Wave Forces and Overtopping on Stepped Seawalls

B Modra[1], I Coghlan[1], J Carley[1], G Blumberg[2], W Boyd[3]
[1]Water Research Laboratory (WRL), School of Civil and Environmental Engineering, UNSW Australia, Manly Vale, NSW
[2]Haskoning Australia (HKA), North Sydney, NSW
[3]Tweed Shire Council (TSC), Murwillumbah, NSW

Abstract

Stepped seawalls are popular in NSW, and are often the hub that connects the beach to foreshore parks, promenades, surf clubs and other amenities. They are also often the most popular places for the public to observe storm waves, so their design must also consider public safety.

The design of stepped seawalls is complex – with relatively smooth and impermeable slopes they are vulnerable to overtopping and wave forces. Designs must balance the need for a high crest elevation without impacting views. Amenity, pedestrian friendly slopes, wave return walls, structural strength and construction costs all need to be considered.

Physical model testing by WRL for a proposed seawall at Kingscliff Beach included detailed testing of both overtopping and wave forces. In addition to the standard measurement of mean overtopping rate, the testing also measured overtopping bore velocity and depth to provide an enhanced insight into the hazards to pedestrians and landward infrastructure.

The testing considered a base design, and adaptive upgrade options in the case of sea level rise or other changes inducing unacceptable overtopping. Adaptive options included raising the effective crest elevation with the addition of a wave return wall and/or an extra step. These options alter the wave impact forces on the structure.

This paper presents the results and outcomes of the testing, and compares the performance of the stepped seawall against a rock seawall. It presents these results against guidelines for safe overtopping as mean rate, and compares depth and velocity of the overtopping bores against tests of pedestrians in flood flows.

With stepped seawalls generally constrained to a standard step slope, the results of this testing will be applicable to many current and potential stepped seawalls in NSW.

Introduction

Stepped seawalls are popular in NSW, and are often the hub that connects the beach to foreshore parks, promenades, surf clubs and other amenities. They feature along many beaches across NSW and continue to be added to new protection works.

They are however a relatively expensive protection measure, and requires careful consideration of wave forces, overtopping, concrete and reinforcement design, piling, groundwater flows, geotechnical constraints, toe scour, safety, amenity, aesthetics and others.

Most stepped seawalls follow the same general profile, with risers at a comfortable seating height and constrained by geotechnical considerations and standards for safe step dimensions. This results in a narrow range of seawall slopes and step heights. As such, a detailed analysis of wave loading and overtopping at a reference site will be broadly applicable at other locations. The results of physical model testing of a proposed seawall at Kingscliff Beach are provided here to aid in the future design of stepped seawalls.

Tweed Shire Council (TSC) engaged the Water Research Laboratory (WRL) of the School of Civil and Environmental Engineering at UNSW Australia to undertake a detailed concept design for the Kingscliff Beach Terminal Seawall. Haskoning Australia (HKA) was provided additional specialist design input on the structural aspects of the seawall.

Physical modelling formed an integral part of the design process, allowing rapid assessment of the structures performance and a high level of optimisation of both seawalls. This aided in achieving the best possible outcome within the design constraints.

This paper addresses the results and outcomes of the physical modelling of the stepped seawall only. The full conceptual design is provided in *Kingscliff Beach Foreshore Protection Works Part B - Detailed Concept Terminal Seawall Design* (Coghlan et al., 2016b).

Kingscliff Beach Seawall Concept Design

The proposed seawall is to protect the Kingscliff Beach Holiday Park and adjacent foreshore park from coastal storms. Kingscliff Beach is subject to episodic erosion and accretion events, and erosion has previously caused damage in the area and threatened the holiday park.

The seawall concept consists of a greywacke rock seawall spanning 274 m and a stepped concrete seawall spanning 144 m. The proposed stepped seawall consists of 450 mm (V) by 1,000 mm (H) steps to 5.0 m AHD. The structure is supported on piles to bedrock and protected from scour at the toe with a secant pile wall to -5.0 m AHD (Figure 1). Additional designs were tested with an added step to raise the crest to 5.45 m AHD, together with a 1 m return wall adaptation, due to high overtopping rates measured in the original tests.

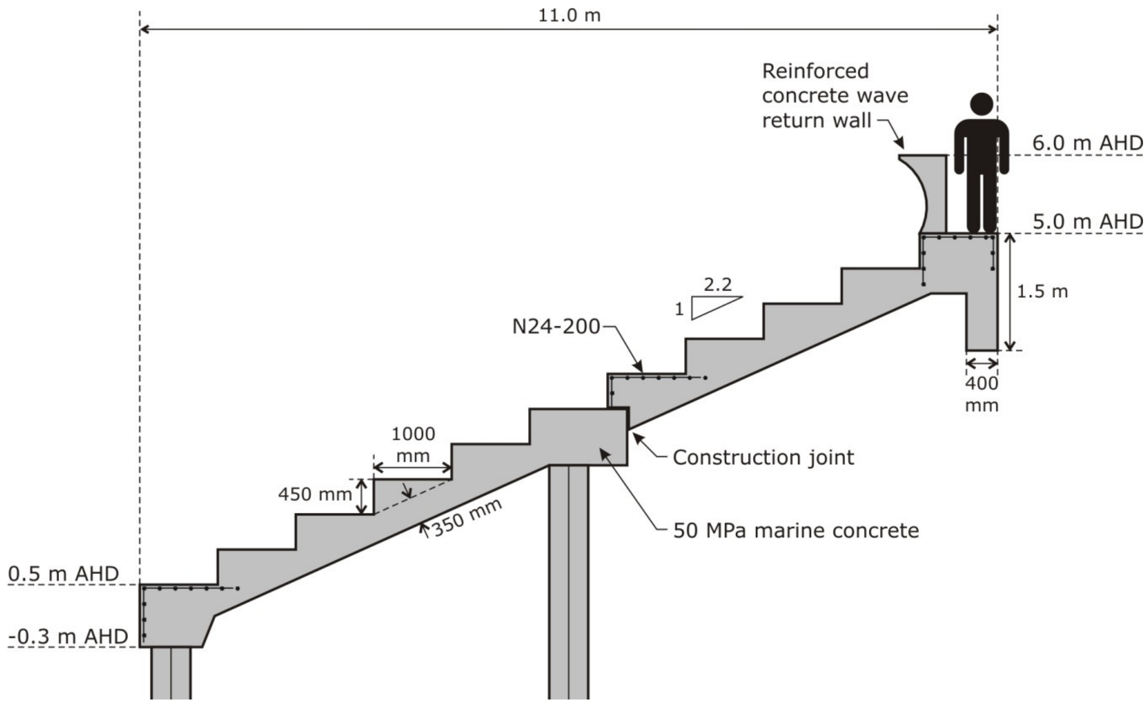

Figure 1: Recommended stepped concrete seawall design for Kingscliff Beach, including 1 m return wall for post-construction adaptation

Adaptation

The inclusion of adaptive responses is an important consideration in contemporary coastal design. While projected sea level rise indicates an increasing hazard, other factors such as changes to storm behaviour means there still remains a level of uncertainty in the timing and magnitude of risk to coastal infrastructure. This is particularly pertinent to structures with a short design life and which are not required to protect critical infrastructure.

To allow for the potential for increased risk to the structure without overdesigning, the seawall was designed for current extreme conditions, but also to withstand the increased forces associated with the addition of a return wall at the crest. It is anticipated that if overtopping becomes problematic at any time within the design life, it will be relatively simple to add the tested return wall to the structure.

Physical Modelling

The physical modelling included an assessment of the performance of a number of rock rubble and stepped concrete seawall design options at a scale of 1:45. The assessment included rock stability, overtopping performance and wave forces.

The stepped seawall was constructed from marine grade plywood as a single unit spanning the flume (54 m prototype width), from -2 m AHD at the secant pile toe, to approximately 11.9 m landward of the top step. A 1 m high return wall was added for the adaptive modification to the structure. A 0.45 m step was added to the top of the structure to test the 5.45 m crest alternate design. The stepped seawall model was constrained with an array of hinged rods such that horizontal forces could be measured by two force transducers attached to each end of the structure section. Figure 2 and 3 show the model under test conditions.

Figure 2: Stepped concrete seawall - Configuration B (6 m AHD Crest, with wave return wall) during 10 year ARI event, present day

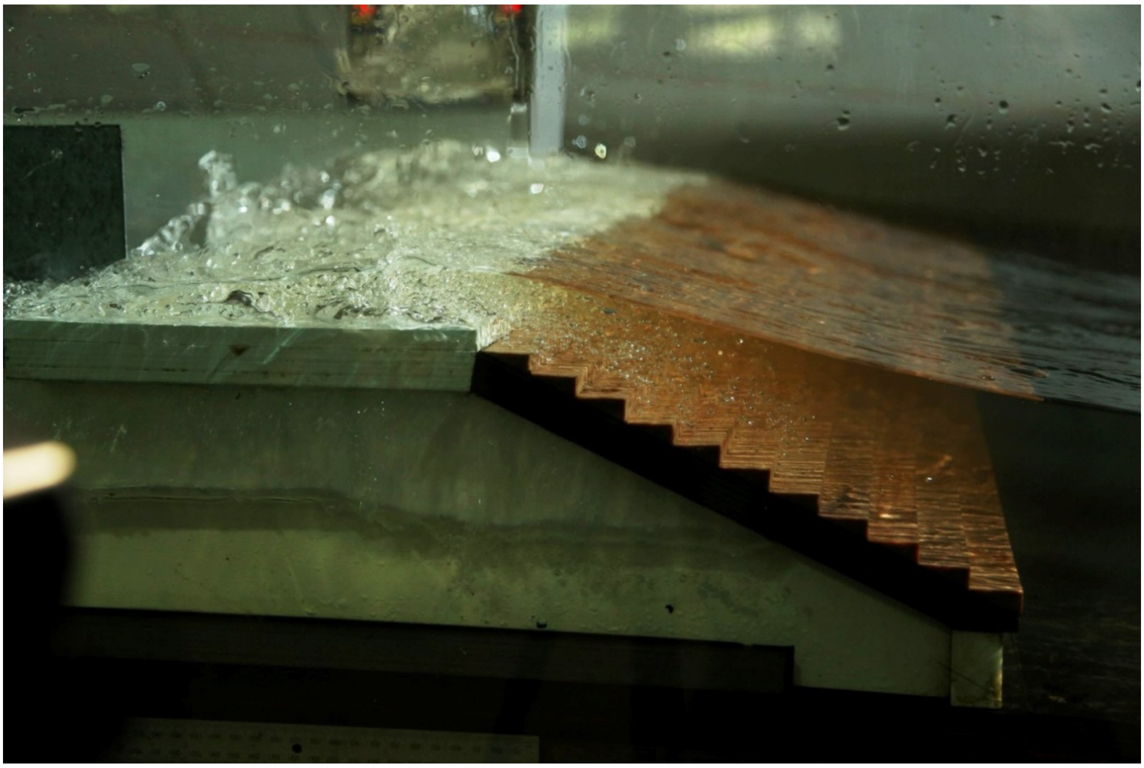

Figure 3: Stepped concrete seawall - Configuration C (5.45 m AHD Crest, no wave return wall) during 500 year ARI event, present day

Design Conditions

Details of the derivation of the design conditions for wave, sea level and sour are provided in the technical reports for the concept design (Coghlan, 2016a and Coghlan, 2016b). A summary of the conditions tested in the modelling program are provided in Table 1.

Table 1: Design Storm Conditions

Nominal Storm Event	**Wave Condition**		**Water Level Condition**			**Scour Level**
ARI (years)	**H_S** (m)	**T_P** (s)	**Basis**	**Elevation** (m AHD)		(m AHD)
				2016	2066	
1	5.2	11.4	Mean High Water (MHW)	0.45	0.99	1.0
10	6.7	12.3	Highest Astronomical Tide (HAT)	1.14	1.68	-0.5
100	8.1	13.1	100 year ARI	1.58	2.12	-2.0
500	9.1	13.6	500 year ARI	1.60	2.14	-2.0

Overtopping

During storm events, wave overtopping of the seawall crest is likely to occur in the form of bores of water being discharged inland or splashes of water being projected upwards and eventually transported inland by onshore winds. Wave overtopping can cause serious structural damage to the seawall crest and to dwellings immediately behind the seawall. Overtopping is also a direct hazard to pedestrians and vehicles on or near the seawall during storm events. As such it forms an important component of the physical modelling.

Overtopping is commonly characterised by the mean overtopping rate, which is the average overtopping volume over a long period. Overtopping flows were collected in the model using a catch tray placed leeward of the crest. Low flow rates were measured by weighing the total overtopping volume. High overtopping rates were pumped through a volumetric flow meter back into the flume, but were limited to approximately 270 L/m/s (prototype) which is sufficient to assess the highest hazard thresholds.

In addition to the mean overtopping rate, individual overtopping bores were also measured. This was done using ultrasonic sensors located 5 m and 10 m leeward of the crest. This provides information on the depth and shape of the bore, and the bore velocity by measurement of the time taken for the bore to traverse the distance between sensors. Characterising the overtopping bore allows a deeper understanding of the overtopping hazard and allows for comparison with other guidelines for pedestrian safety such as Australian Rainfall and Runoff (2016). The results have been reported as "typical bore velocity" and "typical bore depth", which is analogous to the significant wave height, where the highest third of measurable bores have been averaged.

Overtopping Results

Test results for mean overtopping rate and bore characteristics for the stepped seawall are provided in Table 2 for 1 and 10 year ARI events only. The overtopping rates have been correlated to guideline thresholds for pedestrian safety, vehicle stability and property damage provided in EurOtop (2007), see Table 3. The complete set of overtopping results are provided in Table 4.

This indicates that the overtopping of the proposed seawall is at safe levels for the majority of the time. However, unsafe overtopping will occur in moderate storm conditions with an annual recurrence interval between 1 and 10 years. Overtopping results for the rock seawall section (D_{50} = 5 t; 1:2 slope; 5 mAHD crest elevation) are also provided for comparison.

Testing demonstrates that the rock seawall with the same crest level and similar slope provides significantly less overtopping, with rates approximately two orders of magnitude lower than the stepped seawall for identical storm conditions and bathymetry. Similarly the bore depth and velocity are much lower. This is not surprising as the permeability of rock and the roughness of the large units are significant factors in reducing runup. Conversely, the stepped seawall provides an impermeable, relatively smooth slope for the waves to run up.

This has clear implications for the selection of protection types. Stepped seawalls must be higher (with corresponding increase in footprint) to provide the same safety levels as a rock seawall. Alternatively, lower stepped seawall crests can be used where pedestrian safety can be managed and other equipment or structures are not at risk.

For the proposed Kingscliff Beach works, keeping the crest low is highly desirable as it maintains visibility to the ocean from the foreshore area, and crest levels are proposed to be 5.0 m AHD for both rock and stepped seawalls. However, the rock section protects the Kingscliff Beach Holiday Park where overtopping would otherwise be a hazard to lightweight cabins, caravans, vehicles and minor infrastructure. Of particular concern is the potential for storm events and overtopping to occur at night when people could be sleeping at the Holiday Park. Using a rock seawall within the crest level constraints allows a significant reduction in the overtopping risk for this area.

The stepped seawall is backed by the grassed foreshore area. While there is a risk to pedestrians at the crest during a storm, it can be reasonable assumed that those in the vicinity are observing the storm and responding to the immediate hazard. There is little infrastructure in the lee of the stepped seawall and irregular damage to landscaping is tolerable.

The adaptation design, with the addition of a 1.0 m return wall to account for 0.54 m of sea level rise provided a marked decrease in the overtopping rate, with a reduction of approximately two orders of magnitude compared with the design without the return wall. The wave return wall may be added in the case that sea level rise drives overtopping events to become unacceptably damaging to the foreshore area, or the pedestrian safety hazard too great.

A review of the wave bore measurements indicate that the bore changes rapidly as it propagates shoreward, becoming shallow and broader as it travels. Observations of the bore during testing indicate that that the bore is at its deepest at the seaward edge of the crest so will be deeper than that indicated by Table 2. All measurable bore velocities were higher than for the safe limit for pedestrians in flood waters recommended by Australian Rainfall and Runoff (2007). This recommends that flows of any depth above 3 m/s are unsafe for pedestrians in good conditions. This is also broadly consistent with EurOtop (2007) which indicates that tolerable horizontal velocities for pedestrians and vehicles are less than 2.5 - 5 m/s.

Table 2: Comparison of Mean Overtopping Rate Measurements with other EurOtop Thresholds

Terminal Seawall Section	Storm ARI (years)	Planning Period (Water Level)	Q (L/s per m)	Hazard Code: Aware Pedestrians	Hazard Code: Equipment	Hazard Code: Building Elements	Hazard Code: Trained Staff	Hazard Code: Vehicles (Low Speed)	Typical: Bore Depth (m)	Typical: Bore Velocity (m/s)
5 t Rock 1V:2.0H (5 m AHD Crest, No Wave Return Wall)	1	Present Day	**0**						-	-
	10		**0.028**						0.03	3.7
	1	2066	**0**						-	-
	10		**3.4**						0.07	6.0
5 t Rock 1V:2.0H (6 m AHD Crest, With Wave Return Wall)	1	Present Day	**0**						-	-
	10		**0.008**						0.03	3.4
	1	2066	**0**						-	-
	10		**0.21**						0.03	4.7
Stepped Concrete (5 m AHD Crest, No Wave Return Wall)	1	Present Day	**0.02**						<0.03	N/A
	10		**34.9**						0.59	8.0
	1	2066	**1.36**						0.18	8.7
	10		**141**						1.22	16.8
Stepped Concrete (6 m AHD Crest, With Wave Return Wall)	1	Present Day	**0**						-	-
	10		**2.95**						0.14	6.7
	1	2066	**0.01**						<0.03	N/A
	10		**22.2**						1.35	10.7

Table 3: Classification of Wave Overtopping Hazard for People, Vehicles and Property. EurOtop (2007)

Hazard Type	Mean Overtopping Discharge Limit, *Q* (L/s per m): Safe/No Damage	Marginal	Unsafe/Damage
Aware Pedestrians	<0.1	N/A	≥0.1
Equipment Setback 5-10 m	<0.4	N/A	≥0.4
Building Structure Elements	<1	N/A	≥1
Expectant Trained Staff	<1	1-10	>10
Vehicles at Low Speed	<10	10-50	>50

Table 4: Summary of Stepped Concrete Terminal Seawall Overtopping Tests Mean Overtopping Rates

Config.	Wave Return Wall?	Crest Elevation (m AHD)	Storm ARI (years)	Planning Period (Water Level)	Mean Overtopping Rate (L/s per m)
A	No	5.00	1	Present Day	**0.02**
			10		**34.9**
			100		**268**
			500		**>277**
			1	2066	**1.36**
			10		**141**
			100		**>278**
			500		**>300**
B	Yes	6.00	1	Present Day	**0.00**
			10		**2.95**
			100		**79**
			500		**123**
			1	2066	**0.01**
			10		**22.2**
			100		**212**
			500		**247**
C	No	5.45	10	Present Day	**19.7**
			100		**200**
			500		**247**
			100	2066	**>282**
			500		**>300**
D	Yes	6.45	100	Present Day	**63**
			100	2066	**149**

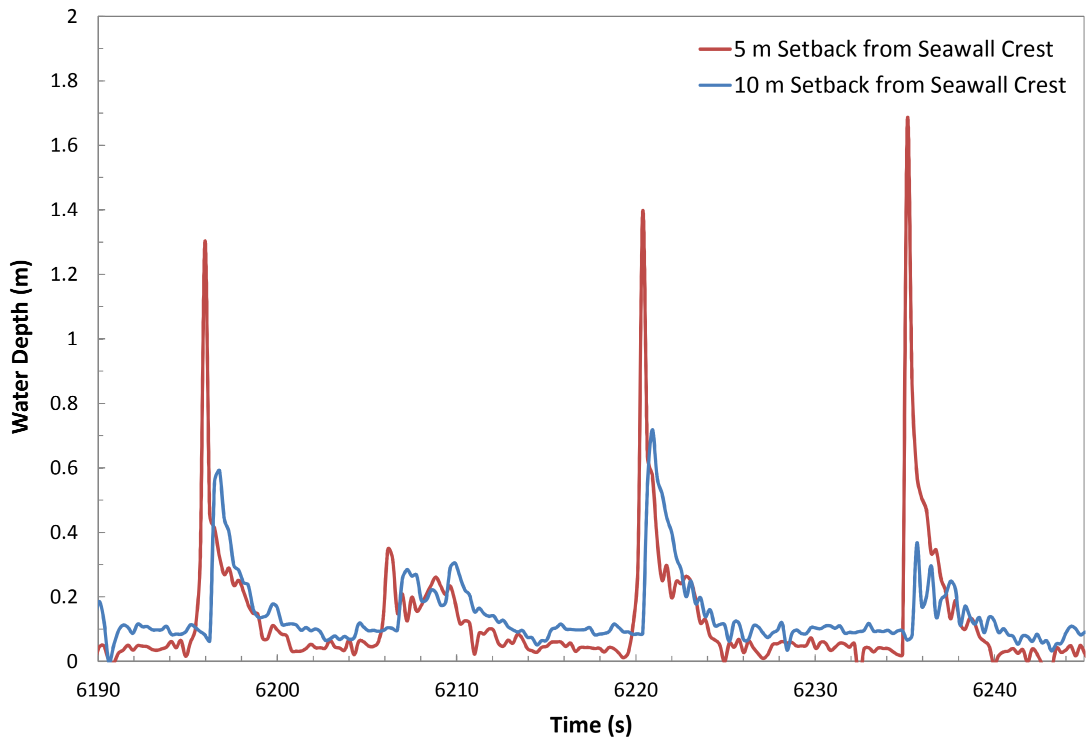

Figure 4: **Sample of Wave Overtopping Bore Measurements 5 and 10 m Setback from Crest for Configuration A (no Wave Return Wall) during 10 year ARI event with Sea Level Rise. Average Overtopping rate = 141 L/s/m, Typical Bore Depth = 1.22 m, Typical Bore Velocity = 16.8 m/s**

Wave Forces

Wave loading is a critical design parameter in the structural design of the seawall. The wall and its foundation must be able to withstand the largest waves likely on the structure. Horizontal wave impact forces on stepped concrete seawall section were measured in the model to provide maximum loading rates to inform the design.

Hinged rods connected between the stepped seawall model and the flume floor were positioned to prevent vertical and lateral movement of the seawall, and prevent twisting of the model in pitch and roll. Two force transducers located at each side of the model measured the longitudinal (horizontal) wave forces and prevented twisting in the yaw axis. Forces were sampled at 500 Hz (74.5 Hz prototype scale) to allow measurement of transient and impact forces on the structure. The accuracy of the load sensing arrangement was assessed to be within 15% of the measured load, and an equivalent adjustment was made to the recommended design loads.

Wave Force Results

Results of the wave impact force testing are provided in Table 5. A sample of the force measurement tests are provided in Figure 5.

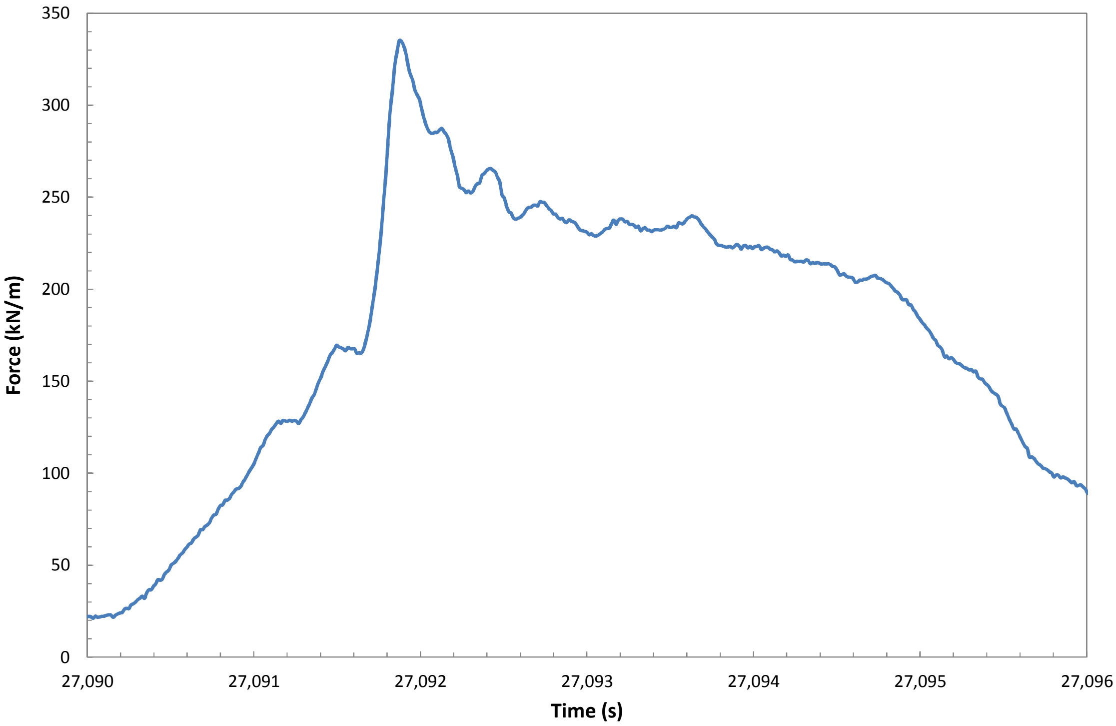

Figure 5: **Sample of Maximum Load Measured on the stepped concrete seawall section for Configuration B (w/ Wave Return Wall) during 100 year ARI event with Sea Level Rise (Test #23)**

?? replace with trace of with & without return wall

With the recommended increase factor of 15%, the highest landward unit force on Configuration B for the 100 year ARI event was 335 kN/m and the highest landward unit force on any test (Configuration B, 500 year ARI) was 662 kN/m.

The wave return wall provides a marked reduction in overtopping rates, but increases peak wave loads by 2-3 times. This is because the act of changing the wave momentum from shoreward to seaward and upwards provides a reaction force in the structure.

While the loads were measured on the structure as a whole, it can be reasonably inferred that the loading experienced at and near the return wall can be approximated by the difference in loading with and without the return wall in place. This gives guideline values for the loading expected on the return wall, though a robust factor of safety would be required without specifically testing the loads on the return wall.

The difference in impact load on the seawall due to the return wall is significant for adaptive structure designs. The original construction must be designed to withstand the higher loading associated with the return wall, even though the return wall is not part of the initial construction. Additional strengthening associated with the addition of the return wall would otherwise be expensive or result in sub-optimal outcomes.

Table 5: Summary of Stepped Concrete Terminal Seawall Wave Force Tests – Maximum Unit Force

Configuration	Wave Return Wall?	Crest Elevation (m AHD)	ARI (years)	Water Level Condition	Maximum Instantaneous Force[1], 0.01 s Duration (kN/m)	
					Raw Test Values	**Design Values** (Raw Test Values Increased by 15%)
A	No	5.00	1	Present Day	56	65
			10		115	132
			100		171	196
			100		165	189 (repeat)
			500		164	189
			1	2066	81	93
			10		147	169
			100		186	214
			500		195	224
B	Yes	6.00	1	Present Day	61	70
			10		133	153
			100		272	313
			500		576	662
			1	2066	95	110
			10		190	218
			100		291	335
			500		493	567
C	No	5.45	10	Present Day	118	136
			100		146	168
			500		183	210
			100	2066	175	201
			500		232	267
D	Yes	6.45	100	Present Day	235	270
			100	2066	318	365

1. WRL recommends increasing the raw, force test values by 15% for design purposes to allow for measurement uncertainty in this test arrangement.

Conclusions

Physical modelling of a concrete stepped seawall was conducted to determine overtopping rates and wave impact forces for a proposed seawall at Kingscliff Beach, NSW. This was performed as part of a broader study to develop a concept design for the protection works.

Wave impact forces determined during the testing provided valuable input to the structural design of the structure. It was noted that a 1 m high wave return wall could increase the wave loads by up to 2-3 times that encountered by a structure without a return wall. This provides additional input to the loads expected at the structure should adaptation to changing sea levels and storm conditions be required.

Testing of the overtopping of the structure provided an accurate assessment of the overtopping hazard likely to be encountered over the life of the structure. This indicates that overtopping is likely to be low for the majority of the time, with moderate storms presenting a hazard to pedestrians and the landscaping in the lee of the structure.

This paper further demonstrates the value of physical modelling as a design tool to provide accurate wave loading parameters, as well as providing a high level of optimisation where there are conflicting objectives, such as minimising both crest height and overtopping.

References

Smith G, and Cox R (2016) Safety Design Criteria, Book 6 in Australian Rainfall and Runoff - A Guide to Flood Estimation, Commonwealth of Australia.

CIRIA; CUR; CETMEF (2007), *The Rock Manual. The Use of Rock in Hydraulic Engineering (2nd edition).* C683, CIRIA, London.

Coghlan, I R, Carley, J T, Shand, T D, Blacka, M J, Cox, R J, Davey, E K and Blumberg, G P (2016a), *Kingscliff Beach Foreshore Protection Works: Part A – Alternative Terminal Seawall Designs and Beach Nourishment*, WRL Technical Report 2011/25, Final, February.

Coghlan, I R, Carley, J T, Modra, B D, Guerry, N, (2016b*) Kingscliff Beach Foreshore Protection Works Part B - Detailed Concept Terminal Seawall Design*, WRL Technical Report 2014/15, Final, September.

EurOtop (2007) *Wave Overtopping of Sea Defences and Related Structures: Assessment Manual*, Environment Agency (UK), Expertise Netwer Waterkeren (NL), and Kuratorium fur Forschung im Kusteningenieurwesen (DE).