Physical modelling of complex overtopping flows at Ōhau Point, Kaikōura and development of mitigation measures

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Abstract
A magnitude 7.8 earthquake struck the Kaikōura coastline in November 2016 causing widespread uplift and landslides closing State Highway 1 and Main North Rail Line. Ōhau Point was the location one of the largest and most challenging landslides, with more than 160,000 cubic meters of rock falling from the surrounding cliffs and inundating the road and rail corridors. Recovery works by the North Canterbury Transport Infrastructure Recovery (NCTIR) Alliance reinstated the roadway further seaward and at a lower level than previous due to residual landslide and rockfall risk. A unique combination of steep offshore bathymetry and rock outcrops resulted in focusing of wave energy and overtopping to occur at a higher frequency and magnitude than was initially expected. This overtopping presented a potential hazard to road users and to the road infrastructure itself during extreme events. This paper presents the results of extensive physical modelling undertaken at the Water Research Laboratory at UNSW Sydney to investigate the overtopping processes and to assist NCTIR in evaluating options for mitigating hazard at the site. The section of road and seawall at the site is fronted by a nearshore zone with highly complex bathymetric features, and as such, a quasi-three-dimensional model was required to simulate the complex 3D effects of the nearshore wave field and overtopping process.

Keywords: physical modelling, overtopping, wave loading, coastal structures.

1. Introduction

Just after midnight on 14 November 2016, a magnitude 7.8 earthquake struck the Kaikōura region in the South Island, New Zealand, causing widespread damage, and closing both State Highway 1 (SH1) and the Main North Line railway between Picton and Christchurch, isolating the surrounding coastal and rural communities.

Ōhau Point was the location one of the largest and most challenging landslides, with more than 160,000 cubic meters of rock falling from the surrounding cliffs and inundating the road and rail corridors (Figure 1). The extend of the landslide resulted the new road alignment being shifted seaward and lower than the previous location. The road is supported by a near vertical seawall, constructed on a mass concrete foundation. The seawall height is limited by seismic considerations and as such is lower than the previous road, reaching a maximum of 10 m above mean sea level. Several wave overtopping events were observed during and following construction and the re-opened road was observed to be affected by overtopping during moderate and larger storm events as shown in Figure 2 [1]. The overtopping was deemed to present a hazard to road users and potentially damaging to the road infrastructure itself.

This paper presents the results of extensive physical modelling undertaken at the Water Research Laboratory at UNSW Sydney to evaluate the coastal and overtopping processes to evaluate options for mitigating hazard at the site, including modification to the toe of the concrete seawall, the installation of a crown wall and the placement of concrete armour units in front of the seawall.

Figure 3 Overview of Ōhau Point post-rehabilitation works [Source: NCTIR]

2.1 Introduction

The section of road and seawall at the site is fronted by a nearshore zone with highly complex bathymetric features (Figure 2). The wave field on approach to the seawall is highly modified by the localised features in this nearshore zone, and as such, a quasi-three-dimensional model (Q3D) was required to properly simulate the 3D effects of the nearshore wave field and overtopping.

The Q3D physical model testing was undertaken in the 3 m flume at WRL. This flume measures approximately 40 m in length, 3.0 m in width and 1.3 m in depth. Several factors were considered in selecting a scale for the physical model, with the main factors in this case being to:

- Ensure that the key bathymetric features influencing the observed wave and overtopping processes could be captured within the flume;
- Ensure that the model was sufficiently large for accurate representation of overtopping processes and resulting overtopping volumes of water over the crest of the seawall;
- Ensure that the model was sufficiently large to reproduce armour stability processes without the influence of scale effects; and
- Ensure that the model was sufficiently large to allow measurement of the wave loads impacting the modelled crown wall.

2.2 Scaling

Model scaling was based on geometric similarity between the prototype (real world) and the model. An undistorted length scale of 1:40 was selected after consideration of the model design constraints.

An analysis of scale effects and scaling limitations was undertaken, which showed that for all proposed test wave conditions, scale effects were minimised and within acceptable values, ensuring that accuracy of the model results was not impacted by scale effects [1].

2.3 Bathymetry

The complex nearshore bathymetry seaward of Ōhau Point was analysed for the model design, using a combination of two bathymetry data sets (Figure 4):

- High resolution (0.5 m) digital elevation model (DEM) derived from photogrammetry, drone-dipping and multi-beam bathymetry [1] covering the area immediately surrounding the rehabilitated Ōhau Point coastal works;
- Medium resolution (20 m) DEM covering the wider area.

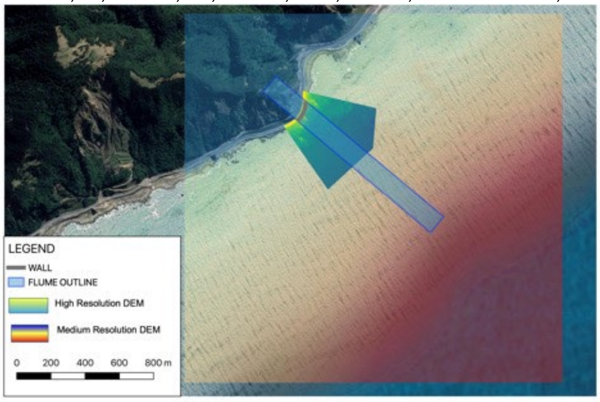

Figure 4 Bathymetry data sources and flume layout

Figure 5).

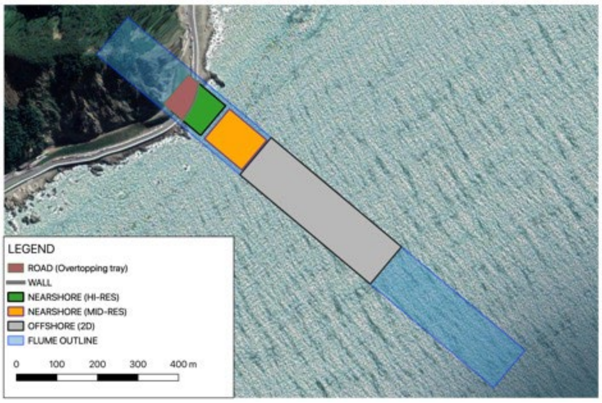

Figure 5 Bathymetry layout

The intermediate nearshore area was modelled with medium resolution using an array of laser cut transects. Based on the review of the bathymetry data, a 12 m transect spacing was adopted to facilitate efficient construction while maintaining model accuracy (Figure 6). The elevation profiles along each of the laser cut transects were derived from the high-resolution DEM (0.5 m resolution) and used to shape the surrounding mortar capping to reflect the intermediate depth bathymetry and allow the introduction of 3D features such as deeper channels and other submerged seabed features.

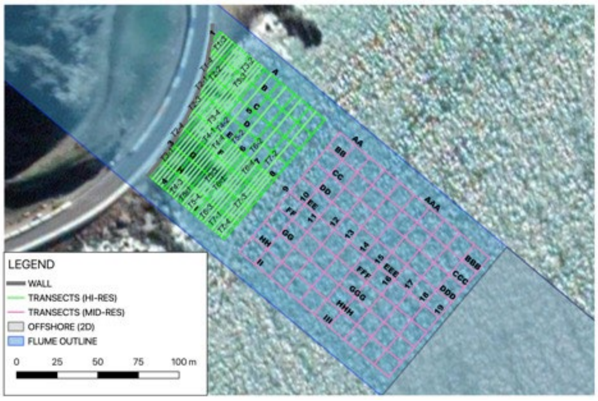

Figure 6 Laser cut transect arrangement for the two nearshore areas

Figure 6.

The tight spacing between the transects combined with the high resolution (0.5 m) of the elevation information along the transects ensured a very precise representation of this complex bathymetry and enabled accurate simulation of the localised wave processes generating the overtopping of the existing coastal protection works at the site.

Specific attention was given to emergent elements such as larger rocks located immediately off the seawall toe as well as the existing concrete slabs, by using detailed elevation information from recently captured LiDAR and onsite photography.

Figure 7 Overview of completed nearshore model bathymetry zones and seawall

2.4 Existing Seawall

The existing seawall face underlying the road is comprised of stepped concrete block layers (including a recurved capping block), with an indicative road/crest level of +9.7 m above New Zealand Vertical Datum (approx. mean sea level). The plan view curvature of the wall was derived from the DEM and used to align the overall 120 m of modelled section into the nearshore bathymetry.

(a) Identification of the individual concrete block layers of the existing seawall

Figure 8 Overview of completed nearshore model bathymetry zones and seawall

Critical elements such as the step offset between each layer and the top recurve were include to ensure accurate modelling of overtopping processes when waves impact the seawall.

The model of the seawall was assembled within the flume and integrated within the nearshore bathymetry area prior to the final pour and shaping of the bathymetry capping, allowing for a seamless bathymetry surface extending right up to the seawall face. The modelled concrete slabs at the toe of the seawall were secured in place over the bathymetry with localised infill based on review of the provided hi-resolution aerial photography and DEM.

3. Data Collection and Instrumentation

3.1 Waves

Waves were measured using capacitance-type wave probes, sampled at 7.9 Hz (prototype scale) and then processed using the least square method described by Mansard and Funke (1980) to separate and interpret incident and reflected waves.

A total of eight (8) wave probes were installed in the flume throughout testing. Two nearshore wave probes were positioned in front of the seawall and two sets of 3 probe array (3PA) were installed further offshore, approximately 200 m and 700 m from the toe of the existing seawall and used for wave climate calibration and reflection analysis [2].

3.2 Overtopping

Average wave overtopping rates were measured using a volumetric catch tray installed leeward of the seawall/road, so that the total volume of overtopping water was captured and then averaged over the test duration.

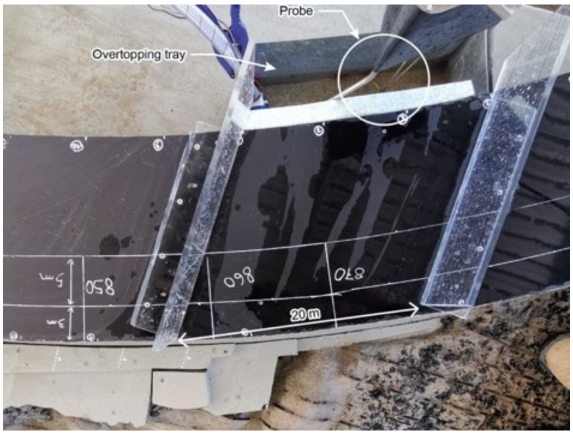

Figure 9 Model overtopping setup

3.3 Wave Loads

The horizontal wave loads on the crown wall were measured using load cells. Two load cells were mounted on a rigid frame at the rear of a 5 m long section (prototype scale) of the crown wall. Forces were measured using Futek LSB210 submersible load cells due to ongoing overtopping of the structure (Figure 10).

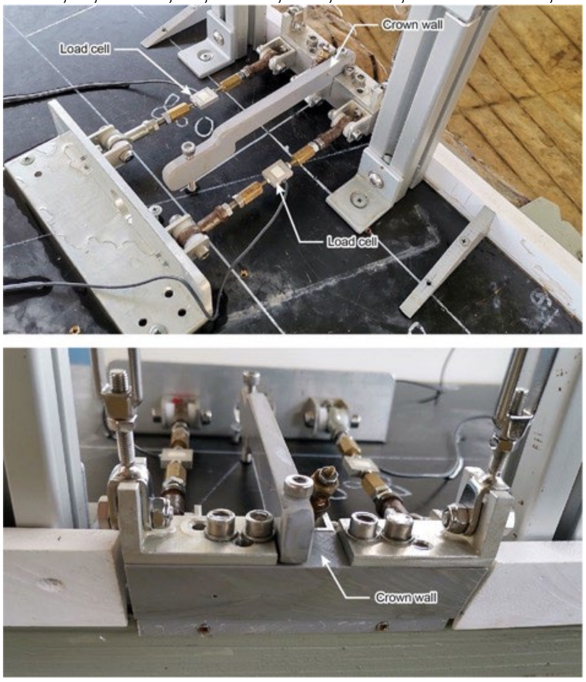

Figure 10 Crown wall section with load cells

(a) Crown wall and concrete slab removed (partially)

(b) Configuration 5 - 6.5 T Hanbars

Figure 11 Examples of some of the tested solutions

The test program predominantly focused on four different test conditions as summarised in Table 1.

Table 1 Modelled Wave Conditions

ID	Description	H_s (m)	T_p (s)	WL[1] (m)
F1	100 year ARI design storm present day	6.4	13	1.23
F2	100 year ARI design storm + 0.5 m SLR	6.4	13	1.73
F3	20 year ARI design storm + 0.3 m SLR	5.4	13	1.47
F4	15 August 2019 storm (control)	2.7	15	0.63

1.WL expressed in mNZVD2016

Additional sensitivity tests were conducted to assess the overtopping sensitivity to various parameters (e.g. water levels, wave period and wave height).

5. Test Program

The physical modelling considered a range of configurations including the existing format of the site and three remediation options to mitigate the overtopping hazard. The first upgrade option was to remove part of the mass concrete slabs located at the toe of the existing concrete seawall.

The second upgrade option considered the installation of a crown wall along the crest of the wall in area affected by overtopping. The third upgrade option consisted of the placement of concrete armour units (Hanbars) in front of the existing seawall to form a toe berm.

6. Overtopping

6.1 Introduction

Total overtopping volumes were measured along a representative 20 m long section of the seawall and averaged over the duration of the test. This location along the crest of the existing seawall was chosen after running several preliminary tests.

Review of the overhead video footage clearly showed that a direct consequence of the interactions between the incoming waves and the complex nearshore bathymetry resulted in focused impulsive jets of water overtopping the crest of the wall in a highly 3D form. These localised overtopping jets were of variable width, predominantly between 5 and 10 metres.

Figure 12 Examples of highly 3D wave overtopping

6.2 Mean overtopping rates

Overtopping volumes were reduced by 50% with the installation of the crown wall (10 L/s/m down to less than 5 L/s/m at prototype scale). Interestingly, the physical model showed that removing sections of the concrete slabs located in front of the seawall also significantly reduced overtopping by 75% to 85%. This was attributed to the slabs acting as a ramp to direct the wave up the seawall as well as reducing the effective freeboard above the wave impact level.

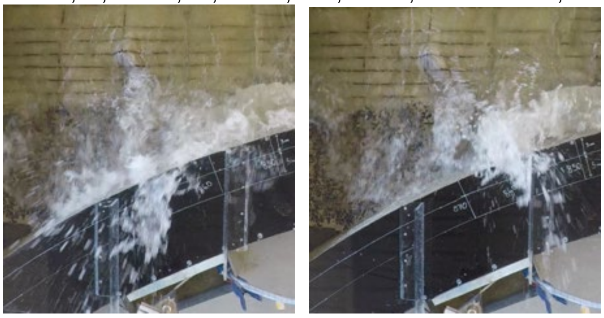

Figure 13 Illustration of spatial variability of overtopping (existing configuration - F1)

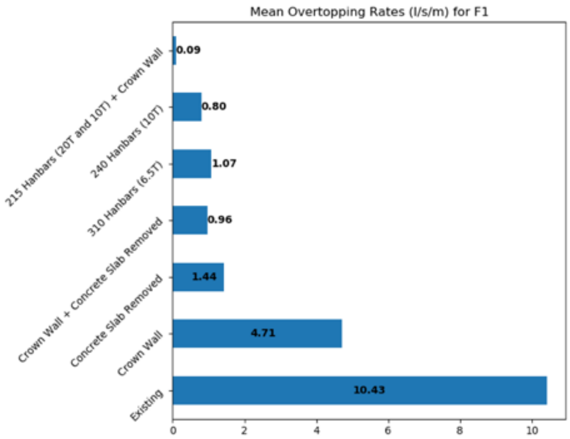

Figure 14 Mean overtopping rates for F1

Further reduction of overtopping volumes was observed with the partial removal of the concrete slabs combined with the installation of a crown wall or the placement of model units representing 10T or 6.5T Hanbar units, with average overtopping volumes around 1 L/s/m (prototype scale) for a 100 year ARI storm for the present day water levels.

6.3 Discussion

Average overtopping discharge is commonly used in coastal engineering to classify the severity of the overtopping and risk to the integrity of the overtopped coastal structure, as well as safety of people [3].

Averaging overtopping volumes along a defined width of crest is a standard practice. This is typically not problematic for cases where overtopping processes are relatively two-dimensional and do not vary extensively along the crest of the considered structure. However, in the case of Ōhau Point, it is important to realise that the necessity of measuring overtopping volumes along 20 m of seawall crest (in order to capture the spatial variability of the overtopping jets, as presented in Figure 15), result in a potential underestimation of mean overtopping rates created by overtopping jets predominantly about 5 to 10 m wide (prototype scale).

The consequence of averaging overtopping over the 20 m (prototype scale) width can be further volumes during flume testing. depicts the evolution illustrated by looking at instantaneous overtopping of total overtopping volumes captured by the catch tray during a short duration test for the existing seawall configuration.

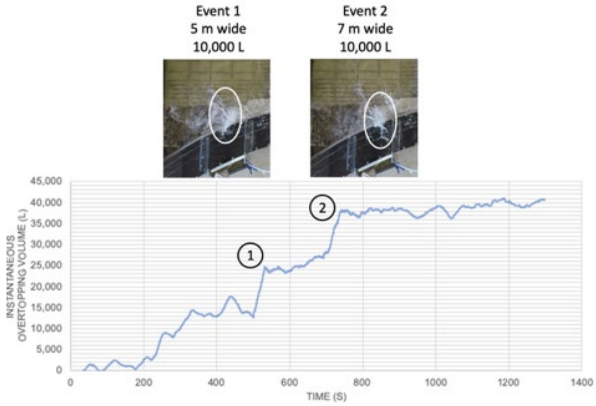

Figure 15 Time series of overtopping volumes for existing condition

Note that two overtopping events (i.e. Events 1 and 2) accounted for nearly 50% of the total volume measured. Review of these two events indicates that the overtopping jet width was between 5 and 10 m, albeit at different locations along the crest.

6.4 Hanbars Stability

A total of 9 Hanbar armour unit stability tests were undertaken for three (3) different armouring configurations, which consisted of 6.5 T (310 units), 10 T (240 units) and a mix of 20 T (35) and 10 T (180) Hanbar units, placed in front of the existing seawall and extending seaward to the -4.0 m NZVD2016 bathymetric contour.

The placement of the model Hanbar units gave due consideration to the specific constraints of the Ōhau Point site, such as the maximum reach of the crane to be used for placement, and the influence of ocean conditions on placement reliability that would be expected during construction.

The overall observed damage process (Figure 16) was similar for the three different armouring configurations and consisted of:

- Progressive peeling of units on both flanks of the armour layer (1);
- Units were pushed up from the seaward edge all the way to the seawall (2);
- The 6.5 T units were observed to hit the seawall during 100 year ARI events (i.e. F1 and F3) (3);
- Units were lost between the two surface piercing rock outcrops (4); and
- Units were also observed to be lost between the seawall and the rock piercing the surface in the north (5).

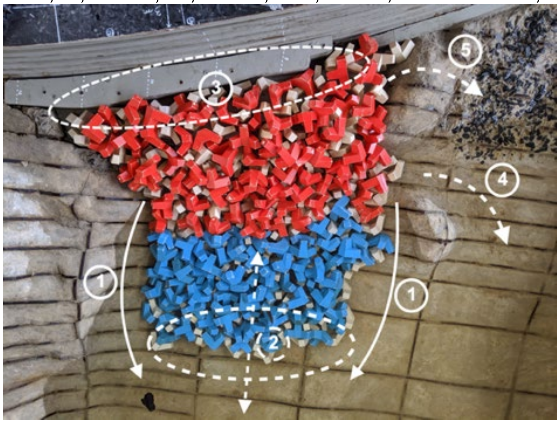

Figure 16 Armour damage processes

All Hanbar armour stability tests showed relatively high levels of damage to the structure under extreme wave conditions (100 year ARI), for the smaller armour units tested. Regardless of unit's size, damage for two consecutive tests, to account for the difficulty of access of the site and undertaking repairs necessity the mobilisation of a specialised mobile crane, were consistently high with damage levels of at least 20% of units displaced.

7. Wave loads

Tests were performed to measure the wave loads on the seawall crown wall for two model configurations (with and without the concrete slab at the toe of the existing seawall). Wave loads were measured on a 5 m long crown wall section (prototype scale) centred on chainage 860 m, which was at the location considered to experience peak overtopping intensity. An example of a wave hitting the crown wall section attached to the load cells is shown in Figure 17.


Figure 17 Wave hitting the crown wall during Test F1 (without concrete slab)

Wave loads on the proposed crown wall were assessed for the 100 year storm event (present day sea level). Maximum instantaneous peak horizontal wave loads of approximately 100 kN/m (prototype scale) were observed for the tested design. While the measured data is considered to provide a reasonable estimate of the magnitude of wave impact loads on the crown wall, it is important to consider that the results contain some inherent scale effects such as altered air entrainment at this scale.

8. Conclusions

Q3D physical modelling was successful in modelling very complex wave and overtopping processes at the Ōhau Point post-Kaikōura earthquake. The coastal road at the rehabilitated site was experiencing an overtopping hazard even in relatively mild wave conditions.

The physical modelling considered a range of configurations including the existing format of the site and three remediation options to address the overtopping hazard. The most efficient and practical options to reduce overtopping identified were the partial removal of some of the already damaged mass concrete slabs located at the toe of the existing concrete seawall and the installation of a crown wall along the crest of the wall in area affected by overtopping.

Removal of the concrete slabs was trialled but could not be achieved. A concrete crown wall was designed informed by the results of the physical modelling and construction on site finalised in late 2020.


Figure 18 Construction of new crown wall

Acknowledgments

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9. References

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