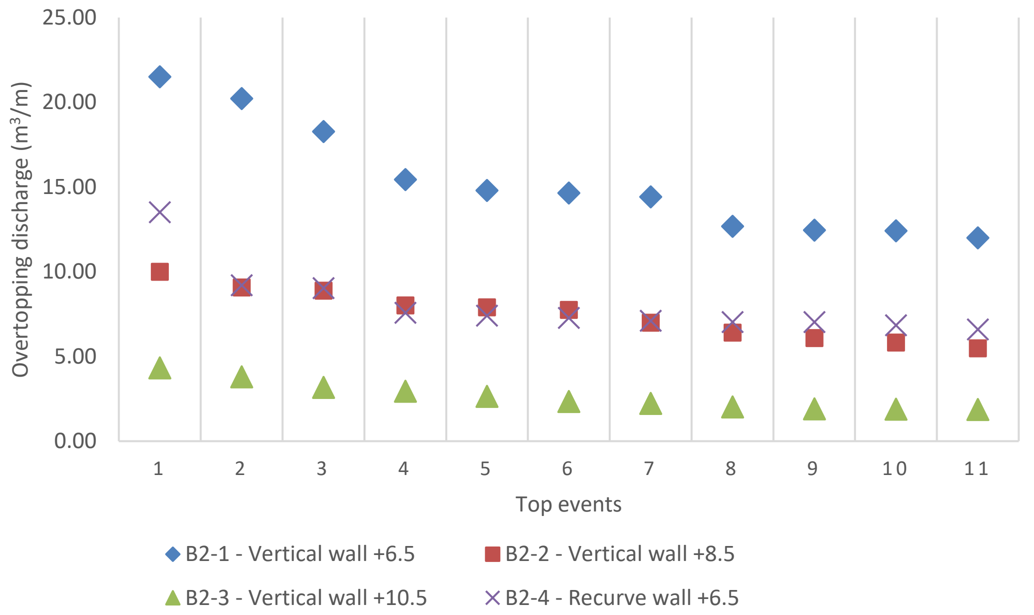


Figure 5.10: Wave by wave overtopping volumes

During the testing, the majority of the observed overtopping was green water, with white water spray falling on to the top of the caisson. Very little water was observed reaching the water column behind the caisson directly.

The recurve wall was observed to work effectively and reduced the overtopping notability. Observations showed waves being redirected seaward by the recurve. It was particularly effective at the lower +6.5 m crest level tested, see Figure 5.11.

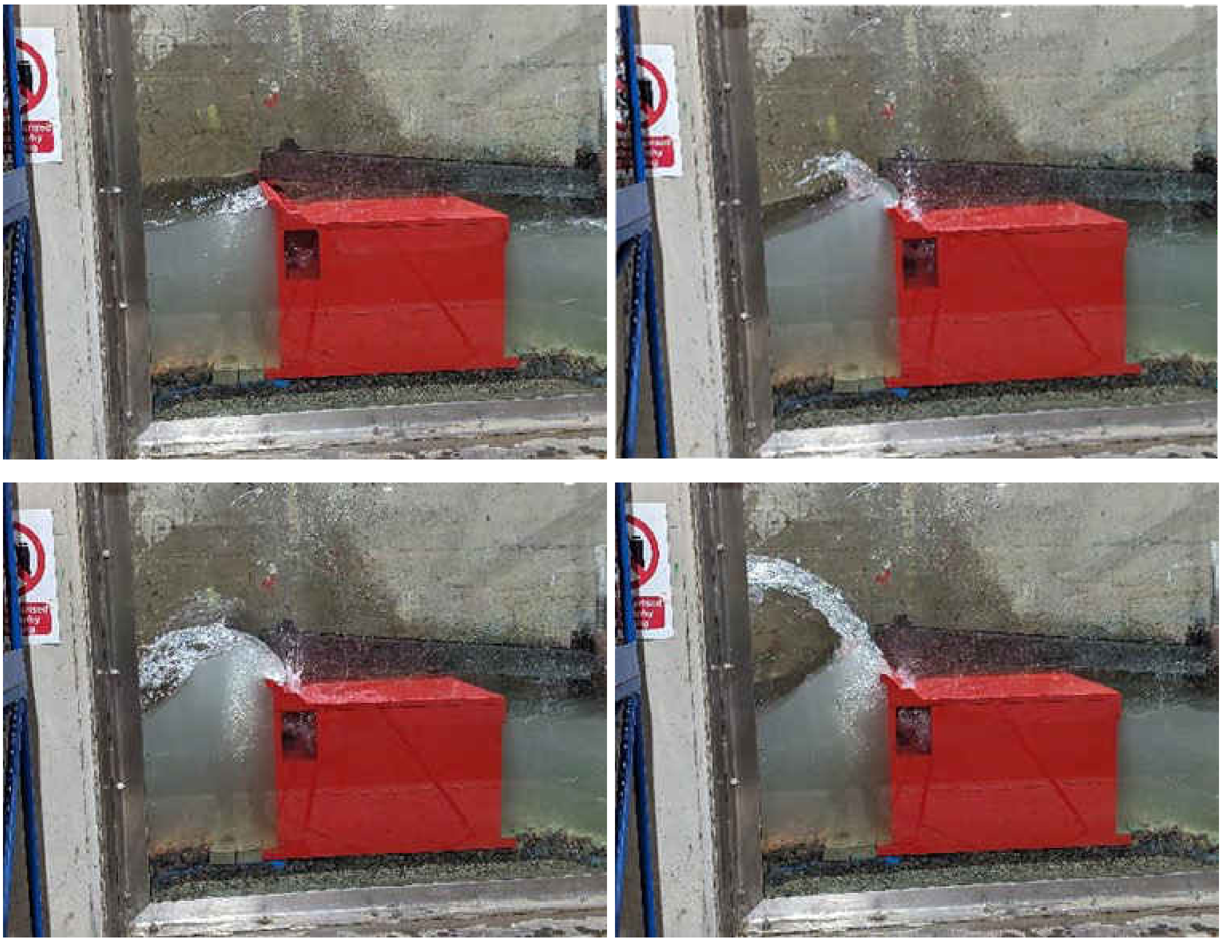


Figure 5.11: Effective recurve wall reducing wave overtopping

5.3 Test Series B3 – Wave transmission during phased construction

Four additional tests were completed to look at the transmission coefficient associated with different phase of construction, see Table 5.12. The caisson configuration with a +3 m and +5 m crest elevation, see Figure 5.12, were testing using two wave conditions representing a 2 year return period and 10 year return period.

Table 5.12: Test Series B3 – test matrix

Test Series	Test	Breakwater configuration		Wave conditions				
		Crest elevation (m MSL)	With or without recurve	Return period Tr (yrs)	H_s (m)	T_p (m)	Wave spectra	Still water level (m MSL)
B3 Transmission tests	B3 -1	+5.0	without	2	3.7	8	JONSWAP γ = 3.3.	+1.0
	B3 -2	+5.0		10	4.5	8.7		+1.2
	B3 -3	+3.0		2	3.7	8		+1.0
	B3 -4	+3.0		10	4.5	8.7		+1.2

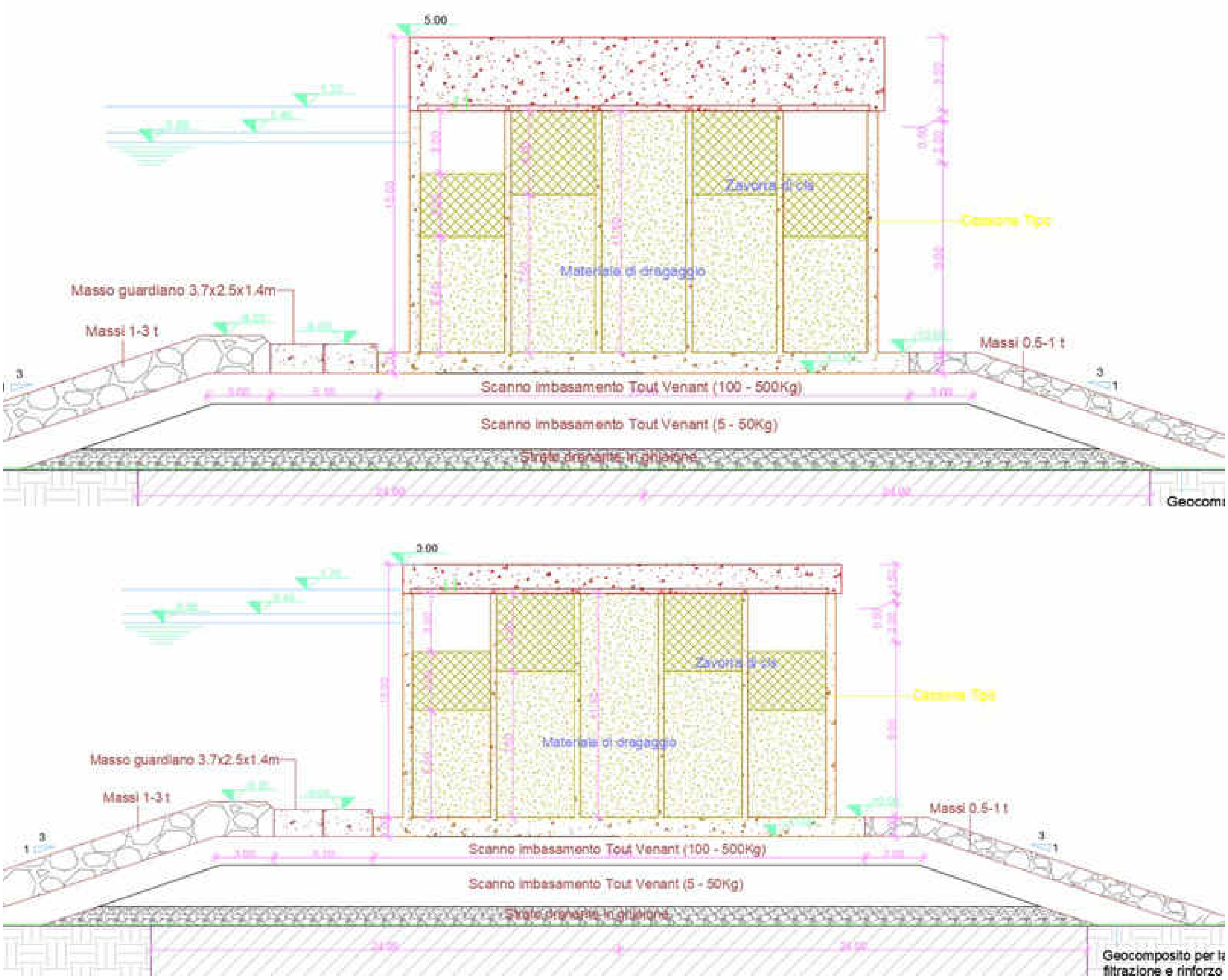


Figure 5.12: Caisson with flat +5 mMSL (top) and flat +3 mMSL (bottom) crest elevation

5.3.1 Transmission coefficient

The transmitted wave was recorded by a wave probes placed on the berthing side of the test section, approximately 50 m, 100 m and 150 m from the caisson harbour-side vertical face. Results for each of the caisson crest elevations tested are given in Table 5.13 and provides the transmission coefficient Ct, measured significant wave height H_s and the maximum wave H_{max}.

Thes values are plotted in Figure 5.13 and show that, as expected, the lower crest elevation +3 mMSL gives the greatest transmission.

Results of the measured wave conditions for these test are given in Appendix D.

Table 5.13: Transmission coefficient during design wave conditions

	Transmission coefficient Tr (%)			Significant wave height behind breakwater H_s (m)			Maximum wave height behind breakwater H_{max} (m)		
Distance back	**50 m**	**100 m**	**150 m**	**50 m**	**100 m**	**150 m**	**50 m**	**100 m**	**150 m**
2 yr return period with +5 m crest	4.0%	3.4%	3.8%	0.15	0.13	0.14	0.75	0.41	0.37
10 yr return period with +5 m crest	7.6%	6.9%	6.8%	0.34	0.31	0.31	1.34	1.07	1.04
2 yr return period with +3 m crest	9.7%	8.3%	8.6%	0.35	0.30	0.31	1.08	1.11	0.80
10 yr return period with +3 m crest	14.4%	12.9%	12.6%	0.64	0.57	0.56	1.93	1.40	1.25

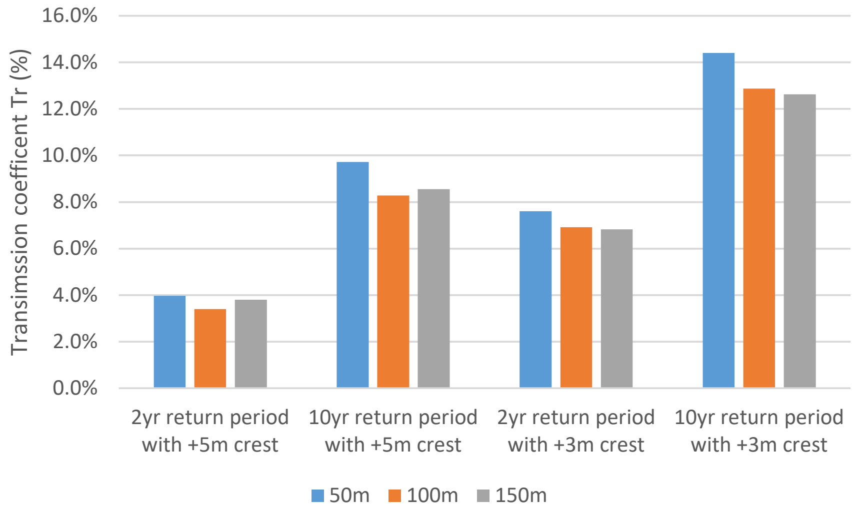


Figure 5.13: Transmission coefficient Ct (top) & transmitted significant wave height Hs (bottom)

5.3.2 Reflection coefficient

The reflection coefficient, C_r, was defined as the ratio of the reflected height, H_{m0r}, to the incident wave height, H_{m0i}, using the spectra resolved by the wave gauge array in front of the caisson breakwater. The resulting coefficients shows that they varied between 67.7% and 79.4%. The results are plotted in Figure 5.14.

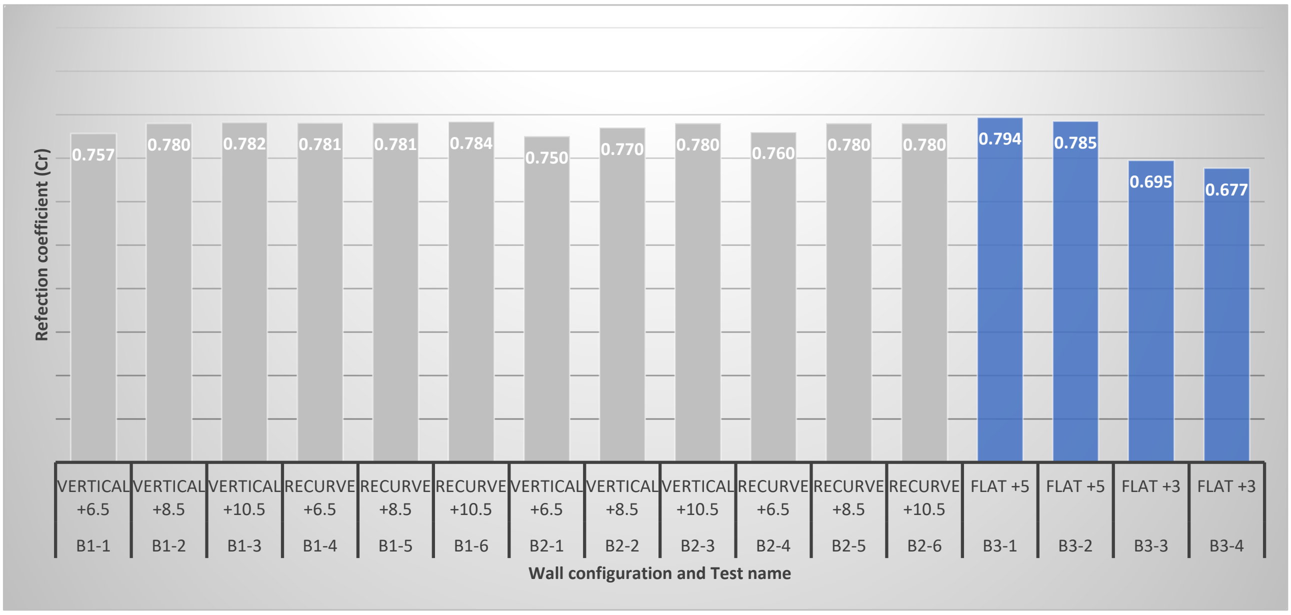


Figure 5.14: Reflection coefficient – Test Series B3

6 Test Series C – scour protection

6.1 Rock armour

The damage to the seaward 1.0-3.0 t rock armour was assessed using the photographic method described in Section 3.5, and the results are presented in Table 6.1, in terms of the N_{od} parameter. The photographs on which the analysis is based are reproduced in Figure 6.1 to Figure 6.3.

Table 6.1: armour damage (1.0-3.0 t rock)

Sea State	Tr (yrs)	H_{m0} (m)	T_p (s)	SWL (m msl)	N_{od}
C1-1	100	5.5	9.6	+0.46	0.50
C1-2	overload	6.6	10.9	+0.46	1.09

The damage level to the 1.0-3.0t remained relatively low throughout testing. Comparing the measured N_{od} values to the performance criteria given in Section 3.5 it can be seen that even after the overload test the recorded damage (N_{od} = 1.09) corresponded to above the 'almost no damage'' limit and below 'acceptable damage' threshold. There was some scouring of the sand placed in the dredged pocket at the toe of the rock armour, however no significant moving of the toe rock was observed.

No measurable movement of the 0.5-1 t rock was observed on the landward side of the caisson.

Figure 6.1: Pre-test C1-1

Figure 6.2: Post C1-1

Figure 6.3: Post C1-2

Figure 6.4: Post test C1-2 (0.5t to 1.0t)

6.2 Concrete toe blocks

No concrete toe blocks were extracted during the test series, although they settled very slightly.

7 Summary and conclusions

7.1 Overtopping discharges, transmission and reflections

The objective of the overtopping tests was to determine the overtopping discharge associated with six different crown wall configurations, made up of vertical and recurve walls. The caisson was tested using the 100 year design wave conditions, with a further two conditions used to investigate the transmission coefficient for more frequently occurring storm conditions.

The mean overtopping discharge was seen to fall between 167 l/s/m and 4.9 l/s/m. With largest volumes of overtopping for the vertical wall set at +6.5 mMSL and the smallest volume for the recurve wall set at +10.5 mMSL. The results have been compared against prediction methods and found to be within the same order of magnitude, however in all case the measured volumes were seen to be less than predicted. The wave by wave overtopping volumes suggest that on average the overtopping is halved for the use of a recurve wall.

The transmitted wave was recorded by a wave probes placed on the berthing side of the test section, approximately 50 m, 100 m and 150 m from the caisson harbour-side vertical face. Results for each of the six crown wall designs provided transmission coefficient Ct in the range of 2% to 7% for the design wave conditions.

For the construction phase breakwater with crest levels set at +3 m and +5 m, the transmission coefficient falls between 3.8% and 14.4%.

The reflection coefficient, C_r, was defined as the ratio of the reflected height, H_{m0r}, to the incident wave height, H_{m0i}, using the spectra resolved by the wave gauge array in front of the caisson breakwater. The resulting coefficients for the design conditions varied between 75%-78.4% and 67.7%-79.4% for the more frequently occurring storms expected to occur when the caisson crest is at +3 m or +5 mMSL.

7.2 Armour damage

The seaward side of the foundation mound was protected by a layer of 1.0-3.0 t rock, as well as concrete toe blocks placed immediately against the caisson. It was tested in the 1:100 year and a 120% overload conditions at low water levels. The rock damage levels remained relatively low throughout testing, reaching a value of N_{od} = 0.5 by the end of the design test and N_{od} = 1.09 by the end of the overload test sequence. This corresponds approximately to the 'almost no damage' threshold defined in the Rock Manual.

None of the concrete toe blocks were extracted or were observed to have moved significantly during the test series.

7.3 Wave loadings

Wave loadings were recorded in the 100 year high water, 100 year lower water events and overload conditions. In determining the loading results, a filter was applied to convert the measured forces to static-equivalent loadings. The filter characteristics were based on the resonant frequencies of the structure in sliding response, which were estimated at 0.5 s to 1.0 s. The results were presented in a series of tables, which give the statistical parameters for the forces and moments on the caisson.

Summary results are given Section 5.1.1 and Section 5.1.2, while details of the coincident loadings are given results data provided electronically.

8 References

1. Guidelines for physical model testing of breakwaters: rubble mound breakwaters, HYDRALAB III Deliverable NA3.1, Version 1.3, August 2007.
2. The Rock Manual, The use of rock in hydraulic engineering, 2nd Edition, CIRA, CUR, CETMEF, 2007.
3. EurOtop (2018) J.W. Van der Meer, N.W.H. Allsop, T. Bruce, J. De Rouck, A. Kortenhaus, T. Pullen, H. Schüttrumpf, P. Troch, B. Zanuttigh (Eds.), Manual on Wave Overtopping of Sea Defenses and Related Structures. An Overtopping Manual Largely Based on European Research, but for Worldwide Application (2018).

Appendices

A Scaling of primary armour

As described in Section 0, there were slight variations between the model and prototype armour densities, as well as between model and prototype fluid densities. If not compensated for these variations would mean that stability of the armour in the model would be different from that in the prototype. It was therefore necessary to correct the size of the units used in the model, so that they exhibited the same stability characteristics as the prototype.

A correction factor for density may be derived by reference to the Hudson equation (Ref. A.1) which states:

$$M \propto \frac{\rho_s H_s^3}{(\rho_s/\rho_f - 1)^3 \cot\theta}$$

where: M is the mass of the armour unit.

H_s is the significant wave height.

θ is the structure slope angle to the horizontal.

ρ_s is the density of the armour.

and ρ_f is the density of the displacing fluid.

The correction factor (C_f) for the armour mass may thus be calculated from the following equation:

$$M_m = M_p \frac{C_f}{\lambda^3}$$

where λ is the model scale. This leads to the following expression for the correction factor:

$$C_f = \frac{\rho_{sm}}{\rho_{sp}}\left[\left(\frac{\rho_{sp}}{\rho_{fp}} - 1\right) \Big/ \left(\frac{\rho_{sm}}{\rho_{fm}} - 1\right)\right]^3$$

where the subscripts p and m respectively refer to parameters in the prototype and model.

Reference

A.1 Coastal Engineering Research Centre. (1984) "Shore Protection Manual." CERC, US Government Printing Office, Washington, Vols 1 & 2.

B Scaling of filter layer and core

At the scale selected for this model study there will be conditions where the flow through the model core is not fully turbulent. Scale effects will thus affect the flow of water through the core. Therefore, in order to reproduce realistically the permeability of the core, it is necessary to compensate for the scale effects which result from the use of Froude's scaling law. Work by Jensen and Klinting (Ref. B.1) suggests a method of compensating for scale effects due to laminar flow by applying a correction factor to the ordinary geometric scale when determining core sizes. The calculation of the correction factor uses a special Reynolds number, ξ_p, which is defined as the ratio of turbulent to laminar hydraulics gradients. The special Reynolds number is defined as:

$$\xi_p = (\beta_0/\alpha_0)1/(n(1-n)^2)U_p\, d/\nu$$

where: α_0 and β_0 are constants derived from experiments.

n is the porosity of the prototype rock mound.

d is the size of the prototype rock.

ν is the kinematic viscosity of water.

U_p is the maximum velocity in the prototype mound.

The ratio of rock size in prototype to model, K, is then given by:

$$K \;=\; (\xi_p/2\sqrt{\lambda})[(1+4\lambda^{3/2}\;(1+\xi_p)/\xi_p 2)^{1/2}-1]$$

where λ is the geometric scale.

Throughout this analysis it is assumed that the porosity of model and prototype rock is identical.

To enable the above equations to be used in calculating a correction factor certain assumptions have to be made. Experimental work by Engelund (Ref. B.2) suggested values for the empirical coefficients of α_0 = 1500 and β_0 = 3.6. The maximum prototype velocity in the mound was estimated at 0.5-1.0 m/s from some simple calculations of wave velocities and comparisons with velocities calculated by a simple mathematical model of flow in rubble. The porosity of the rock mound, n, was also estimated at 35-37%.

References

B.1 Jensen O J and Klinting P. 'Evaluation of Scale Effects in Hydraulic Models by Analysis of Laminar and Turbulent Flows', Coastal Engineering, pp 319-329, 1983.

B.2 Engelund F. 'The Laminar and Turbulent Flows of Ground Water through Homogeneous Sand', Trans. Danish Acad Tech Sci, 3(4), 1953.

C Wave calibration

C.1.1 Wave condition A1-1

Table C.1: Wave calibration results for Design condition 1

Wave Probe	Statistical parameters				Spectral parameters			Incident wave height		
	H_{max} (m)	$H_{1/10}$ (m)	$H_{1/3}$ (m)	T_m (s)	H_{m0} (m)	T_p (s)	$T_{m0,2}$ (s)	H_s avg (m)	Cr	H_{si} (m)
WP1	10.09	7.44	5.92	8.1	5.98	9.5	7.0			
WP2	8.94	6.84	5.60	8.4	5.65	9.1	7.2			
WP3	7.99	6.47	5.36	8.5	5.46	9.5	7.3	5.41	0.17	5.33
WP4	8.08	6.46	5.36	8.6	5.46	9.5	7.3			
WP5	8.73	6.36	5.27	8.6	5.40	9.5	7.3			
WP6	8.74	6.38	5.23	8.4	5.33	9.9	7.2			
WP7	8.05	6.48	5.37	8.6	5.46	9.5	7.2			
WP8	8.64	6.22	5.12	8.5	5.21	9.5	7.2			
WP9	6.93	5.68	4.65	8.4	4.80	9.5	7.2			
WP10	-	-	-	-	-	-	-			

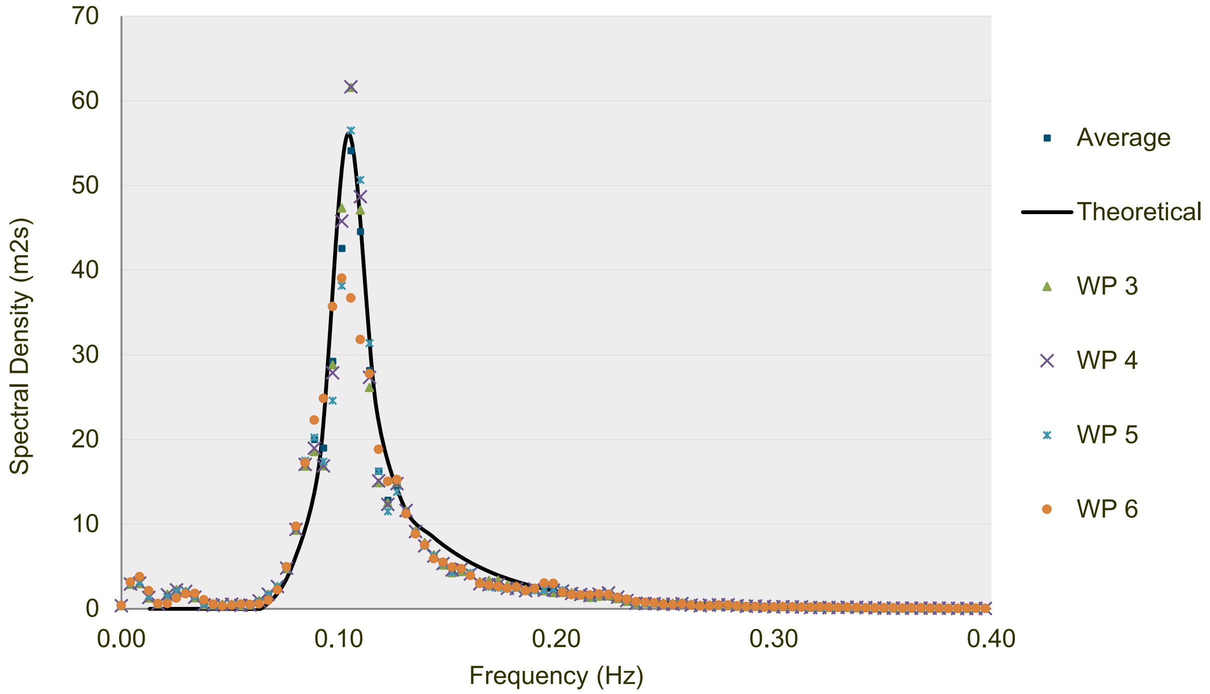


Figure C.1: Measured wave spectra

C.1.2 Wave condition A1-2

Table C.2: Wave calibration results for Design condition 2

Wave Probe	Statistical parameters				Spectral parameters			Incident wave height		
	H_{max} (m)	$H_{1/10}$ (m)	$H_{1/3}$ (m)	T_m (s)	H_{m0} (m)	T_p (s)	$T_{m0,2}$ (s)	H_s avg (m)	Cr	H_{si} (m)
WP1	10.47	7.76	6.20	8.10	6.22	9.47	6.97			
WP2	7.99	6.60	5.59	8.71	5.68	9.47	7.14			
WP3	8.16	6.36	5.35	8.82	5.46	9.47	7.28	5.44	0.10	5.41
WP4	8.04	6.36	5.35	8.79	5.47	9.47	7.28			
WP5	7.79	6.26	5.31	8.81	5.43	9.47	7.32			
WP6	8.05	6.19	5.26	8.69	5.39	9.47	7.28			
WP7	8.16	6.29	5.34	8.72	5.48	9.47	7.26			
WP8	8.20	6.04	5.13	8.64	5.32	9.47	7.30			
WP9	7.56	5.76	4.82	8.80	5.04	9.47	7.47			
WP10	-	-	-	-	-	-	-			

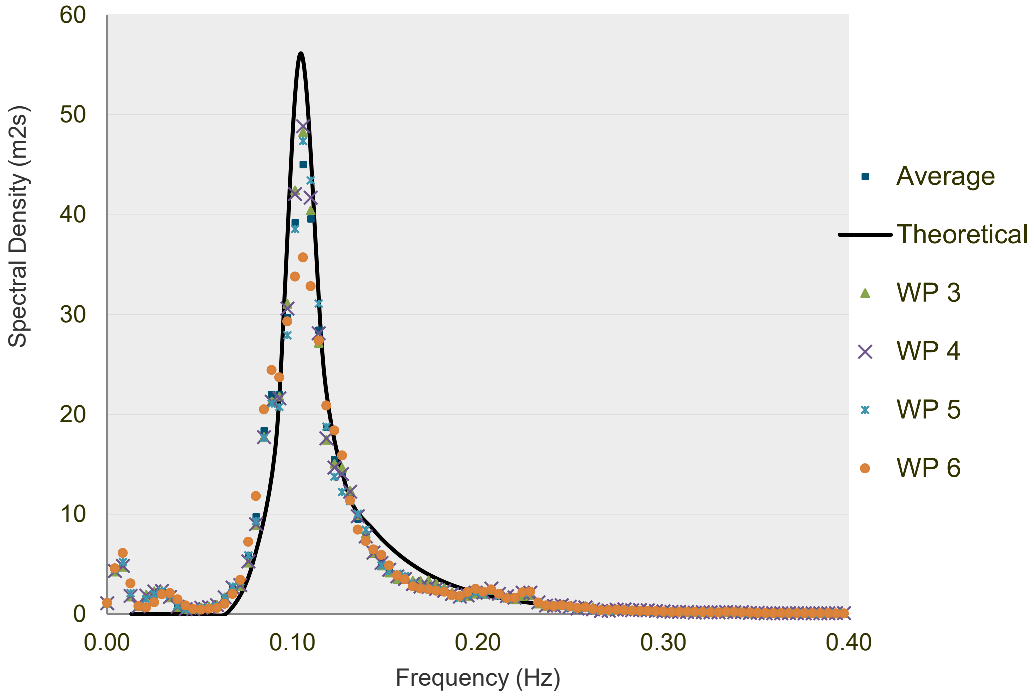


Figure C.2: Measured wave spectra

C.1.3 Wave condition A1-3

Table C.3: Wave calibration results for Overload condition

Wave Probe	Statistical parameters				Spectral parameters			Incident wave height		
	H_{max} (m)	$H_{1/10}$ (m)	$H_{1/3}$ (m)	T_m (s)	H_{m0} (m)	T_p (s)	$T_{m0,2}$ (s)	H_s avg (m)	Cr	H_{si} (m)
WP1	12.47	9.51	7.88	8.87	7.95	9.86	7.49			
WP2	9.59	7.69	6.63	9.63	6.85	10.29	7.51			
WP3	8.80	7.21	6.22	9.48	6.55	10.29	7.72	6.58	0.16	6.51
WP4	9.18	7.20	6.22	9.40	6.56	10.29	7.70			
WP5	9.27	7.19	6.23	9.56	6.54	10.29	7.75			
WP6	9.50	7.29	6.38	9.59	6.68	10.29	7.77			
WP7	8.97	7.20	6.31	9.59	6.67	10.29	7.66			
WP8	9.36	7.09	6.08	9.60	6.43	10.29	7.69			
WP9	8.74	6.56	5.66	9.51	6.09	10.29	7.93			
WP10	-	-	-	-	-	-	-			

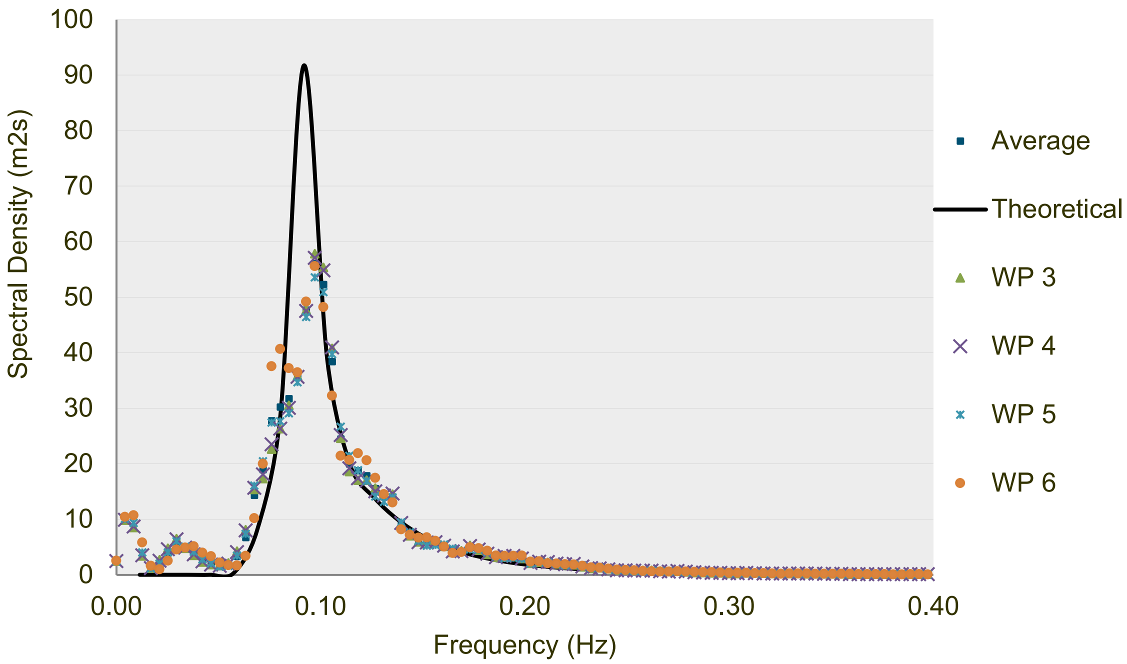


Figure C.3: Measured wave spectra

D Measured wave conditions – Test Series B

Measured wave conditions during Test Series B1 and B2.

Table D.1: Wave conditions measured during Test Series B1

Test	Wave Probe	Statistical parameters				Spectral parameters			Reflection, transmission & incident wave height	
		H_{max} (m)	$H_{1/10}$ (m)	$H_{1/3}$ (m)	T_m (s)	H_{m0} (m)	T_P (s)	$T_{m0,2}$ (s)	Cr / Tr	H_{si} (m)
B1-1	WP1	13.60	8.78	6.91	8.1	6.99	9.9	7.04		
	WP2	12.23	8.71	6.87	8.0	7.01	9.1	7.09		
	WP3	12.37	8.51	6.81	8.1	6.99	9.5	7.14	0.757	5.57
	WP4	12.19	8.73	6.97	8.0	7.18	9.5	7.16		
	WP5	12.32	8.56	6.96	8.2	7.16	9.5	7.24		
	WP6	13.34	8.05	6.47	7.9	6.63	8.8	7.00		
	WP7	11.14	8.39	6.88	8.1	7.09	9.1	7.17		
	WP8	1.20	0.60	0.40	3.4	0.40	236.6	2.80	0.072	
	WP9	1.19	0.52	0.34	3.1	0.37	236.6	2.75	0.066	
	WP10	1.08	0.45	0.31	3.2	0.36	236.6	2.92	0.065	
B1-2	WP1	14.02	8.80	6.90	7.9	7.05	9.9	7.03		
	WP2	12.95	8.83	7.01	8.2	7.10	9.1	7.10		
	WP3	12.20	8.69	6.93	8.1	7.13	9.5	7.11	0.780	5.60
	WP4	13.21	8.84	7.02	8.0	7.30	9.5	7.13		
	WP5	12.59	8.65	7.09	8.2	7.29	9.5	7.21		
	WP6	12.84	8.12	6.48	7.8	6.68	8.8	6.95		
	WP7	11.75	8.61	6.99	8.1	7.22	9.1	7.12		
	WP8	0.91	0.34	0.21	3.9	0.21	236.6	2.73	0.038	
	WP9	0.87	0.25	0.16	2.8	0.19	236.6	2.31	0.033	
	WP10	0.66	0.23	0.15	2.8	0.19	9.5	2.37	0.034	
B1-3	WP1	15.62	8.99	7.10	7.9	7.16	9.9	6.99		
	WP2	13.34	8.93	7.01	8.0	7.12	9.1	7.02		
	WP3	12.10	8.75	6.96	8.0	7.16	9.9	7.03	0.782	5.62
	WP4	12.40	8.92	7.10	8.0	7.32	9.5	7.08		
	WP5	12.52	8.65	7.06	8.1	7.30	9.5	7.17		
	WP6	13.22	8.23	6.61	8.0	6.74	8.8	6.92		
	WP7	11.98	8.59	6.98	8.0	7.23	9.1	7.08		
	WP8	0.55	0.18	0.13	5.1	0.13	8.8	2.86	0.023	
	WP9	0.39	0.12	0.08	2.7	0.11	9.1	1.85	0.020	
	WP10	0.35	0.13	0.09	2.5	0.14	10.8	1.85	0.024	
B1-4	WP1	13.90	8.64	6.76	7.4	6.96	9.1	4.22		
	WP2	12.03	8.59	6.71	7.4	6.98	9.1	4.31		
	WP3	12.29	8.47	6.74	7.6	7.04	9.5	4.16	0.781	5.52
	WP4	13.18	8.65	6.89	7.6	7.21	9.5	4.22		
	WP5	11.85	8.35	6.87	7.7	7.17	9.5	4.31		
	WP6	11.95	7.98	6.34	7.2	6.63	8.8	4.06		

	WP7	11.91	8.29	6.77	7.5	7.10	9.1	4.41		
	WP8	1.46	0.43	0.27	3.4	0.29	236.6	2.40	0.052	
	WP9	1.48	0.35	0.22	2.7	0.25	236.6	2.20	0.046	
	WP10	0.88	0.31	0.20	2.6	0.26	236.6	2.29	0.046	
B1-5	WP1	14.32	8.89	7.01	7.9	7.07	9.9	7.03	0.781	5.63
	WP2	12.71	8.91	7.00	8.1	7.12	9.1	7.07		
	WP3	12.67	8.72	6.91	8.1	7.16	9.5	7.10		
	WP4	13.00	8.93	7.08	8.1	7.34	9.5	7.14		
	WP5	12.08	8.73	7.12	8.3	7.34	9.5	7.19		
	WP6	13.13	8.27	6.57	7.9	6.72	8.8	6.92		
	WP7	11.86	8.52	6.98	8.0	7.21	9.1	7.16		
	WP8	0.30	0.12	0.09	5.7	0.10	11.8	2.76	0.018	
	WP9	0.30	0.08	0.05	2.5	0.09	9.5	1.52	0.015	
	WP10	0.19	0.11	0.07	2.7	0.12	9.5	1.78	0.022	
B1-6	WP1	13.74	8.89	7.02	7.9	7.09	9.9	7.01	0.784	5.63
	WP2	13.32	8.98	7.05	8.0	7.19	9.1	7.05		
	WP3	12.94	8.77	6.94	8.0	7.17	9.5	7.07		
	WP4	14.27	8.90	7.10	8.0	7.35	9.5	7.09		
	WP5	11.90	8.74	7.14	8.1	7.36	9.5	7.18		
	WP6	12.86	8.25	6.58	7.9	6.75	8.8	6.94		
	WP7	11.92	8.59	7.02	7.9	7.27	9.1	7.09		
	WP8	0.20	0.13	0.10	5.9	0.11	8.8	3.07	0.020	
	WP9	0.18	0.10	0.07	2.8	0.10	8.8	1.72	0.018	
	WP10	0.24	0.13	0.08	2.6	0.14	10.3	1.87	0.024	

Table D.2: Wave conditions measured during Test Series B2

Test	Wave Probe	Statistical parameters				Spectral parameters			Reflection & incident wave height	
		H_{max} (m)	$H_{1/10}$ (m)	$H_{1/3}$ (m)	Tm (s)	Hm0 (m)	H_{max} (m)	$H_{1/10}$ (m)	$H_{1/3}$ (m)	Hsi (m)
B2-1	WP1	14.71	8.93	7.04	8.0	7.29	9.1	7.1	0.75	5.65
	WP2	12.39	8.92	7.08	8.3	7.17	9.1	7.1		
	WP3	12.53	8.64	6.89	8.2	7.05	9.5	7.2		
	WP4	12.68	8.78	7.01	8.3	7.16	9.5	7.2		
	WP5	11.93	8.66	7.10	8.3	7.29	9.5	7.3		
	WP6	14.43	8.19	6.53	8.0	6.70	8.8	7.1		
	WP7	11.25	8.50	7.02	8.3	7.17	9.1	7.2		
B2-2	WP1	13.82	9.07	7.13	8.1	7.35	9.5	7.1	0.77	5.65
	WP2	13.09	9.08	7.12	8.2	7.25	9.1	7.1		
	WP3	12.72	8.79	7.04	8.2	7.16	9.5	7.1		
	WP4	12.22	8.90	7.15	8.3	7.27	9.5	7.1		
	WP5	13.27	8.73	7.18	8.2	7.37	9.5	7.2		
	WP6	15.94	8.26	6.57	7.9	6.75	8.8	7.0		
	WP7	12.51	8.71	7.16	8.2	7.28	9.1	7.2		

	WP1	14.45	9.16	7.16	8.0	7.36	9.1	7.1		
	WP2	12.67	9.09	7.16	8.2	7.28	9.1	7.1		
	WP3	12.77	8.82	7.06	8.1	7.21	9.5	7.1	0.78	5.68
B2-3	WP4	13.81	8.98	7.17	8.3	7.32	9.5	7.1		
	WP5	12.60	8.82	7.25	8.3	7.41	9.5	7.2		
	WP6	15.81	8.48	6.69	8.0	6.82	8.8	7.0		
	WP7	11.86	8.78	7.20	8.3	7.31	9.1	7.2		
	WP1	14.07	9.09	7.13	8.1	7.33	9.1	7.1		
	WP2	12.12	8.87	7.03	8.1	7.14	9.1	7.1		
	WP3	13.15	8.79	6.94	8.0	7.11	9.5	7.1	0.76	5.65
B2-4	WP4	13.67	8.88	7.06	8.1	7.23	9.5	7.1		
	WP5	12.96	8.68	7.13	8.1	7.31	9.5	7.2		
	WP6	15.13	8.30	6.56	7.9	6.71	8.8	7.1		
	WP7	11.61	8.56	7.02	8.0	7.22	9.1	7.2		
	WP1	14.62	9.17	7.18	8.1	7.37	9.1	7.1		
	WP2	13.01	9.09	7.09	8.1	7.23	9.1	7.1		
	WP3	14.17	8.88	7.05	8.1	7.18	9.5	7.1	0.78	5.65
B2-5	WP4	14.36	8.95	7.10	8.1	7.28	9.5	7.1		
	WP5	11.83	8.70	7.20	8.3	7.37	9.5	7.2		
	WP6	16.82	8.36	6.62	7.9	6.78	8.8	7.0		
	WP7	12.48	8.69	7.14	8.3	7.27	9.1	7.2		
	WP1	14.40	9.13	7.17	8.1	7.39	9.1	7.1		
	WP2	13.01	9.02	7.12	8.2	7.23	9.1	7.1		
	WP3	13.12	8.84	7.02	8.1	7.17	9.5	7.1	0.78	5.64
B2-6	WP4	13.94	8.95	7.12	8.2	7.29	9.5	7.1		
	WP5	12.74	8.78	7.18	8.2	7.41	9.5	7.2		
	WP6	15.85	8.35	6.61	7.9	6.75	8.8	7.0		
	WP7	13.08	8.78	7.17	8.2	7.28	9.1	7.2		

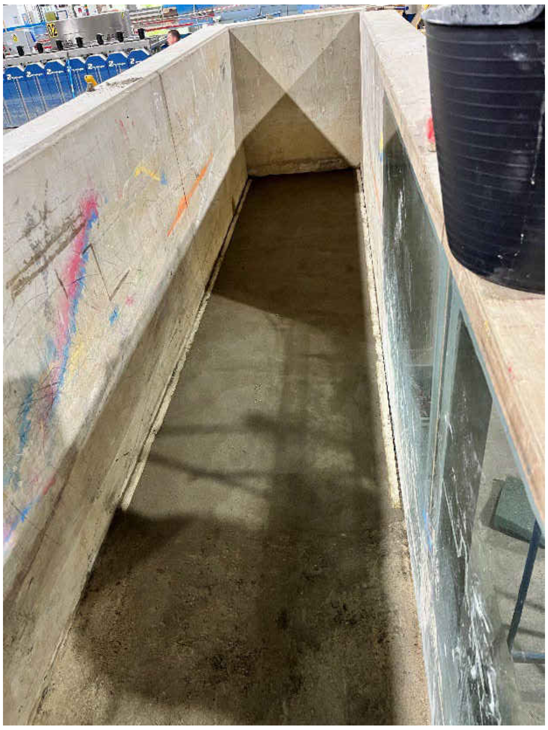
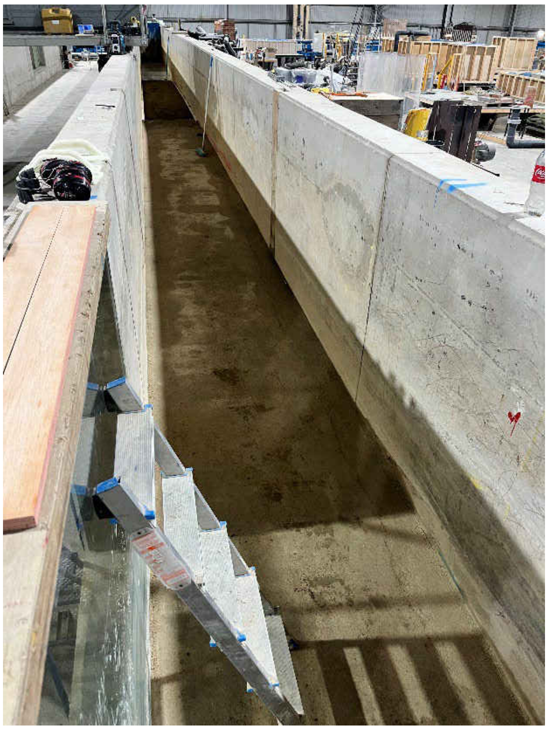
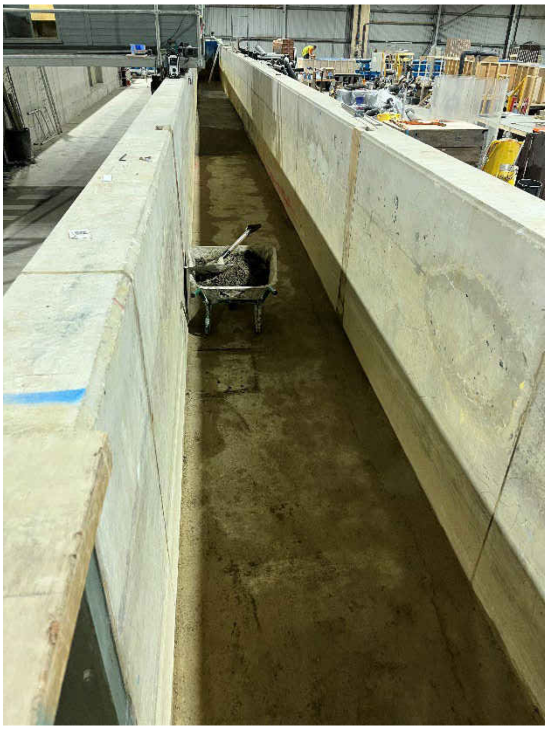
Table D.3: Wave conditions measured during Test Series B3

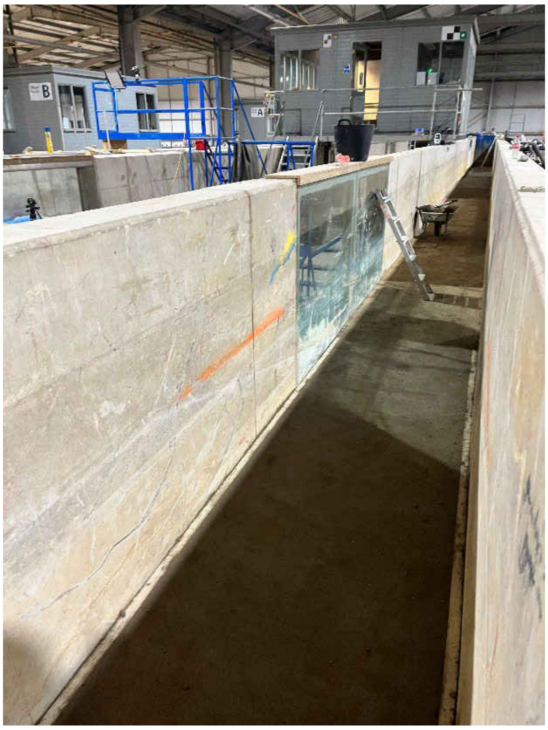
Test	Wave Probe	Statistical parameters				Spectral parameters			Reflection, transmission & incident wave height	
		H_{max} (m)	$H_{1/10}$ (m)	$H_{1/3}$ (m)	Tm (s)	Hm0 (m)	H_{max} (m)	$H_{1/10}$ (m)	$H_{1/3}$ (m)	Hsi (m)
B3-1	WP1	8.24	5.98	4.78	6.9	4.91	7.9	6.12		
	WP2	8.58	5.88	4.65	7.1	4.72	8.5	6.30		
	WP3	9.96	5.92	4.65	7.0	4.77	8.2	6.31	0.794	3.71
	WP4	10.48	5.91	4.69	7.0	4.80	8.2	6.33		
	WP5	8.44	5.73	4.57	7.0	4.70	8.2	6.32		
	WP6	9.63	5.77	4.53	7.0	4.65	8.5	6.27		
	WP7	8.27	5.55	4.35	6.9	4.47	8.5	6.25		
	WP8	0.75	0.24	0.15	4.2	0.15	236.6	2.57	0.040	
	WP9	0.41	0.16	0.10	2.8	0.13	236.6	1.94	0.034	
	WP10	0.37	0.16	0.10	3.2	0.14	29.6	2.28	0.038	
B3-2	WP1	11.32	7.32	5.85	7.5	5.96	8.8	6.61		
	WP2	10.25	7.33	5.84	7.7	5.91	9.1	6.81		

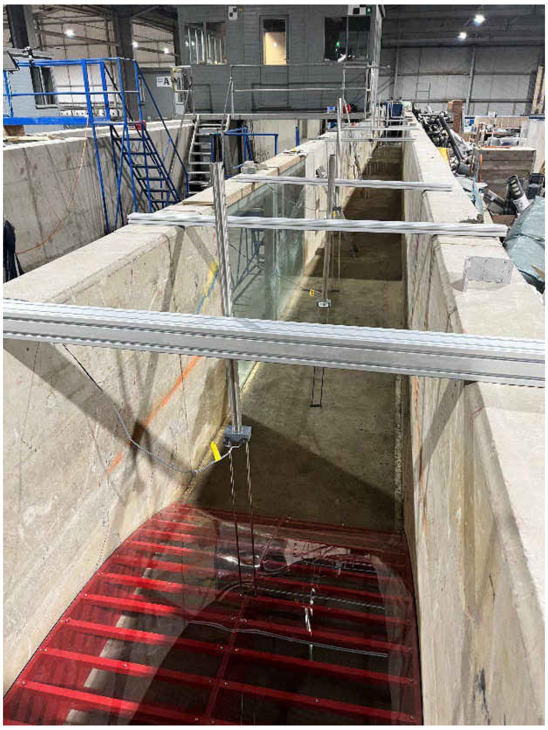
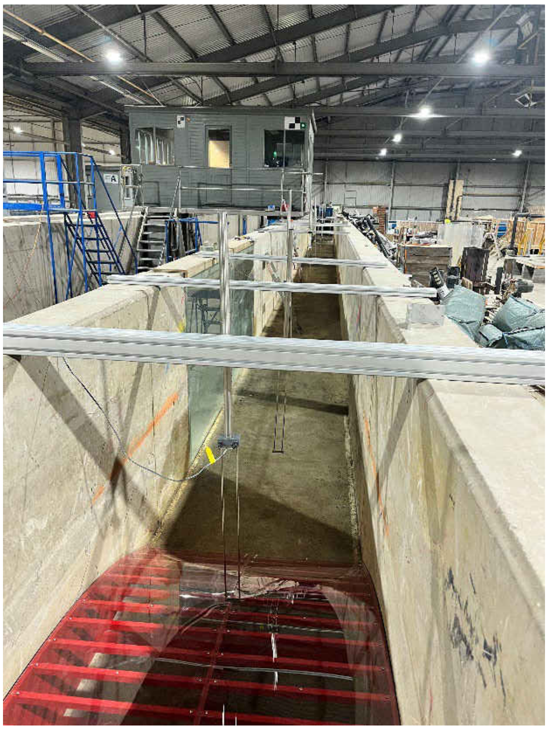
	WP3	9.45	7.02	5.52	7.5	5.66	8.5	6.70	0.785	4.53
	WP4	9.49	6.93	5.50	7.5	5.67	8.2	6.72		
	WP5	10.26	6.72	5.43	7.6	5.61	9.5	6.80		
	WP6	10.38	7.23	5.85	7.6	6.09	8.8	6.87		
	WP7	10.63	7.61	6.07	7.9	6.25	9.1	6.95		
	WP8	1.34	0.54	0.35	3.7	0.34	236.6	2.83	0.076	
	WP9	1.07	0.48	0.31	3.5	0.31	236.6	2.78	0.069	
	WP10	1.04	0.42	0.27	4.2	0.31	29.6	3.30	0.068	
B3-3	WP1	7.66	5.77	4.60	6.9	4.71	7.9	6.08		
	WP2	7.89	5.51	4.38	7.1	4.46	8.5	6.37		
	WP3	8.16	5.46	4.35	7.1	4.45	8.2	6.39	0.695	3.62
	WP4	8.23	5.45	4.36	7.2	4.45	8.2	6.43		
	WP5	7.91	5.37	4.27	7.2	4.40	8.2	6.41		
	WP6	7.94	5.36	4.27	7.0	4.35	8.8	6.36		
	WP7	8.13	5.28	4.11	6.9	4.22	8.5	6.34		
	WP8	1.08	0.51	0.34	3.4	0.35	236.6	2.75	0.097	
	WP9	1.11	0.42	0.29	3.5	0.30	236.6	2.78	0.083	
	WP10	0.80	0.38	0.26	4.1	0.31	29.6	3.41	0.086	
B3-4	WP1	10.84	6.96	5.64	7.6	5.72	8.8	6.63		
	WP2	10.95	6.92	5.47	7.8	5.53	8.5	6.87		
	WP3	8.77	6.42	5.17	7.6	5.26	8.5	6.80	0.677	4.44
	WP4	8.63	6.42	5.16	7.7	5.27	8.5	6.82		
	WP5	8.42	6.30	5.05	7.7	5.20	9.5	6.87		
	WP6	9.04	6.86	5.51	7.8	5.71	8.8	6.97		
	WP7	9.32	7.02	5.61	7.9	5.77	9.1	7.02		
	WP8	1.93	0.90	0.61	3.5	0.64	236.6	2.99	0.144	
	WP9	1.40	0.78	0.54	3.5	0.57	236.6	3.10	0.129	
	WP10	1.25	0.72	0.49	4.0	0.56	236.6	3.57	0.126	

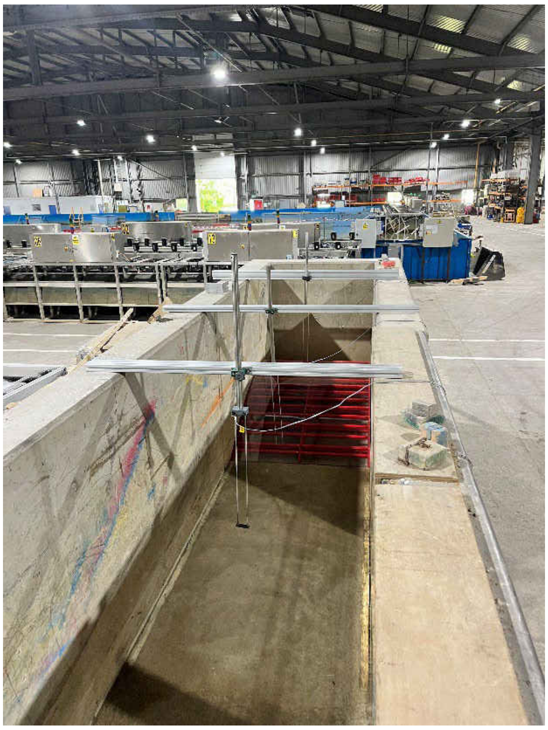
E Photographs

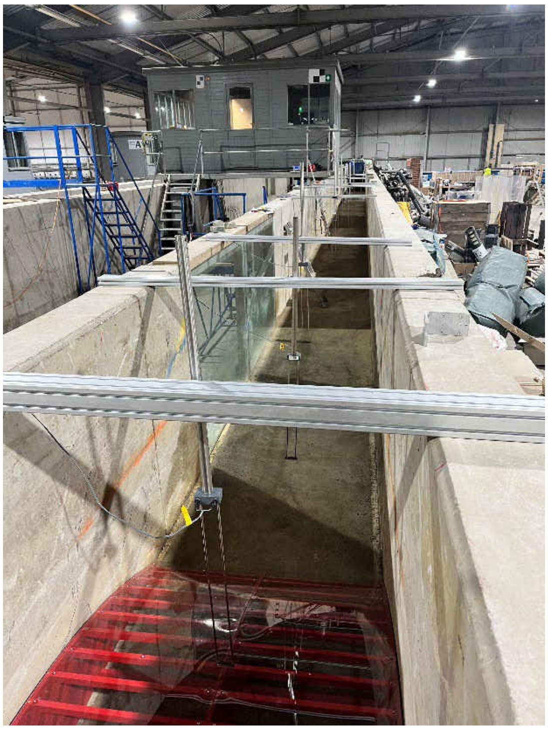
E.1 Bathymetry construction

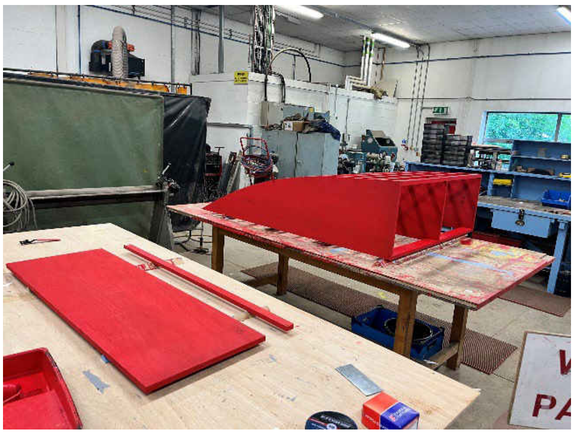


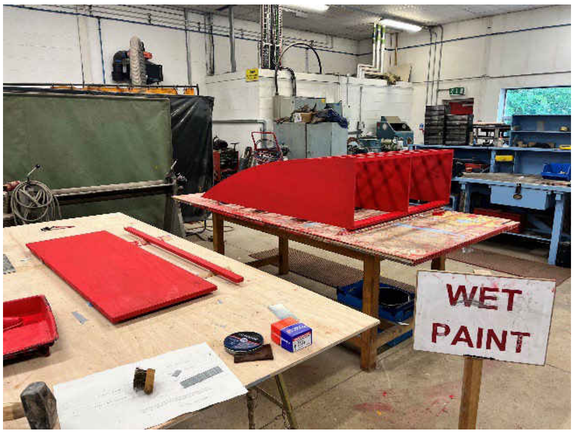



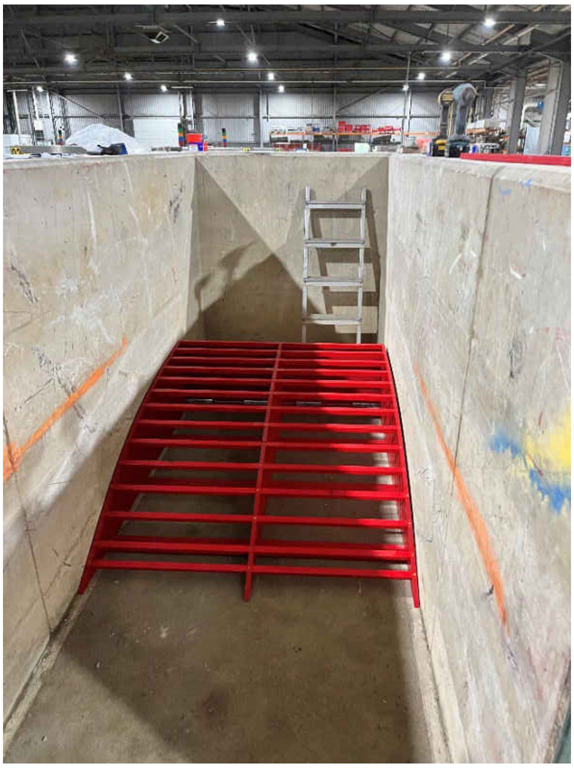


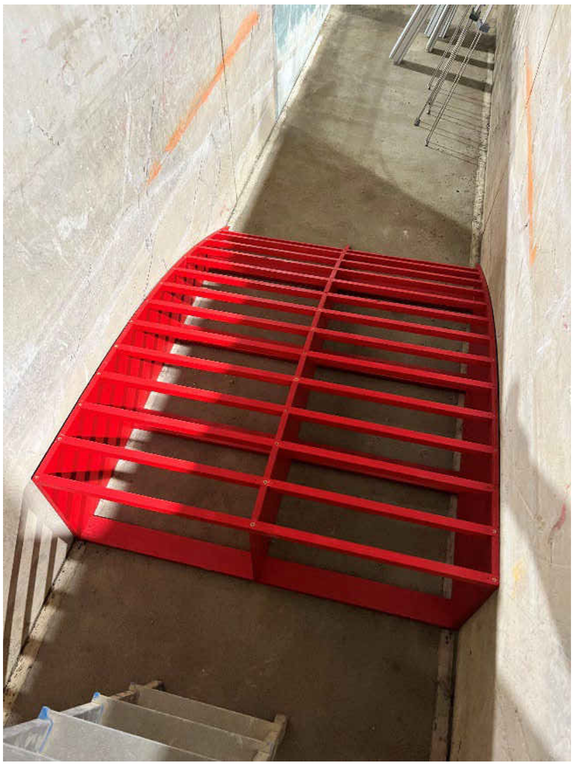


E.2 Passive beach

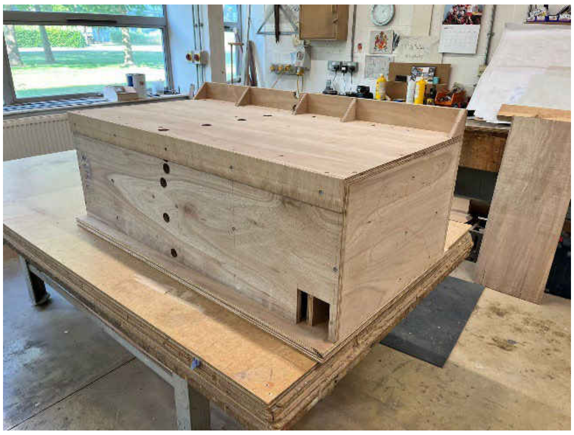




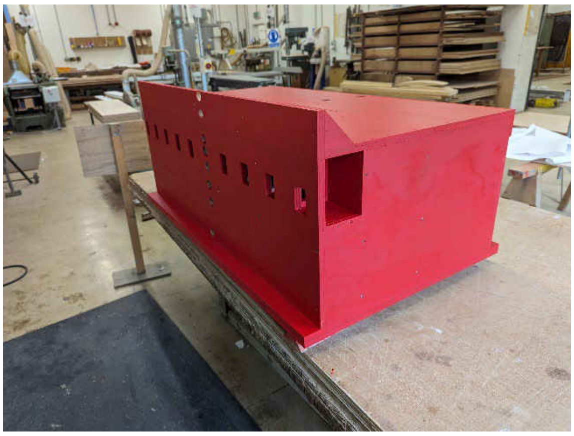




E.3 Model caisson

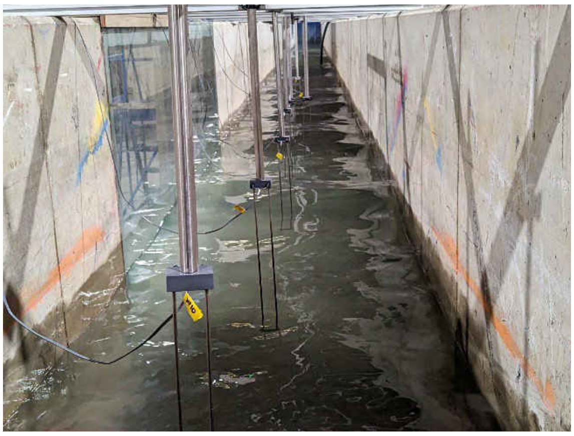




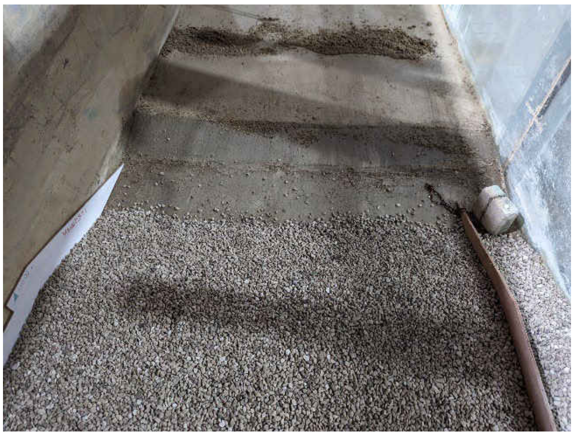


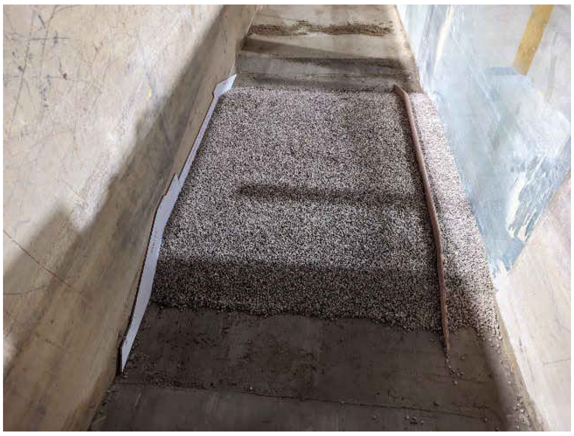

E.4 Wave calibration






E.5 Breakwater construction



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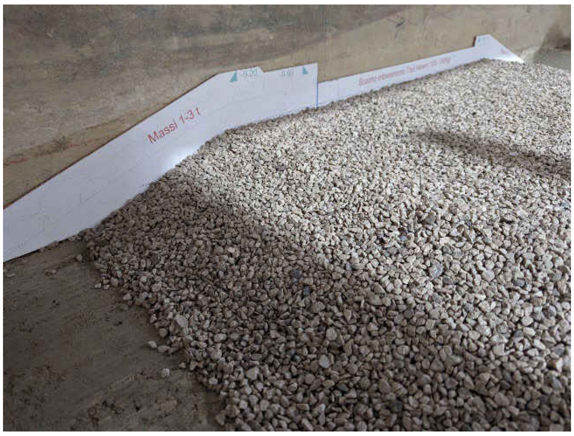
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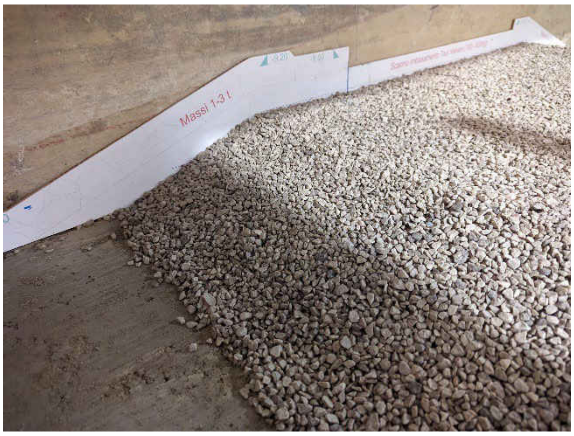
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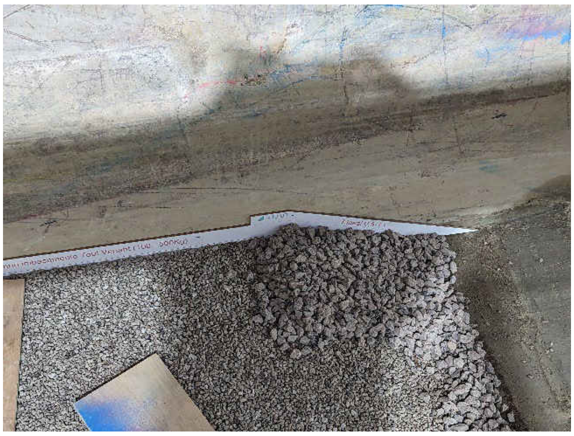
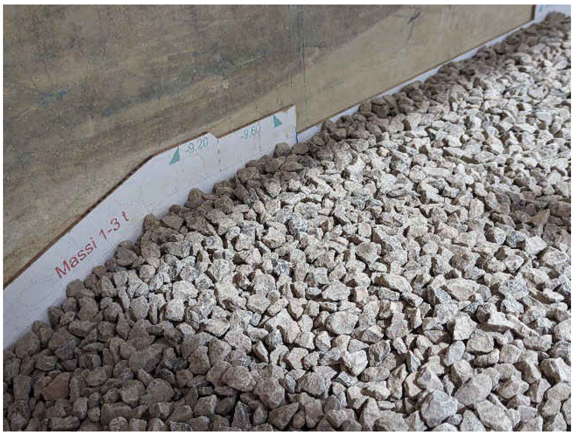



Massi 0.5-1 t


Massi 0.5-1 t


-13.80
Massi 1-3 t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Massi 1-3 t


Massi 1-3 t


Massi 1-3 t


-9.20
-9.60


Massi 1-3 t


Massi 1-3 t
-13.80





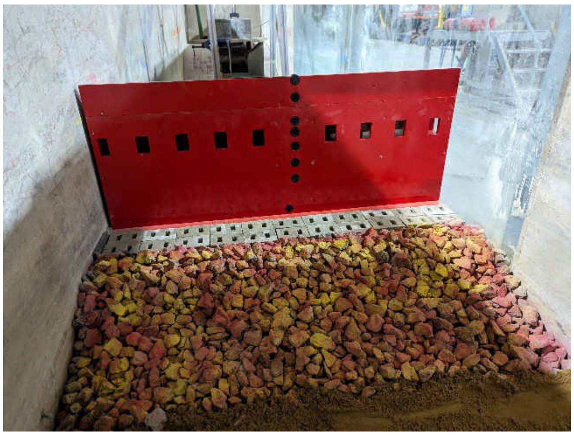
-9.20
Massi 1-31









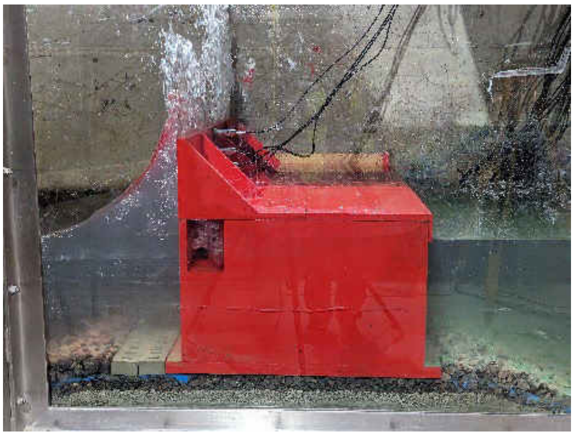
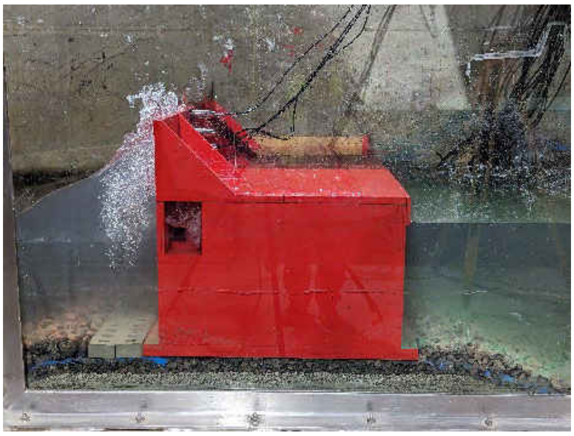




E.6 Toe block casting



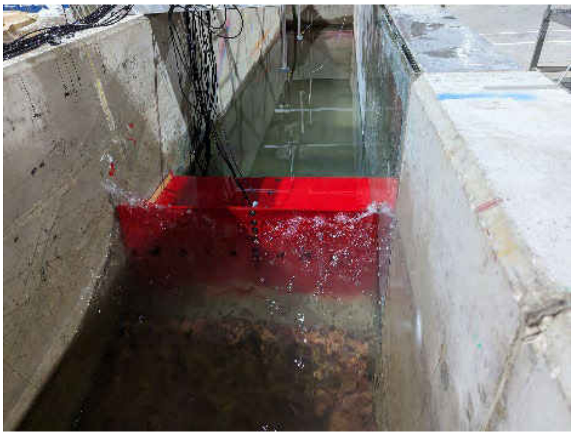

E.7 Test series B1



























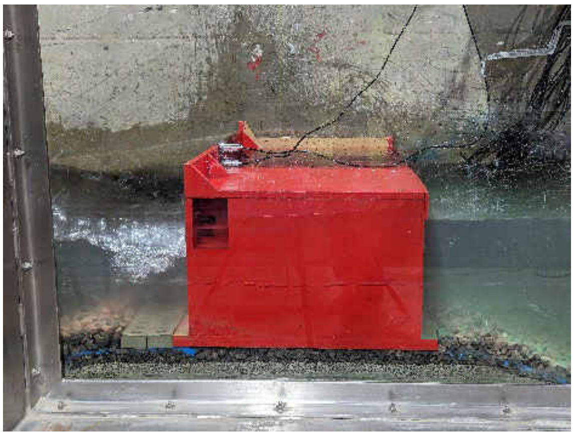
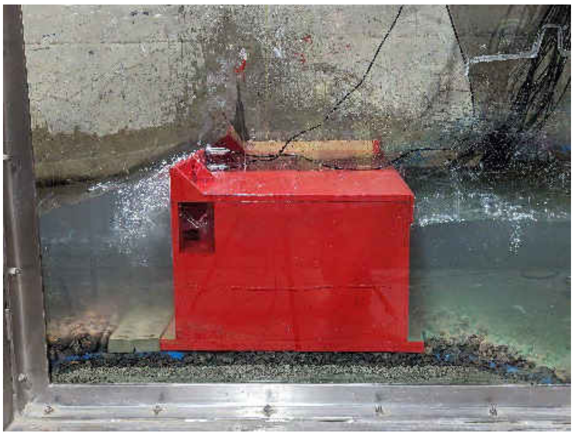


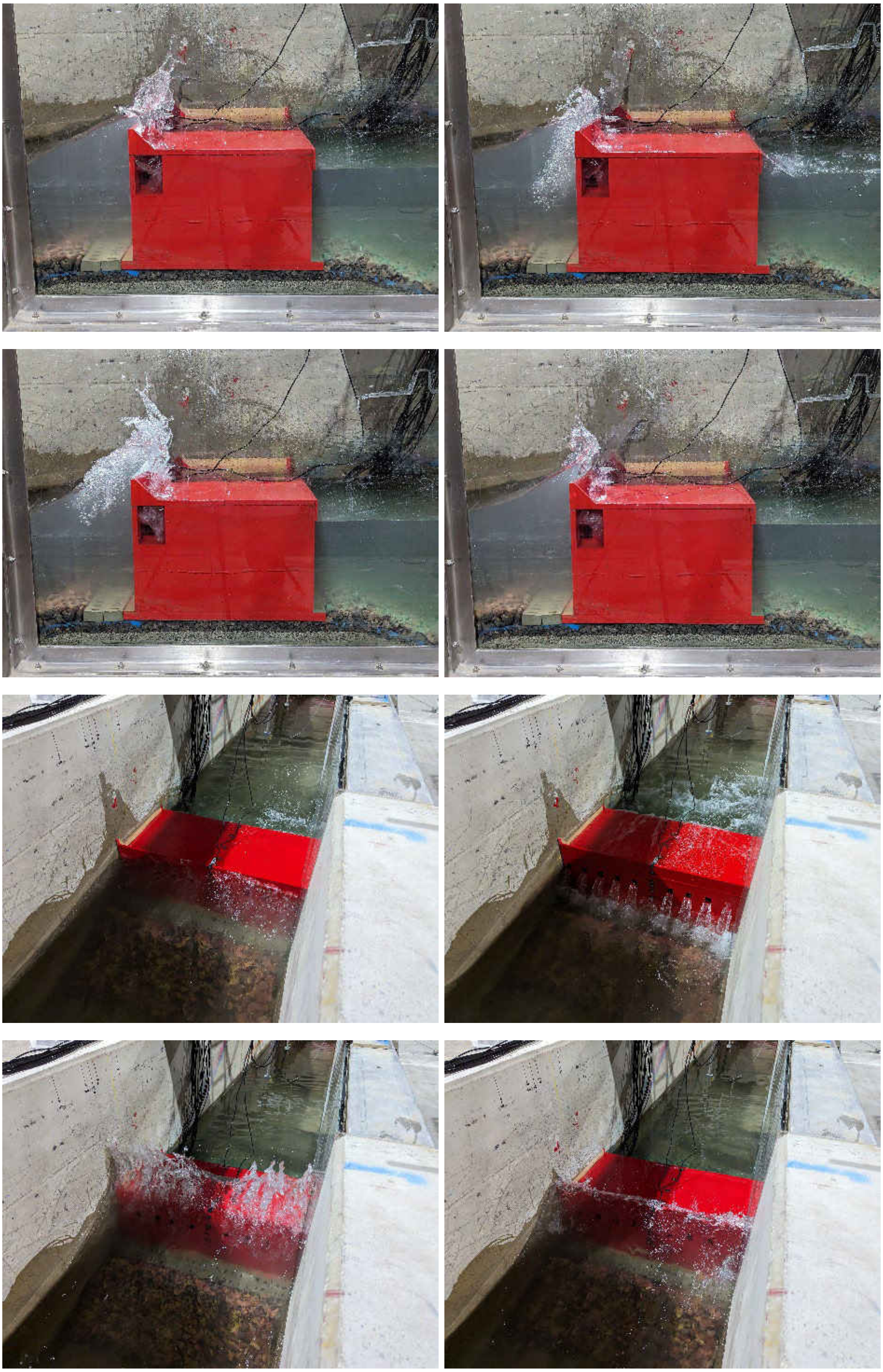








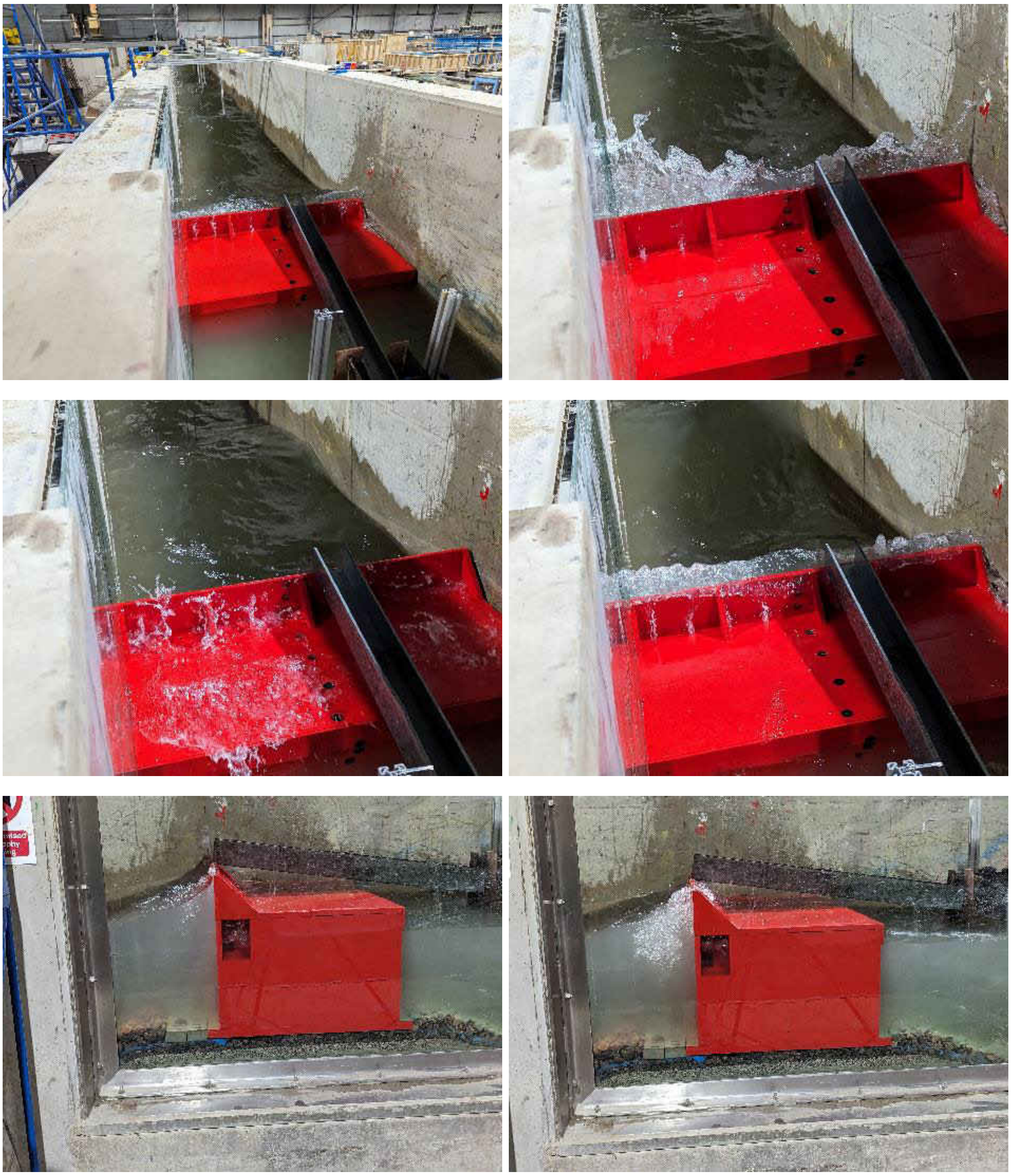


E.8 Test B2-1






E.9 Test B2-2

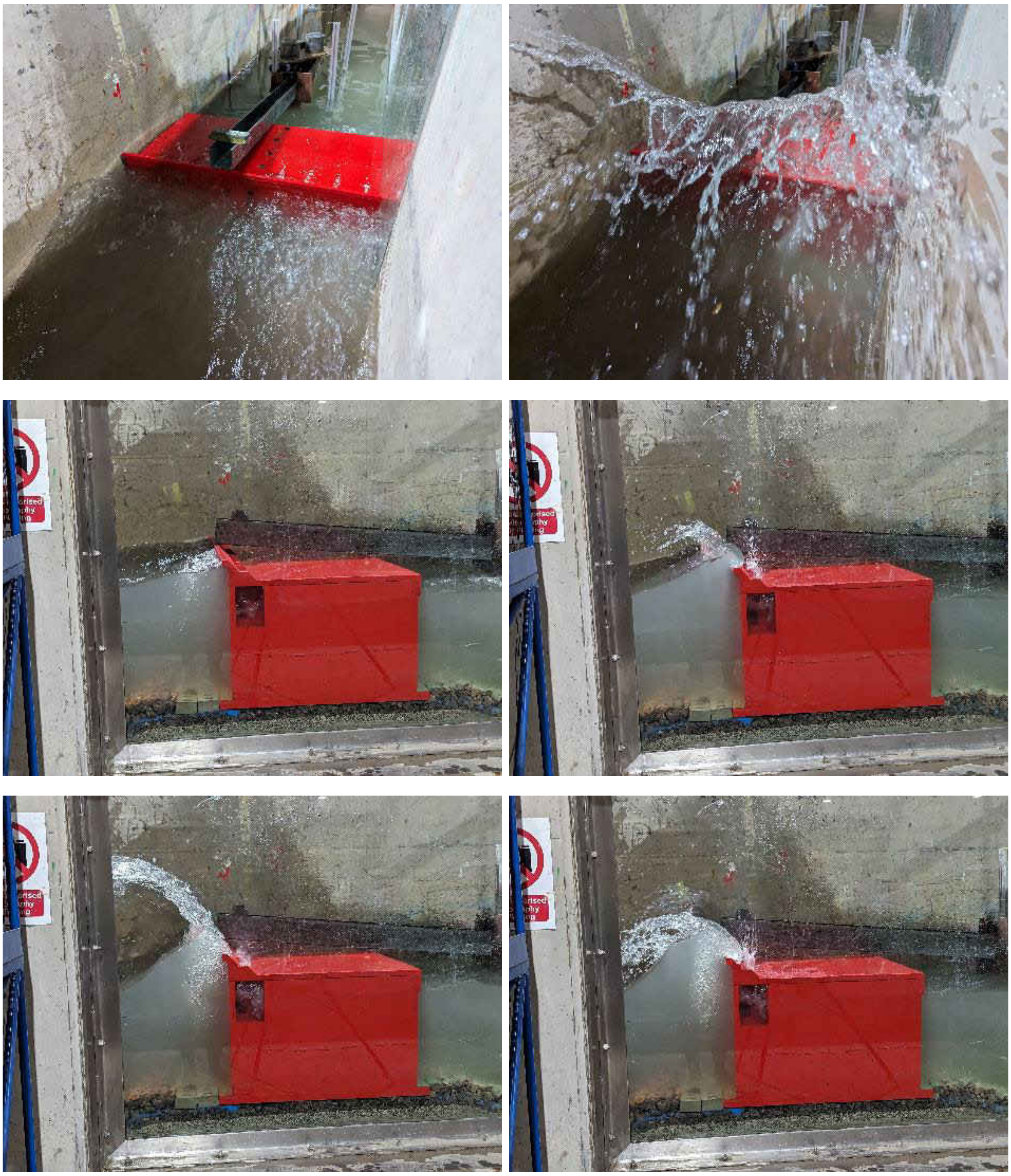


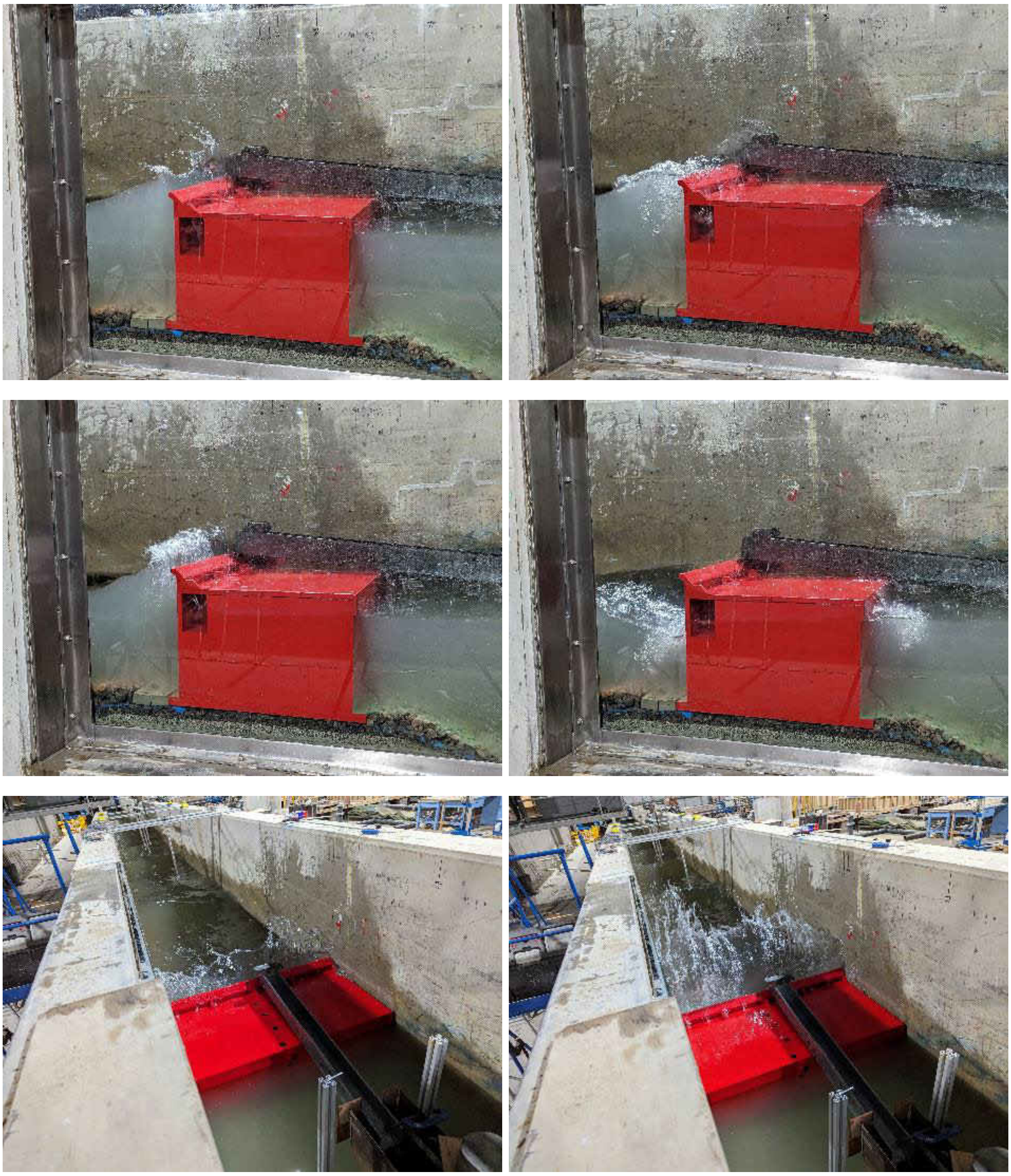





E.10 Test B2-3

E.11 Test B2-4

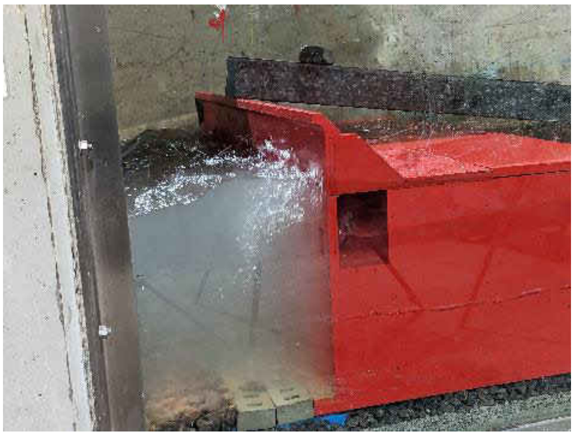




E.12 Test B2-5



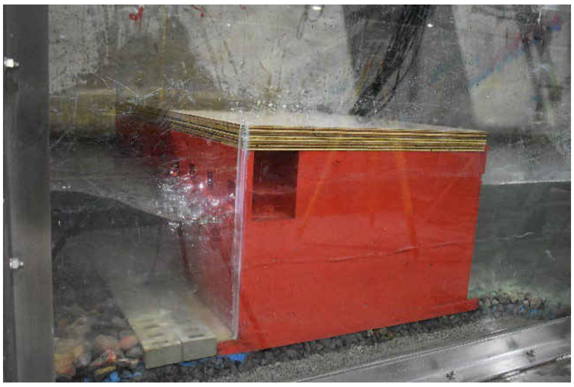
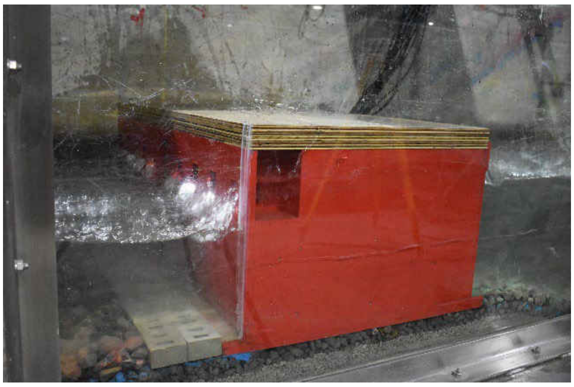
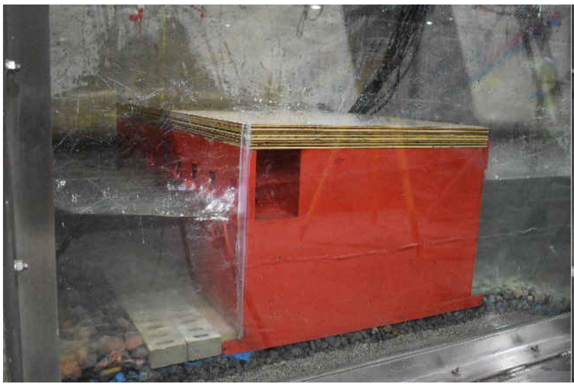


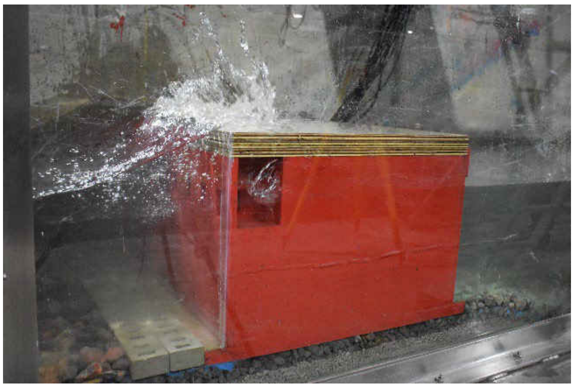


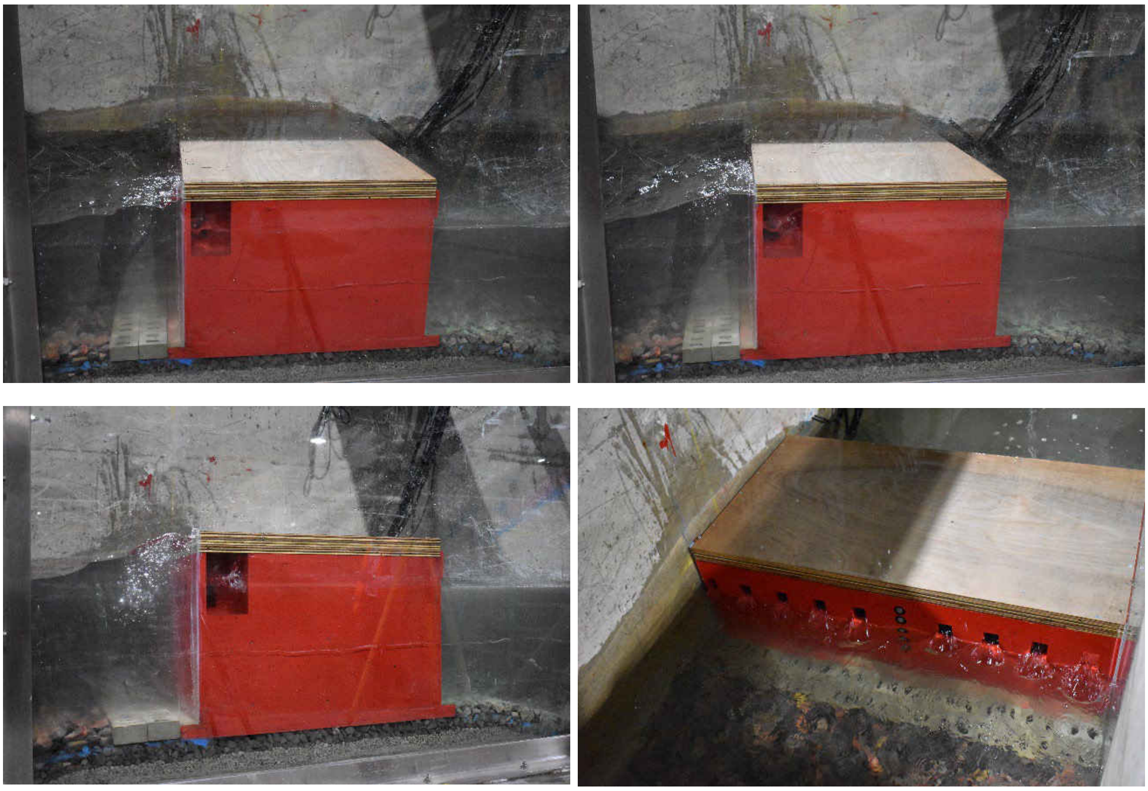

E.13 Test B2-6

E.14 Test B3-1

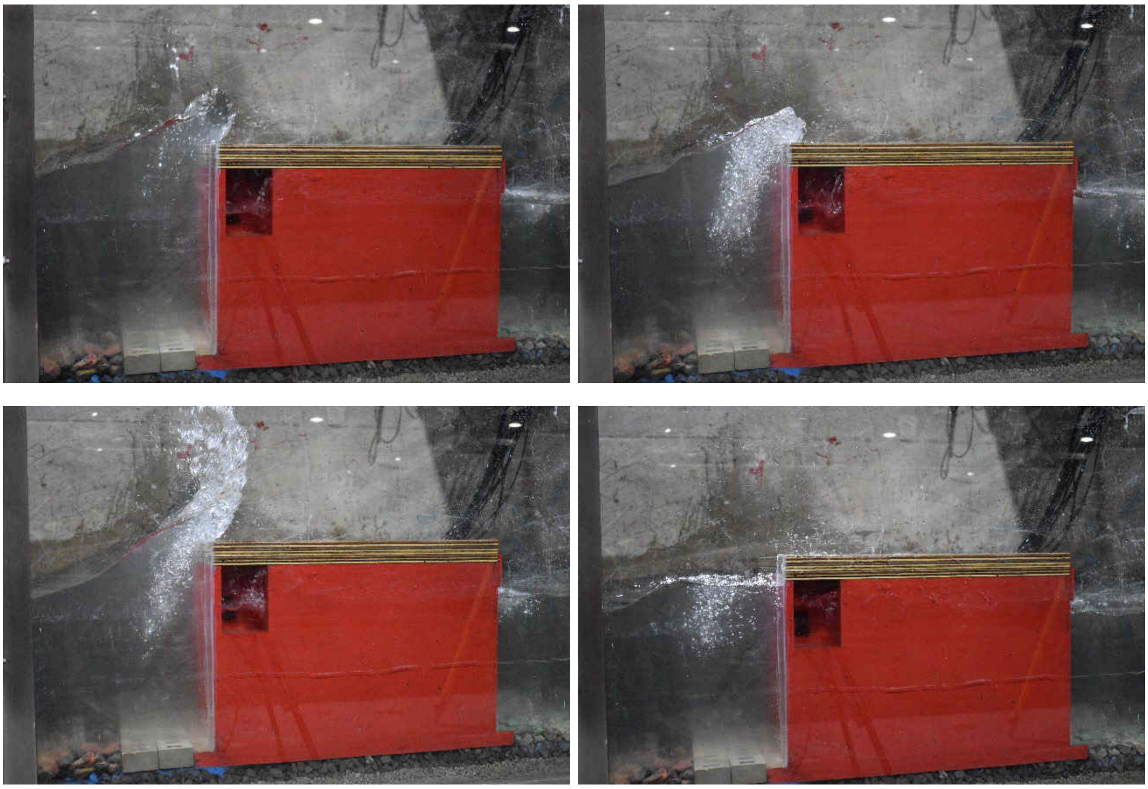




E.15 Test B3-2

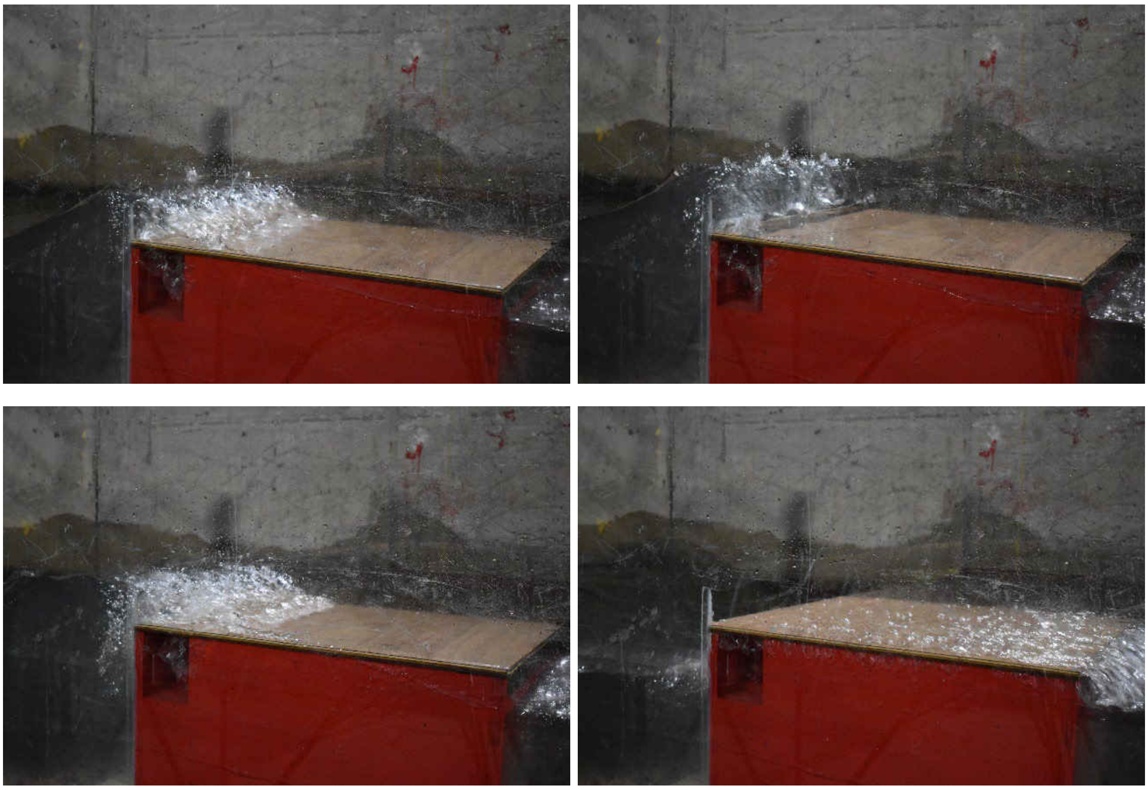






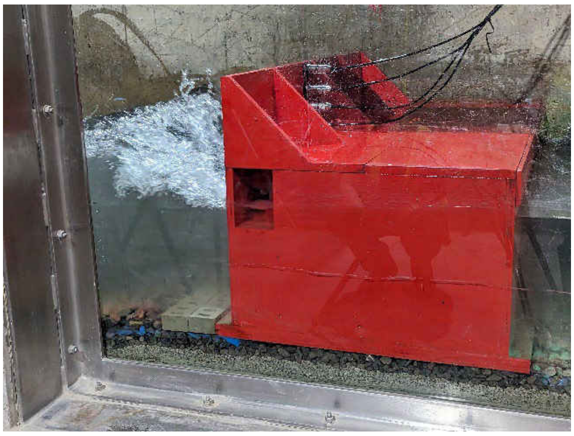

E.16 Test B3-3

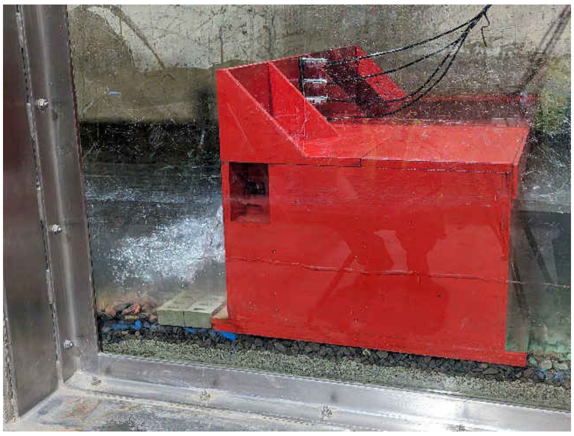

E.17 Test B3-4

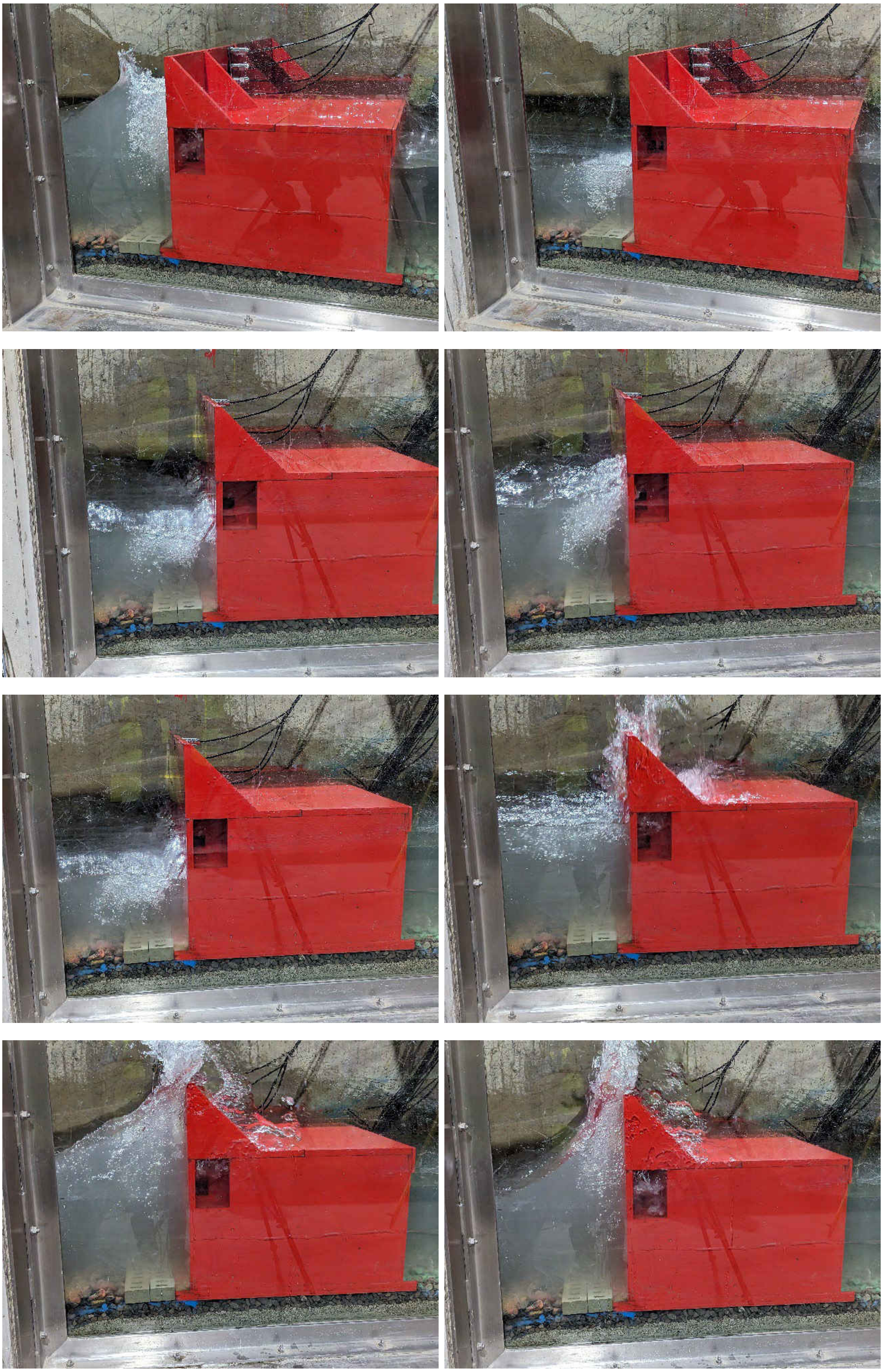


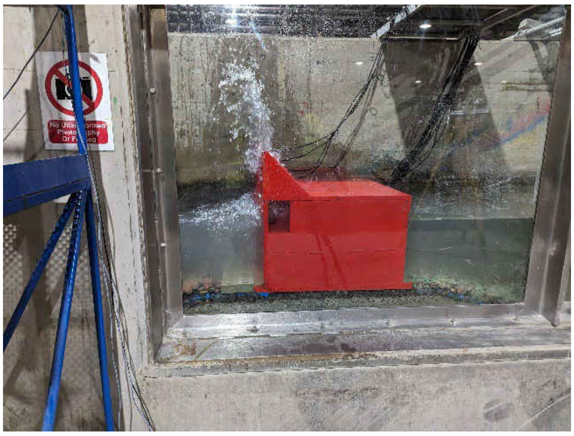
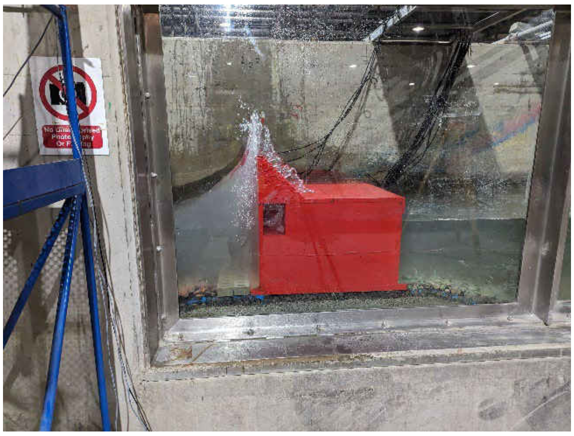
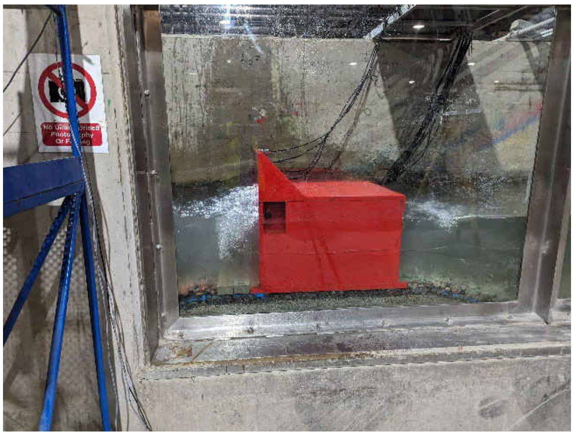
E.18 Test C1-1 and C1-2









F Predicted overtopping calculations

HR Wallingford
Working with water

Cover sheet

Project No.	DJR6894	Calc No.	#001
Project	DJR6894 - 2D physical model study		
Subject	Vertical wall overtopping, with and without recurve		

	Description	Total Sheets	Made by	Date	Checked by	Date	Remarks
Rev 1	Overtopping vertical wall	3	JSA	11/12/2023	ATU	12/12/2023	
Rev							
Rev							
Rev							
Rev							
Rev							

Description of calculation

Subject:

Overtopping of vertical wall, with and without recurve wall

Assumptions:

Probabilistic mean overtopping discharge for a Vertical wall with recurve has been calculated for comparison with measured results.
Calculation completed using EurOtop_II_2018.

Vertical wall has rubble mound toe, which may or may not influence the wave overtopping

References

http://www.overtopping-manual.com/eurotop/downloads/

Results summary:

Breakwater configuration		Wave conditions					Predicted mean overtopping discharge (l/s/m)
Crest elevation (mMSL)	With or without recurve	Return period Tr (yrs)	H_s (m)	T_p (m)	Wave spectra	SWL (mMSL)	
+6.5	without	100	5.5	9.6	Jonswap	+1.7	179
+8.5	without	100	5.5	9.6	Jonswap	+1.7	65
+10.5	without	100	5.5	9.6	Jonswap	+1.7	24
+6.5	with	100	5.5	9.6	Jonswap	+1.7	99
+8.5	with	100	5.5	9.6	Jonswap	+1.7	20
+10.5	with	100	5.5	9.6	Jonswap	+1.7	4.7

Project	DJR6894 - 2D physical model study	Project No.	DJR6894	Sheet	1	of	3
		Calculation No.	#001				
Subject	Vertical wall overtopping	Made by	JSA	Date	11/12/2023		
		Checked by	ATU	Date	12/12/2023		

Calculation | References/Comments

1.0 Methodology and definitions

Probabilistic mean overtopping discharge for a Vertical wall with recurve.
Calculation completed using EurOtop_II_2018

2.0 Input data

2.1 Environment

SWL	1.7	mMSL	Design Still Water Level
H_{m0}	5.5	m	Estimate of significant wave height from spectral analysis, $H_{m0}=4\sqrt{m_0}$
$T_{m-1,0}$	8.7	s	Spectral wave period defined by m_{-1}/m_0, if not available assume $T_{m-1,0} = T_p/1.1$
β	0.0	m	Angle of wave attack relative to normal on structure

2.2 Structure

2.2.1 General

d_{seabed}	-14.3	m	Seabed level
CL	6.5	mMSL	Crest Level

2.2.2 Composite

toe	Yes	-	Presence of absence (yes or no)
d_{toe}	-9.2	m	Top of toe mound level

2.2.2 Bullnose

bullnose	Yes	-	Presence of absence (yes or no)
Br	1.0	m	Horizontal extension of wave return wall
hr	1.0	m	Height of wave return wall
α	90.0	deg	Angle of wave return wall, in degrees, 0 < α < 90

2.0 Input data

R_c	4.8	m	Crest freeboard of structure
d	10.9	m	Water depth above toe mound in front of vertical seawall
h	16.0	m	Water depth in front of toe mound in front of vertical seawall
Lm-1,0	77.6	m	
d/(0.6h)	1.1	m	Toe mound has no significant influence: Plain Vertical Wall case.
h·d/(Hm0 Lm-1,0) < 0.65	0.4	m	None impulsive wave breaking

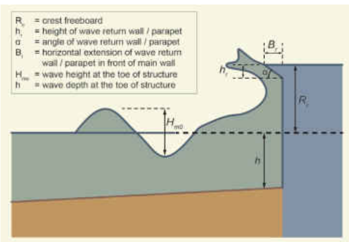
3.0 Method

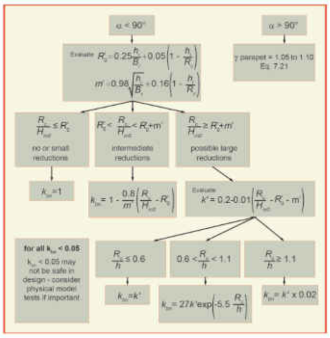
Non-impulsive conditions. For the case of simple vertical walls, with influencing foreshore, under non-impulsive conditions, the mean value approach, Equation 7.5, is developed from the averages of the measured data and should be used for predictions and comparisons with measurements. This is the same as the prediction equation originally proposed by Allsop et al. (1995) for this class of structure under non-impulsive overtopping conditions

$$\frac{q}{\sqrt{gH_{m0}^3}} = 0.05exp(-2.78\frac{R_c}{H_{m0}}) \qquad 7.5$$

Bullnose reduction value Kbn

$$k_{bn} = \frac{q_{with\_bullnose}}{q_{without\_bullnose}} \qquad 7.23$$





4.0 Results

4.1.1 Vertical wall overtopping discharge

Q*	0.00442	-	Dimensionless discharge
Q	179	l/s/m	Mean overtopping discharge

4.1.2 Recurve reduction factor

Ro*	0.25990	-	coefficient
m*	1.10667	-	coefficient
Rc/Hmo	0.87273	-	coefficient
Ro* + m*	1.36656	-	coefficient
Kbn	0.56	-	Reduction factor
Q recurve	99	l/s/m	Mean overtopping discharge

Project	DJR6894 - 2D physical model study	Project No.	DJR6894	Sheet	2	of	3
		Calculation No.	#001				
Subject	Vertical wall overtopping	Made by	JSA	Date	11/12/2023		
		Checked by	ATU	Date	12/12/2023		

Calculation | References/Comments

1.0 Methodology and definitions

Probabilistic mean overtopping discharge for a Vertical wall with recurve.
Calculation completed using EurOtop_II_2018

2.0 Input data

2.1 Environment

SWL	1.7	mMSL	Design Still Water Level
H_{m0}	5.5	m	Estimate of significant wave height from spectral analysis, $H_{m0}=4\sqrt{m_0}$
$T_{m-1,0}$	8.7	s	Spectral wave period defined by m_{-1}/m_0, if not available assume $T_{m-1,0} = T_p/1.1$
β	0.0	m	Angle of wave attack relative to normal on structure

2.2 Structure

2.2.1 General

d_{seabed}	-14.3	m	Seabed level
CL	8.5	mMSL	Crest Level

2.2.2 Composite

toe	Yes	-	Presence of absence (yes or no)
d_{toe}	-9.2	m	Top of toe mound level

2.2.2 Bullnose

bullnose	Yes	-	Presence of absence (yes or no)
Br	1.0	m	Horizontal extension of wave return wall
hr	1.0	m	Height of wave return wall
α	90.0	deg	Angle of wave return wall, in degrees, 0 < α < 90

2.0 Input data

R_c	6.8	m	Crest freeboard of structure
d	10.9	m	Water depth above toe mound in front of vertical seawall
h	16.0	m	Water depth in front of toe mound in front of vertical seawall
Lm-1,0	77.6	m	
d/(0.6h)	1.1	m	Toe mound has no significant influence: Plain Vertical Wall case.
h·d/(Hm0 Lm-1,0) < 0.65	0.4	m	None impulsive wave breaking

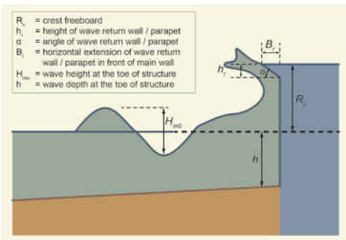
3.0 Method

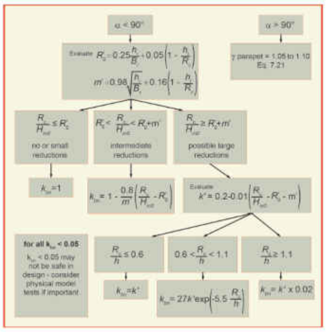
Non-impulsive conditions. For the case of simple vertical walls, with influencing foreshore, under non-impulsive conditions, the mean value approach, Equation 7.5, is developed from the averages of the measured data and should be used for predictions and comparisons with measurements. This is the same as the prediction equation originally proposed by Allsop et al. (1995) for this class of structure under non-impulsive overtopping conditions

$$\frac{q}{\sqrt{gH_{m0}^3}} = 0.05exp(-2.78\frac{R_c}{H_{m0}}) \quad 7.5$$

Bullnose reduction value Kbn

$$k_{bn} = \frac{q_{with\_bullnose}}{q_{without\_bullnose}} \quad 7.23$$





4.0 Results

4.1.1 Vertical wall overtopping discharge

Q*	0.00161	-	Dimensionless discharge
Q	65	l/s/m	Mean overtopping discharge

4.1.2 Recurve reduction factor

Ro*	0.26066	-	coefficient
m*	1.11647	-	coefficient
Rc/Hmo	1.23636	-	coefficient
Ro* + m*	1.37713	-	coefficient
Kbn	0.30	-	Reduction factor
Q recurve	19.5	l/s/m	Mean overtopping discharge

Project	DJR6894 - 2D physical model study	Project No.	DJR6894	Sheet	3	of	3
		Calculation No.	#001				
Subject	Vertical wall overtopping	Made by	JSA	Date	11/12/2023		
		Checked by	ATU	Date	12/12/2023		

Calculation	References/Comments

1.0 Methodology and definitions

Probabilistic mean overtopping discharge for a Vertical wall with recurve.
Calculation completed using EurOtop_II_2018

2.0 Input data

2.1 Environment

SWL	1.7	mMSL	Design Still Water Level
H_{m0}	5.5	m	Estimate of significant wave height from spectral analysis, $H_{m0}=4\sqrt{m_0}$
$T_{m-1,0}$	8.7	s	Spectral wave period defined by m_{-1}/m_0, if not available assume $T_{m-1,0} = T_p/1.1$
β	0.0	m	Angle of wave attack relative to normal on structure

2.2 Structure

2.2.1 General

d_{seabed}	-14.3	m	Seabed level
CL	10.5	mMSL	Crest Level

2.2.2 Composite

toe	Yes	-	Presence of absence (yes or no)
d_{toe}	-9.2	m	Top of toe mound level

2.2.2 Bullnose

bullnose	Yes	-	Presence of absence (yes or no)
Br	1.0	m	Horizontal extension of wave return wall
hr	1.0	m	Height of wave return wall
α	90.0	deg	Angle of wave return wall, in degrees, 0 < α < 90

2.0 Input data

R_c	8.8	m	Crest freeboard of structure
d	10.9	m	Water depth above toe mound in front of vertical seawall
h	16.0	m	Water depth in front of toe mound in front of vertical seawall
Lm-1,0	77.6	m	
d/(0.6h)	1.1	m	Toe mound has no significant influence: Plain Vertical Wall case.
h·d/(Hm0 Lm-1,0) < 0.65	0.4	m	None impulsive wave breaking

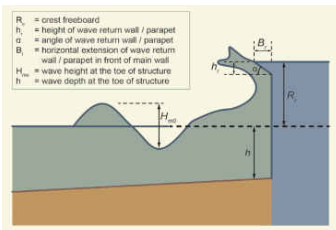
3.0 Method

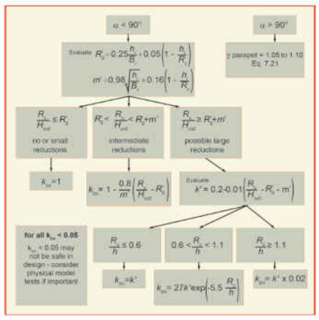
Non-impulsive conditions. For the case of simple vertical walls, with influencing foreshore, under non-impulsive conditions, the mean value approach, Equation 7.5, is developed from the averages of the measured data and should be used for predictions and comparisons with measurements. This is the same as the prediction equation originally proposed by Allsop et al. (1995) for this class of structure under non-impulsive overtopping conditions

$$\frac{q}{\sqrt{gH_{m0}^3}} = 0.05exp(-2.78\frac{R_c}{H_{m0}}) \qquad 7.5$$

Bullnose reduction value Kbn

$$k_{bn} = \frac{q_{with\_bullnose}}{q_{without\_bullnose}} \qquad 7.23$$





4.0 Results

4.1.1 Vertical wall overtopping discharge

Q*	0.00059	-	Dimensionless discharge
Q	24	l/s/m	Mean overtopping discharge

4.1.2 Recurve reduction factor

Ro*	0.26108	-	coefficient
m*	1.12182	-	coefficient
Rc/Hmo	1.60000	-	coefficient
Ro* + m*	1.38290	-	coefficient
Rc/h	0.55	-	coefficient
Kbn	0.20	-	Reduction factor
Q recurve	4.7	l/s/m	Mean overtopping discharge

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HR Wallingford
Howbery Park
Wallingford
Oxfordshire OX10 8BA
United Kingdom

+44 (0)1491 835381

info@hrwallingford.com

www.hrwallingford.com

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