Onslow Seawall 2D Physical Model

Armour Rock Stability Testing

Report MHL3083

11 April 2025

Prepared for: Royal HaskoningDHV

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Department of Climate Change, Energy, the Environment and Water

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Foreword

NSW government's professional specialist advisor, Manly Hydraulics Laboratory (MHL) were commissioned by Royal HaskoningDHV to undertake 2D physical modelling of the Onslow seawall (Golden Beach, Sunshine Coast, Queensland) to inform detailed design structure overtopping and crest height. Tests were carried out in MHL's Two- Dimensional (2D), one metre wave flume.

The report was prepared by Megan Liu, Jared Smith and Indra Jayewardene.

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The version of the report is Final.

Executive Summary

This report presents the results of a 2D physical model study assessing the stability and overtopping performance of two proposed revetment cross-sections: Revetment Scenario 1 (short-term) and Revetment Scenario 2 (long-term).

Model testing was conducted at the Manly Hydraulics Laboratory (MHL) 2D wave flume. A total of seven stability assessment tests and nine overtopping measurement tests were performed on a 1:15 scale model.

The key findings from the model tests are as follows:

- Stability assessment – Revetment Scenario 1 (short-term)

  Most tests showed relative stability, with less than 5% damage to the rock armour. However, Test 17 exhibited approximately 8.8% damage, and the stability coefficient (K_D) for this test was 4.5.

- Stability assessment – Revetment Scenario 2 (long-term)

  All tests showed only minor movements, with translations consistently less than the nominal diameter D_{50} of the armour units.

- Overtopping measurement – Revetment Scenario 1 (short-term)

  Overtopping rates generally matched or were below theoretical predictions, with exceptions in Test 1 and Test 17 where actual overtopping exceeded theoretical rates, likely due to wave groupiness affecting the results.

  Test 1 and Test 2 recorded overtopping rates below 15 l/s per m, classified as safe for pedestrians. Test 17 and Test 3 had rates between 10 l/s per m and 50 l/s per m, deemed unsafe for pedestrians but safe for vehicles. Test 6 (Scenario 1 Optimised Test) had overtopping rates below 15 l/s per m, classified as safe for pedestrians.

- Overtopping measurement – Revetment Scenario 2 (long-term)

  All tests recorded overtopping rates lower than theoretical estimates, likely caused by a high frequency of wave breaking near the structure.

  Test 9 and Test 10 reported rates below 5 l/s per m, safe for pedestrians. Test 11 had a rate between 5 l/s per m and 25 l/s per m, and Test 18 had a rate between 1 l/s per m and 5 l/s per m, considered unsafe for pedestrians but safe for vehicles.

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Classification: Private

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1 Introduction

1.1 Background

NSW government's specialist coastal and water engineering advisor, Manly Hydraulics Laboratory (MHL), was commissioned by Royal HaskoningDHV to undertake 2D physical modelling of the Onslow rubble-mound seawall to inform detailed design, particularly regarding structure crest height and wave overtopping. Onslow seawall is located at Australian Navy's TS Onslow cadet facility at Golden Beach, adjoining Pumicestone Passage in the lee of Bribie Island, south-east Queensland.

1.2 Study objectives

This project involved the set up and construction of the proposed bathymetry, rock revetment structure and vertical concrete landscaping wall(s) in the MHL 2D wave flume. The primary focus of the 2D physical modelling was to investigate the stability, wave transmission and overtopping performance of the Revetment Scenario 1(short-term) and Revetment Scenario 2 (long-term) cross-sections. The scope of the 2D physical model involves testing of various water level and wave parameters to determine the stability of the proposed design cross-sections and observe/investigate any damage to the design cross-sections.

Section 2 and **Section 3** sections of this report provide details of MHL's methodology and the results of the 2D modelling.

In general, the scope of the project compromised the following components:

- Design, set up and construction of the 2D model to a nominal scale of 1:15. Further details on model scales, design and layout are provided in **Section 2**.
- Construction of the rock revetment structure in the MHL wave flume for the physical model testing
- Testing short-term (Revetment Scenario 1) and long-term (Revetment Scenario 2) cross-sections
- Assess stability of the cross-sections
- Measurement of overtopping
- Reporting, including progress reports, and provision of model results including photos and videos.

The following meetings and inspections were scheduled during the project:

- Inception meeting (video/phone conference)
- Weekly emails or progress reports to supervise the progress of the model testing.

1.3 Testing schedules

The test series for the 2D modelling is documented in **Table 1.1**. All tests were completed using JONSWAP[1] waves. The stability test durations were for over 93 minutes which is equivalent to a six-hour storm duration in prototype. The overtopping tests to obtain overtopping rates were carried out for 400 seconds, which is equivalent to approximately 26 minutes in prototype.

Table 1.1 Test series for Onslow seawall 2D physical model (provided by Royal HaskoningDHV per comms, 26/07/24)

Scenario	Test No.	Test Condition		
		WL (m AHD)	Hs (m)	Tp (s)
Scenario 1 (Refer to **Figure 2.2**)	1	1.58	0.75	Up to 13s
	17*	1.58	1.00	Up to 13s
	2	2.08	0.75	Up to 13s
	3	2.08	1.00	Up to 13s
	6**	2.38	0.75	Up to 13s
Scenario 2 (Refer to **Figure 2.3**)	9	1.58	1.66	8s to 13s
	10	2.08	1.66	8s to 13s
	18*	2.58	1.66	8s to 13s
	11	2.08	1.91	8s to 13s

*Adjusted tests as per comms with Royal HaskoningDHV

**Scenario 1 Optimised Test 6 as per comms with Royal HaskoningDHV[2]

[1] Various idealized spectra are used to characterise random ocean waves. Pierson and Moskowitz (1964) assumed that if the wind blew steadily for a long time over a large area, the waves would come into equilibrium with the wind (fully developed sea). Hasselmann et al (1973), after analyzing data collected during the Joint North Sea Wave Observation Project JONSWAP, found that the wave spectrum is never fully developed, continuing to develop through non-linear, wave-wave interactions, even for very long times and distances. Hence they multiplied the Pierson-Moskowitz spectrum by an extra peak enhancement factor to improve the fit to their measurements. The JONSWAP spectrum is used extensively by the offshore industry and applied here by MHL.

[2] Scenario 1 optimised Test 6 modelled through adjusted WL/freeboard rather than full model rebuild to replicate WL = 2.08m AHD, Hs = 0.75m with an ‘optimized crest’ at 2.6m AHD and freeboard 0.52m.

2 2D Physical modelling methodology

Tests were conducted in the MHL wave flume (30 m long, 1 m wide and 1.7 m deep) as shown in **Figure 2.1**.

2.1 Scale and armour configuration

2.1.1 Scale ratios

The model was constructed to a length scale of 1:15 (model: prototype). Time scaling was adopted from the length scale using Froudian similitude. The mass scale was obtained using Sharp and Khader's methodology which considers differences between the density of water in the model and prototype (Hughes 1993). Rock scaling and distributions are discussed below.

Length scale (L_r)	Time scale ($\sqrt{L_r}$)	Mass scale
15	3.87	3596

The model scales selected for the study were:

Length scale $L_r = 15$

Time scale $T_r = \sqrt{L_r} = 3.87$

Sharp and Khader's methodology (Hughes 1993) for determining the mass scale (M_r) is given by:

$$M_r = (H_p / H_m)^3 (\rho_p / \rho_m)/(\Delta_p / \Delta_m).$$

Where:

H_p and H_m = Wave height in prototype and model respectively

ρ_p and ρ_m = Density of rock in prototype and model respectively

Δ_p and Δ_m = Relative density of prototype rock in sea water and model rock in fresh water respectively

The rock armour density used in the model was 2700 kg/m^3. It is understood that the rock at the quarry (prototype) to be used for the armour is 2600 kg/m^3 as specified by Royal HaskoningDHV. It should be noted that some differences between the density of armour used in the model and prototype is acceptable. Investigations carried out by Menz (PIANC 2003) indicate that an increase in model armour density from 2700 to 3100 kg/m^3, with about the same gradation and appropriately scaled[1], had little or no influence on the modelling outcomes.

2.1.2 Rock armour grading

Table 2.1 displays the details of the design prototype and model rock armour grading. **Appendix A** demonstrates acceptable compliance of model rock armour distribution

[1] Model rock size reduced when testing denser materials

Table 2.1 Rock armour grading (prototype grading provided by Royal HaskoningDHV per comms, 2/08/24)

Type	Prototype						Model					
	Rock Mass (kg)			Rock Diameter (mm)			Rock Mass (g)			Rock Diameter (mm)		
	Min	Median	Max	Min	Median	Max	Min	Median	Max	Min	Median	Max
Underlayer*	5	12	25	150	200	250	1.4	3.3	7.0	9.9	3.3	17.0
Armour Stage 1*	100	170	330	400	480	600	27.8	47.3	91.8	27.0	32.2	40.2
Armour Stage 2*	1700	3200	5000	1000	1300	1500	472.8	890.0	1390.6	69.4	85.7	99.5

*Armour distribution details are shown in **Appendix A**

2.1.3 Model rock scale effects

Several investigations have provided indicators of the limiting size of models for reduced Reynolds number effects or scale effects (Dai and Kamel 1969, Jensen 1989). More recently, Cornett (1995) determined the condition for negligible scale effect as Hsig being greater than a required wave height as indicated in the following formula:

$$H_{sig} > \frac{(\frac{v * Re}{D_{n50}})^2}{g}$$

Where:

H_{sig} = Significant wave height of smallest design wave (m)

v = Kinematic viscosity = 1.003 x 10^{-6} m^2/s at 20℃

Re = Reynolds number = 2.0 x 10^4, 1.0 – 4.0 x 10^4 is the range of values for Re suggested by Dai and Kamel (1969)

D_{n50} = Nominal diameter of model armour (m)

g = Acceleration due to gravity (9.81 m/s^2)

Scenario 1: $$H_{sig} > \frac{(\frac{v * Re}{D_{n50}})^2}{g} = 0.0395\ m\ (\text{model})$$

Scenario 2: $$H_{sig} > \frac{(\frac{v * Re}{D_{n50}})^2}{g} = 0.0056\ m\ (\text{model})$$

The median mass of armour for Stage 1 tests was 170 kg, which had an equivalent median model diameter of 3.2 cm. All Scenario 1 tests were conducted with a model H_{sig} greater than 4.8 cm or 0.72 m in the prototype. Since the required minimum wave height estimate from Cornett's condition is 3.95 cm for a Re of 2.0 x 10^4, this indicates negligible scale effects for the Scenario 1 testing.

The median mass of armour for Stage 2 used in tests was 3200 kg, which had an equivalent median model diameter of 8.5 cm. All Scenario 2 tests were conducted with a model H_{sig} greater than 11 cm or 1.65 m in the prototype. Since the required wave height estimate from Cornett's condition is 0.56 cm for a Re of 2.0 x 10^4, this also indicates negligible scale effects for the Scenario 2 testing.

2.1.4 Cross sections

Cross-section details of Revetment Scenario 1 and Revetment Scenario 2 were provided by Royal HaskoningDHV (refer to **Appendix B**) and constructed in the wave flume as shown in **Figure 2.2** and **Figure 2.3**.

Figure 2.4 and **Figure 2.5** provide an overview of the setups for Revetment Scenario 1 and Revetment Scenario 2, respectively.

Figure 2.4: Revetment Scenario 1 shows the placement of the wooden structure to support the seawall in the model, followed by securing a layer of geotextile fabric over it. An appropriate underlayer was then added on top of the geotextile fabric, with landscaping cap

walls situated on this layer. Rock revetment armour was placed over the underlayer and positioned against the cap walls as per the testing specification, complemented by two rows of nested Rock Bags to brace the toe of the structure as designed by Royal HaskoningDHV.

Figure 2.5: Revetment Scenario 2 builds upon Revetment Scenario 1 by adding an additional layer of rock revetment armour, termed Armour Stage 2, and a second stage of landscaping cap walls. Three rows of nested Rock Bags are positioned beneath Armour Stage 2 to replicate the positioning alongside the toe of the armour layer, as confirmed by Royal HaskoningDHV.

2.1.5 Model armour placement density

The calculated placement density of the constructed breakwater trunk and crest was obtained from the following placement density equation from Coastal Engineering Manual (CERC, 2006):

$$\frac{N_a}{A} = n * k_\Delta * \left(1 - \frac{P}{100}\right) * \left(\frac{w_a}{W}\right)^{\frac{2}{3}}$$

Where:

N_a = the required number of individual armour units for a given surface area

A = the surface area

n = the number of quarry stone or concrete armour units in the thickness

k_Δ = the layer coefficient

P = cover layer porosity (porosity of 37% recommended for 2 layers of quarrystone (rough))

w_a = the specific weight of the armour unit material

W = the weight of the individual armour units

The armour placement density for the model tests are provided in **Table 2.2**.

Table 2.2 Placement of Stage 1 and Stage 2 armour on test cross-section

n	k_Δ	P	w_a $(\mathrm{kg/m^3})$	W (kg)	A $(\mathrm{m^2})$	N_a*	Rock classification
2	1	37	2700	0.048	0.383	707	Armour stage 1
2	1	40	2700	0.898	0.269	67*	Armour stage 2

*Note: N_a represents the number of rocks used for placement density checking, related to the certain surface area.

2.2 Instrumentation

2.2.1 Capacitance wave probes

Wave data were collected using seven capacitance wave probes (P1 to P7) positioned at the locations shown in **Figure 2.2** and **Figure 2.3**. P1 was closest to the wave generator. P5, P6, and P7 were arranged using the ‘three-probe’ method (Mansard & Funke, 1980) near the structure. P2, P3, and P4 served as a redundant set for the ‘three-probe’ method. Additionally, all probes (P1 to P7) monitored the consistency and trends of the measured data across the array as the waves propagated. The data from the three inshore probes (P5, P6, and P7) were processed to remove reflected waves and extract the incident wave conditions for model testing, with P6 set as the target probe to ensure its incident wave height matched the test conditions. All probes were mounted to ensure accurate measurements, with the probe closest to the structure (P7) placed at least half a wavelength away from the structure to minimize noise in the measurements.

2.2.2 Cameras

All stability (damage) and overtopping tests were documented using two and three sets of GoPro Hero 10 Black video recording cameras recording high resolution videos providing views of the structure from offshore and close to the structure. Front and side view cameras captured videos at 2.7k resolution at 60 frames per second for all tests. 23MP before and after photos were captured for all damage tests. For overtopping tests, an additional back view camera was installed to capture the water overtopping the back of the structure.

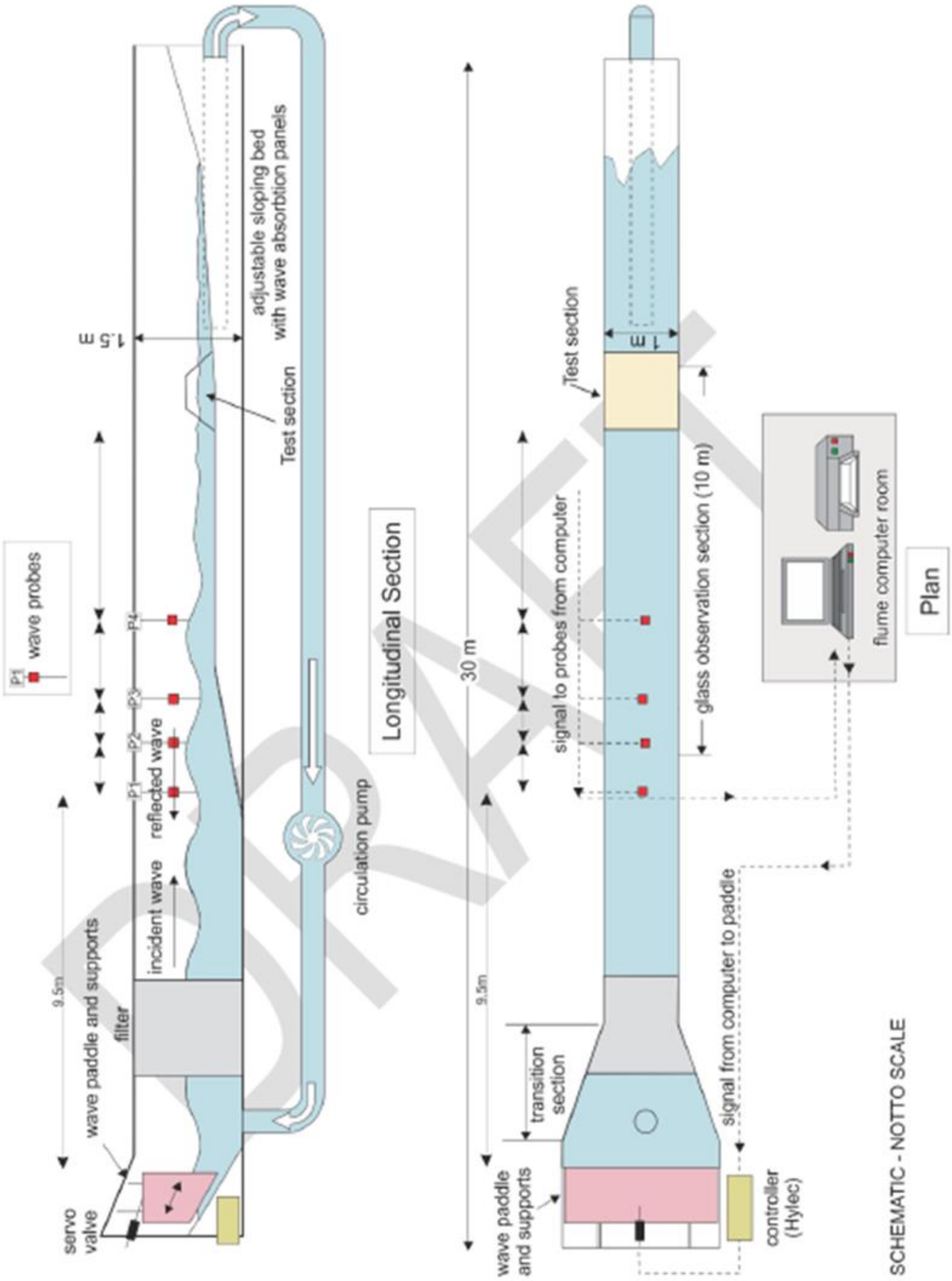


Figure 2.1 MHL wave flume schematic

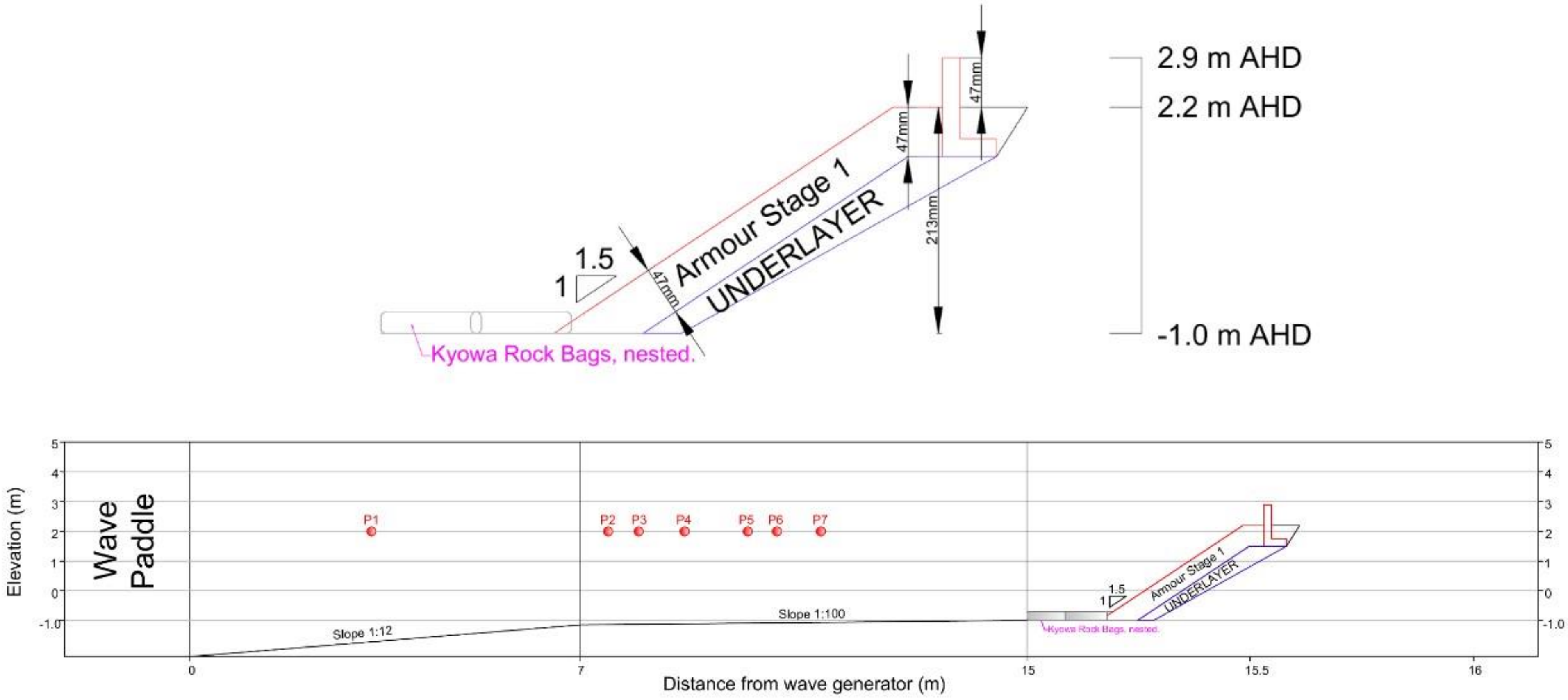


Figure 2.2 Revetment Scenario 1 – Short Term: Structure Layout and Side View of the Model

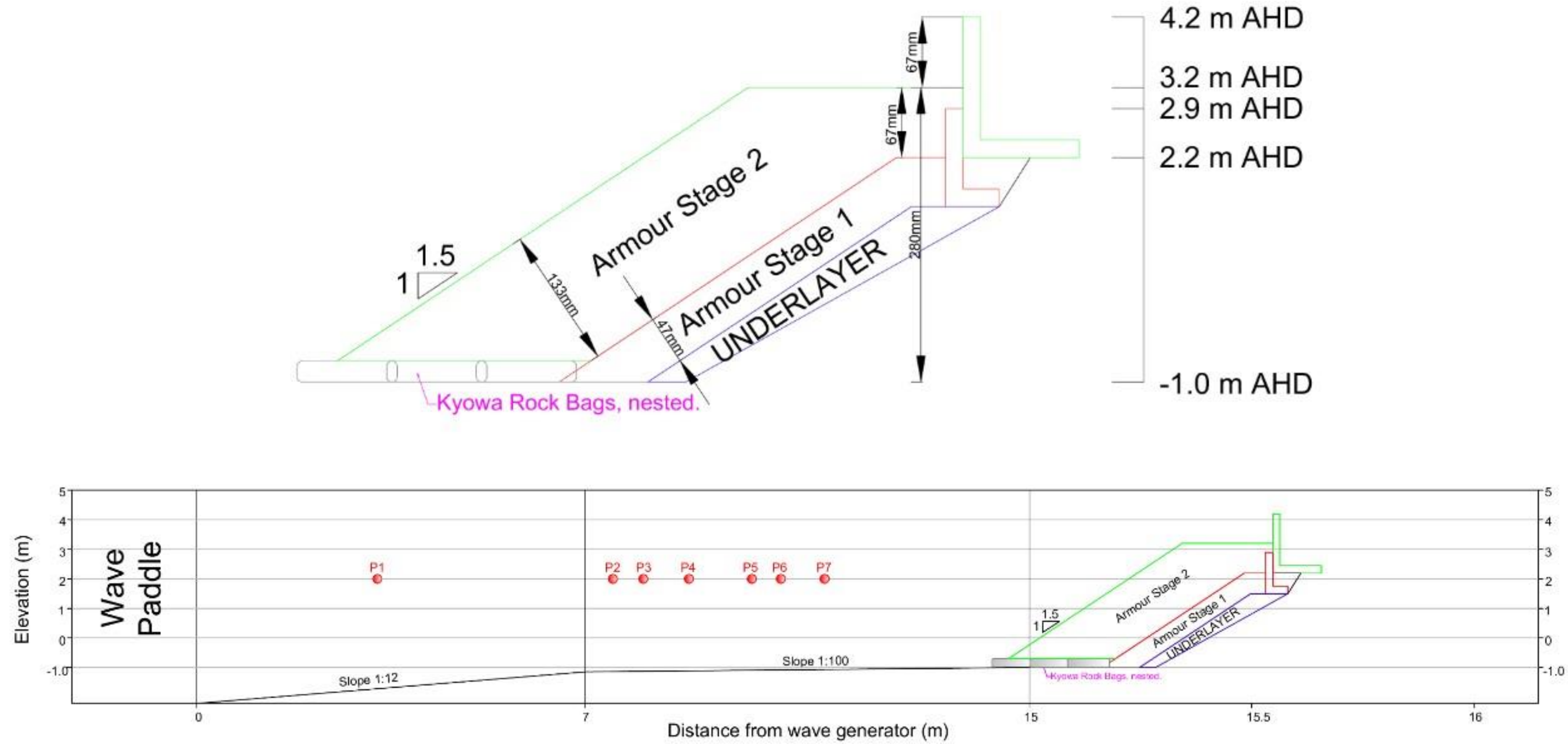


Figure 2.3 Revetment Scenario 2 – Long Term: Structure Layout and Side View of the Model

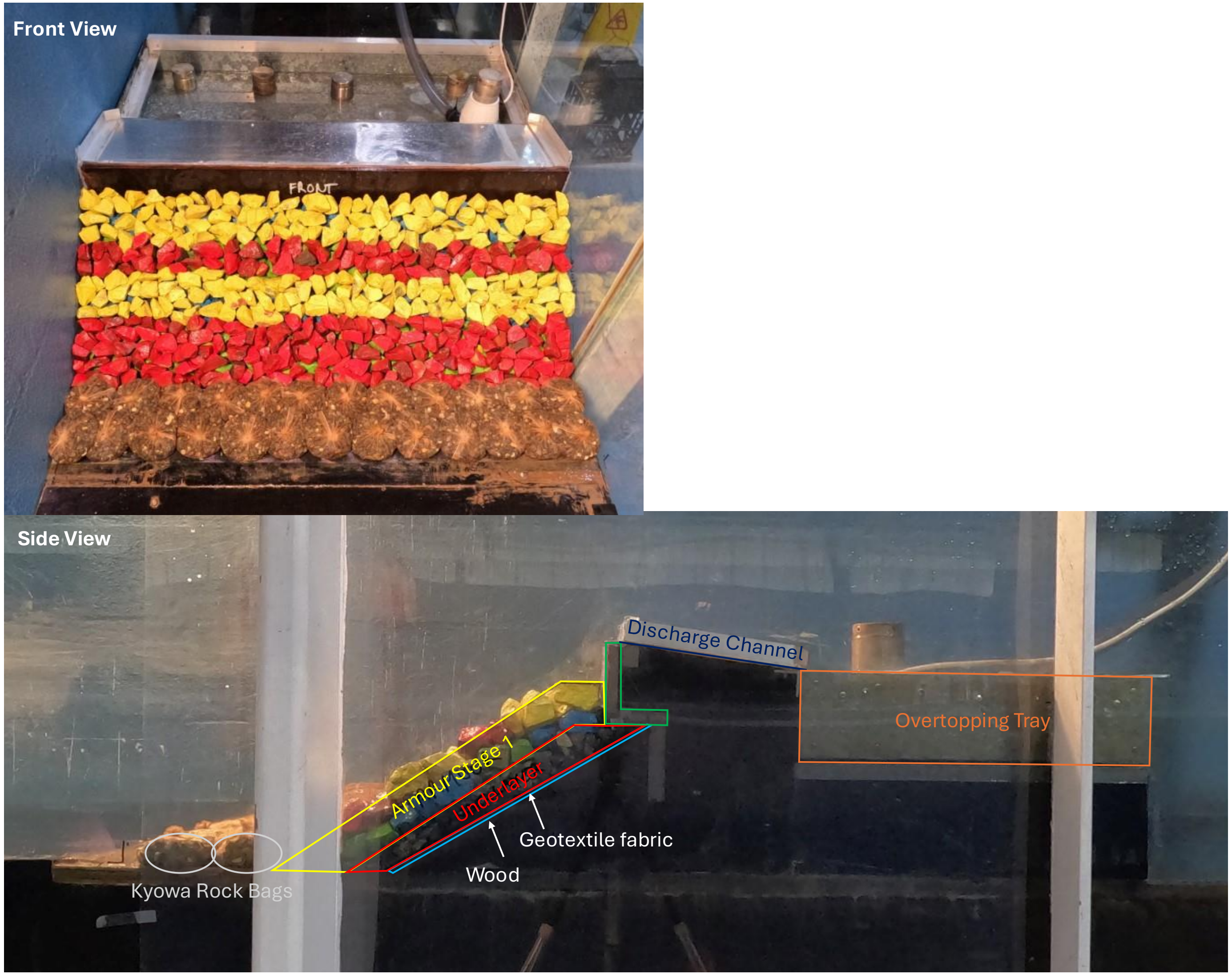


Figure 2.4 Setup Overview for Revetment Scenario 1

Figure 2.5 Setup Overview for Revetment Scenario 2

3 2D model test results

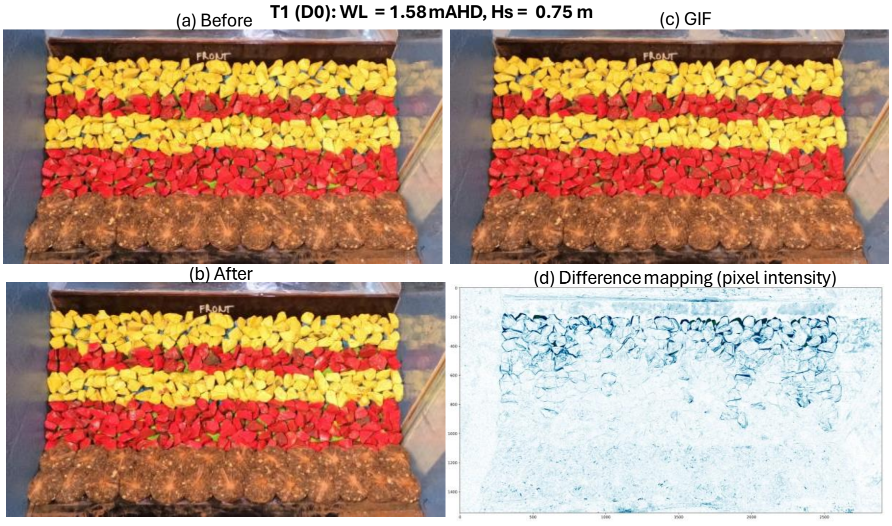
3.1 Stability assessment

The Hudson[1] method for model armour damage assessment was utilised, calculated as:

$$\%\ damage = \frac{no.\ of\ damaged\ units\ in\ the\ tested\ cross\ section}{total\ no.\ of\ units\ exposed\ to\ runup\ and\ rundown\ during\ a\ test} \quad \text{(Eq 3.1)}$$

In accordance with Eq 3.1, armour unit damage of the proposed revetment sections was quantified using the well-recognised method of counting armour units displaced by more than one nominal diameter (D_n). The percentage of damage is the percentage of displaced armour relative to the total number of exposed units within the wave runup/rundown section of the revetment (CERC 2006). This was determined through observation for each structure. All rocks in the top layers were considered exposed.

Rock movement was identified by comparing photographs taken before and after the test from a fixed camera mount. A Python script was developed to process these images and generate both an animated Graphic Interchange Format (GIF) and Difference Mapping (pixel intensity) graphs for each damage assessment test. **Figure 3.1** shows example outputs from the script. The GIF alternated between the "before" and "after" images, providing a dynamic comparison to more easily identify any movement. The images were then processed by applying a convolutional layer to extract key features. The absolute difference between the processed "before" and "after" images was calculated to highlight areas of significant change or movement. Difference Mapping graphs were generated from this difference to visualize areas of rock displacement. The results of this analysis for all stability tests, including the before and after images, GIF, and Difference Mapping graphs for each test, are illustrated in **Appendix C**.



[1] Hudson's methodology was adopted after communication with Royal HaskoningDHV, replacing the use of a damage profiler with a displaced rock count in the testing regime. This was based on Hudson's approach providing a more reliable averaged assessment of damage.

Figure 3.1 Example outputs from the script, including (a) Before image, (b) After image, (c) GIF, and (d) Difference mapping

In general, damage percentages observed during testing are assessed against user defined risk profiles. In the literature, 0% to 5% damage is considered the 'No Damage' condition, or in some cases 'Initial Damage' (CERC 2006). Structural failure is classified as having occurred when the structure core is locally exposed. **Table 3.1** presents damage criteria based on guidance provided in CERC (2006).

Table 3.1 Damage criterion (CERC 2006)

Initial damage	Medium damage	High damage	Failure
0-5%	5-10%	10-15%	>20%, secondary layer exposed

However, the acceptable damage level is dependent on user requirements. It has been identified by Royal HaskoningDHV, that **the damage level considered to represent failure of the rock armour for the TS Onslow seawall installation would be taken as 5% damage**.

The Hudson Stability Equation prescribes a stability coefficient (K_D) which is dependent on the accepted level of damage (CIRIA, 2007):

$$\frac{H_s}{\Delta D_{n50}} = \frac{(K_D cot\alpha)^{1/3}}{1.27} \quad \text{(Eq 3.2)}$$

Where:

K_D = stability coefficient

Δ = relative buoyant density of rock, $\Delta = \rho_r/\rho_w - 1$, ρ_r is mass density of rock, ρ_w is mass density of water

D_{n50} = median nominal diameter, or equivalent cube size, $D_{n50} = (M_{50}/\rho_r)^{1/3}$

α = slope angle

Applied parameters:

$\Delta = 1.7$

$D_{n50} = 480\ mm$ for Armour Satge 1, 1300 mm for Armour Stage 2

$cot\alpha = 1.5$

The results of the model stability assessment are summarised in **Table 3.3**.

For the highest level of damage (T17), the K_D value is 4.5. This exceeds the recommended Hudson damage criteria reported in CERC (2006), indicating that the rock size is below the design armour size according to Eq 3.2.

3.2 Overtopping measurement

Irregular wave runup and overtopping equations, in a manner similar to the stability equations, have evolved over time, with the EurOtop (2018) equations being well established in the literature. The overtopping measurements were made over a model period of 400[1] s using a collection tray width of 1 m.

The following empirical formula (Eq 3.3), for **emergent rubble mound breakwaters** in which the crest of the armour protrudes above the still water level, was utilised to obtain the theoretical estimates of average overtopping rate for **design and assessment purposes** (EurOtop 2018).

$$\frac{q}{\sqrt{g * H_{m0}^3}} = 0.1035 * \exp\left[-\left(1.35 * \frac{R_c}{H_{m0} * \gamma_{f\,mod} * \gamma_\beta}\right)^{1.3}\right] \text{ for steep slopes } 1:2 \text{ to } 1:1.5 \quad \text{(Eq 3.3)}$$

where: q = average overtopping rate/m length of structure

R_c = crest freeboard of structure, from the water level to the top of the rubble (m)

$\gamma_{f\,mod}$ = modified influence factor for roughness (varies from 0.40 to 1.00) - 0.60 was adopted based on EurOtop (2018), max for rubble mound structures with a permeable core given that the breaking parameter > 5, see below.

$\gamma\beta$ = influence factor for oblique wave attack (varies from 0.40 to 1.00) - 1.0 or no obliquity was adopted

α = structure slope (the structural slope of this design is 1:1.5)

H_{mo} = significant incident wave height (m). Hs measured at probe P6 (greater than half a wavelength from the structure) was adopted to estimate theoretical overtopping rates for the revetment

For the purposes of this assessment the influence factor for roughness, γ_f,was taken to be 0.475 (EurOtop 2018). Although ultimately, **the modified influence factor for roughness was adopted**, as the breaking parameter, $\varepsilon_{m-1,0}$ was calculated as being > 5 for all scenarios. From $\varepsilon_{m-1,0} > 5.0$ the roughness factor, $\gamma_{f\,mod}$ increases linearly up to 1 for $\varepsilon_{m-1,0} = 10$, which can be described by the below equation (Eq 3.4), with a maximum of $\gamma_{f\,mod} = 0.60$ for rubble mound structures with a permeable core:

$$\gamma_{f\,mod} = \gamma_f + \left(\varepsilon_{m-1,0} - 5\right) * (1 - \gamma_f)/5.0 \quad \text{(Eq 3.4)}$$

The following empirical formula (Eq 3.5), for **moderate[2] rubble mound breakwaters** under

[1] MHL has found, by experience, that applying a model test duration of 400s is sufficient to ensure that wave variability associated with the random wave test spectrum is generally addressed, but that this duration is not so long as to permit the unwanted development of a seiche in the flume. The 400s is a "sweetspot" which balances these influences to achieve an optimised testing regime.
[2] "Moderate" mounds fall between "small" toe mounds and "emergent" mounds. With crests below the water level like "small" toe mounds, "moderate mounds" however significantly affect wave breaking conditions and therefore overtopping (EurOtop, 2018).

impulsive[1] conditions, was utilised to obtain the theoretical overtopping estimates for Test 6 only (EurOtop 2018).

$$\frac{q}{\sqrt{g*H_{mo}^3}} = 1.3 * \left(\frac{d}{h}\right)^{0.5} * 0.011 * \left(\frac{H_{mo}}{h*s_{m-1,0}}\right)^{0.5} * \exp\left(-2.2 * \frac{R_c}{H_{mo}}\right) \text{ valid for } \frac{R_c}{H_{mo}} < 1.35 \quad \text{(Eq 3.5)}$$

where: q = average overtopping rate/m length of structure

R_c = crest free board of structure, from water level to the top of the vertical wall (m)

d = distance between the still water level and top of the rubble (m)

h = water depth (m)

$s_{m-1,0}$ = wave steepness with L_0

H_{mo} = significant incident wave height at relevant point P6 which is greater than half a wavelength from the structure (m).

It must be noted that all of the theoretical overtopping rates calculated by empirically derived equations can only be regarded as being within, at best, a factor of 1 – 3 of the actual overtopping rate (Van der Meer, et al., 2018).

Guidance on acceptable overtopping rates is provided in EurOtop (2018).

Table 3.2 summarizes the recommended limits for overtopping rates related to pedestrian safety, vehicle safety, damage to parkland, and damage to pavements, categorized by H_s.

Table 3.2 Overtopping rate limits (EurOtop 2018)

H_s (m)	Pedestrian Safety (l/s per m)	Vehicle Safety (l/s per m)	Damage to Parkland (l/s per m)	Damage to Pavements (l/s per m)
3	0.3	<5	5	200
2	1	10 - 20	5	-
1	10 - 20	<75	5 - 10	-
<0.5	No limit	No Limit	No Limit	No Limit

As shown in Table 3.2, overtopping impacts depend on multiple factors including wave height, the type of foreshore and how it is used. Tolerable overtopping increases as wave heights reduce, related to the trajectory of the overtopped jet. Horizontal flows associated with lower wave heights are assessed to be less hazardous even though overtopping flow rates may be higher.

Royal HaskoningDHV has proposed adopting specific overtopping limits for breakwater design, focusing on pedestrian and vehicle safety. For pedestrian safety, the following

[1] “Impulsive” waves tend to break onto the seawall. Under “impulsive” conditions, up-rushing water can significantly overtop even high structures.

thresholds are applied:

- For the H_s = 0.75m overtopping should be limited to 15 l/s per m
- For the H_s = 1m overtopping should be limited to 10 l/s per m
- For the H_s = 1.66m overtopping should be limited to 5 l/s per m
- For the H_s = 1.91m overtopping should be limited to 1 l/s per m

For vehicle safety, thresholds are set to 5 times the pedestrian limits:

- For the H_s = 0.75m overtopping should be limited to 75 l/s per m
- For the H_s = 1m overtopping should be limited to 50 l/s per m
- For the H_s = 1.66m overtopping should be limited to 25 l/s per m
- For the H_s = 1.91m overtopping should be limited to 5 l/s per m

Table 3.4 presents the measured wave overtopping rates from the 2D flume tests, with results categorized by the following safety thresholds:

- Green: Safe for pedestrians (below the pedestrian safety limit)
- Orange: Exceeds pedestrian safety limit but remains safe for vehicles (above the pedestrian safety limit and below the vehicle safety limit)
- Red: Unsafe for vehicles (exceeds the vehicle safety limit, set at 5 times the pedestrian limit)

Theoretical overtopping rates were generally calculated using Eq 3.3, with the modified influence factor determined by Eq 3.4.

The measured wave overtopping rates were less than or close to the theoretical values for most tests, which may be due to the wave breaking in front of the structure. Also, since a single wave can result in overtopping rates that are 100 times greater than the average (Van der Meer 1994) the discrepancy may be attributed to the lack of high waves reaching the structure due to wave breaking. In Test 1, Test 17 and Test 6, where the measured overtopping exceeded theoretical expectations, the variance could be due to wave groupiness[1].

[1] Wave groupiness refers to the phenomenon where waves travel in groups or sets, rather than as individual waves. This principally occurs due to the superposition of waves with slightly different wavelengths and speeds, creating a pattern where waves of higher amplitude are followed by waves of lower amplitude, which can lead to variable overtopping. Non-linear interactions between waves within a group can also lead to complex, nonlinear effects that can enhance the overtopping process. While wave groupiness would be less influential in the flume for a longer test duration, consideration of modelling effort and laboratory experience are used to achieve a cost-effective and acceptable testing outcome.

Table 3.3 Summary of stability assessment results

Scenario	Test No.	Test Condition: WL (m AHD)	Test Condition: Hs (m)	Test Condition: Tp (s)	Measured wave heights and periods, Target Probe (P6)[1]: Measured Wave Height Hs (m)	Reflection Coefficient	Incident Wave Height Hi (m)	Peak Spectral Wave Period Tp1 (s)	Peak Spectral Wave Period Tp2[2] (s)	Percent Damage (Percentage of rocks with more than D50 movement above minimum rundown level)	K_D
Scenario 1	1	1.58	0.75	Up to 13s	0.86	0.57	0.75	8.83	13.68	0.3%	1.9
	17	1.58	1.00	Up to 13s	1.18	0.59	1.01	8.83	13.68	8.8%	4.5
	2	2.08	0.75	Up to 13s	0.86	0.62	0.73	13.05	9.15	0.0%	1.7
	3	2.08	1.00	Up to 13s	1.15	0.63	0.98	13.05	9.15	2.4%	4.2
	6*	2.38	0.75	Up to 13s	NA	NA	NA	NA	NA	NA	1.9
Scenario 2	9	1.58	1.66	8s to 10s	1.84	0.49	1.65	8.75	-	0.0%	1.1
	10	2.08	1.66	8s to 10s	1.83	0.47	1.66	8.49	-	0.0%	1.1
	18	2.58	1.66	8s to 10s	NA	NA	NA	NA	NA	NA	1.1
	11	2.08	1.91	8s to 10s	2.13	0.49	1.91	8.49	-	0.0%	1.6

*Scenario 1 Optimised Test 6 as per comms with Royal HaskoningDHV

Table 3.4 Summary of overtopping test results

Scenario	Test No.	Test Condition: Crest Level (m AHD)	Test Condition: WL (m AHD)	Test Condition: Hs (m)	Test Condition: Tp (s)	Measured wave heights and periods, Target Probe (P6)[1]: Measured Wave Height Hs (m)	Reflection Coefficient	Incident Wave Height Hi (m)	Peak Spectral Wave Period Tp1 (s)	Peak Spectral Wave Period Tp2[2] (s)	Relative Freeboard Rc/Hm0	Theoretical Overtopping Prototype (l/s per m)	Measured Overtopping Prototype (l/s per m)
Scenario 1	1	2.9	1.58	0.75	Up to 13s	0.89	0.58	0.77	9.33	13.08	1.71	0.7	1.3
	17	2.9	1.58	1.00	Up to 13s	1.23	0.60	1.05	9.33	13.08	1.26	7.3	10.8
	2	2.9	2.08	0.75	Up to 13s	0.86	0.63	0.73	9.13	12.95	1.12	7.2	5.2
	3	2.9	2.08	1.00	Up to 13s	1.19	0.62	1.01	9.13	12.95	0.81	36.9	22.6
	6*	2.9	2.38	0.75	Up to 13s	0.88	0.65	0.74	9.06	12.95	0.69	8.6	14.8
Scenario 2	9	4.2	1.58	1.66	8s to 10s	1.87	0.50	1.67	8.57	-	1.57	1.6	0.3
	10	4.2	2.08	1.66	8s to 10s	1.83	0.48	1.65	8.46	-	1.28	6.2	1.2
	18	4.2	2.58	1.66	8s to 10s	1.84	0.47	1.67	8.87	-	0.97	32.0	21.3
	11	4.2	2.08	1.91	8s to 10s	2.13	0.50	1.91	8.52	-	1.11	11.8	4.6

*Scenario 1 Optimised Test 6 as per comms with Royal HaskoningDHV

[1] MHL used P5, P6, and P7 for the three-probe analysis to calculate the reflection coefficient. P6 served as the target probe for removing reflected waves, allowing the extraction of the incident wave and ensuring that the designated wave test conditions were met.
[2] Bimodal wave energy observed during certain tests, attributed to high wave period limitations.

4 Summary and conclusions

Seven physical model tests of rock armour stability and eight tests of wave overtopping were carried out in the wave flume at MHL. The rock armour stability tests considered rock movements above the minimum wave run down level. Overtopping volumes were collected immediately landward of the landscaping cap walls.

4.1 Armour stability tests

Revetment Scenario 1 generally showed relative stability with less than 5% damage in most tests. However, in Test 17, which was conducted at a lower water level (WL 1.58 m AHD), with a significant wave height (Hs) of 1.01 m and peak period (Tp) of 8.83 s, there was approximately 8.8% rock armour damage. This higher level of damage is attributed to reduced submergence of the structure. This test run also equates to a K_D of 4.5 (Eq 3.2), indicating that the rock size is below the design armour size according to Hudson's equation. Revetment Scenario 2 showed overall minor movements, with translations always less than D_{50}.

4.2 Overtopping tests

In Revetment Scenario 1, overtopping results were generally close to or less than theoretical estimates, except for Test 1 and Test 17. In these tests, the measured overtopping exceeded the theoretical expectations, a variance that could be attributed to wave groupiness.

In Revetment Scenario 2, all tests indicated that the measured overtopping rates were lower than the theoretical estimates. The discrepancy between theoretical and measured values can likely be attributed to a high percentage of wave breaking near the structure, a complex process which is not captured in the applied wave theory.

Interpretation of overtopping test results based on the EurOtop (2018) criteria (refer to **Section 3.2 Table 3.2**) is set out below:

Scenario 1:

- Test 1 (WL: 1.58 m AHD; H_s: 0.75 m):1.3 l/s per m – Below the pedestrian limit of 15 l/s per m (safe for pedestrians)
- Test 17 (WL: 1.58 m AHD; H_s: 1.00 m): 10.8 l/s per m – Exceeds the pedestrian limit of 10 l/s per m but remains below the vehicle safety threshold of 50 l/s per m (unsafe for pedestrians, safe for vehicles)
- Test 2 (WL: 2.08 m AHD; H_s: 0.75 m): 5.2 l/s per m – Below 15 l/s per m (safe for pedestrians)
- Test 3 (WL: 2.08 m AHD; H_s: 1.00 m): 22.6 l/s per m – Exceeds 10 l/s per m but below 50 l/s per m (unsafe for pedestrians, safe for vehicles)
- Test 6 (Scenario 1 Optimised Test; WL: 2.38 m AHD; H_s: 0.75 m): 14.8 l/s per m - Below the pedestrian limit of 15 l/s per m (safe for pedestrians)

Scenario 2:

- Test 9 (WL: 1.58 m AHD; H_s: 1.66 m): 0.3 l/s per m – Below the pedestrian limit of 5 l/s per m (safe for pedestrians)

- Test 10 (WL: 2.08 m AHD; H_s: 1.66 m): 1.2 l/s per m – Below 5 l/s per m (safe for pedestrians)
- Test 18 (WL: 2.58 m AHD; H_s: 1.66 m): 21.3 l/s per m – Exceeds the pedestrian limit of 5 l/s per m but remains below the vehicle safety threshold of 25 l/s per m (unsafe for pedestrians, safe for vehicles)
- Test 11 (WL: 2.08 m AHD; H_s: 1.91 m): 4.6 l/s per m – Exceeds the pedestrian limit of 1 l/s per m but remains below the vehicle safety threshold of 5 l/s per m (unsafe for pedestrians, safe for vehicles)

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Appendix A Model rock armour distributions

Figure A1 Underlayer: 5 to 25 kg gravel distribution (Scale 1:15)

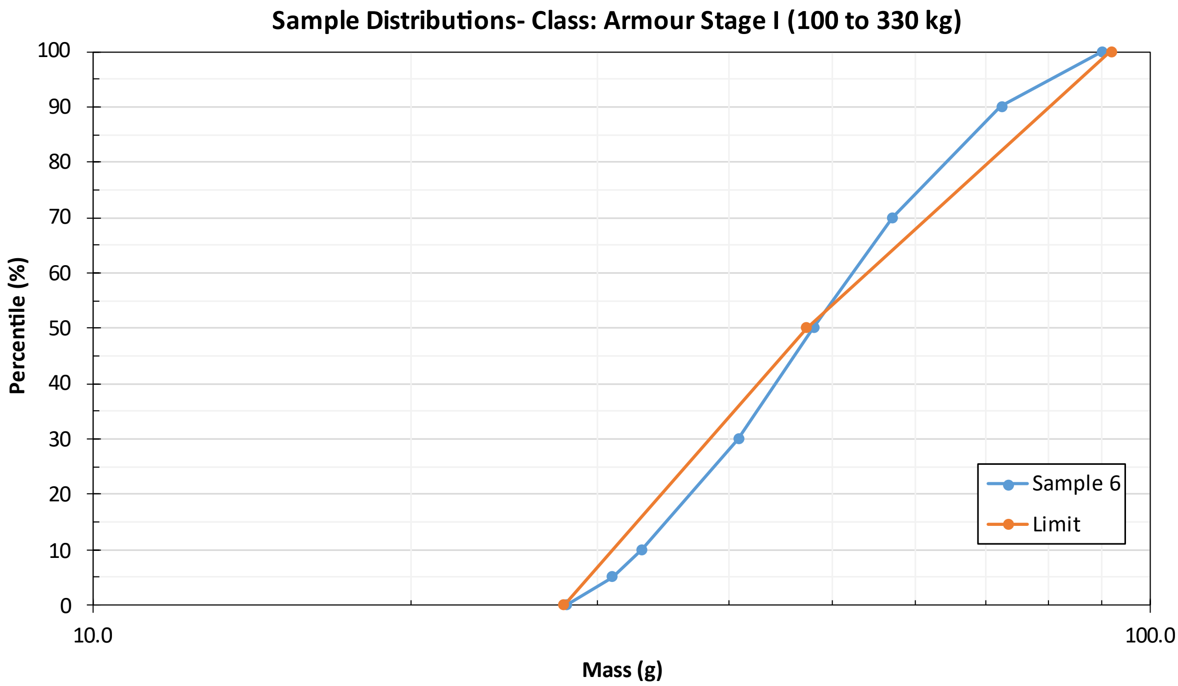


Figure A2 Armour Stage 1: 100 to 330 kg rock distribution (Scale 1:15)

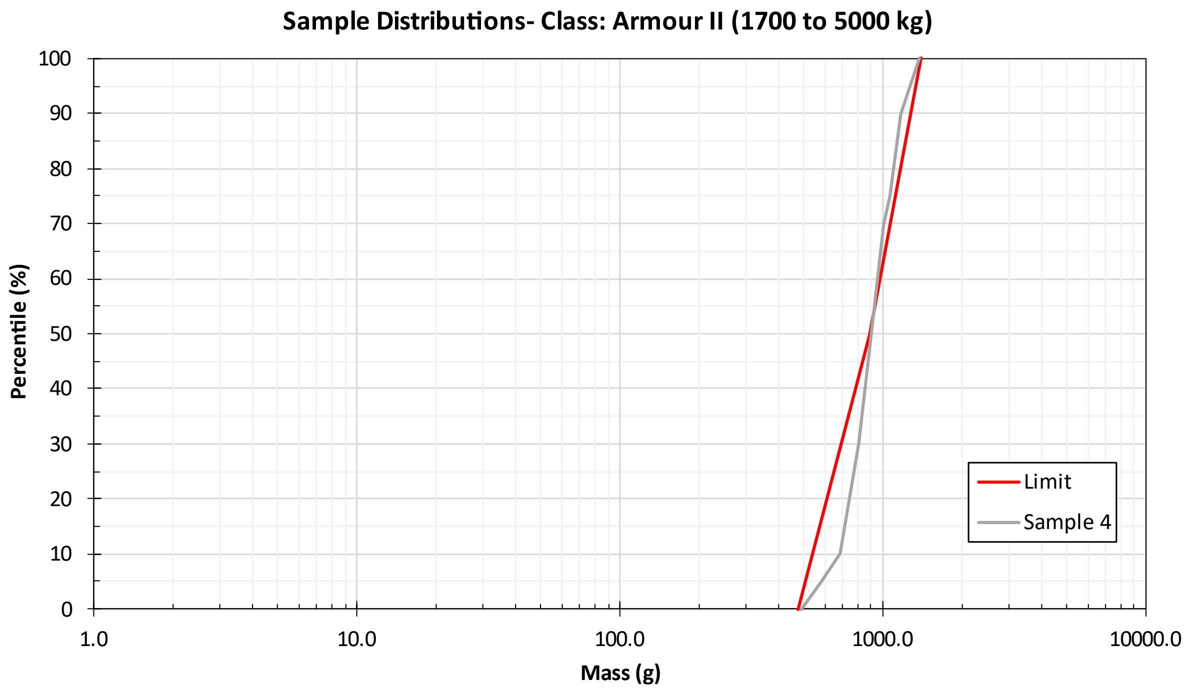
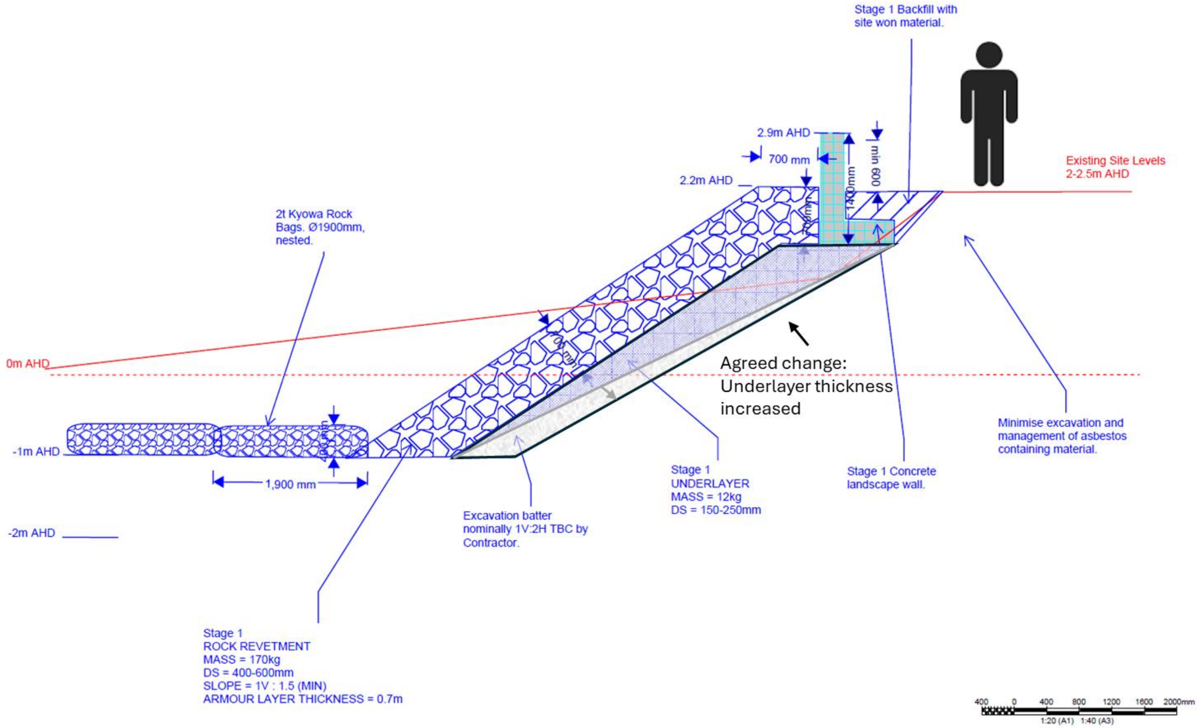


Figure A3 Armour Stage 2: 1700 to 5000 kg rock distribution (Scale 1:15)

Appendix B Revetment design cross sections

B.1 Scenario 1 – short term cross section[1]

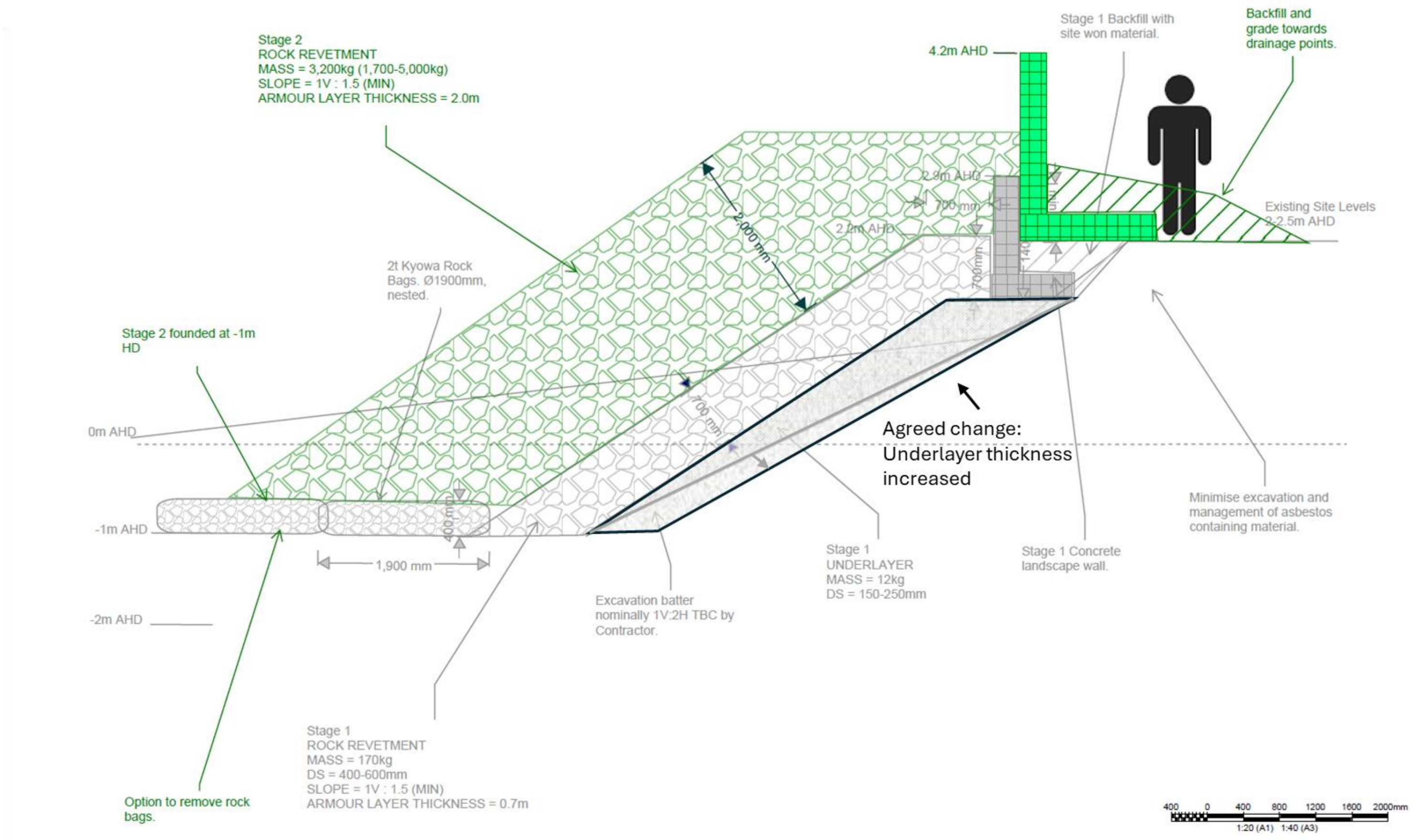
Section C – Revetment
Scenario 1 – Short Term
(Royal HaskoningDHV per comms, 2/08/24))



[1]: Note that the model design included the Rock Bags scaled at 1:20 based on what was available in the laboratory. For this reason an extra layer of Rock Bags was used to replicate their positioning alongside the toe of the armour layer.

B.2 Scenario 2 – long term cross section

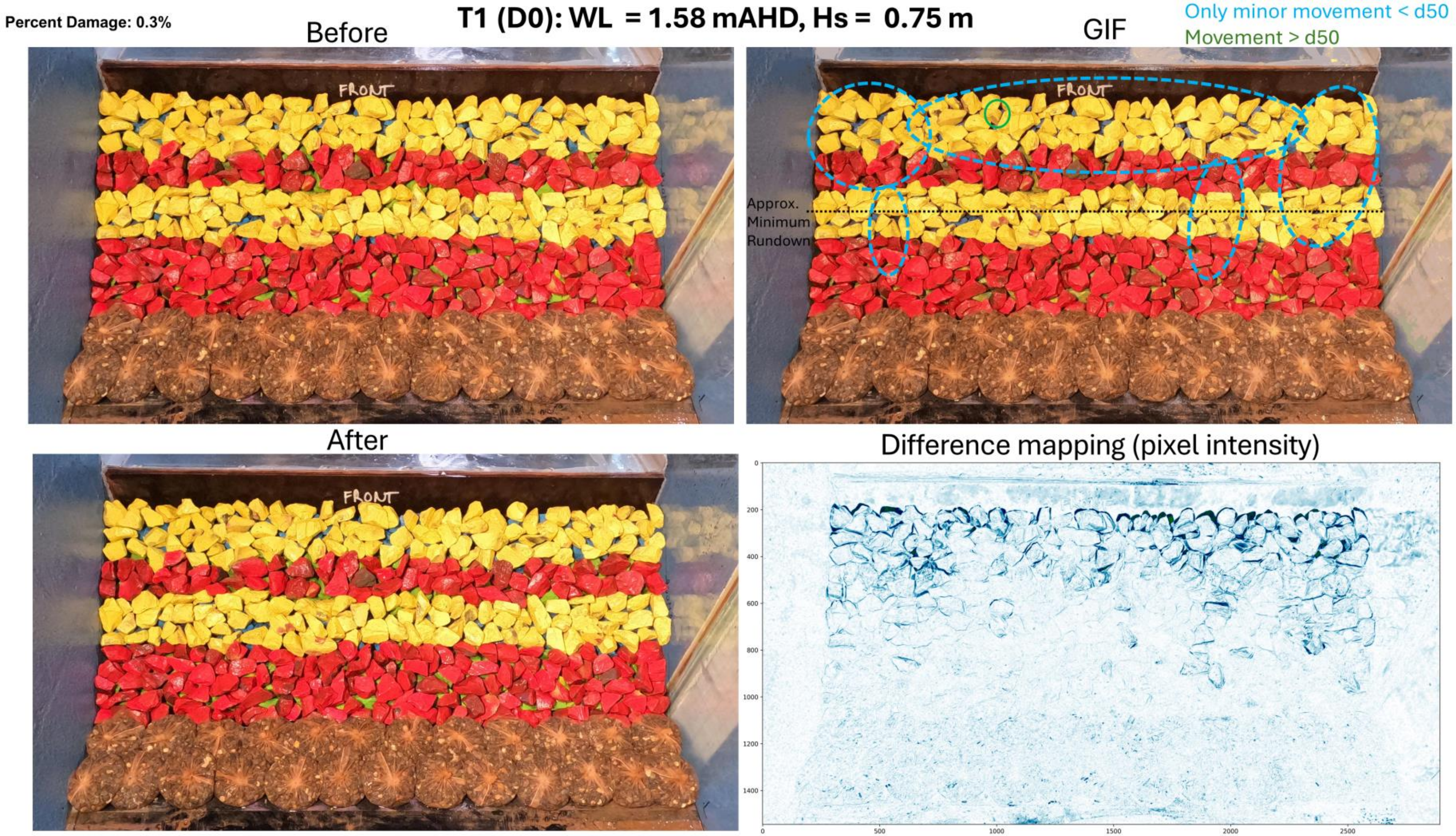
Section C – Revetment
Scenario 2 – Long Term
(Royal HaskoningDHV per comms, 2/08/24))



Classification: Private

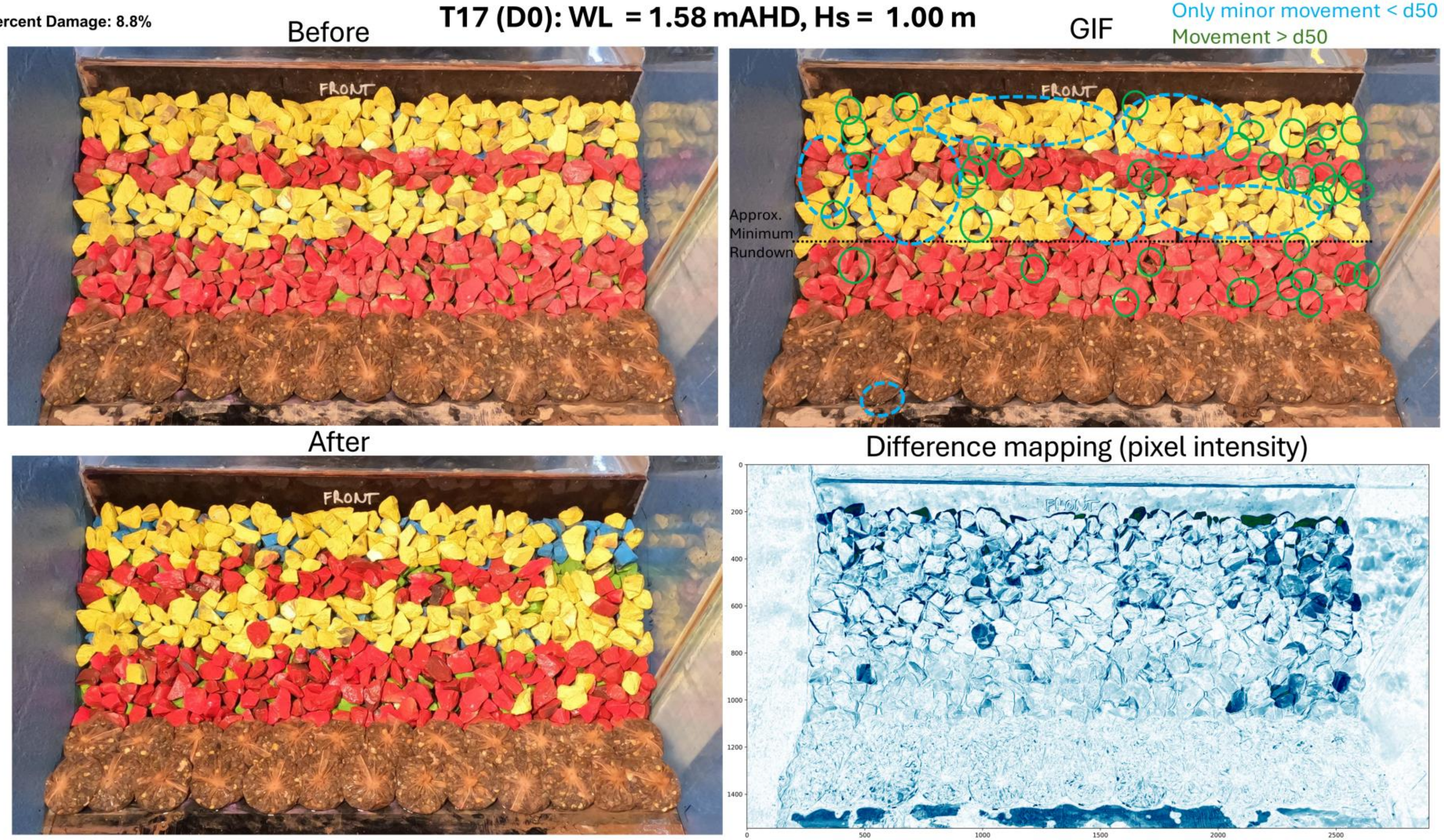
Appendix C Armour stability assessment

C.1 Scenario 1 armour stability assessment



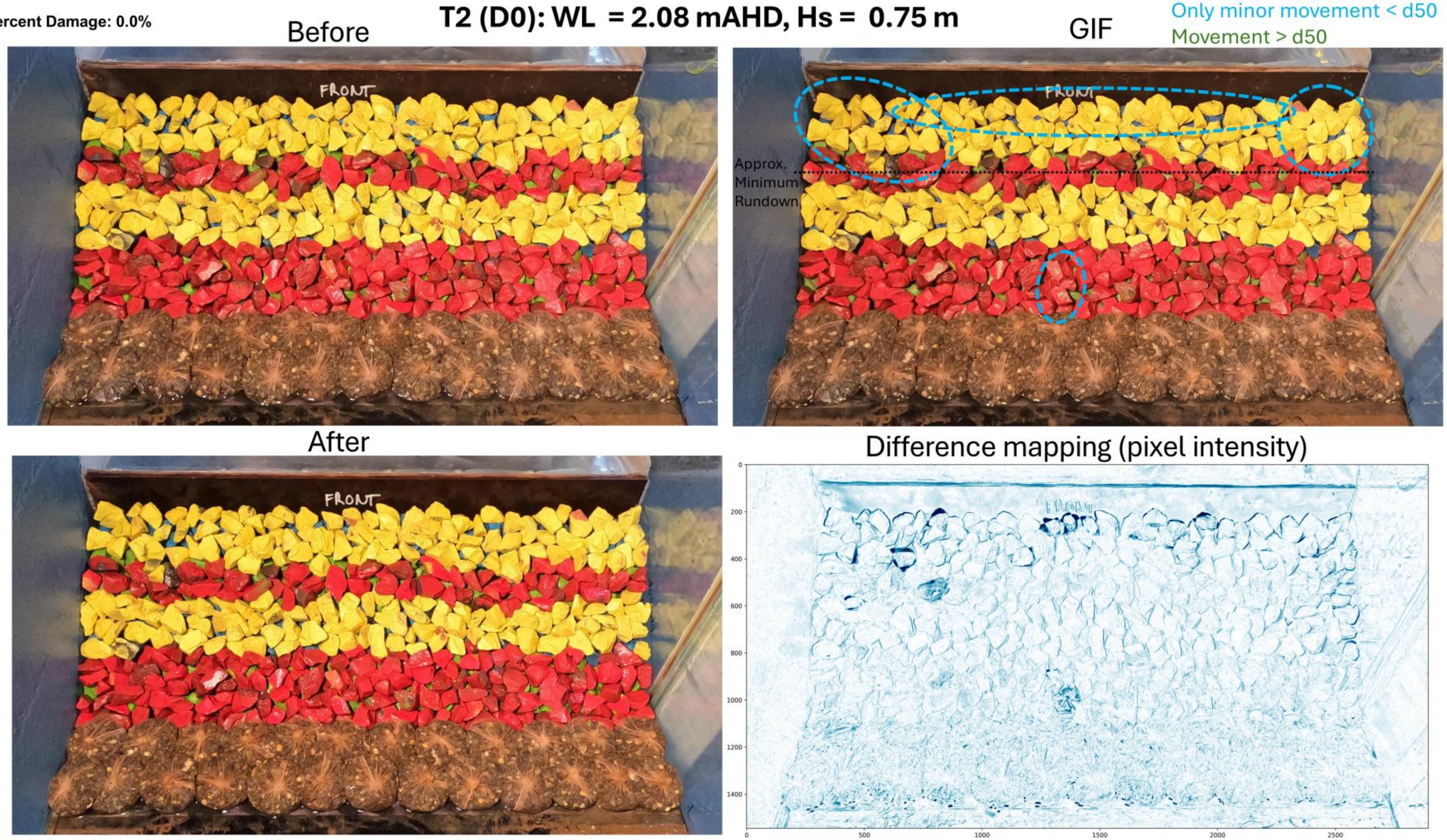
Percent Damage: 8.8%

T17 (D0): WL = 1.58 mAHD, Hs = 1.00 m



Percent Damage: 0.0%

T2 (D0): WL = 2.08 mAHD, Hs = 0.75 m



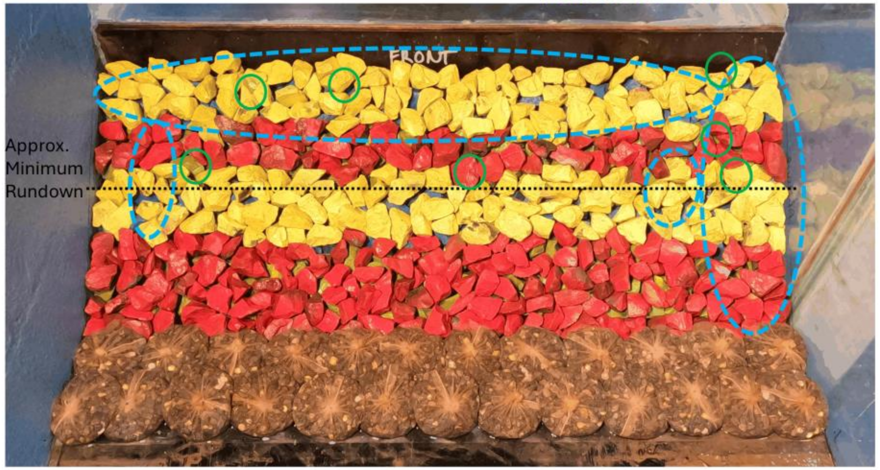
Percent Damage: 2.4%

T3 (D0): WL = 2.08 mAHD, Hs = 1.00 m

Before

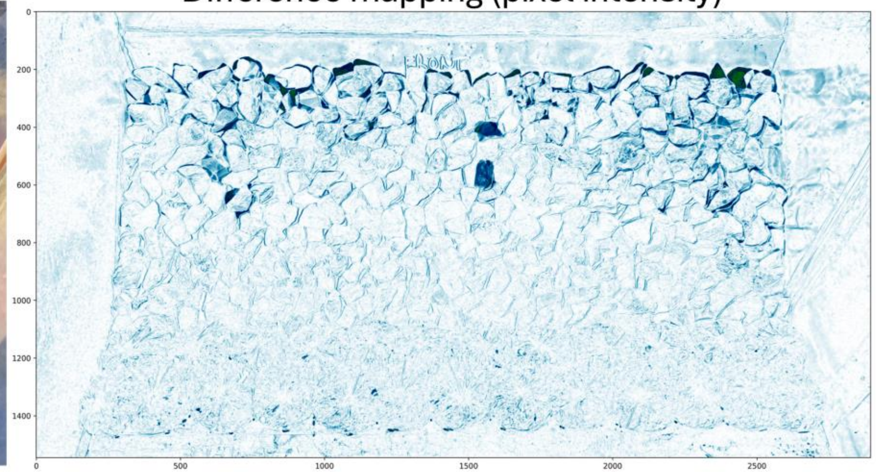
GIF

Only minor movement < d50
Movement > d50



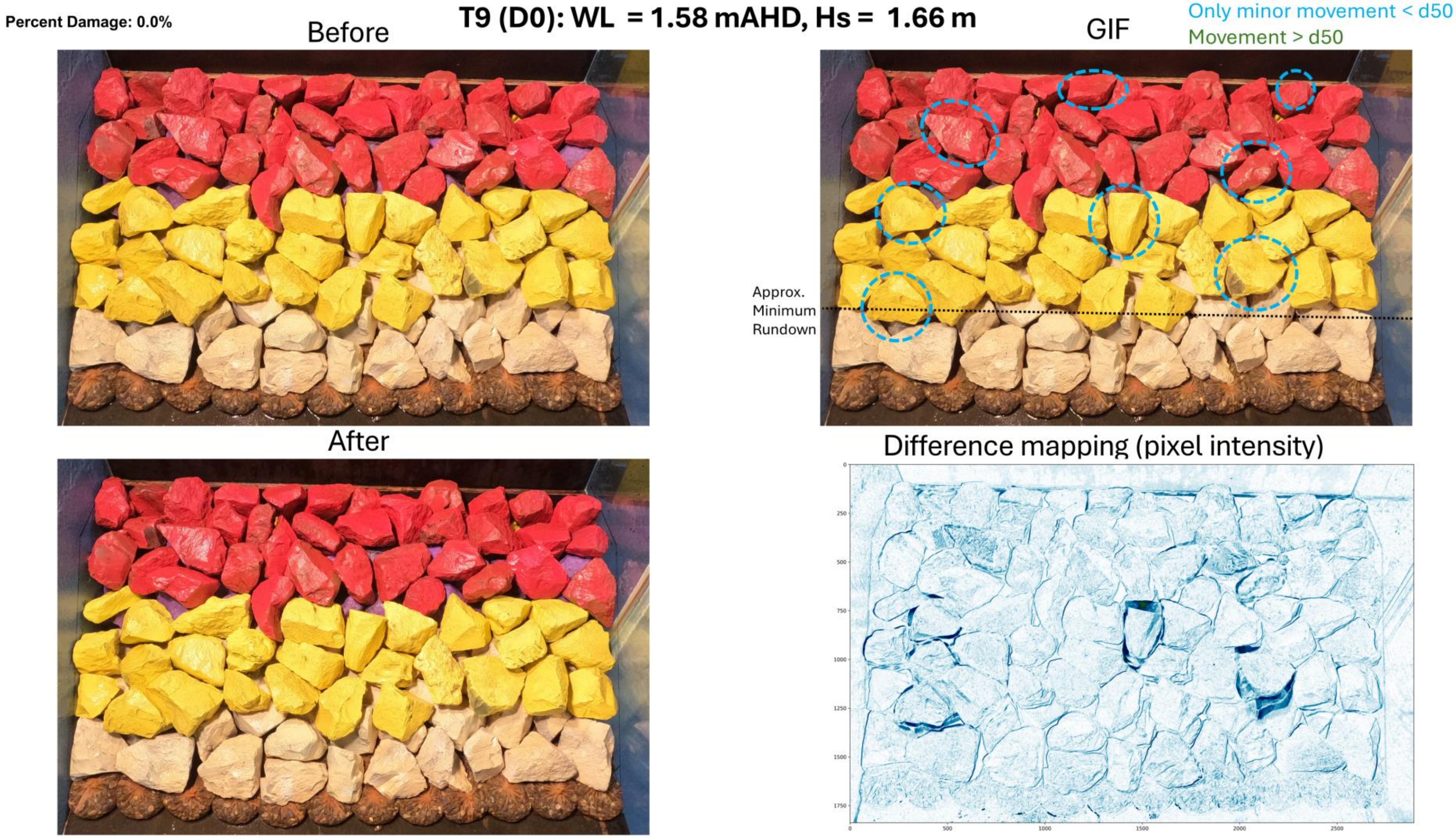
After

Difference mapping (pixel intensity)



Classification: Private

C.2 Scenario 2 armour stability assessment



Percent Damage: 0.0%

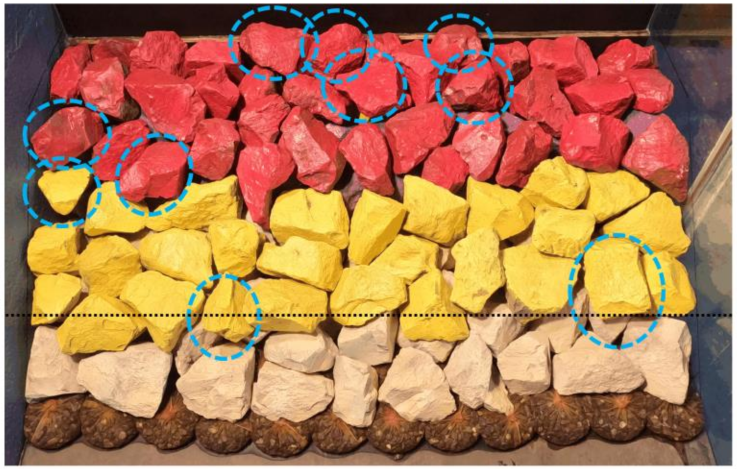
T10 (D0): WL = 2.08 mAHD, Hs = 1.66 m

Before

GIF

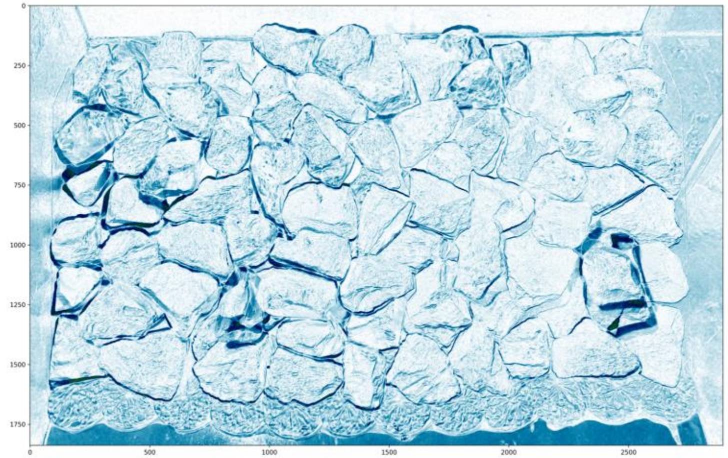
Only minor movement < d50

Movement > d50



After

Difference mapping (pixel intensity)



Percent Damage: 0.0%

T11 (D0): WL = 2.08 mAHD, Hs = 1.91 m

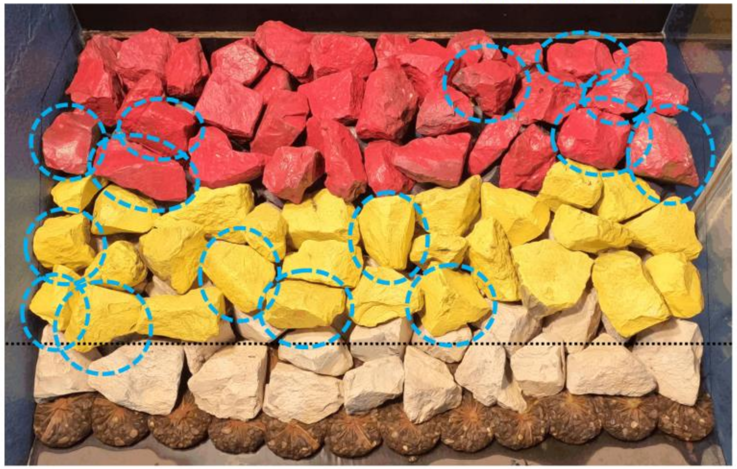
Before

GIF

Only minor movement < d50

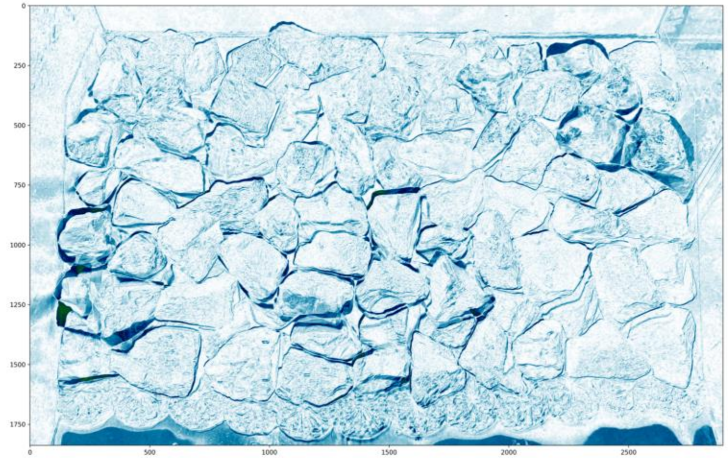
Movement > d50

Approx.
Minimum
Rundown



After

Difference mapping (pixel intensity)



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