structure. This geometry is subsequently refined using physical modelling to ensure hydraulic stability (wit over-conservatism) and reduce the risk of the design criteria being exceeded during its working life. This paper presen designs developed both empirically and through physical modelling for a seawall located at the back of a sandy beac the Australian East Coast. The authors collectively acted as both designer and physical modeller for the client, prov 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, commentar provided as to which processes were reasonably well described empirically for this case study and which proce introduced over-conservatism throughout the ensuing preliminary design. Given the same offshore wave and still v level boundary conditions as the physical model, inaccuracies in the preliminary design water depth at the structure 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.

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	e.8(con	0	0	8.7492	2	1r025

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

3.5 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

4.4 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).

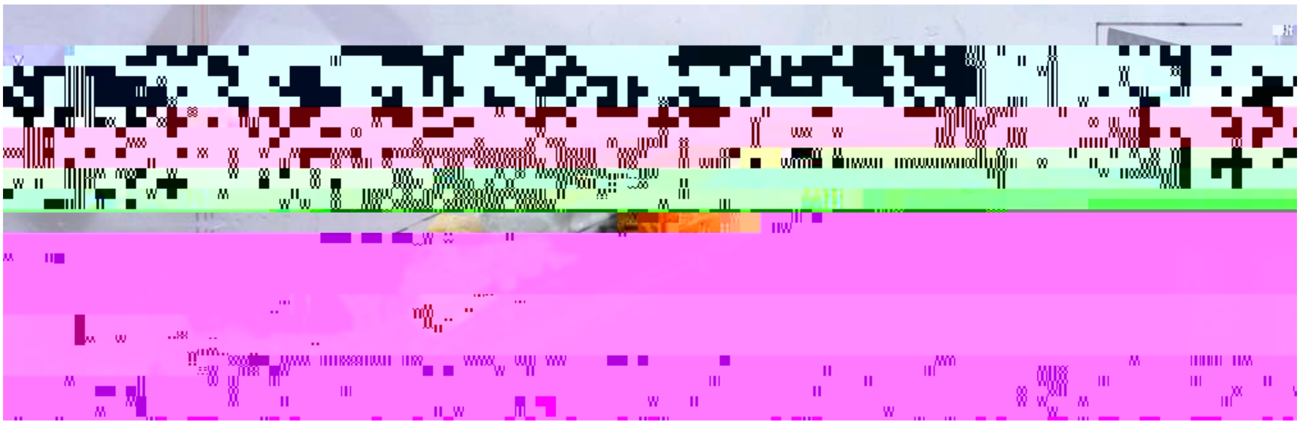

Figure 3. Example photograph of model structure prior to testing in the 2D wave flume (7 t rock, 1V:2.2H slope, no wave return)

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

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

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 meadn(w).9(its i)6(n).4(Tabw).9(il)6(e 10. These)6.2(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

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.

ACKNOWLEDGEMENT

Funding for the investigation presented was provided by Tweed Shire Council, NSW, Australia.

REFERENCES

Battjes, J.A and Groenendijk, H. W., 2000. Wave Height Distributions on Shallow Foreshores, *Coastal Engineering*, 40, 161-182.

BMT WBM, 2013. Tweed Shire Coastal Hazards Assessment, Report Number R.B19094.001.04, Version 04, November.

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

CIRIA; CUR; CETMEF, 2007. The Rock Manual. The use of rock in hydraulic engineering (2nd edition). C683, CIRIA, London.

Dally, W.R., Dean, R.G. and Dalrymple, R.A., 1984. "Modeling Wave Transformation in the Surf Zone". Miscellaneous Paper CERC-84-8, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS

EurOtop, 2008. Wave Overtopping of Sea Defences and Related Structures: Assessment Manual, Environmental Agency (UK), Available: http://www.overtopping-manual.com/

FEMA, 2005. FEMA Coastal Flood Hazard Analysis and Mapping Guidelines Focused Study Report: Wave Setup.

Goda, Y., 2007. Reanalysis of Regular and Random Breaking Wave Statistics, 54th Japanese Coastal Engineering Conference.

Mansard, E.P.D. and Funke, E.R., 1980. The Measurement of Incident and Reflected Spectra Using a Least Squares Method, 17th International Conference on Coastal Engineering, American Society of Civil Engineers, USA.

MHL, 2010. "NSW Ocean Water Levels", Draft Manly Hydraulics Laboratory Report 1881, December.

Patterson, D., 2007. "Comparison of Recorded Brisbane and Byron Wave Climates and Implications for Calculation of Longshore Sand Transport in the Region", Proceedings of the Australasian Coasts and Ports Conference 2007, Melbourne, Australia.

Sharp, J. J. and Khader, M. H. A., 1984. Scale Effects in Harbour Models Involving Permeable Rubble Mound Structures, Symposium