Influence of Water Level on Wave Uplift Loading for a Cantilevered Walkway above a Vertical Seawall

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Abstract
This paper discusses two-dimensional physical modelling of the distribution of vertical wave forces on the underside of a cantilevered walkway slab on top of a proposed vertical seawall in Darwin. The proposed seawall is to be built along the foreshore to protect infrastructure associated with a new luxury hotel from erosion and wave overtopping. The walkway will provide the public with access to and along the foreshore seaward of the hotel.

Since the design water levels are close to the walkway level, it was anticipated that the maximum wave loads would occur at a lower part of the tidal cycle when there is a greater potential for generation of "impulsive" type wave impact loads. The key objective of the model investigation was to estimate the maximum wave-generated uplift loads on the walkway for two different cyclonic wave conditions (to assist structural design), and to understand the influence of water level on the resulting wave loads.

The pressures and total force exerted upwards on the walkway (underside elevation 5.0 m AHD) were measured at multiple water levels between 2.0 m AHD and 5.6 m AHD for both wave height-wave period combinations. The largest uplift forces and pressures were observed at water levels between 2.4 m AHD and 3.0 m AHD. The largest maximum peak force measured was 4,293 kN on a movable plate with area 6.44 m^2 (approximately 670 kPa). This paper also compares the peak wave loads measured in the physical model with those estimated using an empirical design technique.

Keywords: physical modelling, wave loading, vertical seawalls, wave return wall, cantilevered walkway.

1. Introduction

The Water Research Laboratory (WRL) of the School of Civil and Environmental Engineering at UNSW Sydney completed two-dimensional (2D) physical modelling of a proposed vertical seawall in Darwin (Figure 1) to assist Wallbridge Gilbert Aztec (WGA) with their structural design of a cantilevered walkway slab to be secured to the top of the seawall. The seawall will be built along the foreshore to protect infrastructure associated with a new luxury hotel from erosion and wave overtopping. The walkway will provide the public with access to the foreshore seaward of the hotel. The objective of the physical modelling investigation was to identify the peak pressures and forces on the underside of the cantilevered walkway for two different cyclonic wave conditions and the water levels at which they occur.

Since the design water levels are close to the walkway level, it was anticipated that the maximum wave loads would occur at a lower part of the tidal cycle when there is a greater potential for generation of "impulsive" type wave impact loads. The distribution of pressures and total wave forces exerted upwards on the underside of the walkway were measured for two different design wave conditions with a range of water levels which may occur within a tidal cycle during a cyclonic event.

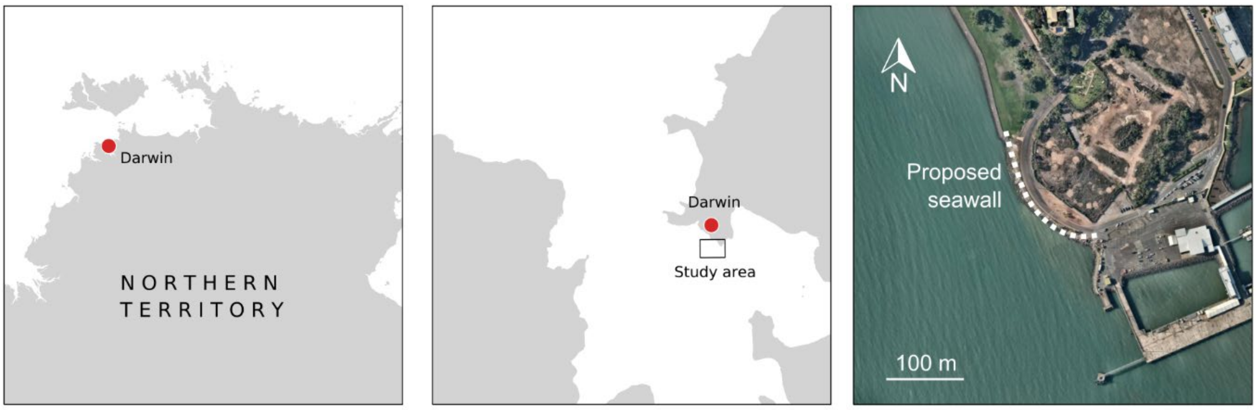

Figure 1 Location of proposed vertical seawall in Darwin

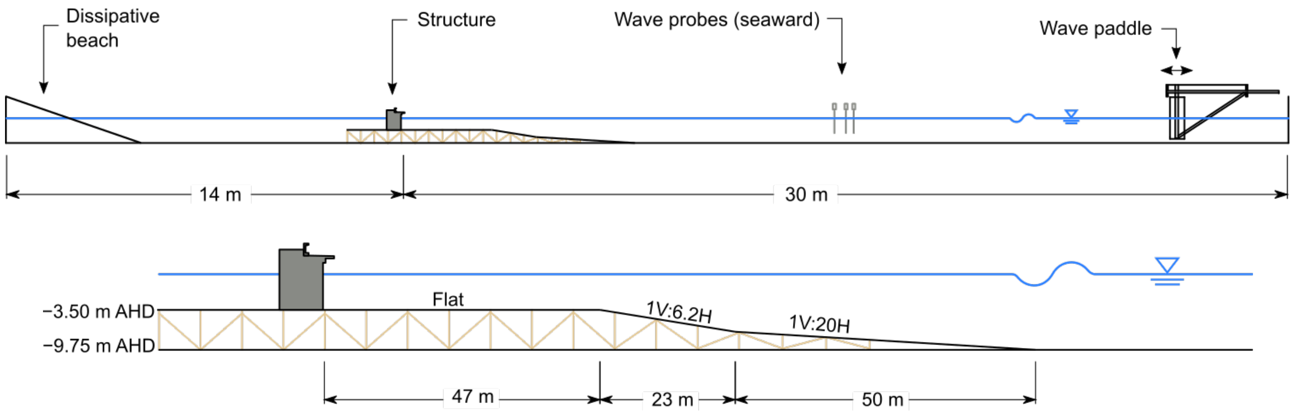

Figure 2 Model side views [structure not present during wave calibration] (top: whole flume [model scale], bottom: bathymetry detail [prototype scale])

2. Model design

2.1 Test facility

Two-dimensional modelling was completed using WRL's 1.2 m wave flume, which is 44 m long, 1.2 m wide and 1.6 m deep (Figure 2 top).

The wave flume was filled with fresh water rather than salt water to avoid corrosion of the hardware and to ensure responsible disposal of drained water. This is standard practice for almost all coastal hydraulic laboratories in the world [3].

2.2 Scaling

2.2.1 Overview

Model scaling was based on geometric similarity between model and prototype, using an undistorted Froude scale of 15. Selection of the length ratio was primarily based on the upper limit wave height able to be generated in the 1.2 m wave flume. Forces and pressures had an additional scaling factor (N_γ) to adjust for the different fluid densities between the model (fresh water; 998 kg/m^3) and prototype (salt water; 1024 kg/m^3). The scale ratios (prototype divided by model) derived for the study are summarised in Table 1. All quantities are reported in prototype scale, unless otherwise noted.

2.2.2 Comments on alternative scaling laws for force and pressure

Wave loads on vertical seawalls and their associated infrastructure (e.g. decks) can be divided between:

- Slowly acting loads, having durations of approximately 0.2 to 0.5 times a wave period, which are referred to as "pulsating" or "quasi-static" loads and are generally associated with non-breaking waves; and
- Short duration (approximately 0.01 times the wave period or less), high intensity loads, which are referred to as "impulsive" or "impact" loads and are generally associated with waves breaking directly on the structure which may entrap and compress an air pocket.

Table 1 Froude scaling factors

Quantity	Froude relation	Scaling factor
Length (m)	λ	15
Time (s)	$\lambda^{1/2}$	3.87
Force (kN)	$\lambda^3 N_\gamma$	3,463
Pressure (kPa)	$\lambda\, N_\gamma$	15.39

It is well accepted that "pulsating" or "quasi-static" loads can be scaled by the simple Froude relationships for force and pressure described in Table 1 with negligible scale effects [1]. However, use of Froude scaling for "impulsive" loads may lead to over-estimation of force and pressure at prototype (real-world) scale (refer to Section 2.2.2 in [2] for a more detailed discussion on this matter).

During the design storm events modelled for the Darwin seawall, individual waves generated both "pulsating" and "impulsive" vertical loads on the underside of the cantilevered walkway. In the absence of a reliable, alternative scaling relationship (with less conservatism) for short duration "impact" vertical loads on the underside of a horizontal overhang protruding from a vertical wall, WRL universally adopted Froude scaling for wave-generated force and pressure in the physical model.

2.3 Model construction

2.3.1 Vertical seawall

Most components of the vertical seawall were constructed from water-resistant plywood. The wave return wall on the landward side of the walkway was constructed from painted timber and the capping beam was constructed from painted PVC plastic sheet. All dimensions were consistent with provided design drawings except that the proposed piles in the prototype were constructed as a simplified smooth vertical wall in the physical model and the balustrade at the seaward edge of the walkway (underside elevation 5.0 m AHD) was omitted. The future earthworks level landward of the wave return wall was 6.5 m AHD and the scoured bed level adjacent to the seawall was assumed to be -3.5 m AHD.

2.3.2 Bathymetry

The seabed geometry adopted in WRL's 1.2 m wave flume was identical to that adopted in a previous physical modelling investigation of wave overtopping for the same structure at an earlier design phase [8], except that the scour level at the seawall was raised based on logging data from boreholes drilled at the site. The false floor was constructed from water-resistant plywood with the following characteristics (Figure 2 bottom):

- Intersected structure at -3.5 m AHD (flat for 47 m seaward from the toe of the seawall);
- 1V:6.2H slope from -3.5 to -7.26 m AHD; and
- Seaward of -7.26 m AHD the false floor sloped at 1V:20H until it intersected the permanent flume floor at -9.75 m AHD.

2.4 Instrumentation

A combination of capacitance wave probes, load cells, and pressure sensors were used during testing (Table 2, Figures 3 and 4). These instruments are typically selected to have a capacity slightly larger than the expected range of each physical quantity to maximise the accuracy of the measurements obtained. However, in this case preliminary estimates of maximum wave height, uplift force and pressure were not available at the time of model design, and as such WRL relied on previous experience from modelling of similar coastal structures to select the most appropriate instrumentation. High-speed oblique videos were recorded for each test.

Table 2 Instruments used in testing

Instrument	Sample rate (Hz)	Measurement Range
Wave probe	13	0 to 6.75 m wave height
Load cell (forces)	258	-1.73 to 1.73 MN (individual) -6.93 to 6.93 MN (total)
Pressure sensor	258	-150 to 700 kPa

A static calibration was performed on each instrument to ensure it was operating correctly across its full measurement range. A series of dynamic verification tests were also completed for the load cells. On the basis of these additional tests, a 10% uncertainty factor was applied to all force measurements to allow for accuracy limitations in the model setup (i.e. all reported forces were multiplied by 1.1).

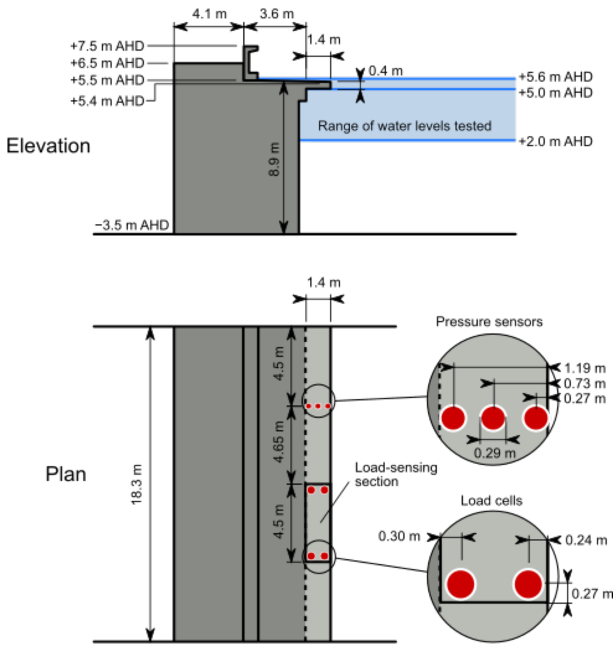

Figure 3 Model instrumentation and dimensions

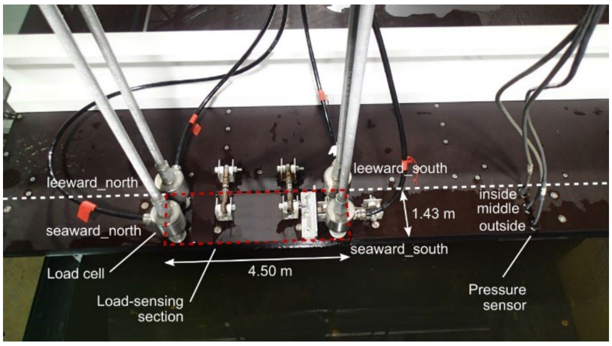

Figure 4 Top view of model sensor arrangement

2.5 Wave climates

Two different cyclonic design conditions [DC] (Table 3) were provided for the testing. Two corresponding irregular drive signals were generated using a JONSWAP spectrum, each with 1,000 waves to be statistically relevant. Two more lower water levels (2.75 m AHD and 4.0 m AHD) were included in addition to those in Table 3, to allow more space for waves to impact with the underside of the structure and potentially develop high uplift forces. The two (2) different drive signals and four (4) water levels yielded eight (8) different calibrated wave climates.

Table 3 Design conditions at the structure (-3.5 m AHD)

Description	SWL (m AHD)	H_S (m)	T_P (s)
Design condition 1	5.0	2.6	8.1
Design condition 2	5.6	2.0	8.7

The wave climates were calibrated with the bathymetry installed in the flume, but with the structure removed, to minimise wave reflections. Waves were measured using two (2) different three-probe arrays: one in deep water offshore; and one at the structure (Figure 2 top) where the design conditions were defined. Incident and reflected irregular wave trains were separated using the Mansard and Funke method [6] during post-processing analysis.

2.6 Test program

A total of 19 tests were completed (Table 4) to identify the water level (to within 0.1 m) at which the peak pressure and force occurred for both wave conditions. The four (4) calibrated drive signals for each design wave condition were used to determine the required drive signal gains by interpolation for other test water levels which were not calibrated prior to uplift testing.

Table 4 Test program

DC#	SWL (m AHD)*
1	2.0, 2.25, 2.4, 2.5, 2.6, **2.75**, 3.0 3.5, **5.6**
2	2.0, 2.5, 2.65, **2.75**, 2.82, 2.9, 3.0, 3.1, 3.5, **5.6**

Note – DC: Design condition
* Note – Bold SWL's were calibrated prior to uplift testing.

Table 5 Qualitative descriptions of waves at different water levels

Water level	Wave breaking	Overtopping of crown wall	Degree of uplift slamming on cantilevered walkway
Low (< 2.25 m AHD)	Offshore	None	Moderate
Intermediate	Onto structure	Extensive spray, and some green water on larger waves	Violent
High (> 5 m AHD)	Over structure	Green water for most waves	Minimal

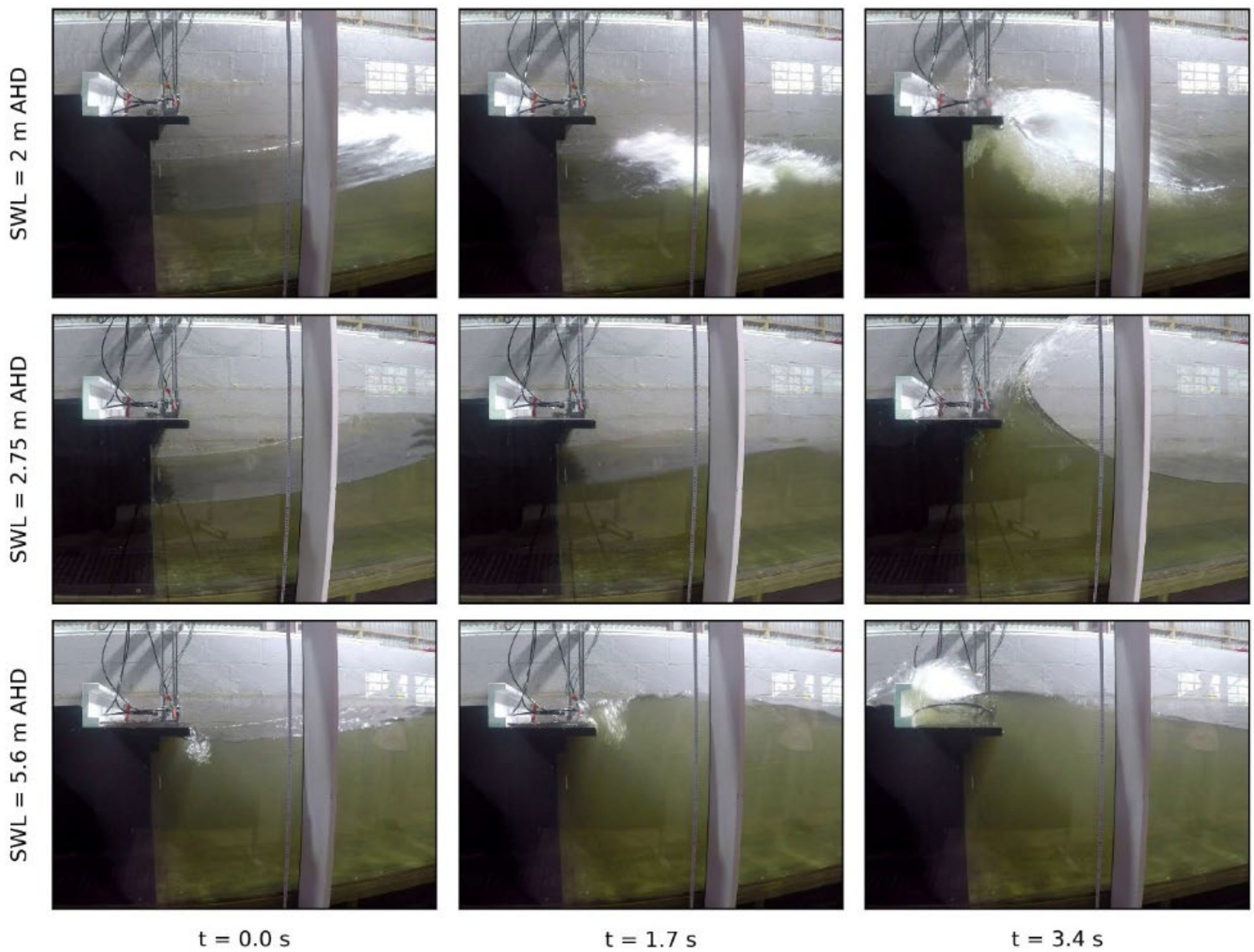

Figure 5 Behaviour of waves impacting on the structure at different water levels

3. Results

3.1 Video records

During testing, it became apparent that the maximum uplift forces were observed when the water level was between 2.4 m AHD and 3.0 m AHD. The oblique video footage was then used to describe the behaviour of waves at different water levels (Table 5, Figure 5).

3.2 Forces

Forces were measured at each corner of the load-sensing section of the cantilevered walkway (Figures 3 and 4), which represented a 4.5 m long section of the walkway. The values from the four (4) load cells were summed to obtain the total force, and the peak total uplift force for each wave was identified (Figure 6). Total force peaks less than 100 kN over the 4.5 m long walkway section were not included in the analysis.

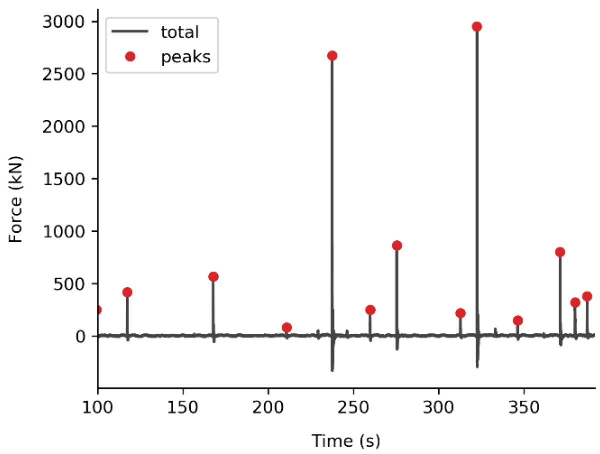

Figure 6 Identification of peak total uplift forces for Design condition 1 with 2.25 m AHD SWL

Two parameters were used to compare the peak uplift forces between tests F_{max} (the highest total force peak) and $F_{1/250}$ (the average of the four highest total force peaks recorded during each test of 1,000 waves).

3.3 Pressures

Pressures were measured at three locations on the underside of the fixed portion of the cantilevered walkway (Figures 3 and 4). Peak uplift pressures for each wave were identified for each sensor (Figure 7). Peak pressures less than 10 kPa were not included in the analysis. Measurements were provided as dynamic pressures where the hydrostatic (still water) and barometric (ambient atmospheric) pressure were subtracted from the absolute (observed) pressure.

Unfortunately, some of the larger waves caused uplift pressures beyond the measurement range of the selected pressure sensors (approximately 700 kPa). These large pressures only occurred for a short time (approximately 20 ms), but the peak values were clipped from the signal (Figure 8). When this clipping was first observed, WGA's structural engineers advised that their design would primarily be based on the load cell results rather than the pressure measurements. As such, testing was completed without acquiring alternative pressure transducers with a higher measurement range.

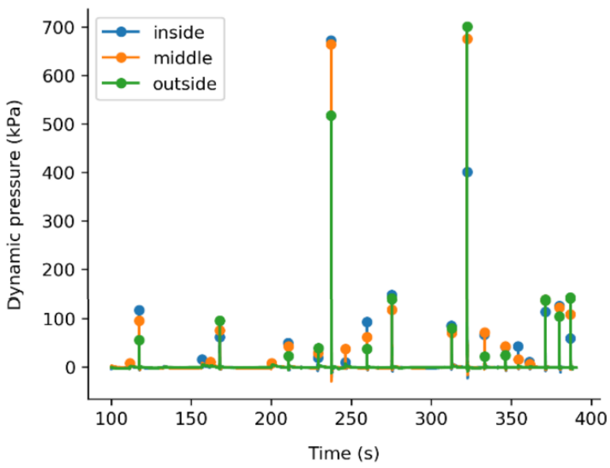

Figure 7 Identification of peak uplift pressures for Design condition 1 with 2.25 m AHD SWL

Since pressure clipping was observed in most tests, P_{max} was not a useful measure to compare different test results. The 97th percentile pressure (P_{97}) was used instead because clipping was not observed at or below this percentile for all tests.

3.4 Peak uplift forces and pressures

The maximum peak uplift forces and pressures were observed at water levels between 2.4 m AHD and 3.0 m AHD (Figures 9 and 10). For tests between these water levels, the typical total duration (rise and fall) of the force and pressure impacts was approximately 0.2 to 0.3 s.

Design condition 1 (H_S = 2.6 m, T_P = 8.1 s) recorded the peak uplift forces at water levels of 2.4 to 2.5 m AHD and Design condition 2 (H_S = 2.0 m, T_P = 8.7 s) had maximum uplift at the 3.0 m AHD water level. This difference in water level is likely to have occurred because Design condition 2 had a lower wave height and so a higher water level was required to achieve similar uplift dynamics. Regardless of this, the maximum recorded uplift force on the underside of the cantilevered walkway was nearly identical for Design condition 1 (4,280 kN) and Design condition 2 (4,293 kN). Averaged over the 6.44 m^2 area of the load sensing section, these maximum forces correspond to inferred uplift pressures of approximately 670 kPa. This corresponds well with the clipped peaks of the individual pressure transducers (700 kPa) which had a much smaller surface area of 0.07 m^2.

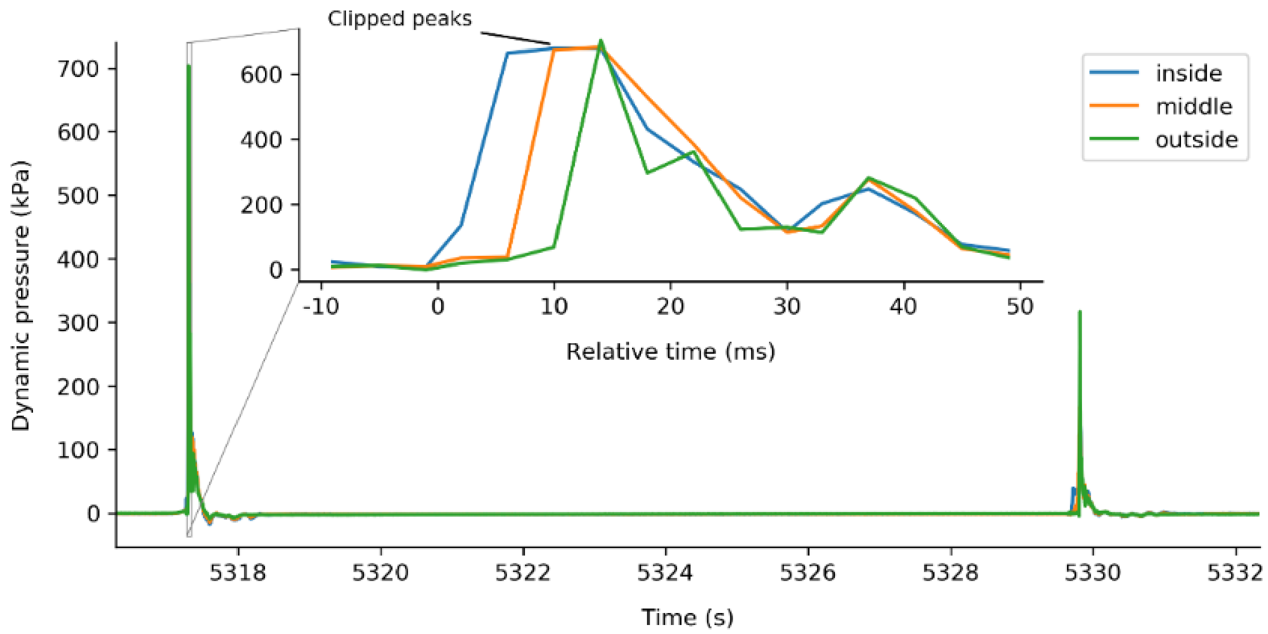

Figure 8 Example clipping of peak pressures for Design condition 1 with 2.5 m AHD SWL

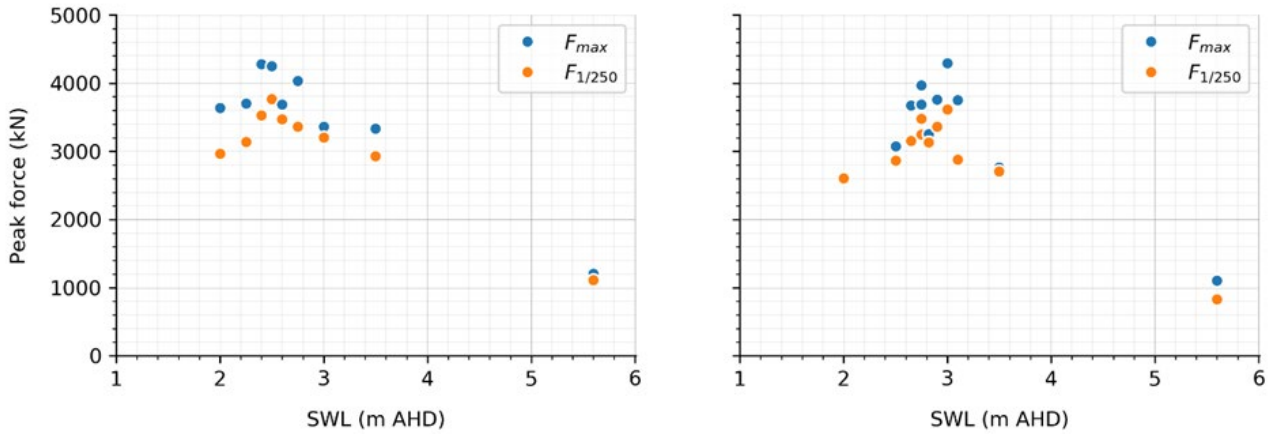

Figure 9 Peak force values (F_{max} and $F_{1/250}$) for different water levels (left: Design condition 1, right: Design condition 2)

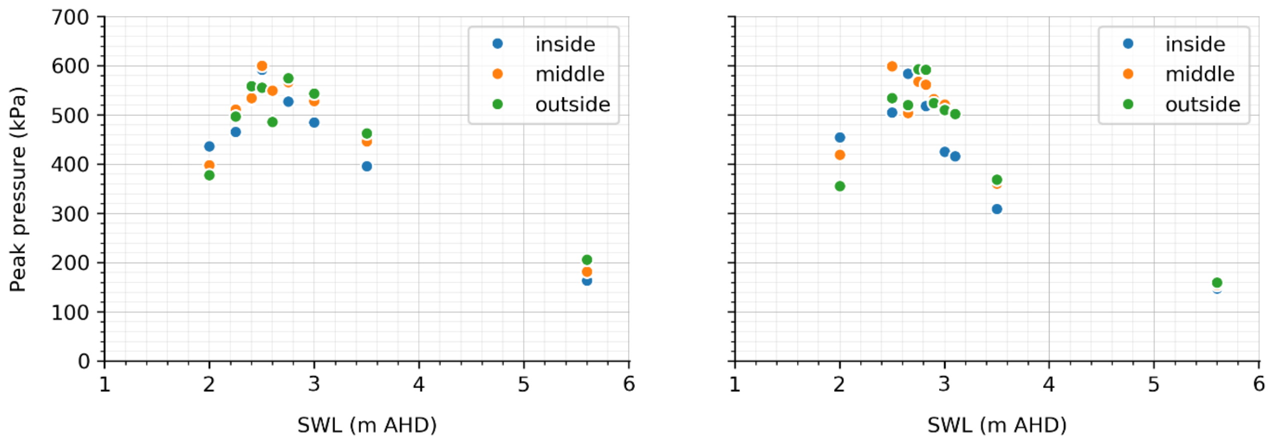

Figure 10 Peak pressure values (P_{97}) for different water levels (left: Design condition 1, right: Design condition 2)

4. Comparison with an empirical design technique

Following the conclusion of the physical modelling program, WRL compared the maximum recorded uplift force on the underside of the cantilevered walkway with an empirical design approach published in a series of papers based on physical modelling tests of vertical seawalls with horizontal overhangs [4, 5 and 7].

Maximum uplift forces on the cantilevered walkway proposed for Darwin were estimated for both design conditions via the following steps:

- Calculation of horizontal impact forces on a vertical seawall without the overhang and without reduction in loading due to overtopping using Equation 3 from [4] (it is acknowledged that the input H_S values (2.0 and 2.6 m) are outside the stated range of validity up 1.2 m);
- Then, increasing these horizontal impact forces by a factor of 1.8 due to the presence of the overhang [5]; and
- Finally, increasing these last horizontal impact forces with overhang by 1.03 (based on a "medium recurve" described in [7]) to calculate the vertical uplift force on the overhang.

Using this approach, the empirical estimates for maximum uplift forces on the Darwin cantilevered walkway were approximately 6,200 kN (Design condition 1) and 3,700 kN (Design condition 2). These results are in the same order of magnitude as that measured in the physical model (approximately 4,300 kN for both design conditions). However, the empirical technique overestimated the maximum uplift forces for Design condition 1 by 45% and underestimated those for Design condition 2 by 15%.

These findings may be considered when choosing whether to establish uplift forces for structural design of similar coastal structures by empirical techniques or physical modelling.

5. Conclusion

WRL completed 2D physical modelling of a proposed vertical seawall with a cantilevered walkway in Darwin, and measured uplift forces and pressures on the underside of the walkway deck to assist structural design. The key objective of the model investigation was to estimate the maximum wave-generated uplift loads on the walkway (underside elevation 5.0 m AHD) for two different design wave conditions, and to understand the influence of water level on the resulting wave loads.

Two different design wave conditions were tested (H_S = 2.6 m, T_P = 8.1 s and H_S = 2.0 m, T_P = 8.7 s) at multiple water levels between 2.0 m AHD and 5.6 m AHD. The largest uplift forces and pressures were observed at water levels between 2.4 m AHD and 3.0 m AHD. The largest maximum peak force (F_{max}) measured was 4,293 kN, on a movable plate with area 6.44 m^2 (approximately 670 kPa). The maximum peak pressures could not be accurately measured, because they exceeded the measurement range of the pressure sensors (approximately 700 kPa).

Note that while all tests were conducted with a scour level of -3.5 m AHD, it is conceivable that similar peak forces and pressures on the underside of the cantilevered walkway may occur for higher sand levels at the vertical seawall.

An empirical technique has been demonstrated to provide "order of magnitude" estimates of maximum uplift forces on a cantilevered walkway above a vertical seawall. If it is advantageous to reduce the uncertainty associated with empirical estimates of wave uplift for similar coastal structures, 2D physical modelling is recommended.

6. References

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