PHYSICAL MODEL TESTS OF NEW LINEAR DEFENCES FOR COLWYN BAY

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This paper describes the physical modelling of the new linear rock armour defences that Conwy County Borough Council intends to use at the eastern extreme of the Colwyn Bay frontage, North Wales coast. Two-dimensional physical model tests of armour stability and wave overtopping were performed at the National Civil Engineering Laboratory, Portugal, for 8 alternative cross-sections of the defences. The results of tests on Alternatives 1 to 8 showed a stable rock revetment profile with respect to rock displacement, which provided adequate protection. Regarding overtopping, for the type of alternatives tested, combining a milder slope, with a recurve crest wall and a permeable crest was very effective in reducing overtopping. The final decision on which defence profile is appropriate, requires design consideration of other aspects, particularly hinterland integration and the level of risk that may be considered as acceptable, set against the additional costs of providing various standards of protection.

Keywords: Physical Models; Rock revetments; Armour stability; Wave overtopping; Colwyn Bay.

1. Introduction

The Colwyn Bay frontage between Rhos Jetty (western extreme) and Beach Rd, Old Colwyn (eastern extreme), is located on the North Wales coast (Figure 1). It comprises a sandy beach, with a surface layer of coarse material, and it is protected by a seawall, a series of shore normal groynes and a detached breakwater at the western extreme.

Figure 1. Study area

The beach in Colwyn Bay has become depleted over the recent past and existing sea defences are no longer performing as they did originally, leading to beach lowering and localised flood damage of the road and the residential areas behind the wall during storms, due to damage to the seawall and wave overtopping (Figure 2).

Figure 2. Damage to the existing seawall and wave overtopping at Colwyn Bay

Conwy County Borough Council is developing the preferred approaches for coastal defence improvements in the area. To achieve this, the first step was the development of a Coastal Strategy for Colwyn Bay, which was concluded in November 2007. From the Strategy, which included one line modelling of beach behaviour, under a range of beach recharge scenarios, conclusions were drawn on the preferred approaches, which comprised a combination of the following elements: i) beach recharge with/without control structures; ii) promenade and wave protection improvements; iii) construction of a linear rock armour defence.

The second step was to undertake detailed modelling studies, based on the preliminary arrangements derived from the Strategy Study, to develop the approach further and provide appropriate criteria to inform detailed design of the construction works. Within the scope of the second step, Royal Haskoning (RH), UK, and the National Civil Engineering Laboratory (LNEC), Portugal, carried out numerical and physical modelling of the study area with the primary objectives of:

- Defining the optimum arrangements for beach recharge and any associated control structures;
- Examining the performance and define arrangements for the new linear defence.

This paper concentrates on the physical modelling of the alternative cross-sections considered for the new linear rock armour defences, which are primarily intended for use at the eastern extreme of the study area. Following this introduction, the paper describes the physical model setup regarding the test facilities, the model scale, the cross-sections, the foreshore, the test programme, the wave conditions, the stability of rock armour and the wave overtopping. Then, the test results are presented and discussed. Finally, there are some conclusions.

2. Model Setup

2.1. Test Facilities and Model Scale

Two-dimensional physical model tests of armour stability and wave overtopping were performed, between September 2009 and February 2010, in one of LNEC's wave flumes. The flume is approximately 50 m long, 1.6 m wide and 1.2 m height. The operating width and operating water depth are 0.8 m.

The flume is equipped with a piston-type wave-maker and an active wave absorption system, AWASYS (Troch, 2005), which allows the absorption of reflected waves. The paddle of the wave-maker is controlled by a computer using the SAM software (Capitão, 2002), developed at LNEC.

The models were built and operated according to Froude's similarity law, with a geometrical scale of 1 : 25. This scale was selected to ensure that: the main aspects of wave-structure interaction were well reproduced in the model; significant scale effects were avoided; and the agreed test conditions could be reproduced in the selected facility with the resources available (Hughes, 1993; De Rouck et al., 2005).

2.2. Cross-Sections and Foreshore

Eight alternative cross-sections were constructed and tested, called here after Alternatives 1 to 8 (Figure 3). Table 1 presents the main characteristics of Alternative 1 and for Alternatives 2 to 8 it shows the differences from Alternative 1. The eight alternatives differed mainly in armour slopes, crest levels, crest widths, crest forms and rear wall levels and forms. Alternative 1 was developed from the preliminary cross-section identified in the Colwyn Bay Coastal Defence Strategy Plan (Conwy County Borough Council, 2007) and from empirical evaluation of overtopping performance. The subsequent arrangements were tested to evaluate the impact of changing different geometrical structure characteristics.

Alternative 1 was basically a rock revetment with a concrete slab maintenance access roadway arrangement and a concrete wave wall on the landward side. The primary armour consisted of 2 layers of 3 to 6 tonne rock, at a 1 : 3 slope, with a crest berm width of approximately 3.50 m (3 rocks) at +7.50 m ODN, constructed on a 300 kg to 1 tonne rock filter layer. The maintenance access was 4.50 m wide, at about +7.00 m ODN. The concrete wall had its crest at +8.00 m ODN.

Table 1. Main characteristics of Alternative 1 and corresponding differences for Alternatives 2 to 8.

CHARACTERISTICS		ALTERNATIVE							
		1	**2**	**3**	**4**	**5**	**6**	**7**	**8**
General		• rock revetment							
Primary armour	• **slope:**	• 1:3		• 1:2.5	• 1:2.5	• 1:2.5			
	• **layers:**	• 2; 3-6 tonne rock							
	• **crest berm width:**	• ≈3.5m (3 rocks)				• ≈11.47m (10 rocks)	• ≈4.70m (4 rocks)	• ≈4.70m (4 rocks)	• ≈4.70m (4 rocks)
	• **crest berm level:**	• +7.50m ODN					• +8.00m ODN	• +9.00m ODN	
Filters		• 300kg - 1 tonne rock layer							
Toe	• **level:**	• ≈ -0.65m ODN					• ≈ -0.61m ODN	• ≈ -0.61m ODN	
	• **top layer:**	• 2 rocks (3-6 tonne)						• 1 rock	• 1 rock
	• **bottom layer:**	• 3 rocks (3-6 tonne; ≈3.5m wide)						• 2 rocks (≈2.22m wide)	• 2 rocks (≈2.3m wide)
Concrete slab maintenance access roadway arrangement	• **level:**	• ≈ +7.00m ODN (landward side)							
	• **slope:**	• 1:40 seaward				Non existent	Non existent	Non existent	
	• **width:**	• 4.5m		• 7.97m	• 7.97m				
Concrete wave wall	• **type:**	• rectangular section					• recurve wall	• recurve wall	• recurve wall
	• **crest level:**	• +8.00m ODN	• +8.50m ODN		• +8.50m ODN	• +8.50m ODN	• +9.00m ODN	• +10.00m ODN	• +8.15m ODN

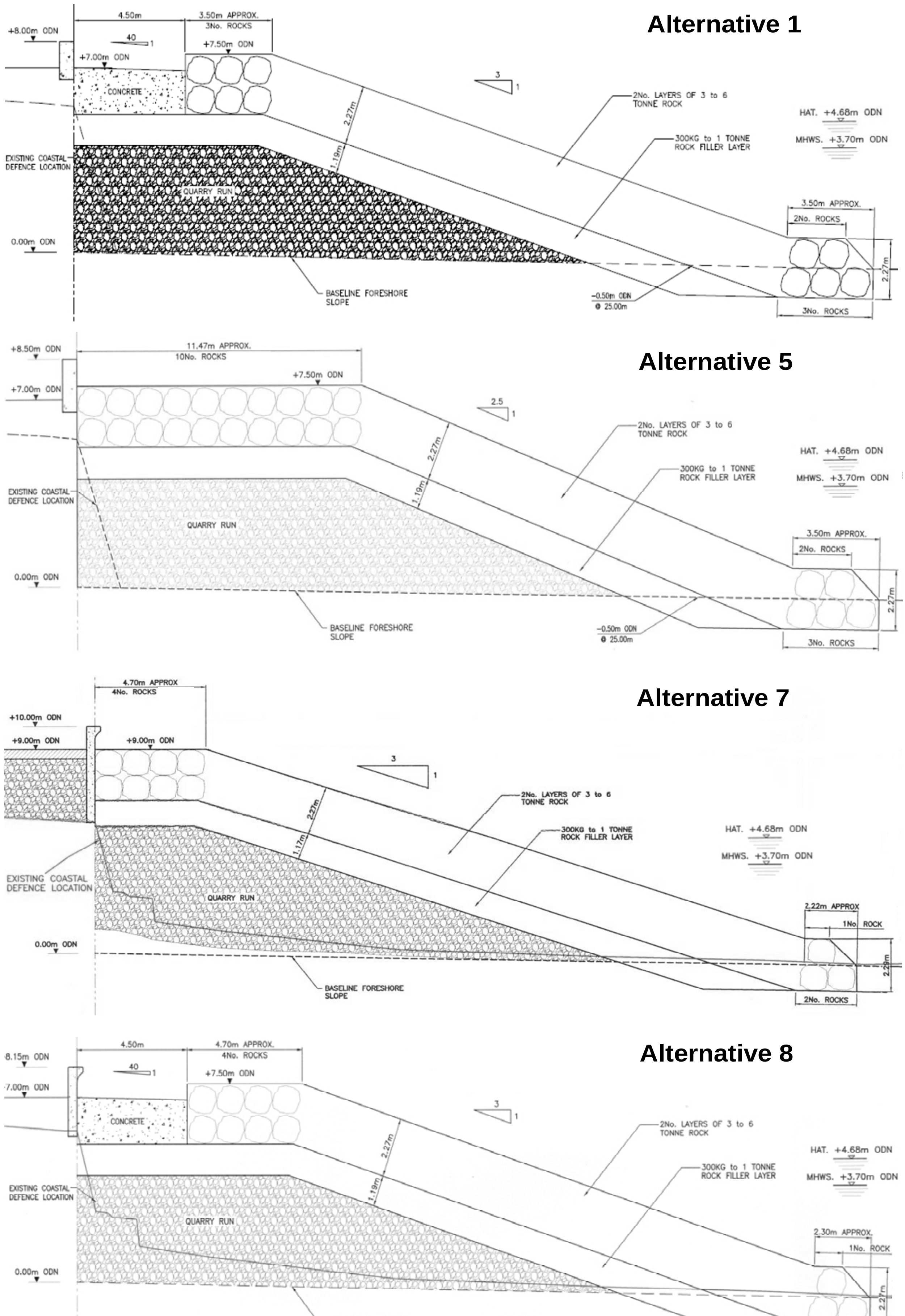

Figure 3. Example of cross-sections tested: Alternatives 1, 5, 7 and 8.

Alternative 2 differed from Alternative 1 in the crest area only: the wall had its crest at +8.50 m ODN, instead of at +8.00 m ODN.

For Alternative 3, the armour slope was 1 : 2.5, instead of 1 : 3, and the concrete maintenance access roadway was 7.97 m wide, instead of 4.50 m.

Alternatives 4 and 5 differed from Alternative 3 in the crest area only: in both alternatives, the concrete wall had its crest at +8.50 m ODN, instead of at +8.00 m ODN; in Alternative 5, the crest berm of approximately 3.50 m (3 rocks) had been extended to an approximately 11.47 m wide rock berm (10 rocks) (there was no concrete maintenance access roadway).

In Alternative 6, the armour slope was 1 : 3 and the crest berm, located at +8.00 m ODN, was approximately 4.70 m wide (4 rocks). The concrete wall was changed to a recurve wall with its crest at +9.00 m ODN.

In Alternative 7, the levels of the crest berm and of the recurve wall had been raised to +9.00 m ODN and +10.00 m ODN, respectively. The toe detail differed from previous alternatives.

Alternative 8 was similar to Alternative 1 but the concrete wall changed to a recurve wall, with its crest at +8.15 m ODN (instead of at +8.00 m ODN), and the crest berm width had been extended from approximately 3.50 m (3 rocks) to about 4.70 m (4 rocks). The toe detail was similar to that of Alternative 7.

The foreshore in front of the model structures was represented by a fixed bed foreshore from the toe of the linear defence down to a level of -0.164 m, which corresponded to a prototype level of 0 m CD (-4.1 m ODN, Figure 4). Two different slopes were used: 1 : 50, for the 5 m immediately in front of the rock structures, and 1 : 100, in the last 4.4 m of the foreshore.

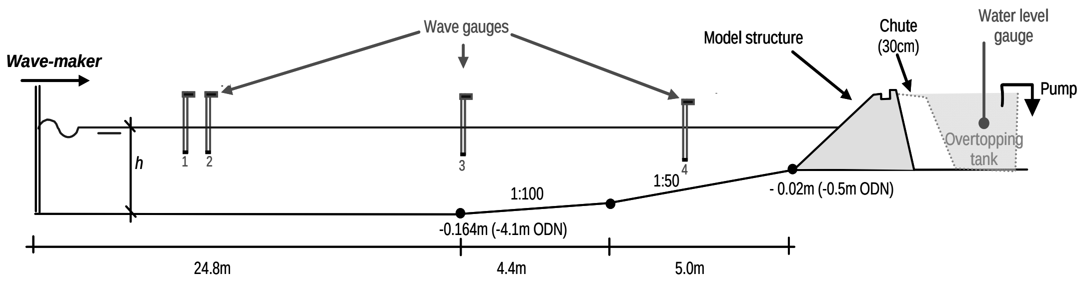

Figure 4. Sketch of wave flume and location of experimental equipment (not to scale).

2.3. Test Programme and Wave Conditions

For each alternative cross-section, the test programme specified a sequence of runs, each with predefined target values of significant wave height, H_{os}, and mean wave period, T_{om}, at -4.1 m ODN for each of the six water levels considered (Table 2). These conditions were derived from a Joint Probability wave and tide analysis produced for the Conwy frontage and the water levels included consideration of future sea level rise based on the current DEFRA guidance. Irregular waves conforming to the mean JONSWAP spectrum (with a peak enhancement factor of 3.3) were employed in the study, with H_{os} and T_{om} varying between 1.8m and 4m, and 5.6s and 8.6s, respectively, covering a wide range of conditions with return periods between 1 and 200 years, and which included potential worst case joint-probability combinations identified from preliminary empirical overtopping assessment. In total, 96 test runs were carried out, each with a duration of approximately 1000 waves.

To measure the free-surface elevation, the flume was equipped with four resistive-type wave gauges (Figure 4): a fixed array of two gauges (gauges 1 and 2), located in front of the wave-maker and required for the dynamic wave absorption system; gauge 3, located at the toe of the foreshore; and gauge 4, located in front of the structure. A computer collected and stored the data in digital format at a frequency of 40 Hz (model scale). The recorded signals were analysed using the SAM software, developed in-house (Capitão, 2002).

2.4. Stability of Rock Armour and Wave Overtopping

For each alternative, armour stability was analysed by counting the number of displaced 3 to 6 tonne rocks per test run and by determining the corresponding percentage, calculated by dividing this number by the total number of rocks used in the model structure. A displaced rock is a rock that has moved from its original position more than the nominal rock diameter.

The number of displaced rocks per test run was assessed by visual observation of the tests, by comparing photographs taken before and after each test run and by analysing the corresponding video.

The percentage of displaced rocks was compared with the maximum acceptable percentage recommended in CIRIA/CUR/CETMEF (2007) referred to as the *no damage* condition: 5 %.

To determine the mean overtopping discharges per metre length of structure, Q (l/s/m), an overtopping tank, located at the back of each structure, was used to collect the overtopping water (Figure 4). The water was directed to the tank by means of a chute, 30 cm wide. A pump and a water-level gauge were deployed in the overtopping tank and connected to a computer that monitored and recorded the water level variation within a test run at a frequency of 40 Hz. Once a preset maximum water level was reached in the tank, the pump was activated for a fixed period. The pumped volume of water was derived from a pump calibration curve.

Table 2. Target conditions at -4.1 m ODN (prototype values).

Water Level (m ODN)	T_{om} (s)	H_{os} (m)	Target Condition Nr.
3.5	8.5	2.5	TC1
		3.0	TC2
		3.5	TC3
		4.0	TC4
4.0	7.5	2.5	TC5
		3.0	TC6
		3.5	TC7
		4.0	TC8
4.8	8.0	2.5	TC9
		3.0	TC10
		3.5	TC11
		4.0	TC12
5.8	5.6	1.8	TC13
		2.0	TC14
		2.5	TC15
		3.0	TC16
4.8	7.0	2.7	TC17
	8.4	3.8	TC18
5.8	7.0	2.7	TC19
	7.5	3.0	TC20
	8.0	3.4	TC21
4.3	7.4	2.9	TC22
	8.0	3.4	TC23
	8.6	3.9	TC24
5.3	5.6	1.8	TC25
	7.4	2.9	TC26
	8.0	3.4	TC27

The mean overtopping discharges per metre length of structure obtained for the test runs were compared with the acceptable overtopping discharge of 0.1 l/s/m that was identified as being appropriate for the primary hinterland receptors (pedestrians and/or vehicles) across the frontage, based on current guidance (Pullen *et al.*, 2007).

3. Test Results

Regarding rock armour stability, the physical model tests showed that Alternatives 1 to 8 were very stable: the percentage of rock displacements was always smaller than 1 %. When damage did occur it was only in the upper armour layer and no gaps down to the filter layer were visible. Furthermore, throughout the course of the tests, RH/LNEC felt there was potential for decreasing rock size without compromising the structure's stability, although this was not tested.

Figures 5 to 8 illustrate the obtained overtopping results for the six still-water-levels tested. The figures show that the still-water-level of +4.8 m ODN was, for all alternatives, the more unfavourable water level with respect to wave overtopping. When considering the overtopping results of Alternatives 1 and 2, they showed that overtopping of Alternative 1 exceeded the acceptable criterion of 0.1 l/s/m by considerable margins. However, visual observation of the model indicated that the crest wall was quite effective in limiting the volume of water overtopping the wall. By increasing the wall height by only 0.5 m to +8.5 m ODN in Alternative 2, the overtopping rates were very significantly reduced, although they were still above the criterion in TC10 to TC12 and TC16 (see Table 2 and Figure 5). Increasing the crest level of the rear wall improved the standard of service from below 1 in 50 years to 1 in 50-100 years without consideration of sea level rise and from 1 in 10 year to above 1 in 20 year with consideration of sea level rise in the next 75 years based on the current DEFRA guidance.

The results from Alternatives 3 and 4 demonstrated that a steeper slope of 1 : 2.5 with a wider concrete berm in front of the crest wall was not as effective in reducing overtopping as the milder slope of 1 : 3 with a narrower concrete berm.

The results of Alternative 5 showed that a wider rock berm was effective in reducing overtopping. Alternative 5 would provide similar performance to Alternative 2 with above 1 in 50 year standard of service without consideration of future sea level rise, reducing to 1 in 20-50 year standard of service with consideration of sea level rise in the next 75 years.

In Alternatives 6 and 7 (without the concrete element of the berm), to control overtopping rates within the required criterion, the rock berm needed to be at +9 m ODN with 1 m high recurve crest wall on top (crest level of +10 m ODN). Although the overtopping rates were above the criterion in TC12 and TC16 in Alternative 7 and the wind was not reproduced in the model, they were considered acceptable, bearing in mind that they only exceeded the target rate by a small margin.

Among all tested profiles, Alternative 7 produced the best performance with respect to overtopping. It would provide generally a 1 in 100 year standard of service without consideration of future sea level rise, or 1 in 50 year standard of service with consideration of sea level rise in the next 75 years.

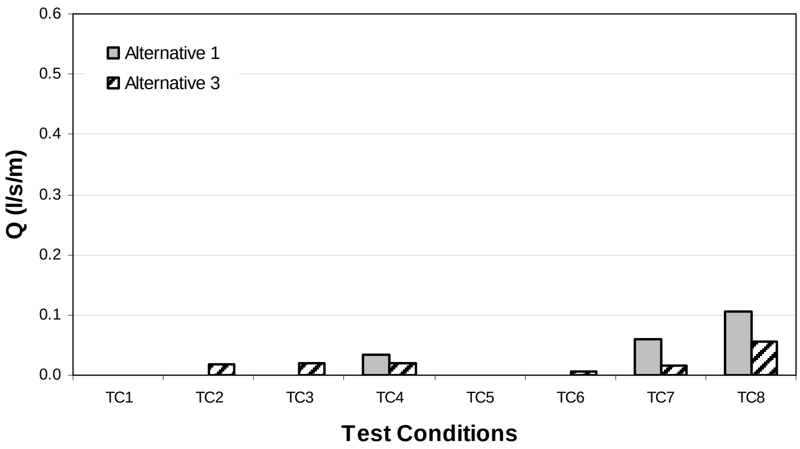

Figure 5. Mean overtopping discharges obtained for the still-water-levels of +3.5 m ODN (TC1 to TC4) and +4.0 m ODN (TC5 to TC8)

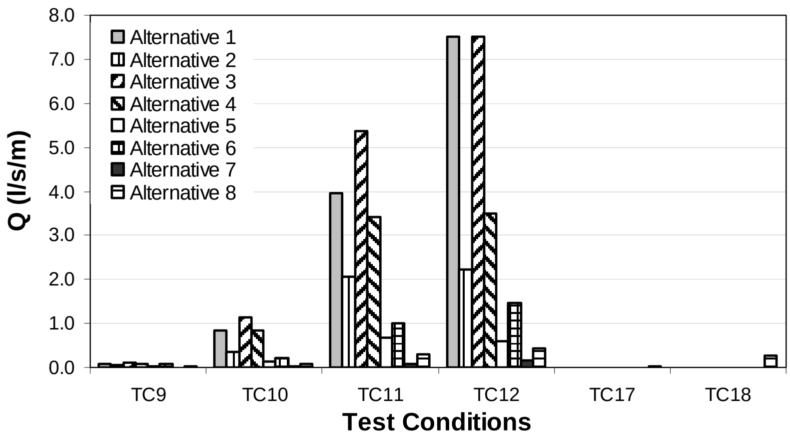

Figure 6. Mean overtopping discharges obtained for the still-water-level of +4.8 m ODN (TC9 to TC12, TC17 and TC18)

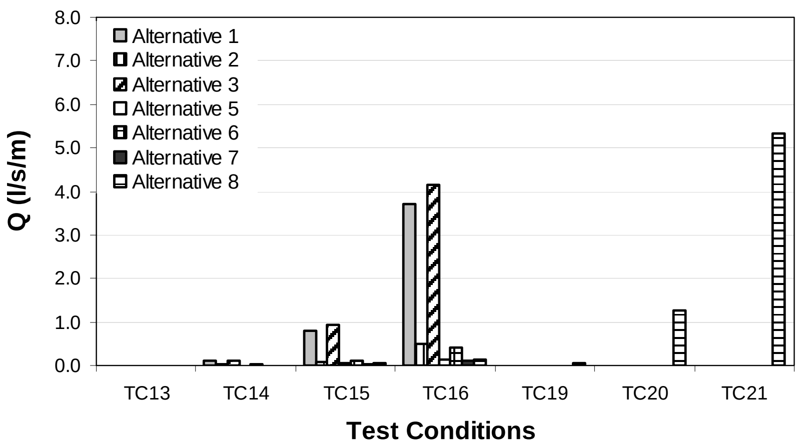

Figure 7. Mean overtopping discharges obtained for the still-water-level of +5.8 m ODN (TC13 to TC16 and TC19 to TC21)

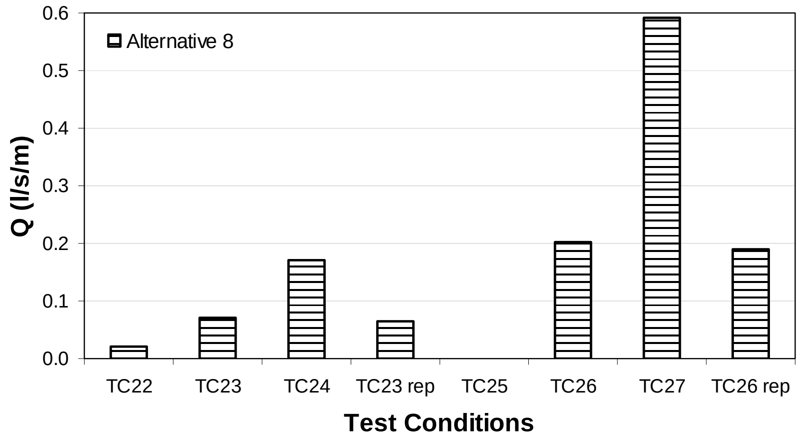

Figure 8. Mean overtopping discharges obtained for the still-water-levels of +4.3 m ODN (TC22 to TC24 and repetition of TC23) and +5.3 m ODN (TC25 to TC27 and repetition of TC26)

Alternatives 6 and 7 however required significantly higher crest levels than other alternatives, which would potentially prove problematic when integrating with present hinterland arrangements.

In Alternative 8, the effectiveness of the milder slope, of the recurve crest wall and of the permeable crest was used to try to reduce overtopping whilst still keeping crest levels to a minimum. Overtopping rates were above the criterion for TC11, TC12, TC16, TC18, TC20, TC21, TC24, TC26 and TC27. Alternative 8 would provide above 1 in 50 year standard of service without consideration of future sea level rise, reducing to 1 in 20-50 year standard of service with consideration of sea level rise in the next 75 years.

4. Conclusions

Conwy County Borough Council, Wales, aims to develop the preferred approaches for coastal defence improvements at Colwyn Bay frontage, North Wales coast, including linear rock armour revetments in front of the existing sea walls. In this respect, Royal Haskoning (RH), UK, and the National Civil Engineering Laboratory (LNEC), Portugal, carried out numerical and physical modelling of the study area with the primary objectives of: i) defining the optimum arrangements for beach recharge and any associated control structures; and ii) examining the performance and define arrangements for new linear defences. This paper has described the physical modelling of the alternative cross-sections considered for the new linear rock armour defences intended for use at the eastern extreme of the area.

The physical modelling carried out at LNEC has tested a wide range of conditions in respect of both wave and water level combinations (1 in 1 year to 1 in 200 year return periods), and structure profiles (with varying armour slopes, crest levels, crest widths, crest forms and rear wall levels and forms). Eight alternative cross-sections were considered for the new linear defences.

The results of tests on Alternatives 1 to 8 showed a stable rock revetment profile with respect to rock displacement. Based on the results, the proposed primary amour, having 2 layers of 3 to 6 tonne rock at a 1 : 2.5 or 1 : 3 slope together with a 300 kg to 1 tonne rock filter layer, provides an adequate protection and a stable structure. Throughout the course of the tests, RH/LNEC felt there was potential for decreasing rock size without compromising the structure’s stability.

Regarding overtopping, for the type of alternatives tested, combining a milder slope, with a recurve crest wall and a permeable crest was very effective in reducing overtopping. Alternative 7 was the least overtopped structure, followed by Alternatives 8 and 5.

Alternatives 1, 2 and 8 were considered to represent the preferred form of Works that would meet interface most favourably with proposed hinterland arrangements whilst providing an appropriate standard of coastal defence, subject to the cost of the defences meeting required economic criteria for investment.

The final decision on which defence profile is appropriate requires design consideration of other aspects, particularly the level of risk that may be considered as acceptable, set against the additional costs of providing various standards of protection.

Acknowledgments

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