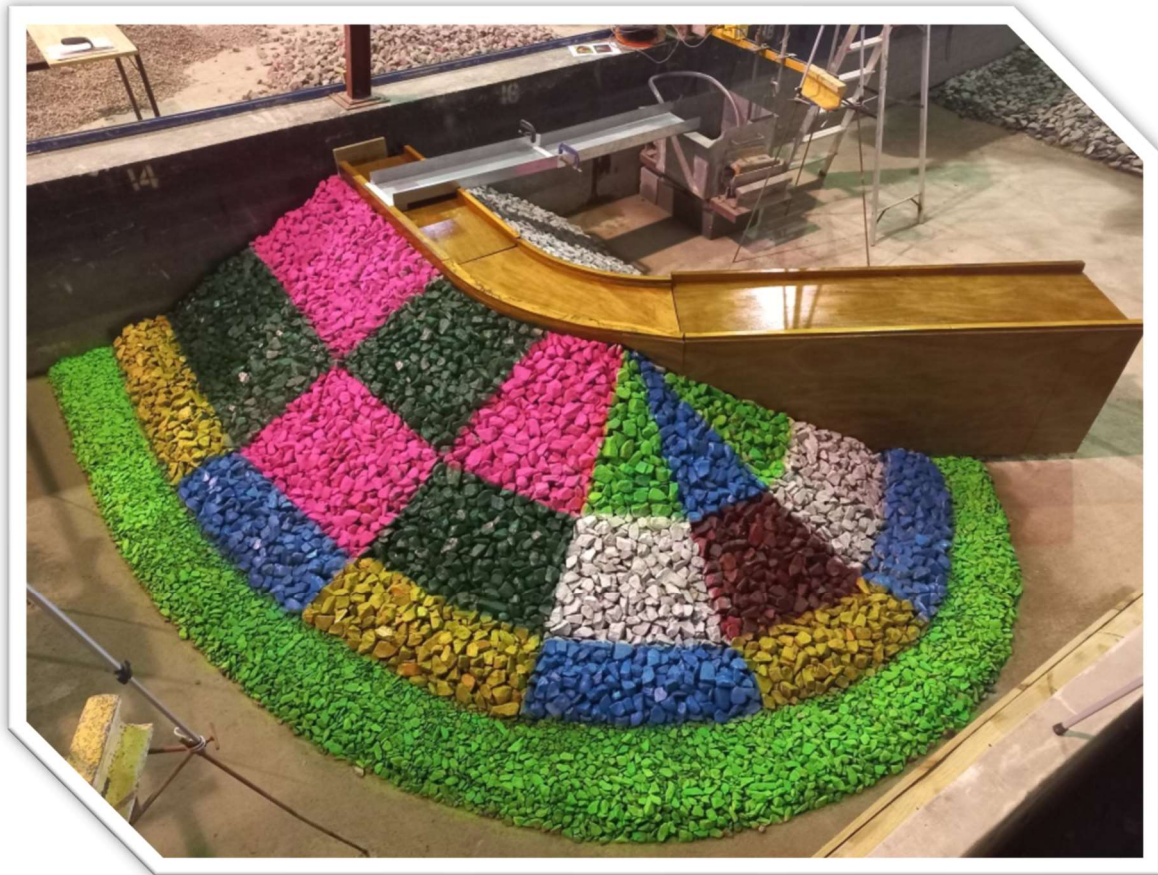


Lubec Breakwater Project, USA

2D Physical model studies

Project report



July 2024

Prepared by:

Prepared for:



Jacobs

Lubec Breakwater Project, USA

2D Physical model studies

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1. INTRODUCTION

1.1. General

Lubec is part of the state of Maine in the United States of America (USA) and is located on a peninsula which lies adjacent to a narrow strait that forms the border between Canada and the USA. The Maine Department of Transportation (MaineDOT) is intending to construct a breakwater at Lubec to provide a safe harbour for fishing and recreational vessels. Jacobs Engineering Group Inc. was appointed by MaineDOT to fulfil the role of consulting engineers and was responsible for the planning and detailed design of the breakwater structure. The Council for Scientific and Industrial Research's Coastal Engineering and Ports Infrastructure group (CSIR) was contracted by Jacobs Engineering Group Inc. (Client) to commission a 2D physical model study to assess and verify the stability of key elements of the design.

The project site is located on the eastern shore of Johnson Bay, Maine, and the project will include the construction of a breakwater approximately 925 ft (282 m) in length. About 630 ft (192 m) of the structure will comprise a rubble mound structure and 295 ft (90 m) of a King Pile Sheet Pile Combi wall breakwater. The approximate location of site is indicated in **Figure 1**.

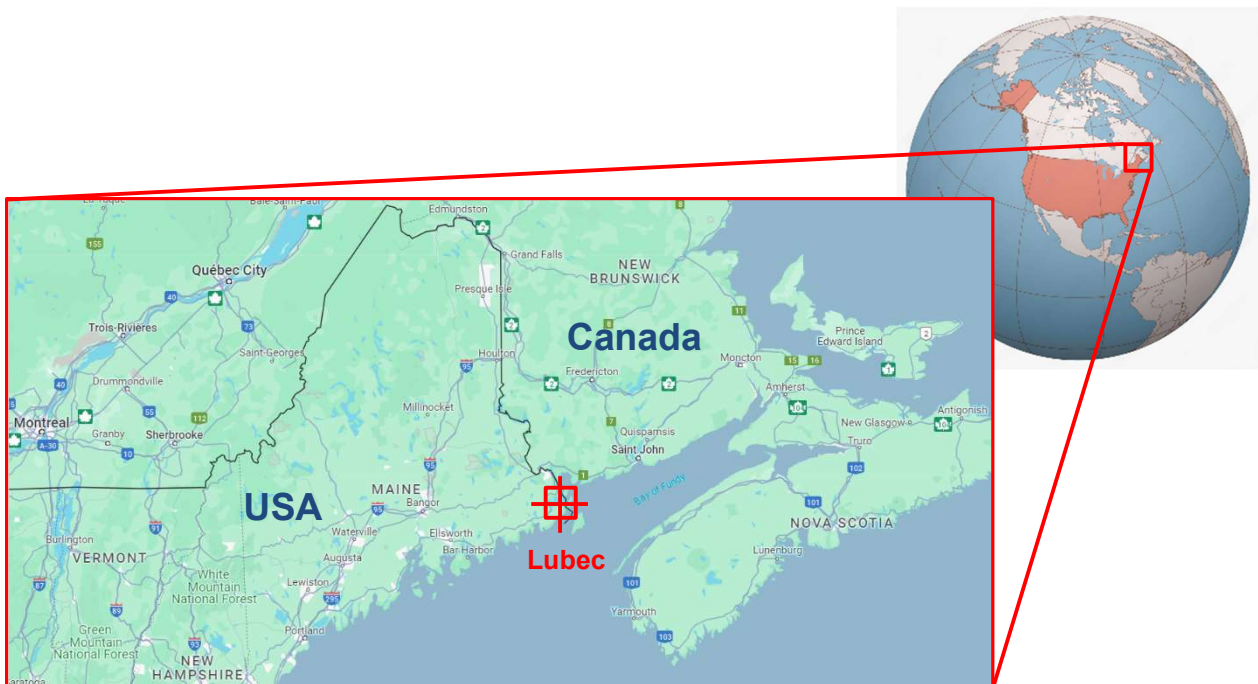


Figure 1: Project locality

The observations and results obtained during these physical model studies will be used to supplement the detailed design process by validating the performance of the proposed design options.

1.2. Study objectives

The objective of this 2D physical model study was to evaluate the performance and behaviour of key components of the Lubec breakwater design. This included verifying the stability of the rock armour layer and toe design, assessing potential interaction between the rubble mound and the combi wall structures, and measuring wave overtopping at a critical location on the trunk.

The study objectives included the following:

- Testing the breakwater design against of a range of wave scenarios, including moderate intensity events, the design wave conditions and extreme overload conditions. These also included different seawater level elevations for the various events.
- Measurements to identify damage to the design armour layer (seaward and harbour side slopes) and rock toe.
- The stability of the transition from the rubble mound structure to the King Pile Sheet Pile Combi wall structure.
- Measuring wave overtopping at the trunk section behind the parapet wall (STA_15+30).

To achieve the study objectives, a 2D model setup was commissioned inside a 4 m wide flume. The model coverage, test structure detail and test schedule for each model setup was confirmed by the Client prior to commissioning. All physical model testing was successfully conducted at the hydraulics laboratory of the CSIR in Stellenbosch, South Africa.

1.3. Report layout

This report summarises the construction and setup of the physical model, the test results obtained and the observations made during the study. The content of this document are as follows:

- Section 2 provides a description of the facility and equipment used to set up the physical model;
- Section 3 provides details on the test results; and
- Section 4 provides a testing summary and conclusions.

All parameters given in this report refer to prototype unless otherwise stated.

2. STUDY METHODOLOGY

This section provides details of the project, as well as of the physical model facilities and equipment used during this study.

2.1. General

A physical model study was required as input for the detailed design of the Lubec Breakwater Project. This study comprises a 2D physical model setup and the model scale was selected in cooperation with the Client. The model scale for this study was 1:20 and was based on a best fit of key components of the design to be included in the model setup.

The scope of this model study was defined by the Client and included verification of key design features of the Lubec Breakwater Project.

2.1.1. Rock armour layer stability

The Lubec breakwater design comprised a rock armour layer, where the rock size was defined by standard commercially available rock material. The same rock size was selected for the breakwater front and rear slopes (seaward side and harbour side), the roundhead, as well as the breakwater toe. The rock size for the breakwater was selected based on the design wave conditions near the structure toe.

During this study, the stability of two different armour rock sizes were investigated. This included 1-3 t rock (Series A) and 600-1900 kg rock (Series B). The underlayer rock was varied depending on the armour layer size and included 300-1000 kg and 60-300 kg rock for the two series respectively. The breakwater core comprised 5-40 kg material and was kept constant throughout this study.

2.1.2. Rock toe stability

The Lubec breakwater design included the same rock sizes for the structure toe as was used for the armour layer design. Filter material also extended beyond the toe, which comprised the same material used for the underlayer.

2.1.3. Overtopping

Wave overtopping was measured at a single location within the model. The measurement location was selected at the end of the trunk just before the transition to the combi wall. This location was considered the most critical since it will be located in deeper water depths. The breakwater trunk was orientated to be perpendicular to the direction of wave approach, which is commonly regarded as yielding a more conservative outcome.

2.2. Physical modelling

The methods and procedures described in this section were used during this physical model study.

2.2.1. Model bathymetry

The bathymetric detail used for this study was extracted from survey detail provided by the Client. The survey detail was provided in AutoCAD format, which allowed it to be easily scaled and transformed to a 1 m by 1 m grid in the x-y plane.

Grid lines for constructing the model floor were set out inside the flume using a Leica Total Station, where all points were referenced to a local coordinated grid system. Point elevations were adjusted and verified using a dumpy level surveying instrument. This combination allowed all x-y-z points to be staked out with millimetre precision.

Bulk filling sand was hauled into the flume, and then accurately shaped and compacted to form the correct contour detail of the model floor. All nodal points were fixed in place using construction templates and then covered with a 50 mm thick layer of cement-sand topping mix. Once the topping had dried and hardened, it was sufficiently strong to work on. This method of construction assumes a constant seabed roughness for the model and does not take minor seabed features into account, e.g. sand ripples and small outcrops.

Construction of the model bathymetry is shown in **Figure 2**. The cement-sand topping is constructed in 1 m wide strips, which span the length of the model flume. Every second line is constructed first and then once dry, the areas between lines are filled.

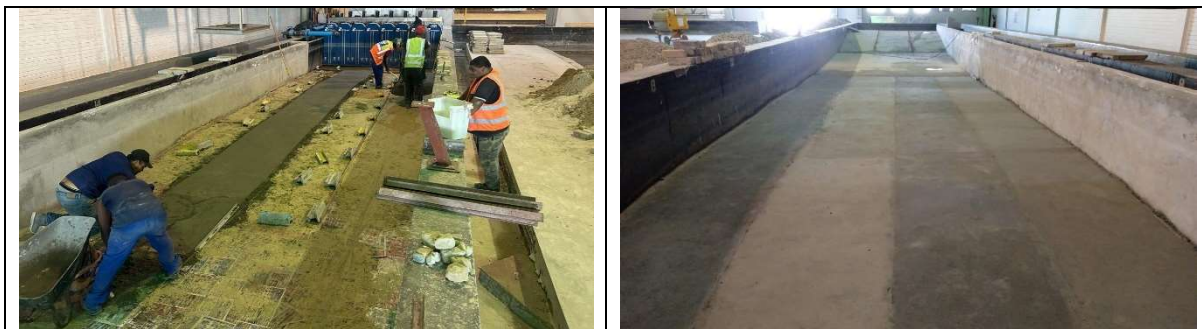


Figure 2: Construction and completion of the model bathymetry

The model floor for this setup included sufficient bathymetric cover in front of the test structure, which was equal to at least three times the longest wavelength (HYDRALAB III, 2007a). For this study, the longest wavelength was determined to be no more than 35 m, which is based on $T_P = 4.71$ s at 13.5 m water depth at the structure toe. A flat area was included in front of the wave generator to accommodate the maximum paddle stroke, and a gentle transition slope of 1:15 was constructed to link the flat area to the model bathymetry.

2.2.2. Model test structure

Once completed, the Lubec breakwater will be approximately 925 ft (282 m) in length, which includes a 630 ft (192 m) long rubble mound structure and a 295 ft (90 m) King Pile Sheet Pile combi wall breakwater. The model test structure only focused on the area closer to the rubble mound roundhead and included detail between stations STA_14+85 and STA_17+50. The coverage of the model test structure within the 4 m wide flume is shown in **Figure 3**.

The model test structure was constructed to the detail provided by the Client. The layout and positioning of the structure was determined from plan drawings (in AutoCAD format) and accurately staked out in the model flume using a Leica total station. Templates containing cross sectional detail were fabricated from hardboard and were accurately positioned and levelled in place. Rock material for the structure core, underlayer, main armour and toe were placed to the detail according to the templates, and once all material were in place, the templates were carefully removed. Surface indentations where templates were located were then filled and reshaped to match the adjacent structure detail.

The concrete roadway and combi wall breakwater was fabricated from wood in three main sections to accurately replicate the straight parts and complex corners. The vertical piles were replicated using \varnothing 48 mm PVC pipes. The top elevation of all parapet walls was accurately surveyed and then fixed in

place to the top of the rubble mound structure. The combi wall breakwater was joined to the adjacent capping, as well as to the model floor via the PVC piles.

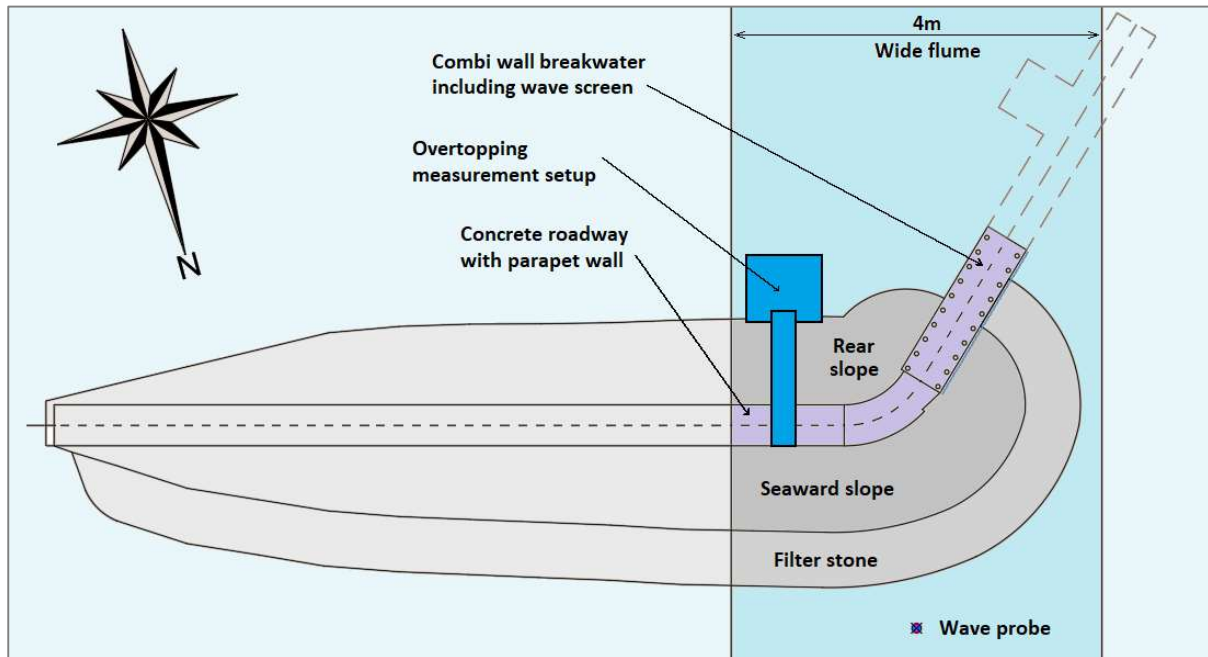


Figure 3: Coverage of the model test structure within the 4 m wide flume

Model rock for the toe, underlayer and armour layer were scaled according to Froude's law, whilst the core material was scaled in accordance with Burchardt (1999). Both methods take the difference between the specific density of seawater (prototype) and that of fresh water (model) into account.

During this study, the stability of two different armour rock sizes was investigated. This included 1-3 t rock (Series A) and 600-1900 kg rock (Series B). The underlayer rock was varied depending on the armour layer size and included 300-1000 kg and 60-300 kg rock for the two series respectively. The breakwater core comprised 5-40 kg material and was kept constant throughout this study. Apart from the different sized rock material, all structural dimensions and other cross-sectional detail for the two breakwaters remained the same.

The prototype material classification and corresponding model sizes are presented in **Table 1**.

Table 1: Rock material classification and sizes

Description	Prototype classification	Model classification
Armour layer & toe (Series A)	1-3 ton	40 – 80 mm
Underlayer (Series A)	300 – 1000 kg	35 – 40 mm
Armour layer & toe (Series B)	600 – 1900 kg	36 - 60 mm
Underlayer (Series B)	60 – 300 kg	12 – 28 mm
Core	5 – 40 kg	12 – 19 mm

Rock samples were taken to ensure that the model rock was representative of the prototype material included in the design. Grading curves for the different rock material are included in **Appendix F**.

The setup and implementation of the model test structure is shown in **Figure 4**. Image A shows placement of the underlayer rock on top of the finer core. All underlayer rock was spraypainted a bright green colour for the purpose of aiding its positive identification if this material became exposed during testing. Similarly, as seen in Image B, the armour layer was also spraypainted in different colours to aid identifying and tracking of rock movement. The different colours conformed to different areas of the design, e.g. trunk, roundhead, etc., as well as separating the top and bottom halves. More images of the model setup and implementation are included in **Appendix E**.

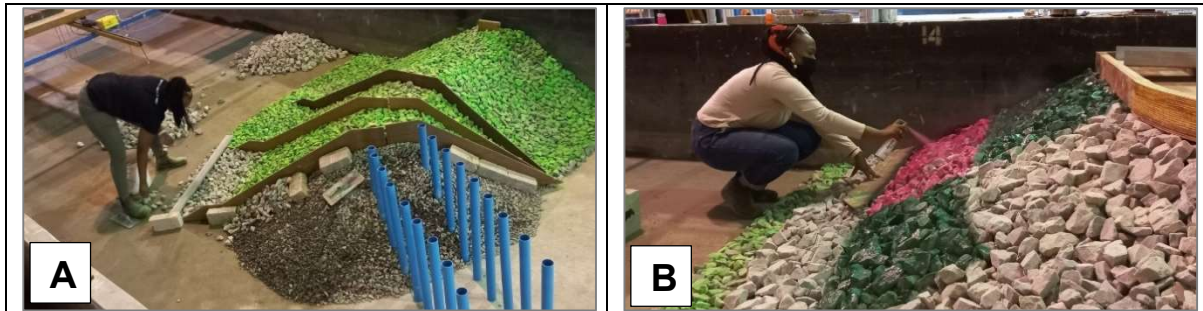


Figure 4: Set up of the model test structure

All design drawings for the test structures were provided by the Client in .dwg format. A typical cross-section of the rubble mound breakwater is included in **Appendix G**. The final rock placement for both test structures was inspected by the Client prior to the start of testing.

2.2.3. Wave generators

Waves inside the 4 m wide flume were generated using a multi-element wave generator manufactured by HR Wallingford, UK. The wave generator comprise a rack and pinion paddle system and waves are generated by synchronised pulsating movements of the paddles. A single wave module measuring 4 m wide was used for the Lubec study.

Wave conditions for this study were generated as irregular (random) long crested waves. Waves were defined by the standard JONSWAP spectral shape using a peak-enhancement factor (γ) of 3.3. All waves were generated perpendicular to the wave generators.

2.2.4. Wave measurement

Wave measurements were taken using capacitance probes. These are twin wire gauges that measure the capacitance difference between the air-water interface as it fluctuates with passing waves. The output datasets captured from the probes were analysed using GEDAP analysis software developed by the Canadian Hydraulics Centre (Miles, 1997 and Miles & Funke, 2013) to provide usable outputs such as H_{mo} , T_P , etc. Before the start of each model test series, all probes were checked and calibrated to ensure that they functioned properly.

Four probe locations were identified for this study. This included a single probe close to the structure toe, as well as a three-probe reflection array that was used to measure wave reflection inside the flume. The data from this setup was analysed using the method developed by Mansard & Funke (1980) to separate the reflective waves from the incident waves.

All probe locations were verified by the Client.

2.2.5. Wave calibration

Wave calibration is required to validate the input parameters for wave generation and to achieve a desired wave condition at a specified calibration location. The calibration location for this study was selected at P-04 close to the structure toe.

Wave calibration was conducted without the presence of the test structures. An absorption beach was placed at the back of the flume, which comprised of large coarse rock. Absorption rock was placed at a slope of 1:12. The wave measurements recorded during calibration are included in **Appendix A**.

2.2.6. Test conditions

The test program for this study required each model setup to be subjected to multiple sea states. The intensity of the sea states ranged from a calmer shake down condition, to more severe design and overload conditions. The design waves were associated with a 100-year return period, while H_s for the overload conditions were 10% greater than that of the design conditions and included an additional storm surge. All water levels were referenced to NAVD88.

The test schedule is shown in **Table 2**. These include different conditions simulated for both Series A and Series B.

Table 2: Model test conditions

	Test ID	Return Period (yr)	Water Level (m NAVD88)	Hm0 (m)	Tp (s)	Duration (Hrs)	Design aspect verification
Series A	A1	1	4.57	0.90	3.69	10	Rock stability (slope and toe) Wave overtopping
	A2	10	4.83	1.44	4.10	10	
	A3	50	4.99	1.87	4.46	10	
	A4	100	2.03	2.07	4.71	10	
	A4B	100	-3.66	1.50	4.05	10	
	A5	100	5.05	2.07	4.71	10	
Series B	B1	1	4.57	0.90	3.69	10	Rock stability (slope and toe)
	B5	100	5.05	2.07	4.71	10	Wave overtopping
	B6	100*	5.55	2.28	4.71	10	

Note(*): Overload condition

The wave measurements recorded during testing are included in **Appendix A**. It should be noted that wave condition A4B was not calibrated prior to the start of testing.

2.2.7. Stability analysis

Rock displacements were tracked and quantified using the image-overlay flicker technique. This technique checks for displacements by comparing photographs taken before and after each test. To limit any shifts or changes between the before and after photos, each camera was set up on a tripod and triggered remotely to avoid any unnecessary handling. Four digital cameras were used to capture all areas of the test structure. Each camera was set up overlooking the structure at an angle that was nearly perpendicular to the test slope to achieve an orthogonal view of the structure slopes.

Rock movements are classified according to its magnitude. Only full displacements are considered as damage, which include those where rock was shifted a distance greater than D_{n50} regardless of whether it remained on the slope or not. Rocks that were dislodged from the slope were identified and its migration was tracked to prevent double counting of full displacements in subsequent tests. The classification used to quantify rock displacements is given in **Table 3**.

Table 3: Rock displacement (X) classification

Classification	Displacement	Damage description
1	$\frac{1}{4} D_{n50} < X < \frac{1}{2} D_{n50}$	Settlement
2	$\frac{1}{2} D_{n50} < X < D_{n50}$	Rocking / flipping over
3	$X > D_{n50}$	Full displacement (rock remains on slope or unit lost from slope)

By quantifying the rock displacement at the end of each test, it can then be defined as percentage damage ($D\%$) sustained after a typical storm. This represents the number of rocks that were displaced from the armour layer (Class 3 displacements) divided by the total number of rocks in that section. The percentage damage is defined as shown in **Equation 1**.

$$\text{Percentage damage: } D\% = \frac{\text{Number of stones displaced over a distance greater than } D_{n50}}{\text{Total number of stones at that section}} \quad (1)$$

The breakwater surface was split into multiple zones, which allows areas of interest to be identified based on the amount of damage observed. Separation lines were mainly drawn to identify the main slope, roundhead, rear slope and toe. The different zones are shown in **Figure 5**.

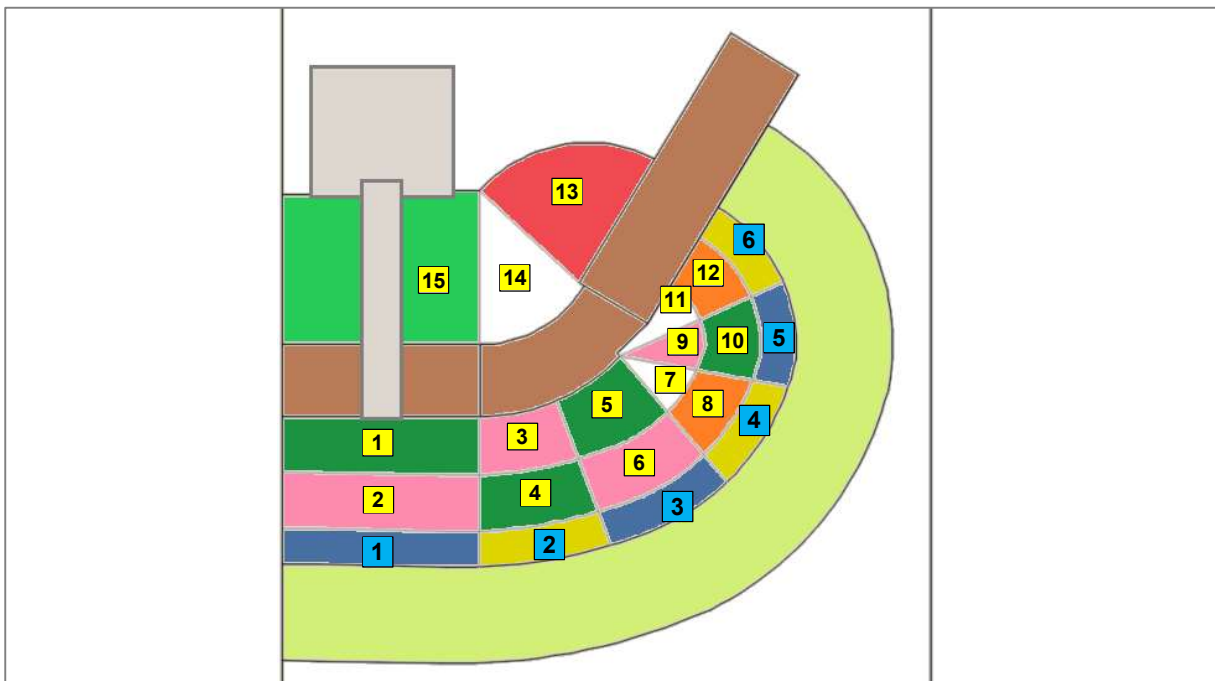


Figure 5: Structure separation zones

All test images are included in **Appendix B**, while the zones for each series defined during the post-test analysis are shown in **Appendix C**.

2.2.8. Wave overtopping

Wave overtopping was measured at the breakwater trunk section at STA_15+30, which is located immediately before the start of the breakwater bend. An overtopping chute, measuring 274 mm in width (5.48 m prototype), was fitted immediately behind the seaward parapet wall to collect wave overtopping.

The overtopping collection bin was placed at the rear of the breakwater and was fitted with a needle gauge to measure the total accumulated overtopping. A single wave probe was also placed inside the overtopping bin to collect data on individual wave overtopping events. The probe data would also allow the measurement and verification of the total overtopping per test. The location of the overtopping measurement setup is shown in **Figure 3**.

The data from the probe that was located inside the overtopping bin were analysed using the O/T-Track software, which was developed by the CSIR. This allowed the identification and quantification of individual overtopping events, as well as to determine its corresponding overtopping volumes. The total overtopping measurements for each test were converted to average overtopping rates in litre per second per metre (l/s/m). This was quantified for the full 134 minute (10-hour prototype) test duration.

3. MODEL TESTING AND RESULTS

On completion of this study, a total of two test series were successfully completed. This comprised verifying the stability of two different armour layer rock sizes for the same rubble mound structure. Each test series included different test conditions to verify the behaviour of the rock material.

3.1. Test Series A

3.1.1. Armour and toe rock stability

This section describes the stability and behaviour of the armour layer and toe at different parts of the model test structure. This included observations for the trunk and trunk transition, the roundhead and the rear slope. The before and after images of all tests are included in **Appendix B**.

Trunk and trunk transition

The trunk and trunk transition made up the largest portion of the seaward slope. This section was completely exposed to the approaching waves, where the trunk was also orientated perpendicular to the direction of wave approach. The design comprised 1-3 t rock for the main armour and the toe, while the underlayer comprised 300-1000 kg rock.

Rock displacements were mainly focussed near the still water level, thus movement on the armour layer varied as the water level was increased or decreased. Testing of the lowest water level of -3.66 m NAVD88, however, resulted in no observable rock displacements and the relative damage remained zero during this test. As can be expected, the greatest number of rock displacements were recorded during the high water 1:100 yr storm condition (Test A5).

The condition of the trunk and trunk transition before and after Test Series A is shown in **Figure 6** and **Figure 7**.

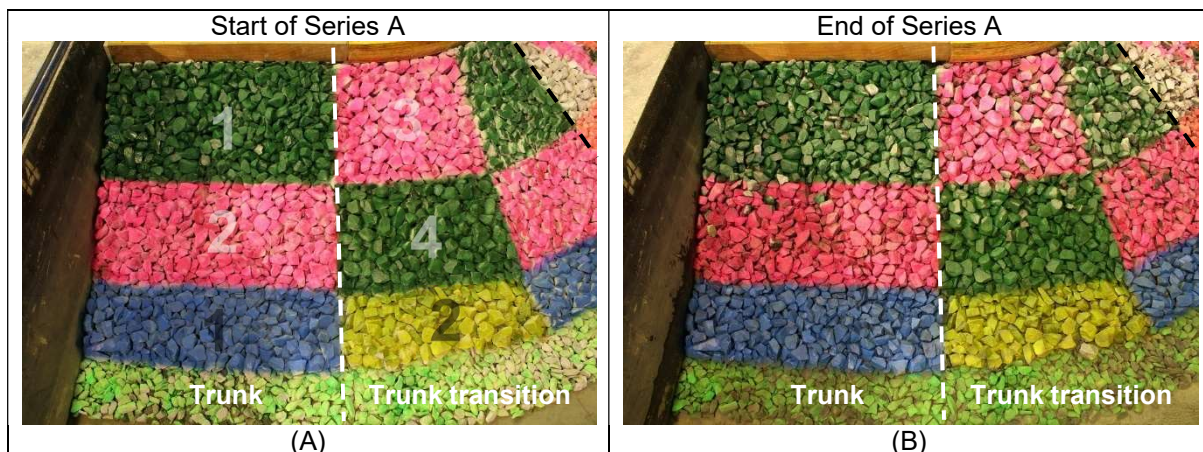


Figure 6: Series A - Trunk and trunk transition

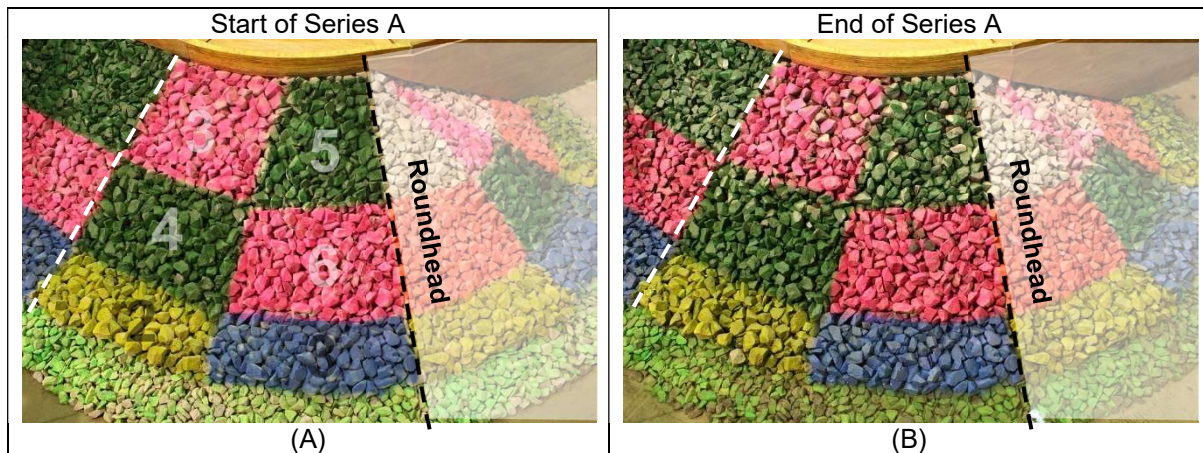


Figure 7: Series A - Trunk transition

By the end of the test series, the greatest number of rock displacements recorded for the armour layer were at Zone 5 (see **Figure 7**), where the cumulative damage was $D\% = 6.25\%$. The toe rock sustained negligible displacements, where the cumulative damage was $D\% = 0.57\%$ for the most critically affected area (Zone 2).

The damage summary for the trunk sections is presented in **Table 4**.

Table 4: Series A – Damage summary for trunk sections

	Zone	Damage ($D\%$) recorded after test						Cumulative
		A1	A2	A3	A4	A4B	A5	
Armour slope	1	0.38%	0.38%	0.76%	0.57%	0%	0.76%	2.85%
	2	0%	0%	0%	0.50%	0%	0%	0.50%
	3	0%	0.38%	0.38%	0%	0%	3.03%	3.79%
	4	0%	0%	0%	0.69%	0%	0%	0.69%
	5	0%	0.96%	0.96%	0.96%	0%	3.37%	6.25%
	6	0%	0%	0%	0.83%	0%	0%	0.83%
Toe	1	0%	0%	0%	0%	0%	0%	0%
	2	0%	0%	0%	0.57%	0%	0%	0.57%
	3	0%	0%	0%	0%	0%	0%	0%

The damage values shown in **Table 4** correspond to that of the Damage Tables for Camera 01 and 02 included in **Appendix D**.

Roundhead

Since the breakwater design included a combi wall with a wave screen that extended from the rubble mound roundhead, only about half of the rock on the roundhead was exposed to direct wave attack. The design comprised a continuation of 1-3 t rock for the roundhead armour layer and toe, while the same underlayer rock was also used as for the trunk sections.

Similar to the trunk sections, rock displacements were mainly focussed near the still water level, however for the roundhead, the greatest number of rock displacements were recorded during the low water 1:100 yr storm condition (Test A4). Test A4B comprising the lowest water level of -3.66 m NAVD88 resulted in no observable rock movements.

The condition of the roundhead before and after Test Series A is shown in **Figure 8**.

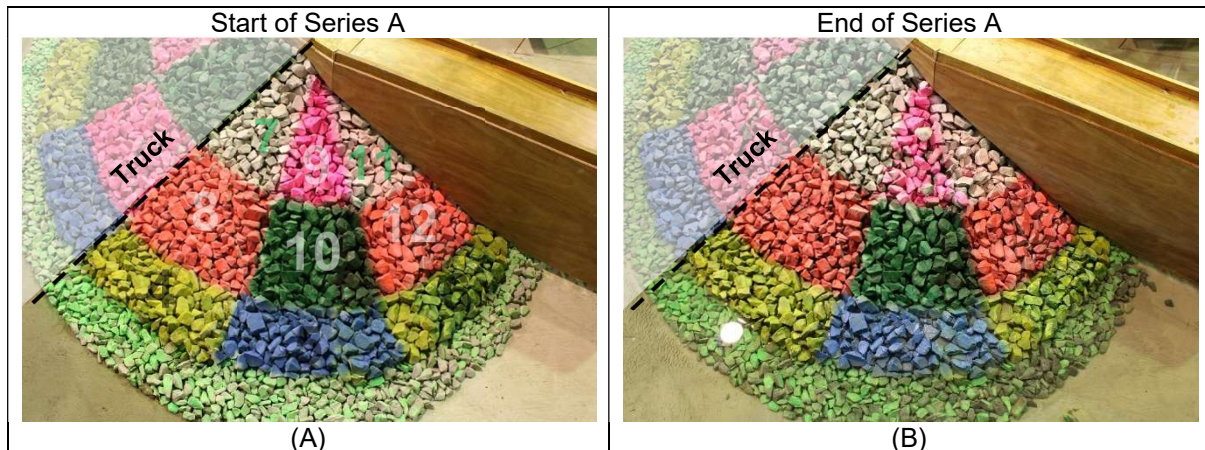


Figure 8: Series A - Roundhead

By the end of the test series, the greatest number of rock displacements recorded for the armour layer were at zones 7 and 8, where the cumulative damage was $D\% = 2.11\%$ and 4.44% respectively. The toe rock sustained no observable movements, thus $D\%$ remained zero.

The damage summary for the roundhead sections is presented in **Table 5**.

Table 5: Series A – Damage summary for roundhead sections

	Zone	Damage ($D\%$) recorded after test						Cumulative
		A1	A2	A3	A4	A4B	A5	
Armour slope	7	0%	0%	0.7%	1.41%	0%	0%	2.11%
	8	0%	0%	0%	0%	0%	0%	0%
	9	0%	0%	0%	4.44%	0%	0%	4.44%
	10	0%	0%	0%	0%	0%	0%	0%
	11	0%	0%	0%	0%	0%	0%	0%
	12	0%	0%	0%	0%	0%	0%	0%
Toe	4	0%	0%	0%	0%	0%	0%	0%
	5	0%	0%	0%	0%	0%	0%	0%
	6	0%	0%	0%	0%	0%	0%	0%

The damage values presented in **Table 5** correspond to that of the Damage Tables for Camera 03 included in **Appendix D**.

Rear slope

The rear sections of the breakwater included part of the roundhead, the trunk and a transition section, and comprised a continuation of the same rock as used on the seaward slopes. Since the combi wall and wave screen protected the rear part of the roundhead, no portion of the rear slope was exposed to direct wave attack.

The rear slopes were well protected and nearly all rocks remained completely still. Some negligible rock movements were observed at the top corner of the rear roundhead and transition, however these small shifts were noticed during the initial, less severe storm conditions (Test A2 and A3) and can thus be regarded as part of the initial settlement of the structure. This suggestion is further supported since

no rock movements were observed at the rear slope during any of the more severe 1:100 yr storm conditions.

The condition of the rear slope before and after Test Series A is shown in **Figure 9**.

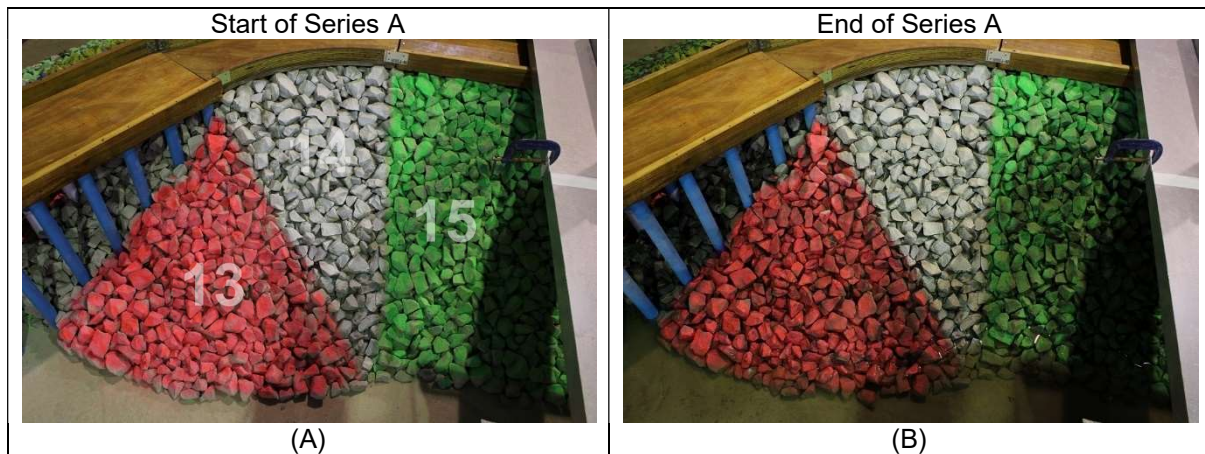


Figure 9: Series A – Rear slope

No significant movements were recorded at the rear slopes, therefore $D\%$ for all areas remained zero. The damage summary for the rear sections is shown in **Table 6**.

Table 6: Series A – Damage summary for rear sections

	Zone	Damage ($D\%$) recorded after test						Cumulative
		A1	A2	A3	A4	A4B	A5	
Slope	13	0%	0%	0%	0%	0%	0%	0%
	14	0%	0%	0%	0%	0%	0%	0%
	15	0%	0%	0%	0%	0%	0%	0%

The damage values shown in **Table 6** correspond to that of the Damage Tables for Camera 04 included in **Appendix D**.

3.1.2. Overtopping measurements

Wave overtopping was measured using the setup as described in **Section 2.2.8**. It is worth noting that during Test Series A, no significant wave overtopping events were observed visually. The wave overtopping collected was mainly as a result of wave splash, which merely caused a constant trickle of water into the overtopping collection bin. This was also confirmed from the data collected by the wave probe inside the bin, since no significant peaks in water level increase could be identified.

A summary of the average overtopping rates recorded during Series A is provided in **Table 7**.

Table 7: Series A - Wave overtopping measurements

Test ID	Water Level	Target H_s	Target T_p	Total overtopping (Litre)	Rate of overtopping (l/s/m)
A01	4.57	0.90	3.69	220	0.001
A02	4.83	1.44	4.10	10 117	0.051
A03	4.99	1.87	4.46	67 292	0.341
A04	2.03	2.07	4.71	174.4	0.001
A04B	-3.66	1.50	4.05	-	0.000
A05	5.05	2.07	4.71	114 616	0.581

No individual wave overtopping events were identified; therefore, no data could be presented on single events.

3.2. Test Series B

3.2.1. Armour and toe rock stability

Subsequent to the completion of Test Series A, the rock size for the breakwater armour and toe rock was reduced from 1-3 t to 600-1900 kg, while the underlayer rock size was reduced from 300-1000 kg to 60-300 kg. The breakwater core material remained unchanged at 5-40 kg.

This section describes the stability and behaviour of the armour and toe rock at different parts of the model test structure. Similar to Test Series A, this mainly included observations for the trunk and trunk transition, the roundhead and the rear slope. The test images are included in **Appendix B**.

Trunk and trunk transition

Similar to Series A, rock displacements were mainly focussed near the still water level. For Series B, however, only high water conditions were tested, thus damage to the armour layer was mainly concentrated around the upper slopes. As can be expected, the greatest number of rock displacements were recorded during the high water overload storm condition (Test B6).

The trunk and trunk transition before and after Test Series B is shown in **Figure 10** and **Figure 11**.

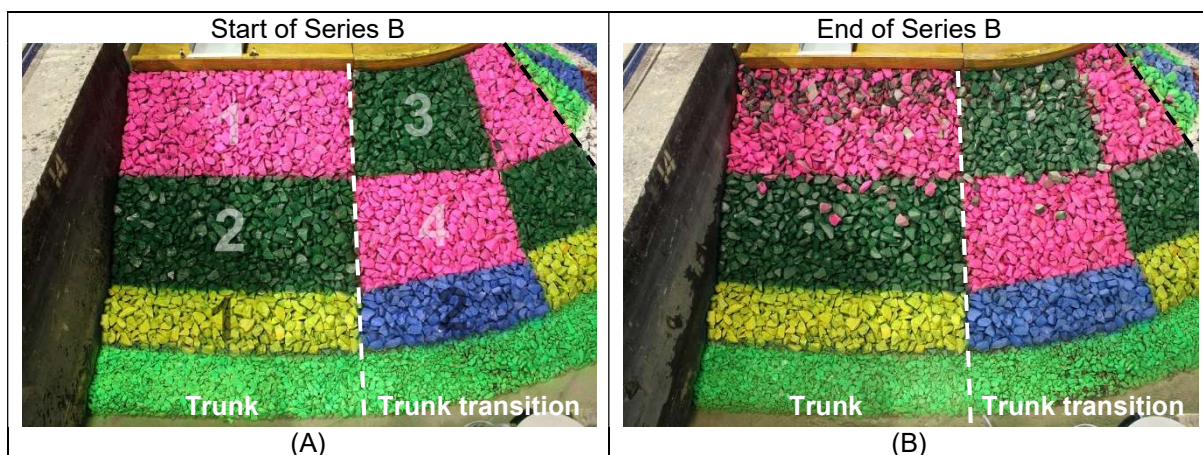


Figure 10: Series B - Trunk and trunk transition

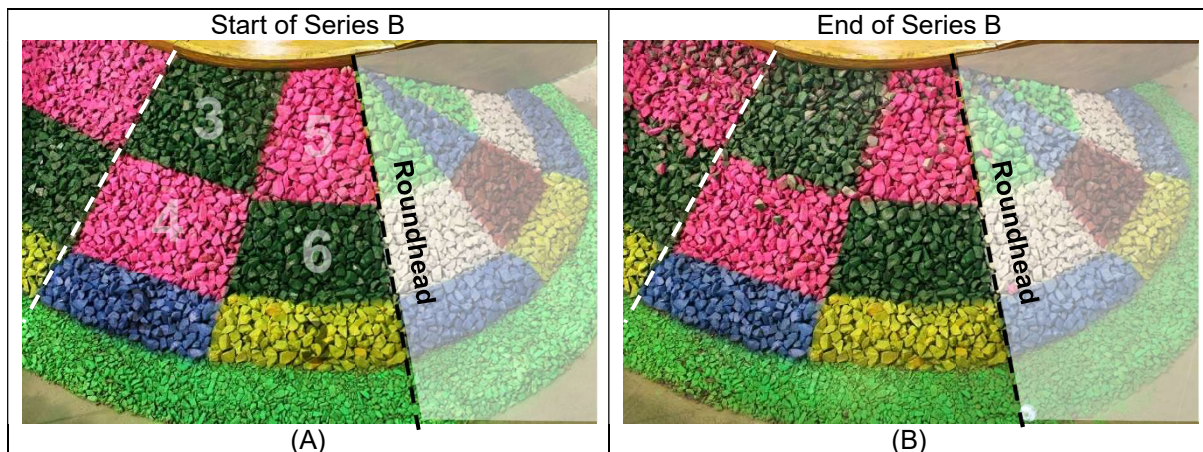


Figure 11: Series B - Trunk transition

The greatest number of armour rock displacements were recorded at Zone 1 (see **Figure 10**) and by the end of the test series, the cumulative damage to this section was $D\% = 10.63\%$. The rock toe had negligible displacements and thus the cumulative damage remained zero for all toe sections.

The damage summary for the trunk sections is presented in **Table 8**.

Table 8: Series B – Damage summary for trunk sections

	Zone	Damage ($D\%$) recorded after test			Cumulative
		B1	B5	B6	
Armour slope	1	0.57%	3.59%	6.47%	10.63%
	2	0%	0%	0.14%	0.14%
	3	0%	0%	5.68%	5.68%
	4	0%	0%	0%	0%
	5	0%	1.45%	2.03%	3.49%
	6	0%	0%	0%	0%
Toe	1	0%	0%	0%	0%
	2	0%	0%	0%	0%
	3	0%	0%	0%	0%

The damage values presented in **Table 8** correspond to that of the Damage Tables for Camera 01 and 02 included in **Appendix D**.

Roundhead

Similar to the trunk sections, the only design changes that were made to the roundhead were the resizing of the rock material. The combi wall design remained unchanged and still provided protection to a large portion of the rubble mound roundhead.

As before, rock displacements were mainly focussed near the still water level, however since only high water conditions were tested, rock displacements were observed mainly on the upper slopes. The greatest number of displacements were recorded during the overload storm condition (Test B6).

The condition of the roundhead before and after Test Series B is shown in **Figure 12**.

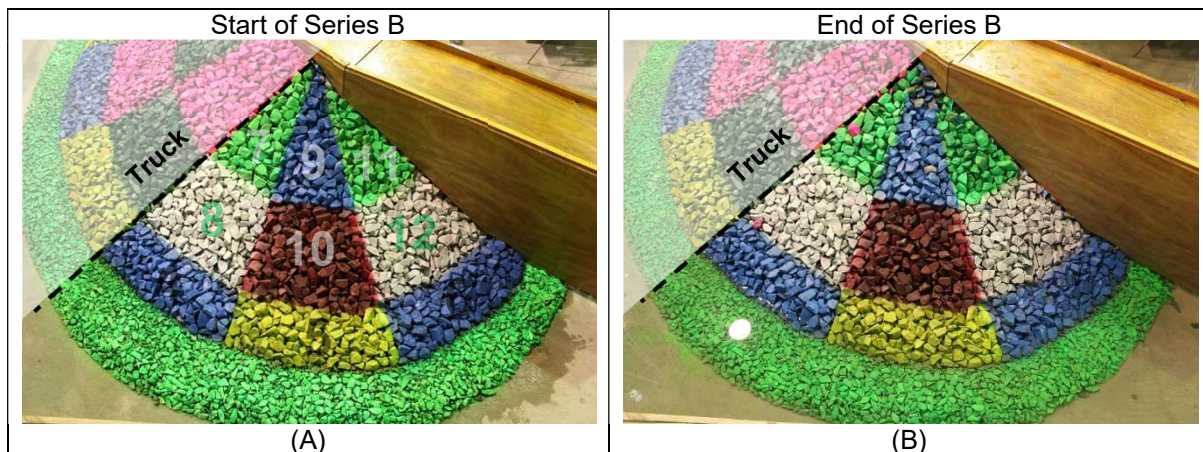


Figure 12: Series B - Roundhead

The greatest number of rock displacements were recorded at Section 11, where the cumulative damage was $D\% = 4.48\%$, closely followed by $D\% = 3.47\%$ for the adjacent section. The rock toe sustained no observable movements, thus $D\%$ remained zero.

The damage summary for the roundhead sections is presented in **Table 9**.

Table 9: Series B – Damage summary for roundhead sections

	Zone	Damage ($D\%$) recorded after test			Cumulative
		B1	B5	B6	
Armour slope	7	0%	0%	0%	0%
	8	0%	0%	0%	0%
	9	0.69%	0.68%	2.08%	3.47%
	10	0%	0%	0%	0%
	11	0%	2.99%	1.49%	4.48%
	12	0%	0%	0%	0%
Toe	4	0%	0%	0%	0%
	5	0%	0%	0%	0%
	6	0%	0%	0%	0%

The damage values presented in **Table 9** correspond to that of the Damage Tables for Camera 03 included in **Appendix D**.

Rear slope

Similar to Test Series A, the rear slopes remained mostly protected from direct wave attack and nearly all rocks remained completely still. Some negligible rock movements were observed at the top of the slope, however these minor movements were too small to be considered as damage. $D\%$ for the rear slopes therefore remained zero for Series B.

The condition of the rear slope before and after Test Series B is shown in **Figure 13**.

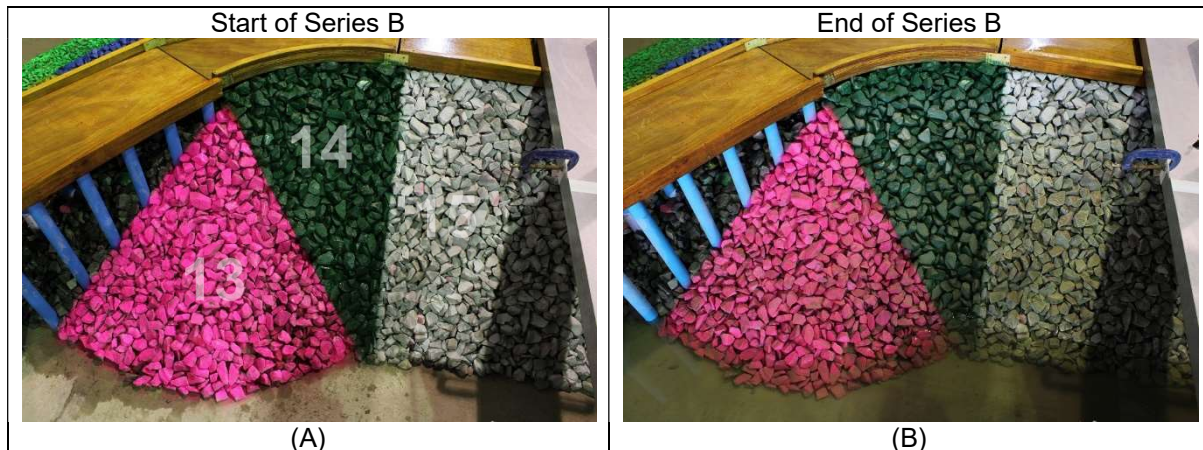


Figure 13: Series B – Rear slope

The lack of wave overtopping also aided the preservation of the rear slopes. The damage summary for the rear sections is presented in **Table 10**.

Table 10: Series B – Damage summary for rear sections

	Zone	Damage (<i>D</i> %) recorded after test			Cumulative
		B1	B5	B6	
Slope	1	0%	0%	0%	0%
	2	0%	0%	0%	0%
	3	0%	0%	0%	0%

The damage values presented in **Table 10** correspond to that of the Damage Tables for Camera 04 included in **Appendix D**.

3.2.2. Overtopping measurements

It is worth noting that, similar to Test Series A, no significant wave overtopping events were observed visually during Test Series B. The wave overtopping was mainly as a result of wave splash, which was also confirmed from the wave probe data since no significant peaks in water level increase could be identified.

A summary of the average overtopping rates recorded during Series B is provided in **Table 11**.

Table 11: Series B - Wave overtopping measurements

Test ID	Water Level	Target H_s	Target T_p	Total overtopping (Litre)	Rate of overtopping (l/s/m)
B01	4.57	0.90	3.69	2 535	0.013
B05	5.05	2.07	4.71	579 508	2.937
B06	5.55	2.28	4.71	1 064 380	5.395

No individual wave overtopping events were identified; therefore, no data could be presented on single events.

4. SUMMARY AND CONCLUSIONS

A 2D physical model study was commissioned at the CSIR to evaluate the performance and behaviour of key components of the new Lubec breakwater design. The objectives of this study included the verification of the rock armour stability, toe stability, assessing the interaction between the rubble mound and the combi wall structures, and quantifying the wave overtopping at the trunk.

Testing of two structures were successfully completed by the end of this study. The stability trends were similar for both structures, which indicated that damage occurred mainly around the still water level and that damage would increase as the wave heights were increased. The overall damage to Structure B were, however greater than that of structure A, where the maximum cumulative damage percentage ($D\%$) recorded were 10.6% and 6.25% respectively. This was somewhat expected since the size of the armour rock for Structure B ($M_{50} = 1.25$ t) was about 40% lighter than that of Structure A ($M_{50} = 2.0$ t). A different combination of wave conditions was however simulated for each structure, which should be taken into consideration in the final engineering assessment.

No adverse interactions between the rubble mound and combi wall structures were observed, since the rock displacements adjacent to the wave screen were of the same magnitude as those recorded at the trunk. Other factors such as the water level, significant wave height and rock size had far greater effect on the overall stability.

The rear slopes on the harbour side of the breakwater remained completely protected from direct wave attack. This included a critical part of the breakwater roundhead, which was located in the lee of the combi wall and wave screen. As a result, the recorded damage to the rear slopes for both structures remained zero. This positive outcome does allow for some potential optimisation of the rear where, for instance, the rock size could be further reduced, or the design could include a steeper rear slope.

The wave overtopping results remained reasonably low for both structures, where the average rate of overtopping for the 1:100 yr high water design condition was 0.58 l/s/m and 2.94 l/s/m for structure A and B respectively. The higher overtopping rates for Structure B was anticipated, since the elevation of the seaward parapet wall was reduced by 2 ft (0.61 m). The highest overtopping rate of 5.4 l/s/m was recorded for Structure B during the 10% overload condition.

APPENDIX A – WAVE RESULTS

APPENDIX B – TEST IMAGES

APPENDIX C – ZONES FOR STABILITY ANALYSIS

APPENDIX D – DAMAGE TABLES

APPENDIX E – MODEL IMPLEMENTATION

APPENDIX F – ROCK GRADING CURVES

APPENDIX G – TYPICAL CROSS-SECTION