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HINDSIGHT IS 20/20? A REVIEW OF PRELIMINARY EMPIRICAL SEAWALL DESIGN AT KINGSCLIFF BEACH AFTER PHYSICAL MODELLING

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ABSTRACT

During the design process for a seawall, preliminary structural geometry is rapidly estimated using a range of empirical techniques. These accepted techniques are generally intended to be conservative, resulting in an initially over-designed structure. This geometry is subsequently refined using physical modelling to ensure hydraulic stability (without over-conservatism) and reduce the risk of the design criteria being exceeded during its working life. This paper presents the designs developed both empirically and through physical modelling for a seawall located at the back of a sandy beach on the Australian East Coast. The authors collectively acted as both designer and physical modeller for the client, providing the unique opportunity to review the assumptions inherent in their own preliminary empirical design following the conclusion of a physical modelling program.

While the use of physical modelling for coastal engineering projects is almost always recommended, commentary is provided as to which processes were reasonably well described empirically for this case study and which processes introduced over-conservatism throughout the ensuing preliminary design. Given the same offshore wave and still water level boundary conditions as the physical model, inaccuracies in the preliminary design water depth at the structure were found to have significant impacts on subsequent design parameters.

KEWORDS: empirical techniques, wave setup, armour stability, wave overtopping.

1 INTRODUCTION

A rubble mound seawall is proposed to be constructed at the back of a sandy beach in eastern Australia to protect a section of coastline with built assets at immediate risk from coastal hazards. For the case study site at Kingscliff Beach on the far north coast of NSW (Figure 1), ongoing erosion in the last decade has resulted in substantial loss of beach amenity and community land.

The authors were engaged by Tweed Shire Council to prepare a conservative preliminary design, using a range of empirical techniques, for the proposed rock seawall. Following the development of this preliminary design, a physical modelling program was used to refine the structural geometry to ensure that the final design is hydraulically stable (without unnecessary conservatism), to confirm capital construction costs and reduce the ongoing risk of the design criteria being exceeded during its working life. During the physical modelling program, the authors collectively acted as both designer and physical modeller for the client, providing the unique opportunity to review the assumptions inherent in their own preliminary empirical design following the conclusion of the physical modelling program. While the methodology of the physical modelling program followed established practices, its value lies in verifying the empirical estimates.

Kingscliff Beach is located on the open coast at the southern end of a long bay with littoral drift transport generally northward but occasionally southward resulting in net annual longshore sand transport of approximately 520,000 m^3/year northward (Patterson, 2007). Tides are classified as semi-diurnal, microtidal with a mean spring tide range of 1.20 m. The dominant wave generation sources include a range of cyclone types and onshore sea breezes. The beach is composed of fine to medium sand with a median grain size of 0.30 mm, which forms an energetic double bar surf zone and numerous rips (Short, 2007). A 100 year average recurrence interval (ARI) storm event is expected to erode 200 m^3 of sand above mean sea level per metre of shoreline (BMT WBM, 2013).

Unless otherwise specified, data represented are given in prototype equivalent units. Reduced levels refer to the present day, local Mean Sea Level (MSL) datum.

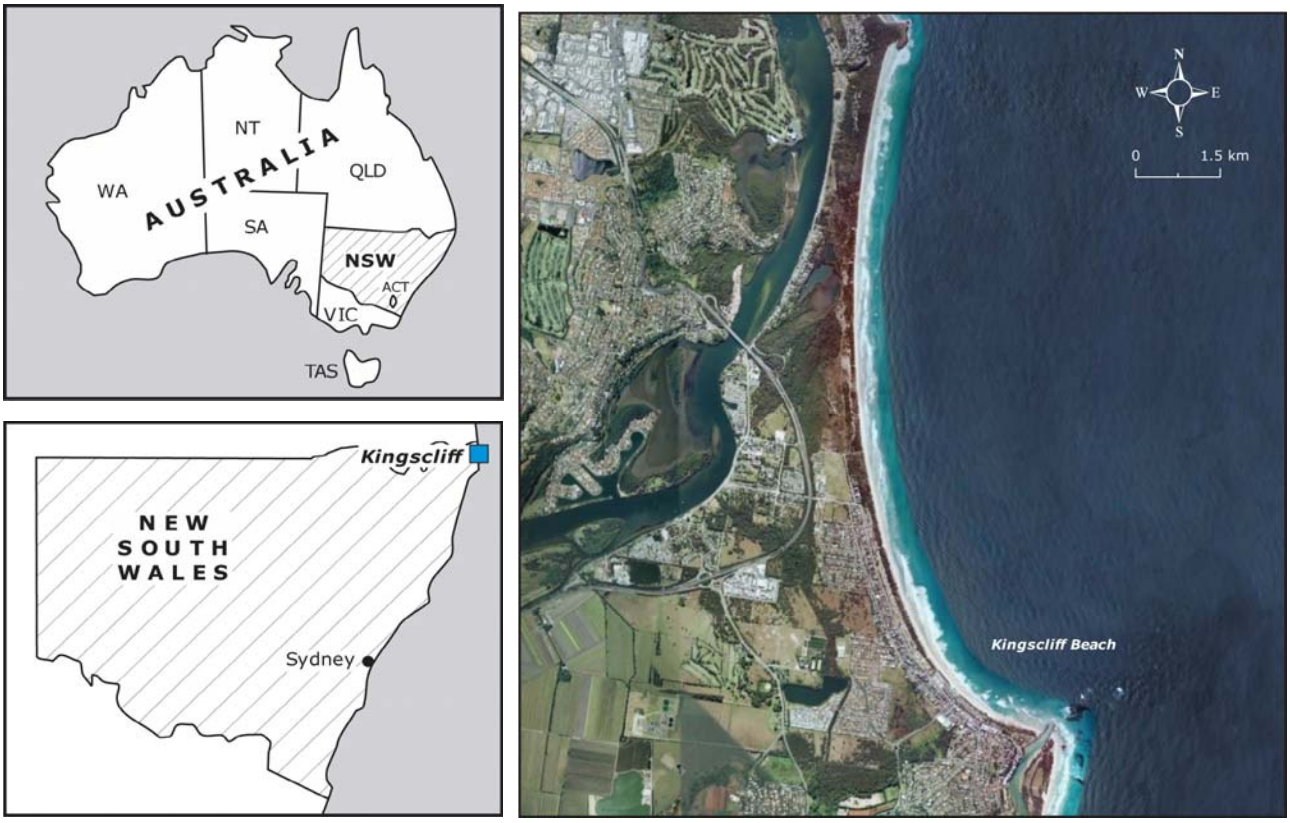

Figure 1. Location.

2 DESIGN CONDITIONS

2.1 Planning Horizon

Establishing the design working life of the proposed seawall was critical for determination of subsequent design parameters. A nominal design life of 50 years was adopted for the structure. A further consideration is that the maximum significant wave height that can reach the structure is a function of design water level due to depth limited wave conditions. The 100 year ARI event, equivalent to 1% annual exceedance probability (AEP), was selected for both wave conditions (height, period and direction) and water level conditions (tide plus anomaly).

2.2 Offshore Wave Conditions

The offshore design wave conditions (Table 1) were based on wave buoy measurements located in offshore depths of 60 to 80 m. In the absence of a comprehensive numerical wave modelling study for the area, a K value (the combined coefficient of refraction, diffraction, friction and shoaling) of 1 was adopted.

Table 1. 100 year ARI offshore wave conditions.

Variable	Value
Offshore H'_{OS} (m)	8.1
Offshore T_P (s)	13.1
Offshore f_P (Hz)	0.076
K	1.0

2.3 Still Water Levels

Elevated water levels consist of (predictable) tides, which are forced by the sun, moon and planets (astronomical tides), and a tidal anomaly. The present day 100 year ARI extreme still water level (excluding wave setup) adopted for preliminary design was 1.58 m MSL based on long term water level records in MHL (2010). Allowing for 0.54 m of sea level rise over its design life, the design still water level at the end of the life of the proposed seawall is 2.12 m MSL.

2.4 Scour Level

During fair weather, the proposed seawall is expected to be buried (or at least partly buried) landward of the active beach for extended periods. However, during a design event, the beach is assumed to be in an extremely eroded condition. The adopted design sand level at the seawall is -2 m MSL during a design event throughout its design life. This assumption is supported by numerical erosion modelling (detailed in Coghlan et. al., 2016) which found similar scour levels for both the present and future (pre-storm bathymetry profile raised and receded, water level raised) planning periods.

2.5 Geotechnical Conditions

Following a geotechnical fieldwork investigation including drilling boreholes, dynamic cone penetrometer tests, laboratory particle size analysis of sediment samples and associated numerical model slope stability analysis, it was recommended that the rock armoured seawall have a slope no steeper than 1V:2H to maintain long term stability.

2.6 Structural Criteria

Under design conditions a small amount (up to 5%) of damage to the primary rock armour was considered acceptable. An acceptable design mean overtopping rate of 200 L/s per m was also adopted to prevent damage to paved surfaces behind the seawall which might ultimately lead to failure of the seawall itself if allowed to occur. This threshold value is consistent with published values (USACE, 2006; CIRIA, 2007; EurOtop, 2008).

3 PRELIMINARY EMPIRICAL DESIGN

3.1 Wave Setup

While the design still water levels incorporate allowance for tides, barometric setup and regional wind setup (i.e. storm surge), wave setup is excluded and needed to be determined. Total wave setup is the increase in water level above the still water level (defined as including the effects of all other forcing except wave setup) due to momentum transfer to the water column by waves that are breaking or otherwise dissipating their energy (FEMA, 2005). Total wave setup includes a static component (the increase in mean water level) and a dynamic component, a type of infragravity wave, which varies much more slowly than the dominant wave period. “Wave setup” is loosely equated with static wave setup only in various literature, while the dynamic component is often termed “surf beat”, infragravity waves or long waves (if considered).

To determine the static wave setup at the proposed seawall, the effective offshore significant wave height (H'_{OS}) was adjusted to the root mean square wave height (H_{RMS}) according to USACE (2006) and CIRIA (2007) in Equation (1).

$$H_{RMS} = 0.706 \times H'_{OS} \tag{1}$$

An offshore H_{RMS} of 5.72 m was applied as a deep water boundary condition to the Dally et al. (1984) two-dimensional surf zone model to estimate static wave setup. The corresponding 100 year ARI peak spectral wave period and still water level were also applied. The corresponding wave setup, setup water surface level and setup water depth at the present day -2 m MSL contour for present day and future (with SLR) conditions were determined as shown in Table 2.

Table 2. Dally et al. (1984) static wave setup estimates at the present day-2 m MSL contour.

	Still Water Level (m MSL)	Wave Setup (m)	Setup Water Surface Level (m MSL)	Setup Water Depth (m)
Present Day	1.58	0.58	2.16	4.16
Future with SLR	2.12	0.50	2.62	4.62

It is assumed that the wave setup values calculated using Dally et al. (1984) exclude the dynamic wave setup component but this is not explicitly specified, particularly because the model was forced with H_{RMS}.

3.2 Depth Limited Wave Heights at the Structure

For the 100 year ARI wave, water level and eroded profile condition (design scour level), depth limited nearshore wave heights at the proposed seawall were estimated using the method of Goda (2007) for significant wave height and Battjes and Groenendijk (2000) for $H_{1/10}$ and $H_{2\%}$. Both methods rely on an estimate of the water depth (d_b) at the -2 m MSL contour. Note that static wave setup (estimated in Section 3.1) was included in all calculations involving d_b. The nearshore slope was assumed to be 1V:35H based on site surveys. Based on Goda (2007), the breaker depth index (ratio of H_S to d_b) was 0.60 (present day) and 0.59 (future with SLR). Wave height estimates for present day and future (with SLR) conditions are summarised in Table 3.

Table 3. Depth limited wave heights at the -2 m MSL contour using Goda (2007) and Battjes and Groenendijk (2000).

	Present Day Condition (m)	Future with SLR Condition (m)
H_S	2.48	2.74
$H_{1/10}$	3.03	3.35
$H_{2\%}$	3.20	3.54

3.3 Armour Geometry and Stability

Preliminary enquiries with several local quarries indicated that it was not possible to acquire greywacke rock (density = 2,650 kg/m^3) in sufficient quantities for construction with a median mass greater than 7.0 t. The median required rock armour size (M_{50}) was estimated using several different empirical methods as detailed in CIRIA (2007): Hudson (SPM, 1977), Hudson (SPM, 1984) and Van der Meer (shallow water, Van Gent, et al., 2004) for structure slopes of 1V:2.0H (maximum slope to maintain geotechnical stability) and 1V:2.2H in Table 4.

Table 4. Estimated armour size for future with SLR condition.

Technique	Estimated Armour Size M_{50} (t)		Notes
	1V:2.0H Slope	1V:2.2H Slope	
Hudson (SPM, 1977)	2.0	1.8	H_S = 2.74 m, K_D = 3.5 (rough, angular, random, n = 2)
Hudson (SPM, 1984)	6.3	5.7	$H_{1/10}$ = 3.35 m, K_D = 2.0 (rough, angular, random, n = 2)
Van der Meer (shallow water)	4.0	3.9	$H_{2\%}$ = 3.54 m, $T_{m-1,0}$ = 11.9 s, S_d = 2, N = 3,000, P = 0.4

Since a moderately wide rock grading (M_{85}/M_{15} ratio of 3.43) will be adopted to maximise yield from the selected local quarry (and hence slightly more damage may be expected compared with narrower gradings on which the empirical methods are based), the preliminary design conservatively adopted a structure slope of 1V:2.2H comprising two layers of 7.0 t primary armour rocks overlying two layers of 0.7 t secondary armour rocks. The adopted preliminary primary armour rock mass of 7.0 t is heavier than the largest mass (5.7 t) suggested by the empirical methods for a 1V:2.2H slope. Adoption of the flatter slope and larger rock size was undertaken in case physical modelling was not undertaken and the client constructs the preliminary design without detailed design (not recommended). A non-woven, heavy grade geotextile was included between the in-situ material and the secondary armour to prevent the loss of fines (mostly in situ beach sand).

3.4 Crest Level and Wave Overtopping

Following consideration of a range of crest elevations for the proposed seawall, the preliminary crest elevation adopted was 5 m MSL, largely because this was the upper envelope of the existing dune crest foreshore topography. It is proposed that the crest be raised at some point during its design life with the addition of a 1 m high wave return wall (effective crest elevation 6 m MSL) when the extent of measured sea level rise necessitates it.

To estimate the mean wave overtopping rate at the proposed seawall for the two structure slopes (1V:2.2H and 1V:2.0H) and two effective crest elevations (5 and 6 m MSL), the EurOtop (2008) "Overtopping Manual" was used. Empirical calculations were undertaken using a purpose built spreadsheet rather than the online empirical calculation tool. Based on USACE (2006), the spectral wave height, H_{m0}, was determined to be 2.76 (present day) and 3.04 m (future with SLR). As recommended in EurOtop (2008), wave setup was not included in the input water levels as the empirical equations are based on physical model test results which implicitly reproduced wave setup against the test structures.

The empirically derived mean wave overtopping rate estimates are presented in Table 5 for present day and future (with SLR) conditions. The values presented in the table are based on the "deterministic" equations in EurOtop (2008) which are recommended for preliminary design as they are considered more conservative. "Probabilistic" values are also presented in brackets in Table 5 and represent the average of all raw EurOtop datasets.

Table 5. Mean wave overtopping rate estimates using EurOtop (2008).

Structure Slope	Mean Overtopping Rate, Q (L/s per m)			
	No Wave Return Effective Crest 5 m MSL Present Day	No Wave Return Effective Crest 5 m MSL Future with SLR	With Wave Return Effective Crest 6 m MSL Present Day	With Wave Return Effective Crest 6 m MSL Future with SLR
1V:2.2H	16.5 (9.0)	68.4 (42.5)	3.0 (1.3)	14.1 (7.5)
1V:2.0H	20.9 (11.6)	82.6 (52.1)	4.0 (1.9)	17.9 (9.6)

Note: Deterministic (average estimate of raw EurOtop datasets plus one standard deviation) mean overtopping rates shown. Probabilistic (average estimate of raw EurOtop datasets ~ 50% exceedance) values are also shown in brackets.

3.5 Preliminary Seawall Design

Figure 2 shows a cross-section of the preliminary design for the greywacke rock seawall. The design still water levels (excluding wave setup) for present day and future (with SLR) conditions and a typical beach profile (in an accreted state) are also illustrated on Figure 2. Note that the typical beach profile will intersect the proposed structure at approximately 2 m MSL but the eroded sand level during a design storm event is -2 m MSL.

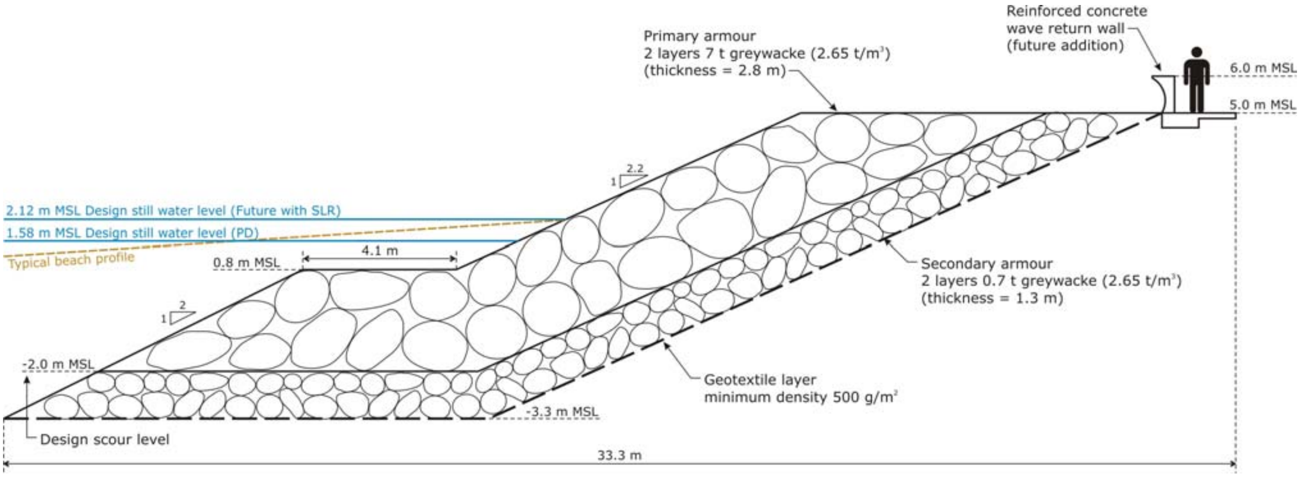

Figure 2. Preliminary greywacke rock seawall cross-section.

4 TWO-DIMENSIONAL PHYSICAL MODELLING SETUP

4.1 Testing Facility

Two dimensional testing was undertaken in a flume at the UNSW Water Research Laboratory measuring approximately 35 m in length, 0.9 m in width and 1.4 m in depth. The wave generator is a hydraulic, piston-type paddle.

4.2 Design and Scaling

Model scaling was based on geometric similarity with an undistorted scale of 1:45 being used for all the tests. Selection of the length ratio was primarily based on the limitations of the wave machine to generate the required range of wave conditions at the largest possible scale. The scaling relationship between length and time was determined by Froudian similitude. The prototype density for the salt water at Kingscliff Beach, the proposed armour (greywacke rock), model water density and model armour density are shown in Table 6. While the model greywacke rock armour had the same density as the prototype, fresh water was used in the flume. The scaling relationship for armour mass (1:95,145) considered the difference in prototype and model water densities and was determined using the empirical scaling modification of Sharp and Kader (1984). The selected scale was large enough to ensure that the flow through primary armour layers remained turbulent, eliminating viscous scale effects on armour stability.

Table 6. Prototype and model density values.

Parameter	Value		Units
	Prototype	Model	
Water Density	1024	998	kg/m^3
Rock Density	2650	2650	kg/m^3

4.3 Test Program

A flexible, iterative test program was adopted at the commencement of the physical modelling program. 20 separate tests on five (5) different seawall designs were undertaken, with the results for each structure informing the next tested model structure. For present day and future (with SLR) conditions, each seawall design was tested with and without a wave return wall in place to test the sensitivity of the design to wave overtopping (i.e. 4 tests per design). Model seawall structures were first tested without a wave return wall (test 1 – present day, test 2 – future with SLR) and secondly with a wave return wall (test 3 – present day, test 4 – future with SLR). The 6 t, 1V:2.2H model seawall was an exception to this sequence; the tests with a wave return wall preceded the tests without a wave return wall (i.e. tests 3,4,1,2). Primary armour rock masses of 4 t, 5 t, 6 t and 7 t were tested with structure slopes of 1V:2.0H or 1V:2.2H. The first tested section was based on the preliminary design (Figure 2) with each subsequent design having either a reduced primary armour size or steepened structure slope.

4.4 Construction

4.4.1 Bathymetry

The model bathymetry constructed from water resistant plywood was representative of the proposed site bathymetry for a distance of at least 6.1 wavelengths (470 m) seaward of the test structures with the following characteristics:

- intersected structure at -2.0 m MSL;
- 1V:35H slope from -2.0 m MSL to -4.5 m MSL;
- 1V:85H slope from -4.5 m MSL to -9.0 m MSL;
- seaward of -9.0 m MSL false floor sloped at 1V:5H until it intersected the permanent flume floor at -44.8 m MSL.

As discussed in Section 2.4, the model bathymetric profile used for testing the seawall designs was considered to be representative of an eroded state for Kingscliff Beach throughout its design life. Only one model bathymetric profile was used for both present day and future (with SLR) tests. While the bathymetry intersected the structure at -2.0 m MSL (the bottom of the primary armour layer), the model structure was built down to -3.3 m MSL (the geotextile underlayer).

4.4.2 Seawall Backfill Material

Based on geotechnical fieldwork, the material on which the geotextile underlayer (and the seawall itself) will be supported is expected to be fine to medium sand. For model design purposes, this sand was considered to be impermeable. Since it is not possible to correctly scale such fine material in the model, the batter slope for the seawall was constructed with an impermeable hollow timber frame. This frame was covered with geotextile material to separate the backfill material and the secondary armour (Figure 3). The modelling approach for the backfill material was expected to yield conservative stability results for the primary and secondary armour since the model has lower permeability relative to the prototype (higher reflections off the backfill material in the model will lead to higher seaward loads on the armour layers).

Figure 3. Example photograph of model structure prior to testing in the 2D wave flume (7 t rock, 1V:2.2H slope, no wave return)

4.4.3 Rock Primary and Secondary Armour

Model greywacke rocks for the primary (7 t, 6 t, 5 t and 4 t) and secondary armour gradings were sorted according to rock mass distributions commensurate with the grading adopted for the preliminary seawall design (M_{85}/M_{15} ratio of 3.43).

4.5 Wave Sequence Preparation and Generation

The 100 year ARI deep water wave sequence to be reproduced by the wave generator was assumed as JONSWAP spectrum with a peak enhancement factor of 3.3. The same sequence (with adjustment of flume water level) was used for both present day and future (with SLR) conditions.

4.6 Data Collection and Analysis

4.6.1 Wave Data

Waves that reflect from model structures towards the wave generator were not actively absorbed by the wave generator. Instead, test wave climates were first calibrated both in deep water and at the proposed structure location, without a model structure in place. Wave conditions were set in deeper water near the model boundary, based on the deep water wave data described in Section 2.2, and then allowed to shoal and break across the model bathymetry. Reflections from the far end of the wave flume (without a model structure in place) were minimised through the use of low gradient, dissipative materials. Waves were measured using two, three probe arrays to allow for the separation of incident and reflected waves using the method of Mansard and Funke (1980). Use of this technique further reduced the influence of reflected waves on the calibrated wave climates. The same calibrated test wave climates were then reproduced with the model structure in place. The wave conditions measured in deep water during the structural tests were then compared with the measurements without a structure in place to ensure that the influence of wave reflections from the structure were minimal. Each armour stability test included 1,000 incident waves; this related to a prototype storm duration of 2.8 hours.

4.6.2 Overtopping Data

Throughout the duration of the tests, the volume of water overtopping the crest of the seawall was collected using a catch tray placed leeward of the crest for a portion of the flume width. The exact location where overtopping was measured (distance landward of the crest) varied between 9.4 m and 11.9 m depending on the test setup. This setup allowed the measurement of mean overtopping discharge, Q (L/s per m of crest length). The impact of wind on wave overtopping was not considered in the model. Each of the 20 overtopping tests was conducted in parallel with an armour stability test.

4.6.3 Rock Armour Layer Damage Assessment

To aid in the tracking of individual primary armour units, a four colour banding system was used. Damage was defined as rocks which were displaced a distance greater than the D_{n50} (median nominal rock diameter, equivalent cube). The damage percentage for rock was determined by relating the number of rocks displaced as a proportion of the total number of rocks in each colour band and the complete primary armour layer. Prior to armour stability testing each model seawall for the first time, a settlement test was run with the wave climate at 50% strength for a prototype duration of approximately 1 hour to allow the rock armour to “bed” into place (i.e. finding an equilibrium position).

5 TWO-DIMENSIONAL PHYSICAL MODELLING RESULTS

5.1 Wave Setup

Wave setup was measured in the physical model using the raw water level recorded at the location of the proposed seawall toe (-2 m MSL) during the wave climate calibration tests without the model structure in place. This record was taken from the seaward probe in the three probe array at the proposed seawall location. Note that the influence of wave reflections in this probe record was not removed prior to undertaking wave setup analysis but they were low due to the presence of a dissipative beach rather than a structure.

Over the full duration of each test, the average increase in water level was 0.22 m for both design wave sequences (present day and future with SLR). This is considered to be equivalent to the static wave setup component. Note however that this level is exceeded many times throughout each test due to the dynamic component of wave setup. While dynamic wave setup analysis has been omitted for brevity, the maximum total wave setup (static plus dynamic) for present day and future (with SLR) conditions was 1.60 m and 1.84 m above the still water level, respectively.

The physical model results (0.22 m) show that the Dally et al. (1984) model overestimated (0.58 and 0.50 m) the static wave setup at the -2 m MSL contour. Interestingly, these estimates of static wave setup are less than the maximum total wave setup (static plus dynamic) measured in the physical model. It is also noteworthy that the Dally et al. (1984) model estimated comparatively lower wave setup for the future (with SLR) condition, but that identical static wave setup was measured in the physical model as well as higher dynamic wave setup. This exercise highlights the importance of strictly defining “wave setup” and the difficulties associated with defining water depth in the surf zone.

5.2 Depth Limited Wave Heights at the Structure

Measured wave statistics for the present day and future (with SLR) design conditions are summarised in Table 7. These results show that the empirical estimates for wave height are approximately 13% higher (range 4-23%) than that measured in the physical model. With static wave setup included in the measured breaker depth, H_S to d_b was 0.56 (present day) and 0.59 (future with SLR). The similarity of these indices to those estimated using Goda (2007); demonstrate that the inaccuracies in the preliminary depth limited wave statistics are primarily attributable to the inaccuracy in the wave setup estimates, rather than the wave height equations themselves. The spectral wave height, H_{m0}, was also determined for re-analysis with the EurOtop overtopping equations and, as expected, was 11% greater than H_S for both design wave conditions which is in accordance with the assertion that H_S can be 10-15% smaller than H_{m0} in shallow water (EurOtop, 2008 and USACE, 2006); even though these parameters are approximately equal in deep water.

Table 7. Depth limited wave height measurements at the -2 m MSL contour.

	Present Day (m)	Future with SLR (m)
H_S	2.11	2.55
H_{m0}	2.34	2.84
$H_{1/10}$	2.47	2.96
$H_{2\%}$	2.88	3.42

The spectral wave period, $T_{m-1.0}$, which is an input parameter for the Van der Meer (shallow water) and EurOtop empirical equations, was found in the flume testing to be prototype 11.6 s and 11.5 s for the present day and future (with SLR) conditions, respectively. These compared well with the empirical estimate of 11.9 s (within 3%).

5.3 Armour Geometry and Stability

Each model structure (median rock armour mass and structure slope combination) was exposed to four design wave sequences (4,000 total incident waves, 11.2 hour prototype duration) without repair. Percentage damage to rock armour was measured as both progressive (damage induced per test) and cumulative (total overall damage). For brevity, only the cumulative percentage damage results are summarized below in Table 8.

Table 8. Summary of primary armour damage measurements.

Median Rock Armour Mass, M_{50} (t)	Structure Slope	Total Primary Armour Rock Cumulative % Damage			
		No Wave Return Wall Effective Crest 5 m MSL Present Day	No Wave Return Wall Effective Crest 5 m MSL Future with SLR	With Wave Return Wall Effective Crest 6 m MSL Present Day	With Wave Return Wall Effective Crest 6 m MSL Future with SLR
7.0	1V:2.2H	1	2	3	4
6.0	1V:2.2H	5*	5*	2*	4*
5.0	1V:2.2H	2	3	3	4
5.0	1V:2.0H	2	4	4	5
4.0	1V:2.0H	5	9	10	12

* For the 6 t primary armour model structure, the tests with a wave return wall preceded the tests without a wave return wall.

Cumulative primary armour damage increased with decreased median rock mass and, for the two sets of tests with 5 t rock, damage slightly increased with steepened structure slope. The damage induced per test generally decreased for the third and fourth tests in the wave sequence.

With reference to the design criteria of damage not exceeding 5% under design conditions (including future sea level rise), selection of a median rock armour mass of 4 or 5 t may be suitable for a 1V:2.0H structure slope, depending on the number of design events the structure is expected to withstand during its design life. The 4 t rock design provides acceptable performance for a single event but not for two or more events as the damage induced per test did not decrease for the third and fourth tests in the wave sequence. Following discussions with the client, the 5 t primary armour mass was adopted for the detailed design with a view to minimising ongoing maintenance of the seawall over its design life.

The physical model results demonstrate that the preliminary estimates (with over-conservative input wave parameters) for required median rock armour mass were non-conservative (2.0 t) for Hudson (SPM, 1977), over-conservative (6.3 t) for Hudson (SPM, 1984) and reasonable (4.0 t) for Van der Meer (shallow water, Van Gent, et al., 2004).

5.4 Crest Level and Wave Overtopping

Measured mean wave overtopping rates are summarised in Table 9. As with the preliminary estimates, wave overtopping increases with steepening structure slope, decreases with increased crest elevation and increases with future sea level rise. No trend was discernable for the influence of median rock armour mass on overtopping from this limited dataset.

Table 9. Mean wave overtopping measurements.

Median Rock Armour Mass, M_{50} (t)	Structure Slope	Mean Overtopping Rate, Q (L/s per m)			
		No Wave Return Wall Effective Crest 5 m MSL Present Day	No Wave Return Wall Effective Crest 5 m MSL Future with SLR	With Wave Return Wall Effective Crest 6 m MSL Present Day	With Wave Return Wall Effective Crest 6 m MSL Future with SLR
7.0	1V:2.2H	4.4	26.5	1.8	9.3
6.0	1V:2.2H	3.9	17	1.5	6.7
5.0	1V:2.2H	7.3	36.8	1.9	10.8
5.0	1V:2.0H	10.5	40.2	3.1	11.7
4.0	1V:2.0H	9.5	47.6	2.7	12.2

With reference to the design criteria of the mean wave overtopping not exceeding 200 L/s per m under design conditions (including with future sea level rise), all structures tested were well below this threshold. Accordingly, damage to paved surfaces behind the seawall is not expected during design conditions. Indeed, all structures had mean overtopping rates below the threshold (50 L/s per m) for initiating of damage to grassed areas behind the seawall (USACE, 2006; CIRIA, 2007; EurOtop, 2008).

For the structure without a wave return wall (crest elevation 5 m MSL), the measurements in the physical model were always less than both the deterministic and probabilistic values from EurOtop. This indicates that the empirical equations were overly conservative for this arrangement. In fact, for 3 out of the 10 tests, the preliminary overtopping estimates were outside the suggested upper accuracy factor of 3 times the actual overtopping rate measured in the physical model (EurOtop, 2008). This is likely due to preliminary spectral wave height at the proposed structure being over estimated by approximately 13%. However, for the structure with a wave return wall (crest elevation 6 m MSL), the measurements in the physical model were between the deterministic value and the probabilistic value for 7 out of the 10 tests.

6 DISCUSSION

Following the conclusion of the physical modelling program, the assumptions inherent in the preliminary empirical designs were reviewed in light of the model measurements. It is clear that the inaccuracies in the preliminary design water depths (including static wave setup) at the structure for present day and future (with SLR) conditions have the most significant impacts on subsequent design parameters (wave height at the structure, required median rock armour mass and expected mean overtopping rates). Future efforts to improve empirical design of seawalls located in shallow water should focus on this aspect as, for this project; it introduced over-conservatism throughout the ensuing preliminary design.

To demonstrate the importance of accurately estimating wave setup, the physical model measurements (wave setup, wave height and wave period) were substituted as input parameters to the same empirical design equations for re-analysis. With inclusion of reduced static wave setup measured in the physical model, revised empirical wave height estimates for present day and future (with SLR) conditions are compared with the wave height measurements in Table 10. These results show that the revised empirical estimates for wave height are generally within 5% (reduced from 13%) of the physical model measurements, with particularly good accuracy for the future (with SLR) condition. Based on Goda (2007), the ratio of H_S to d_b (including static wave setup) was unchanged at 0.60 (present day) and 0.59 (future with SLR).

Table 10. Revised empirical depth limited wave heights at the -2 m MSL contour using Goda (2007), Battjes and Groenendijk (2000) and USACE (2006) compared with measurements.

	Present Day Condition		Future with SLR Condition	
	Revised Emprical (m)	Physical (m)	Revised Emprical (m)	Physical (m)
H_S	2.27	2.11	2.58	2.55
H_{m0}	2.52	2.34	2.87	2.84
$H_{1/10}$	2.78	2.47	3.16	2.96
$H_{2\%}$	2.94	2.88	3.34	3.42

The median required rock armour size was re-estimated using the same three empirical methods combined with the wave height and period measurements from the physical model as summarised in Table 11. As stated earlier, based on the physical modelling the adopted M_{50} was 5.0 t for a 1V:2.0H structure slope. While estimates using Hudson (SPM, 1977) remain non-conservative (1.6 t), the estimate using Hudson (SPM, 1984) is remarkably good (4.3 t). The estimate using Van der Meer (shallow water, Van Gent, et al., 2004) becomes slightly non-conservative (3.6 t).

Table 11. Revised empirical armour size estimates for future with SLR condition.

Technique	Estimated Armour Size M_{50} (t)		Notes
	1V:2.0H Slope	1V:2.2H Slope	
Hudson (SPM, 1977)	1.6	1.4	H_S = 2.55 m, K_D = 3.5 (rough, angular, random, n = 2)
Hudson (SPM, 1984)	4.3	3.9	$H_{1/10}$ = 2.96 m, K_D = 2.0 (rough, angular, random, n = 2)
Van der Meer (shallow water)	3.6	3.5	$H_{2\%}$ = 3.42 m, $T_{m-1,0}$ = 11.5 s, S_d = 2, N = 3,000, P = 0.4
Physical Model Value	4.0 to **5.0**	<5.0	

Mean overtopping rates were similarly re-estimated using EurOtop (2008) and compared with the overtopping measurements from the physical model in Table 12. The average mean overtopping rate measured in the physical model for each structure slope tested is presented; the range of test values is also shown in brackets. For the structure without a wave return wall (crest elevation 5 m MSL), the empirical overtopping estimates are substantially improved, with the average physical model value falling in between the deterministic and probabilistic values for all cases except one (1V:2.0H, 5 m MSL crest, present day). For the 6 m MSL structures under future (with SLR) conditions, the revised deterministic values were approximately 10% lower than the average physical model value which is also quite reasonable. However the physical model mean overtopping rates were 2-3 times the revised deterministic values for 6 m MSL structures under present day conditions.

Table 12. Revised empirical mean wave overtopping rates compared with measurements.

	Structure Slope	Mean Overtopping Rate, Q (L/s per m)			
		No Wave Return Wall Effective Crest 5 m MSL Present Day	No Wave Return Wall Effective Crest 5 m MSL Future with SLR	With Wave Return Wall Effective Crest 6 m MSL Present Day	With Wave Return Wall Effective Crest 6 m MSL Future with SLR
EurOtop	1V:2.2H	5.5 (2.7)	44.5 (26.8)	0.8 (0.3)	8.2 (4.1)
Physical	1V:2.2H	5.2 (range 3.9-7.3)	26.8 (range 17-36.8)	1.7 (range 1.5-1.9)	8.9 (range 6.7-10.8)
EurOtop	1V:2.0H	7.2 (3.6)	54.3 (33.2)	1.1 (0.4)	10.6 (5.4)
Physical	1V:2.0H	10.0 (range 9.5-10.5)	43.9 (range 40.2-47.6)	2.9 (range 2.7-3.1)	12.0 (range 11.7-12.2)

Note: Primary EurOtop values shown are deterministic, probabilistic values are shown in brackets.
Primary physical model values are the average of all tests, the range of test values is shown in brackets.

7 CONCLUSIONS

As a result of the two-dimensional physical modelling program, the rock armour mass for the sloping rubble mound seawall was reduced by more than 25%, the structure slope was steepened (reducing its footprint) and the expected wave overtopping rates were found to be lower compared with the preliminary designs. However, it is acknowledged that the armour size and structure slope adopted for preliminary design were relatively conservative (in case physical modelling was not undertaken). The extent of optimisation through the physical modelling would have been reduced if a less conservative preliminary design had been initially adopted. Obviously, outcomes from the physical model came at greater expense and effort than that expended on the preliminary empirical designs. Upon review of the assumptions inherent in the preliminary empirical designs, inaccuracies in the preliminary design water depth (specifically static wave setup) at the structure were found to have substantial impacts on subsequent design parameters. Future efforts to improve empirical design of seawalls located in shallow water should focus on reducing the inaccuracy in wave setup estimates. Depth limited wave heights, rock armour stability and wave overtopping may be reasonably estimated empirically if this eventuates.

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