APIA PORT BREAKWATER RECONSTRUCTION

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Samoa is located in the South Pacific Ocean, about halfway between Hawaii and New Zealand. The first wharf at Apia (185m long) was constructed in 1966 and a 100m long breakwater with 8t Dolos armor units was constructed in 1988-89 to provide shelter. Following large cyclones in the early 1990s, the breakwater was rehabilitated in 1996 with larger 20t Dolos armor units. In 2003 a solid concrete crown wall added to the breakwater to mitigate wave transmission into the harbor. There is limited toe and scour protection and multiple units have broken legs, cracks and rust stains from corroding steel reinforcement. The design intent of the breakwater upgrading project is to improve resilience to extreme climatic and natural disaster events. Reconstructing the breakwater with a higher crest, larger roundhead, new concrete armor units, toe support and a scour apron was selected as the most appropriate option to meet the design criteria. Physical modelling was undertaken to verify and optimize the design. Wave calibration indicated good correlation with numerical modelling and showed that for offshore wave heights of ~11.5m and greater the near structure wave height is depth limited (H_s ~9.1m). The physical model testing indicated that the existing 20t Dolos units on the rear slope are unstable under the 1% AEP event as the base units have limited rock toe support. Further model testing was undertaken using 16m^3 Xblocs on the leeside which was successful and detailed design was completed accordingly.

Keywords: Dolosse armored breakwater; design resilience; physical modelling

INTRODUCTION

Apia Port Location

Samoa is located in the South Pacific Ocean, about halfway between Hawaii and New Zealand. The country consists of two main islands (Upolu and Savai'i) and several smaller islands and uninhabited islets. Apia Port, on the north coast of Upolu is a commercial port and provides essential transportation links for imports and exports of mixed cargo, bulk petroleum, LPG, fish trans-shipment, liquid bulk storage and warehousing of dry freight [Figure 1]. The port regularly services container and Ro-Ro ships, as well as fuel tankers, gas tankers, fishing boats, interisland ferries and cruise ships.

Figure 1. Apia Port, Samoa

The Enhancing the Safety, Security and Sustainability of Apia Port Project is an Asian Development Bank funded project that involves upgrading the breakwater, navigation aids,

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reorganization of the container yard terminal and new port administration buildings. The design intent of the breakwater upgrading is to improve resilience to extreme climatic and natural disaster events.

Port Development

The first wharf at Apia (185m long) was constructed in 1966. Due to swell waves affecting the harbor, a 100m long breakwater with 8t Dolos armor units was constructed in 1988-89 to provide shelter.

Following large cyclones in the early 1990s, the breakwater was rehabilitated in 1996 with larger 20t Dolos armor units. In 2003, the wharf was extended north by 166m and a 70m solid concrete crown wall added to the breakwater. In 2018 a wharf extension to the south was constructed to accommodate longer ships, multiple ships or a cruise ship. Refer to Figure 2.

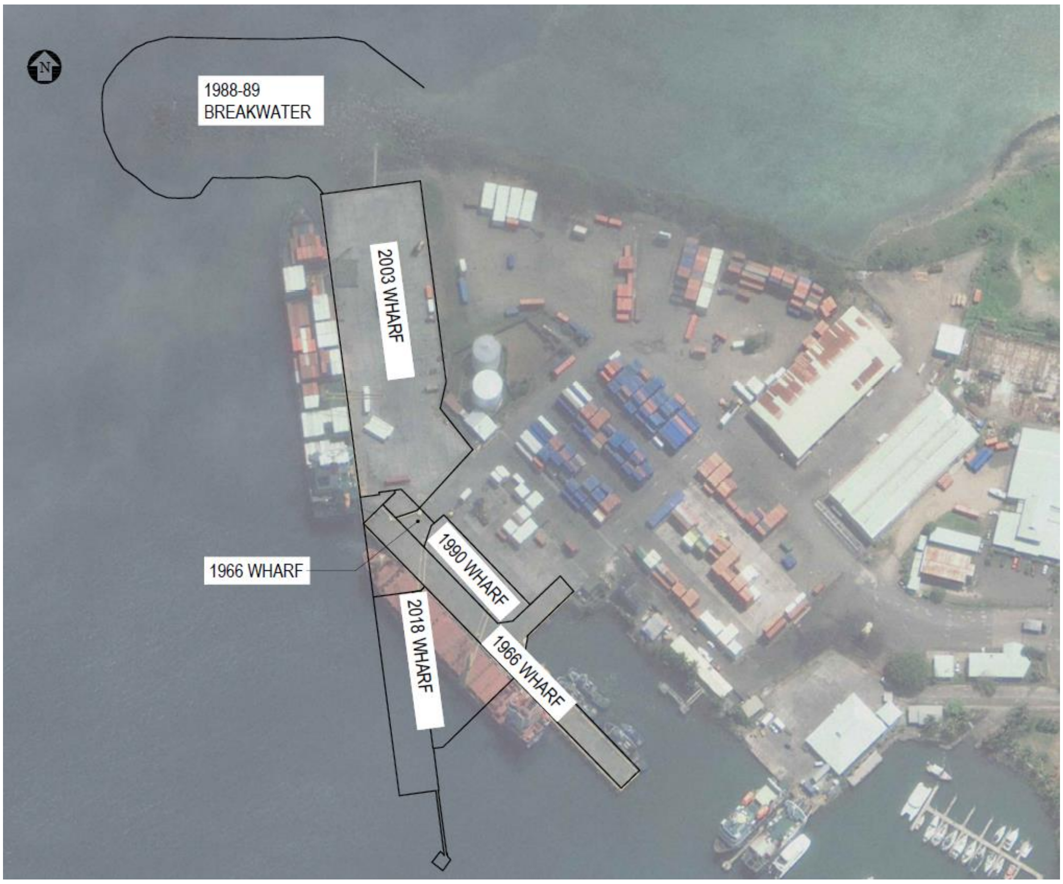

Figure 2. Port Development Timeframes

BREAKWATER HISTORY

The original 1988-89 breakwater extends from a shallow water reef approximately 100m into Apia Harbor to a water depth 14m below Chart Datum (CD). The original breakwater cross section comprised a rubble mound core, rock underlayers and 8t Dolos units at a 1.2H:1V slope with a crest level of 2.8m CD [Figure 3].

In the early 1990s, cyclones damaged the breakwater. Cyclone Ofa in 1990 was a large event (estimated H_s ~7m near the breakwater) and Samoa Port Authority personnel recall that the breakwater was overtopped by waves. In 1996 the breakwater was rehabilitated with 20t steel reinforced Dolos units at a 1.5H:1V slope. Additional rock material was placed at the toe to support the upper layer of Dolosse however there was no bench or scour apron incorporated. The crest level was raised to 4.6m CD [Figure 4].

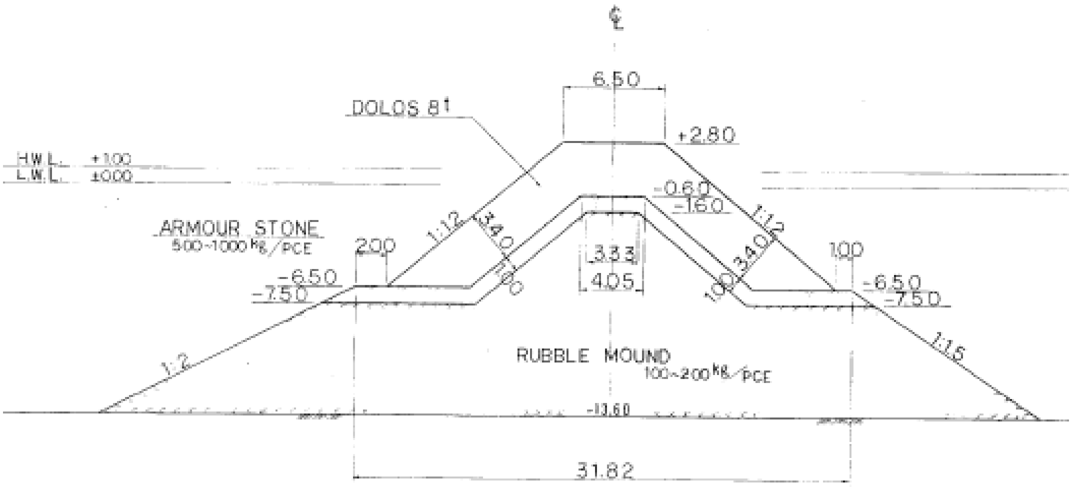

Figure 3. Original 1988-89 Cross Section

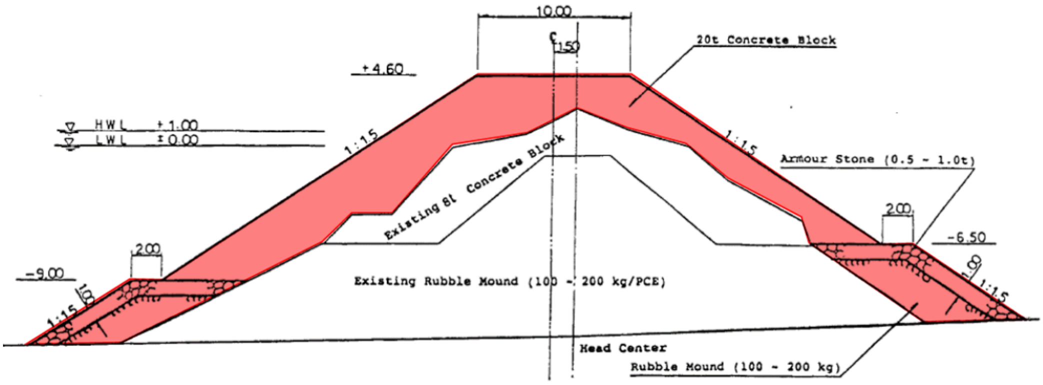

Figure 4. Upgraded 1996 Cross Section

In 2003, a 70m long concrete crown wall was constructed to mitigate against wave transmission through the upper section of the breakwater into the harbor basin. A rock and bag concrete foundation was constructed for the crown wall which is 3.1m high (crest level 2.5m CD) and is made from precast T-units with insitu concrete stitching. The Dolos units were then replaced, covering the crown wall [Figure 5].

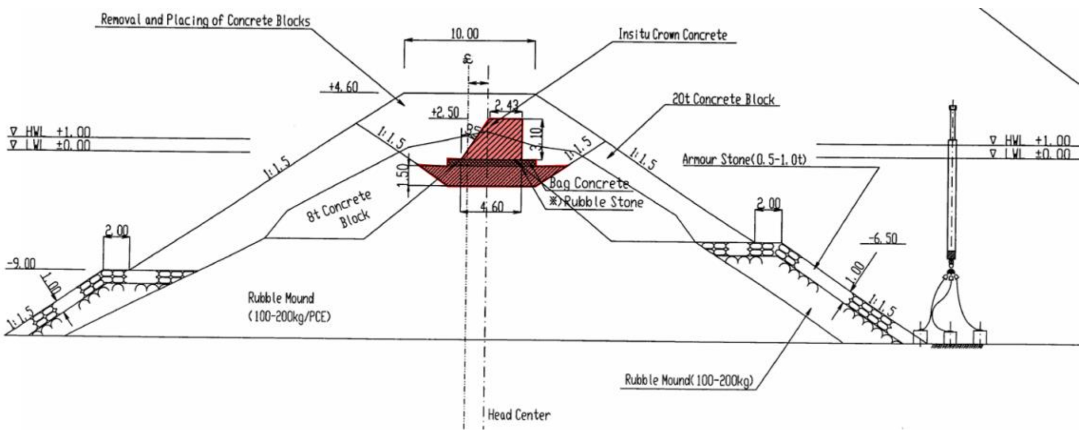

Figure 5. Upgraded 2003 Concrete Crown Wall

EXISTING BREAKWATER CONDITION ASSESSMENT

Inspections

Evaluation of the breakwater [Figure 6] was undertaken by review of design drawings, seaborne and walkover inspections, bathymetric survey and side scan sonar. The bathymetric survey and side scan sonar indicated minor reshaping at the lower section of the breakwater, particularly at the roundhead. A diving inspection was not undertaken for the initial inspections and was not pursued once the decision was made for the reconstruction option.

Figure 6. Existing Breakwater and Harbor Approach

Geometry

The outer half of the breakwater crest is lower by approximately 1m. This indicated that settlement has occurred in the outer section as the design crest level (4.6m CD) is the same along the breakwater. The concrete crown wall, which was installed in 2003, is in good condition and does not appear to have incurred differential settlement, noting that the crown wall starts at the breakwater root. At the root of the breakwater oceanside the outer slope units are orientated in the same direction which is not desirable. The breakwater is low crested and there is limited toe and scour protection.

Geotechnical Ground Conditions Summary

The existing breakwater was constructed from the reef edge seawards. As well as investigations undertaken in 2018 for the project, previous investigations from 1988 to 2016 were analyzed. The site consists of Quaternary Marine Sediments (very loose fine to medium sand and very soft to soft fine to medium sandy silt) overlying two un-weathered to moderately weathered basaltic lava flows which are separated by a zone of completely weathered basalt (gravelly silt/silty gravel). The basalt layer is approximately 20m below seabed.

Armor Unit Condition

The original 8t Dolos units are predominately placed on the harbor side and the root of the breakwater. The 20t Dolos units are placed elsewhere on the breakwater. Multiple units had broken legs, cracks and rust stains from corroding steel reinforcement. The 20t Dolos units are steel reinforced as evidenced by rusting steel and the 1996 design drawings. There were no signs of steel rusting observed in the 8t units and the design drawings were not available to confirm the presence of steel.

From the walkover inspection the estimated number of damaged 20t units above the waterline follows:

- 20-25 units with broken legs [Figure 7].
- 10-15 units with visible cracks.
- 5-10 units with signs of corroding steel reinforcing.

Figure 7. Broken Leg on 20t Dolos Unit

Hydraulic Stability of Existing Dolosse Armor

In the absence of records or reports documenting the design wave climate for the existing 20t Dolosse primary armor on the existing breakwater, back calculation was undertaken. Using the stability equation provided by Burcharth and Liu (1993) in The Rock Manual, the significant design wave height is approximately 7m. This also correlated using Hudson's Equation with a stability coefficient K_D of 12 as recommended by BS6349-7. This assessment assumes unbroken units and appropriate unit placement. As broken units are present the hydraulic capacity will be less than 7m.

METOCEAN ASSESSMENT

Numerical Models

Two numerical model approaches were adopted:

- The spectral wave model SWAN to transform offshore extreme events to the outer harbor and at the breakwater; and
- The free surface, terrain following, multi-dimensional hydrodynamic model SWASH to simulate extreme events from SWAN to the breakwater.

Offshore Wave Climate

The best available wave offshore dataset for Apia was the Centre for Australian Weather and Climate Research (CAWCR) numerical model study that comprises of hourly hindcast wave data from 1979 to 2013. The Secretariat of the Pacific Community Waves and Coasts Programme (WACOP) developed extreme wave height estimates using the CAWCR data based on a general pareto distribution. The CAWCR dataset was adopted by Hoeke, et al, (2014) and supplemented by a synthetic tropical cyclone dataset. Comparison of the Hoeke et al (2014) synthetic cyclone database extreme estimates and the WACOP analysis provided consistent results giving confidence that the WACOP estimates are representative of the offshore Apia wave climate. The 1% Annual Exceedance Probability (AEP) offshore H_s was assessed to be 15.2m with an associated T_p of 14s.

Bathymetry

The wave climate within Apia Harbor is dependent on the nearshore bathymetry and exposure to the incident wave climate. Extensive LIDAR land and bathymetric data out to -50m Chart Datum (CD) was available for Apia. The LIDAR data was validated with the 2018 project bathymetric and side scan survey conducted in the vicinity of the breakwater. For water depths greater than -50m CD data was sourced from hydrographic charts NZ8655 and NZ86501 via the LINZ Data Service. The offshore and local bathymetry is shown in Figures 8 and 9.

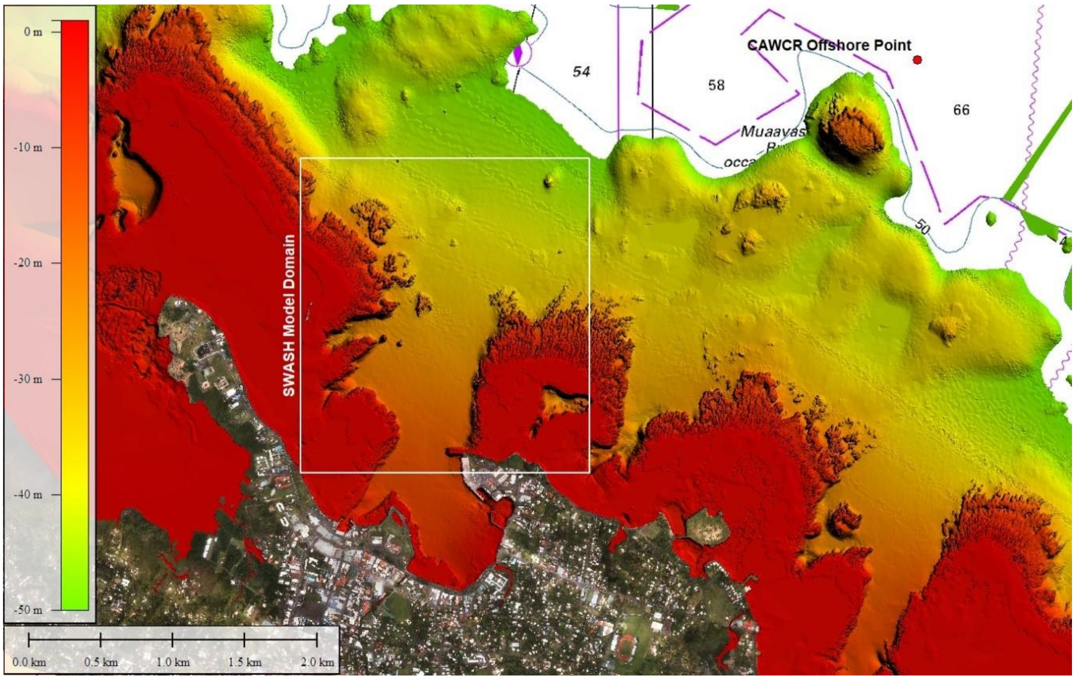

Figure 8. CAWCR Offshore Point and Bathymetry

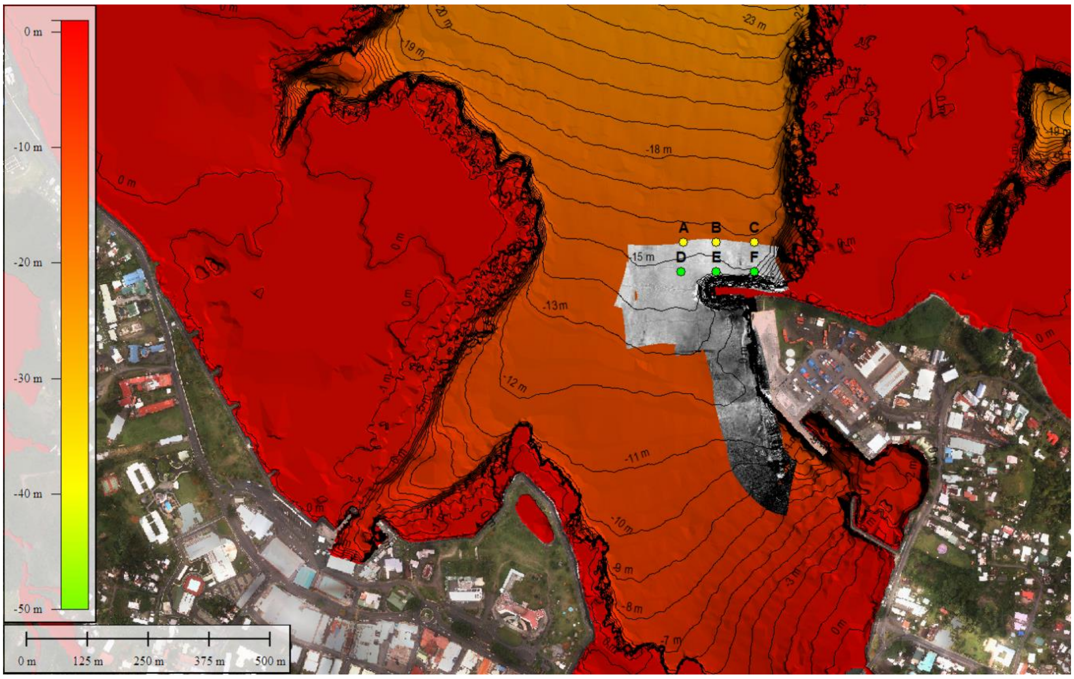

Figure 9. Local Bathymetry and Wave Height Locations

SWAN Modelling

To transform the offshore wave CAWCR data to the inner harbor a high-resolution flexible mesh SWAN model was developed. The computational mesh comprised of typical element lengths of 50m offshore transitioning to approximately 10m at the breakwater. Due to the circular nature of the wind fields during tropical cyclones an incident wave direction from due north was adopted for conservatism with a direction spread of 21 degrees as per Young (2006).

Sensitivity to wave breaking gamma indicated increasing wave height with increasing gamma as more energy is allowed into the harbor. For the design water level of 2.04m CD at location D, the 1% AEP H_s varies between 8.2m (gamma = 0.73) and 9.0m (gamma = 0.82) with a narrow directional spread of 8 degrees.

SWASH Modelling

For breakwater design the incident wave component is critical. SWASH, a phase resolving model, is ideally suited to simulating the complex wave breaking processes within Apia Harbor which includes the steep reef transition to deep water on both sides of the harbor and adjacent to the breakwater. SWASH also inherently accounts for wave reflection from rapid changing bathymetry and structures, such as the breakwater that can locally amplify wave heights. To quantify the effects of wave reflection and identify only the incoming wave energy for design, simulation scenarios including and excluding the breakwater were completed. The option for excluding the breakwater was achieved by adopting a seabed level of -14m CD for breakwater footprint.

The SWASH modelling demonstrated that wave transformation processes within the inner harbor are extremely complex. The distribution of H_s throughout the harbor is significantly affected by wave reflection off the fringing reefs and shallow water wave breaking particularly along the reef edge seaward of the breakwater. The numerical modelling confirmed a zone of high reflected wave energy fronting the breakwater which coincides with a localized bathymetric depression consistent with seabed scour.

Figure 10 shows outputs from SWASH with the 1% AEP H_s = 8.9m at location D off the breakwater roundhead. Note the zone of extreme wave breaking at the eastern reef edge offshore of the breakwater indicated by the ellipse. Excluding, wave reflection the 1% AEP H_s = 8.7m at location D.

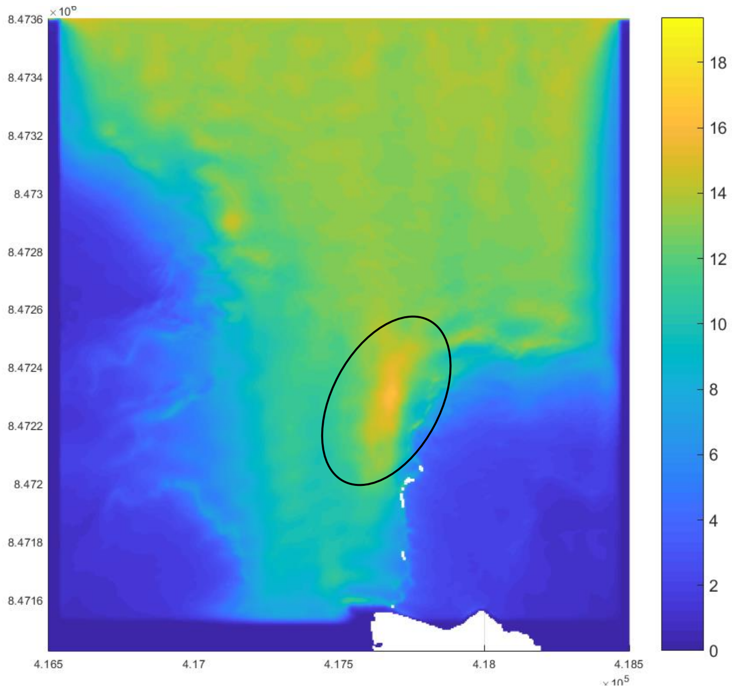

Figure 10. Wave Penetration into Apia Harbor

Commentary on Numerical Modelling Results

The understanding of the extreme wave climate offshore of Apia and within the inner harbor is limited to numerical hindcasts. While these hindcast models have been regionally validated for ambient conditions they remain uncalibrated for extreme conditions. In the absence of recorded wave data, during both ambient and extreme conditions, the hindcast is the best available data. Due to the hindcast data uncertainty the model approaches were erred to conservatism particularly with incident wave direction.

SWAN was primarily used to provide boundary conditions to the SWASH model. However, it was also used to simulate wave heights at the breakwater acknowledging the model limitations with a phased averaged approach, simplistic wave breaking and the absence of wave reflection.

Despite the different model physics the results from SWAN were consistent with SWASH at the breakwater trunk (location E) for standardized wave breaking assumptions (gamma 0.73) and at the breakwater head (location D) for conservative wave breaking assumptions (gamma = 0.82). This gives confidence that in the lieu of measure wave data the results are within a narrow uncertainty band.

Due to the ability of SWASH to simulate higher order wave processes the SWASH results were adopted for engineering design, subject to validation in the physical modelling stage.

Design Engineering Parameters

The 1% AEP design wave criteria considering only incoming wave energy and associated wave period of T_p = 14.2s adopted for design follows with locations as noted in Figure 9:

- Base case: Breakwater head H_s = 8.8m (Location D); breakwater trunk H_s = 8.1m (Location E)
- Sensitivity: Breakwater head H_s = 8.9m (Location A); breakwater trunk H_s = 8.4m (Location B).

OPTIONS ASSESSMENT

Function of the Breakwater and Design Intent

The function of the breakwater is to protect the inner harbor basin and attenuate wave conditions such that ship maneuvering and berth operability time is acceptable. Overtopping of the breakwater in extreme conditions is acceptable as the port is closed and ships are at sea. The criterion in this instance is to avoid significant damage to the breakwater, wharf and terminal infrastructure.

A design working life of 50 years was selected in accordance with guidance in BS6349-1. (category 4 common port infrastructure for commercial ports). The 1% AEP event was selected for hydraulic stability performance of the breakwater armor.

Options

The following options for the rehabilitation of the existing breakwater were considered:

1. Option B1 – Do nothing.
2. Option B2 – Localized patch repairs to oceanside slope with additional concrete armor units.
3. Option B3 – Overlay oceanside slope Dolosse with additional concrete armor units and rock toe support.
4. Option B4 – Reconstruct the oceanside breakwater face – remove existing outer face Dolosse, place quarry run core and construct a new concrete armor revetment. Options to maintain berth pocket space are a) Rotate breakwater roundhead seaward, b) Relocate entire breakwater footprint seaward and c) Construct the toe as a trench on the leeside of the breakwater.
5. Option B5 – Berm breakwater constructed on the oceanside face of the existing breakwater – Fully reshaped mass armored type.
6. Option B6 – Attached submerged berm and patch repairs to existing breakwater.

Option B1 – Do Nothing

Given that the stated aim of the project is to improve resilience and disaster preparedness the do-nothing option B1 was not considered further.

Option B2 – Patch Repairs

The condition assessment has identified that 35-50 20t Dolos units at the crest to waterline level are broken, cracked or showing signs of steel corrosion. Below water inspection by divers was not undertaken but it would be expected that lower hydraulic loading would mean less damage to units below water. Replacement of damaged units lower down the breakwater slope would involve removal and replacement of many overlying undamaged units. Such an operation would need to be carefully considered due to the risk of further damage occurring during re-handling and replacement. This work would replace the current defective units, however further deterioration of other units could be expected to occur progressively over time which will require further patch repairs. Future repairs at approximately 10-year intervals, or four times over the 50-year design life were assessed to be required.

Option B3 – Oceanside Armor Overlay

Repair of Dolosse armored breakwaters has been undertaken previously in several countries by overlaying the existing armor face with new concrete armor units. The replacement units could be new Dolosse however other concrete armor units could be used such as Core-locs, cubes or cubipods. Infill rock armor in gaps between Dolosse armor will likely be required to provide a suitable profile to accept new concrete armor units.

Option B4 – Reconstruct Oceanside

This option involves removing the existing Dolosse armor on the oceanside and constructing new concrete armor units with an underlayer and quarry run core. The crest level would be set to achieve resilience to sea level rise and wave overtopping. The Dolosse armor that is removed could be crushed and the concrete pieces returned to lower sections of the quarry run core. Alternatively, the units could be reused on another section of coastline.

Option B5 – Berm Breakwater

Icelandic type breakwaters have a higher crest level and several rock armor classes. As there is reduced reshaping expected compared to a fully reshaping berm type the front face armor rock is larger. Due to the large design wave height, armor rock greater than 2m diameter is required for both the limited reshaping and fully reshaping berm types, which was not feasible for the required rock volumes. An alternative is to use concrete cubes for the front slope and berm armor.

The berm breakwater option results in greater quantities of materials and a greater footprint area which would need to be considered from an environmental perspective. Protecting and upgrading the roundhead is required as this section is the most vulnerable part of the breakwater. The bench and scour apron need to be gradually reduced to zero at the roundhead, which would need confirmation by physical model testing.

Option B6 – Attached Submerged Berm and Patch Repairs to Existing Armor

An attached submerged berm concept reduces material quantities compared to the berm breakwater concept. The submerged berm breaks the wave thereby reducing the hydraulic loading on the existing structure. However, damaged units on the existing breakwater still need replacement and further deterioration of other units can be expected to occur progressively over time which will require further patch repairs. Future repairs will be required at approximately 10-year intervals, or four times over the 50-year design life. Protecting and upgrading the roundhead is required as this section is the most vulnerable part of the breakwater, which would need confirmation by physical model testing.

Wave Overtopping and Transmission

Wave overtopping was assessed using the EuroTop Manual for ambient and extreme conditions for the options described above. With the raised crest options B4 and B5, overtopping in extreme conditions could reach up to 1,000 l/s/m. In this scenario, ships will be at sea and the port closed. Breakwater survivability is the criterion in this instance. However, stability of the crest and rear slope units under such conditions need to be examined under physical model testing.

Wave transmission was assessed to be reduced compared to the existing situation with additional rock material and geotextile placed seaward of the existing breakwater. Installation of wave buoys to measure offshore and harbor conditions were planned for the construction phase which will enable assessment of ship operability and any improvements required for mooring arrangements.

Recommended Option

The patch repairs option (B2) did not meet the design criteria of improving resilience to extreme climatic events and climate change and is not recommended. The attached submerged berm option (B6) was approximately 30% more expensive than options B3 and B4 and there would be durability issues with keeping existing Dolosse armor and environmental disadvantages to this option.

The overlay option (B3) and reconstruction of the oceanside option (B4) were similar in cost. Option B4 offers improved resilience and reduced overtopping and wave transmission compared to Option B3. Considering all factors, option B4 (reconstruct the oceanside face with new concrete armor units) was selected for physical modelling.

Option B4 Preliminary Design

The main features of the reconstruction option are shown in Figure 11 and noted as follow:

- Removal of existing oceanside armor and quarry run core placement.
- Existing leeside armor, core and crown wall kept.
- New oceanside quarry run core.
- New oceanside geotextile, rock underlayer and scour apron (double layer D_{n50} = 1.1m).
- New oceanside concrete armor Xbloc units 16m^3 (or equivalent unit).
- New rock toe support to Xblocs (double layer D_{n50} = 1.7m).

At the roundhead location the extended footprint due to the scour apron and toe armor support means that the navigable channel width is reduced by 50m. This reduced channel width was modelled by ship navigation simulation and found to be acceptable using the design vessels. Encroachment into the berth pocket space was not acceptable as ships often berth with the bow north of the wharf edge. To account for this the breakwater axis was rotated northwards by approximately 10 degrees thereby maintaining the existing leeside footprint.

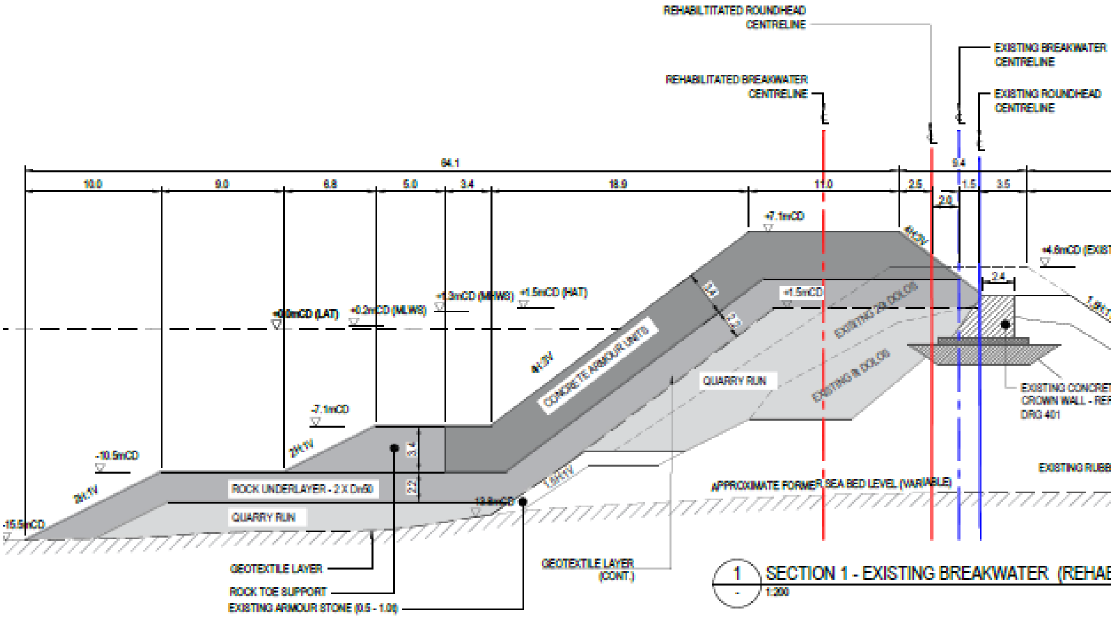

Figure 11. Reconstructed Option Typical Cross Section (Preliminary Design)

PHYSICAL MODEL TESTING

The University of New South Wales (UNSW) Water Research Laboratory (WRL) carried out physical modelling for the reconstructed breakwater option B4. Quasi 3D testing was undertaken in a flume 32.5m long, 3 m wide and 1.3m depth. The model scale for length was selected at 1:92 and scale ratios for time, velocity and mass calculated. The layout of model domain is show in Figure 12.

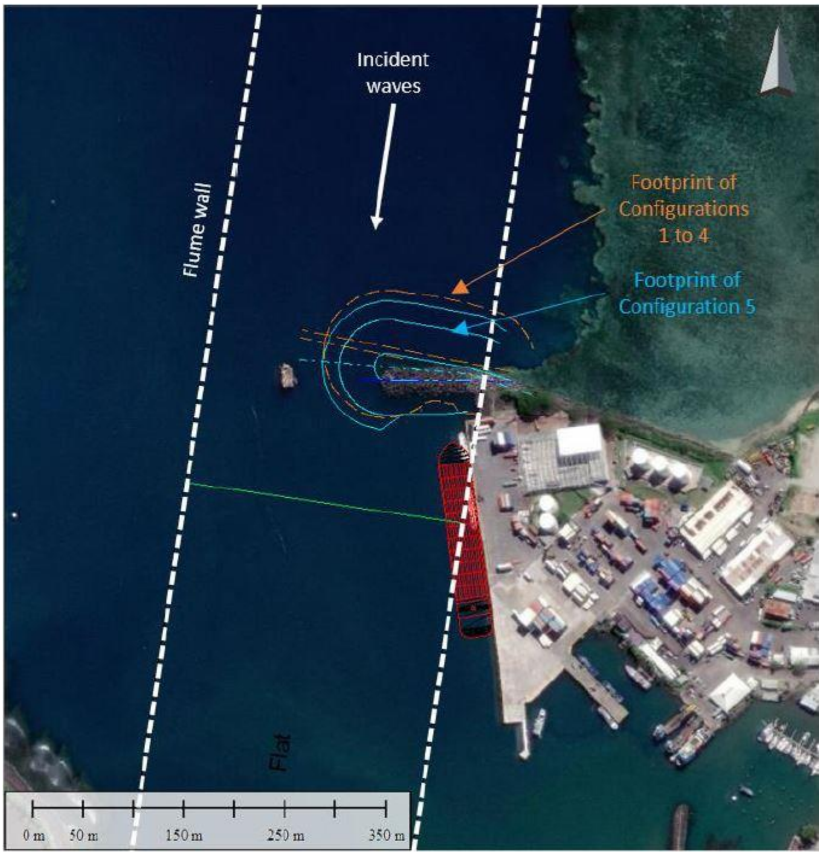

Figure 12. Physical Model Layout (Source: WRL)

Calibration

Wave conditions were calibrated to target offshore conditions and measured near structure conditions compared with the numerical modelling results. The calibration indicated good correlation with numerical modelling. For the 1% AEP offshore H_s of 15.2 m, the physical model has a nearshore H_s measured at location A of ~9.1 m. This is in between the SWAN model results with gamma of 0.78 and 0.82, and what the SWASH model predicted with unidirectional waves, which is more consistent with the unidirectional physical model simulation. Offshore wave conditions with H_s of 11.5 m (2% AEP) and greater are depth limited at the breakwater and all result in the same nearshore H_s of ~9.1 m.

Initial Test Runs

Initial test runs are shown in Table 1. A settlement run at 50% wave climate was undertaken prior to testing.

Table 1. Initial Physical Modelling Test Runs.

Test	Condition	SWL (m CD)	Hs, offshore (m)	Hs, roundhead (m)	Breakwater Configuration
1	A – 1% AEP low water present day	0.5	15	8.9	Option B4
2	B – 1% AEP + SLR	2.04	15.2	9.2	Option B4
3	B – 1% AEP + SLR	2.04	15.2	9.2	Crest row of 100t concrete cubes replacing back row of xblocs
4	C – overload	2.04	17.4	9.2	Crest row of 100t concrete cubes replacing back row of xblocs
5	A – 1% AEP low water present day	0.5	15	8.9	Crest row of 100t concrete cubes without lee side crest support
6	A – 1% AEP low water present day	0.5	15	8.9	Crest row of 100t concrete cubes with 13t armor stone for leeside crest support
7	B – 1% AEP + SLR	2.04	17.4	9.2	Crest row of 100t concrete cubes with 13t armor stone for leeside crest support

Commentary on Initial Test Runs

The existing 20t Dolosse armor on the rear slope were unstable under the 1% AEP event. The base row units have limited rock toe support and these units were displaced, followed by slumping of slope units, which in turn caused instability for the crest Xblocs [Figure 13]. The addition of the 100t cubes and rock armor at the crest was to investigate the sensitivity of the rear slope failure. The alternative crest arrangements had minimal effect on the failure of the rear slope Dolos units.

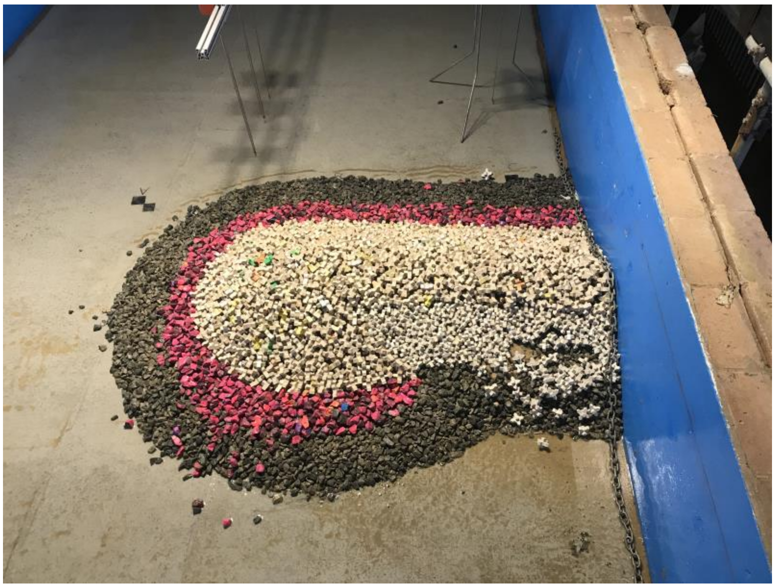

Figure 13. Flume drained after Test Run 2

The rock toe support to the Xblocs generally performed well. Some displaced rocks were observed after the low water test, particularly in the lee quadrant of the roundhead.

The Xblocs generally performed well, particularly the roundhead. On the trunk section, there were instances of unit rocking where units were placed in the same orientation or were poorly interlocked. Two units were displaced in testing which was assessed to be the result of poor placement. One unit was sitting proud on a ply section of the model and the other unit displacement was due to poor interlock. The concept to retain the leeside Dolosse armor was shown to be not feasible. Two alternative options were considered; 1) extend the rock toe on the leeside to support the existing Dolosse and 2) remove all existing units and the crown wall and replace with 16m^3 Xblocs. As option 1 requires reliance on the existing units, option 2 was selected as this provided more certainty on the stability and durability of the leeside armor.

To reduce the cross section width as much a possible the toe (top bench level) of the Xblocs was raised from -7.1m CD to -4.8m CD on the leeside. The existing leeside footprint extent was maintained, and the breakwater axis adjusted accordingly.

Supplementary Test Runs

Supplementary test runs are shown in Table 2. A settlement run at 50% wave climate was undertaken prior to testing.

Table 2. Supplementary Physical Modelling Test Runs.

Test	Condition	SWL (m CD)	Hs, offshore (m)	Hs, roundhead (m)	Breakwater Configuration
8	A – 1% AEP low water present day	0.5	15	8.9	Revised Option B4 (all Xblocs)
9	B – 1% AEP + SLR	2.04	17.4	9.2	Revised Option B4 (all Xblocs)

In general, the revised design performed well, and the rear slope stability issue was solved by extending the Xblocs and the toe support rock. Rocking of Xblocs (3 no. for test 8 and 6 no. for test 9) was observed on the crest due to poor unit interlocking. This was assessed to arise from the varying crest width and rising toe level leading to less than optimal interlocking. Unit placing positions on the crest will need to be reviewed during construction to achieve acceptable interlock.

For the Xbloc toe rock support, greater damage occurred for the low water test (total of 36 toe rocks displaced) compared to the high-water test (17 toe rocks displaced), which is expected.

DETAILED DESIGN ADJUSTMENTS

The concrete armor unit layer thickness used in the preliminary design development was 3.4-3.6m. This represented a range of armor unit types, sizes and suppliers. For the physical model testing, 16m^3 volume Xblocs were selected which have a unit height of 3.63m and a layer thickness of 3.5m. Cross section geometry was adjusted accordingly.

The toe rock support size developed in preliminary design was M_{50} = 13.2t, D_{n50} = 1.7m, layer thickness (t) = 3.4m. Given that the Xblox layer thickness is 3.5m and that the modelling indicated moderate concentrated toe rock displacement at the lee quadrant of the roundhead, the toe rock size was increased to M_{50} = 14.4t, D_{n50} = 1.75m, t = 3.5m.

The change in layer thickness results in other minor changes as listed below and shown in Figure 14:

- The toe rock bench width is increased from 5m to 5.3m ($3 \times D_{n50}$).
- The top of the toe rock bench is higher (-7.1m CD to -7.0m CD).
- The crest height has increased from 7.1m CD to 7.2m CD.

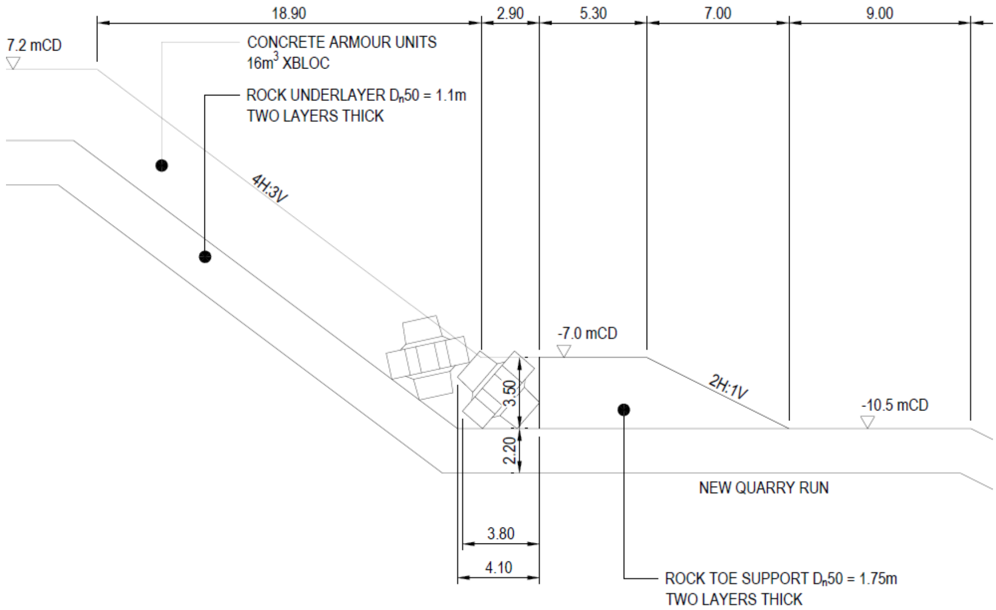

Figure 14. Adjusted Cross Section Geometry

A 3D graphic of the final design of the reconstructed breakwater outer surface profile is shown in Figure 15.

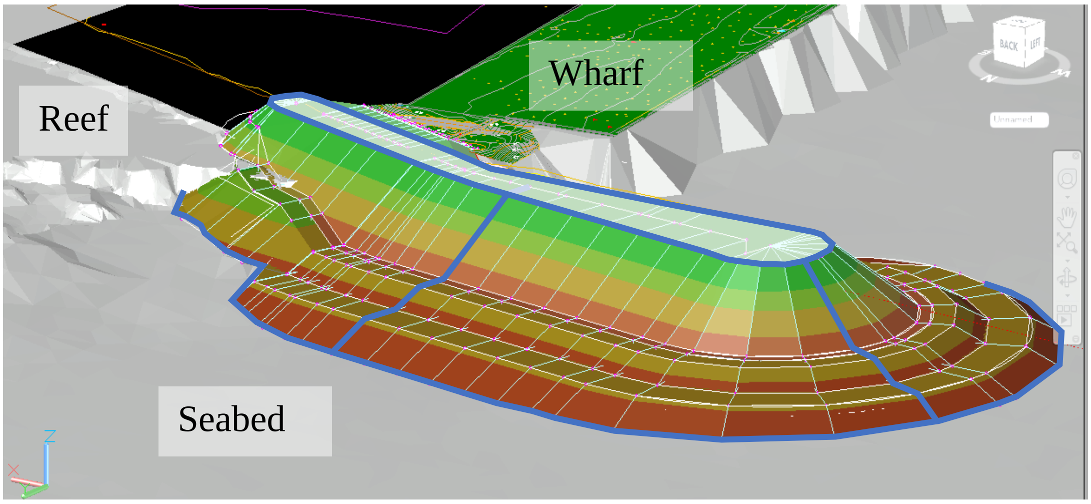

Figure 15. Breakwater Design Model

CONCULSIONS

Key conclusions follow:

- Where there is limited measured metocean data, numerical and physical modelling techniques reduce uncertainty with respect to engineering design parameters.
- Numerical and physical modelling are design tools that complement each other and enable verification and optimization of design.
- Reliance on only one of these design tools would not facilitate an optimal design.
- A multicriteria assessment process that accounts for technical, cost, performance and environmental categories is required for selection of the preferred option.
- Substantial overtopping of breakwaters may result in crest and rear slope instability which needs to be accounted for in the design.
- Toe support for concrete armor units is crucial for stability.

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