Detailed Modelling Studies For Colwyn Bay Coastal Defence Scheme

Physical Model Tests of New Linear Defences

Conwy County Borough Council
Bodlondeb
Conwy
LL32 8DU
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Final

Royal Haskoning
Stanley Hall
Edmund Street
Liverpool L3 9NG

ROYAL HASKONING

Laboratório Nacional de
Engenharia Civil, IP
Av. do Brasil 101
1700-066 Lisboa
Portugal

LABORATÓRIO NACIONAL DE ENGENHARIA CIVIL

A COMPANY OF

ROYAL HASKONING

HASKONING UK LTD.
COASTAL & RIVERS

Stanley Hall
Edmund Street
Liverpool, L3 9NG
United Kingdom
+44 (0) 151 236 2944 Telephone
Fax
info@haywards-heath.royalhaskoning.com E-mail
www.royalhaskoning.com Internet

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Author(s)	Maria Teresa Reis, Maria da Graça Neves, Luís Gabriel Silva, Keming Hu, Paul Winfield
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1 INTRODUCTION

Conwy County Borough Council (CCBC) formally accepted the revised tender for **Detailed Modelling Studies For Colwyn Bay Coastal Defence Scheme**, dated 9th July 2009, submitted by Royal Haskoning (RH) and Laboratório Nacional de Engenharia Civil, IP (LNEC).

Following the submission of the Inception Report, the technical meeting held at LNEC on the 3rd November 2009 and the preliminary results sent by LNEC to the Client since then, this Report has been prepared to summarise for the Client the outcome of the physical modelling of the alternative cross-sections considered for the new linear defences. These defences are primarily intended for use at the eastern frontage of the study area (Figure 1).

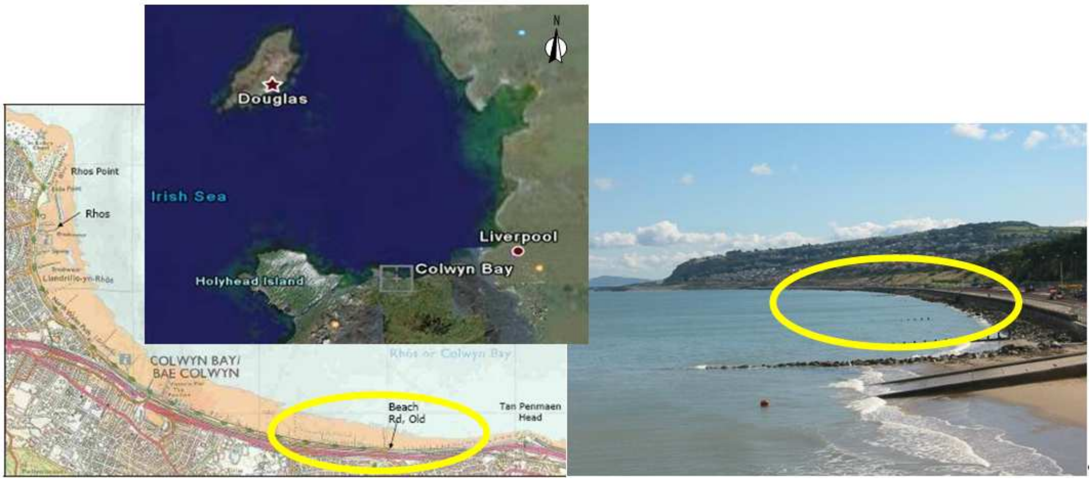


Figure 1 – Study area (adapted from Tender Brief).

The objective of the physical model tests is the analysis of the armour stability and wave overtopping performance of eight different cross-sections of the defences for the agreed combinations of water level and wave conditions.

This report has been prepared by LNEC and Royal Haskoning, in compliance with the Detailed Modelling Studies for Colwyn Bay Coastal Defence Scheme – Technical Proposal. It includes:

- a derivation of the design wave and water level conditions used in the tests;
- a brief description of the test facilities;
- a description of the physical models, including model scale adopted, cross-sections tested and foreshore profile constructed;
- the agreed test programme and wave conditions;
- a description of the equipment used in the model and measurements taken during the experiments;
- the results of the physical model tests; and
- the conclusions and recommendations.

2 DERIVING THE DESIGN WAVE AND WATER LEVEL CONDITIONS

The derivation of the joint probability of design wave and water level conditions is described in Appendix E. The joint probability of extreme waves and water levels with and without consideration of future sea level rise is presented in Tables 1 to 3.

Table 1 – Joint probability of design waves and water levels without sea level rise

Joint Return Period	Water Level (m ODN)	Wave Period (s Tm)	Wave Height (m Hs)	Target Condition with similar Hs/Tm
1 in 10 year	2.84	7.70	3.15	NOT NEEDED
	3.29	7.70	3.15	NOT NEEDED
	3.75	7.61	3.08	TC6
	4.21	7.38	2.91	TC22
	4.67	7.02	2.65	TC17
	5.13	5.63	1.81	TC25
1 in 50 year	3.29	8.21	3.56	NOT NEEDED
	3.75	8.17	3.54	NOT NEEDED
	4.21	8.03	3.42	TC23
	4.67	7.82	3.25	Between TC10 & TC11
	5.13	7.35	2.88	TC26
	5.58	5.58	1.71	TC13
1 in 200 year	3.29	8.68	3.97	NOT NEEDED
	3.75	8.67	3.96	TC4
	4.21	8.60	3.90	TC24
	4.67	8.41	3.74	TC18
	5.13	8.04	3.42	TC27
	5.58	7.57	3.05	TC20

Table 2 – Joint probability of design waves and water levels with 25 year sea level rise

Joint Return Period	Water Level (m ODN)	Wave Period (s Tm)	Wave Height (m Hs)	Target Condition with similar Hs/Tm
1 in 10 year	3.20	7.70	3.15	NOT NEEDED
	3.65	7.70	3.15	NOT NEEDED
	4.11	7.61	3.08	TC6
	4.57	7.38	2.91	
	5.03	7.02	2.65	
	5.48	5.63	1.81	TC25
1 in 50 year	3.65	8.21	3.56	NOT NEEDED
	4.11	8.17	3.54	
	4.57	8.03	3.42	TC23
	5.03	7.82	3.25	Between TC10 & TC11
	5.48	7.35	2.88	TC26
	5.94	5.58	1.71	TC13
1 in 100 year	3.65	8.42	3.74	NOT NEEDED
	4.11	8.34	3.68	
	4.57	8.25	3.60	
	5.03	8.03	3.42	TC11
	5.48	7.70	3.15	
	5.94	7.04	2.65	

Table 3 – Joint probability of design waves and water levels with 75 year sea level rise

Joint Return Period	Water Level (m ODN)	Wave Period (s Tm)	Wave Height (m Hs)	Target Condition with similar Hs/Tm
1 in 10 year	3.84	7.70	3.15	NOT NEEDED
	4.30	7.70	3.15	
	4.76	7.61	3.08	
	5.22	7.38	2.91	
	5.67	7.02	2.65	
	6.13	5.63	1.81	
1 in 50 year	4.30	8.21	3.56	TC23
	4.76	8.17	3.54	TC11
	5.22	8.03	3.42	TC27
	5.67	7.82	3.25	TC21
	6.13	7.35	2.88	
	6.59	5.58	1.71	
1 in 100 year	4.30	8.42	3.74	TC24
	4.76	8.34	3.68	TC11
	5.22	8.25	3.60	
	5.67	8.03	3.42	
	6.13	7.70	3.15	
	6.59	7.04	2.65	

It was impossible to run all above combinations of waves and water levels within the agreed budget. Table 4 presents the agreed 27 test conditions undertaken in this commission. The tests were chosen to allow evaluation of performance for a range of conditions which included potential worst case JP combinations identified from preliminary empirical overtopping assessment.

The last column of Tables 1 to 3 gives the numbers of the target conditions that have the similar wave and water level characteristics. The results of initial tests on “Alternative 1” show that wave overtopping almost did not happen for water level below +4.00 m ODN. The reason is that waves broke before reaching the toe of the structure based observation during physical model testing. Therefore, “NOT NEEDED” is marked on those conditions requiring no further tests on other alternative defence profiles.

Table 4 – Test conditions

Water Level (m ODN)	Tom (s)	Top (s)	Hos (m)	Target Condition No
3.5	8.5	10.6	2.5	TC1
			3.0	TC2
			3.5	TC3
			4.0	TC4
4.0	7.5	9.4	2.5	TC5
			3.0	TC6
			3.5	TC7
			4.0	TC8
4.8	8.0	10.0	2.5	TC9
			3.0	TC10
			3.5	TC11
			4.0	TC12
5.8	5.6	7.0	1.8	TC13
			2.0	TC14
			2.5	TC15
			3.0	TC16
4.8	7.0	8.8	2.7	TC17
	8.4	10.5	3.8	TC18
5.8	7.0	8.8	2.7	TC19
	7.5	9.4	3.0	TC20
	8.0	10.0	3.4	TC21
4.3	7.4	9.3	2.9	TC22
	8.0	10.0	3.4	TC23
	8.6	10.8	3.9	TC24
5.3	5.6	7.0	1.8	TC25
	7.4	9.3	2.9	TC26
	8.0	10.0	3.4	TC27

3 MODEL SETUP

3.1 Test Facilities

Two-dimensional (2D) physical model tests were performed at LNEC, between September 2009 and February 2010, in one of LNEC's wave flumes (Appendix D). 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, and it is possible to generate regular and irregular waves.

The different cross-sections tested were constructed close to the end of the flume, adjacent to a glass window, allowing visual observations to be made during testing (Appendix D).

3.2 Model Scale

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 are well reproduced in the model; significant scale effects are avoided; and the agreed test conditions can be reproduced in the selected facility with the resources available.

Froude scaling implies that the Froude number should be the same in the prototype and in the model. For the most relevant parameters used in the physical model, the scaling laws, defined as the ratio of the prototype to model measure, are:

- Length (m): $\lambda = 25$
- Volume (m^3): $\lambda^3 = 15625$
- Time (s): $\lambda^{0.5} = 5$
- Mass (kg): $\lambda^3 = 15625$
- Overtopping rate (l/s/m): $\lambda^{1.5} = 125$

3.3 Cross-Sections and Foreshore

Eight alternative cross-sections were constructed and tested, called here after Alternatives 1 to 8 (see Appendix A1). Table 5 presents the main characteristics of Alternative 1 and for Alternatives 2 to 8 it shows the differences from Alternative 1.

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 5 – 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

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.

In the physical model structures, the rock was chosen based on the rock gradings agreed with the Client's Technical Advisor, particularly M_{15}, M_{50} and M_{85} (Appendix A2).

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 corresponds to a prototype level of 0 m CD (-4.1 m ODN, Figure 2). 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.

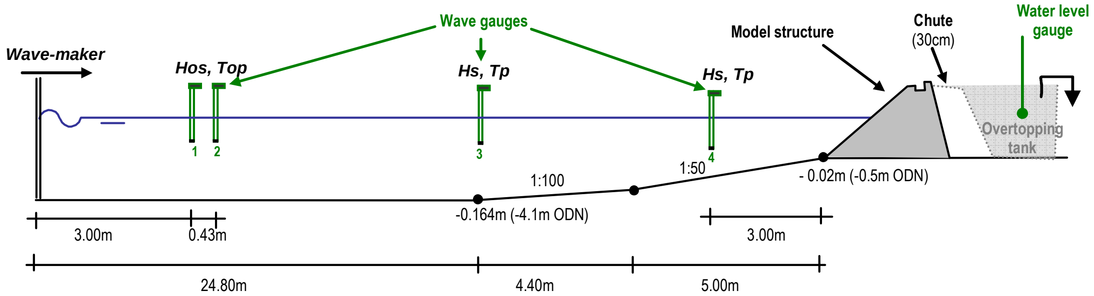


Figure 2 – Sketch of wave flume and location of experimental equipment (not to scale).

3.4 Test Programme and Wave Conditions

For each alternative cross-section, the test programme was agreed with the Client's Technical Advisor and specified a sequence of runs, each with predefined target values of significant wave height, Hos, and mean wave period, Tom, at -4.1 m ODN for each of the six water levels considered: +3.50 m ODN, +4.00 m ODN, +4.30 m ODN, +4.80 m ODN, +5.30 m ODN and +5.80 m ODN (see Table 4). Irregular waves conforming to the mean JONSWAP spectrum (with a peak enhancement factor of 3.3) were employed in the study. The target mean wave periods were used to determine the target peak periods, Top, using the relationship defined for a mean JONSWAP spectrum, Top=1.25 * Tom (Goda, 2000).

To measure the free-surface elevation, the flume was equipped with four resistive-type wave gauges (Figure 2 and Appendix D): a fixed array of two gauges (gauges 1 and 2), located in front of the wave-maker, 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, and spectral characteristics (significant wave heights and peak periods) were obtained for the four gauges (Appendix B1). In all test runs, the obtained significant wave height, Hos, and peak wave period, Top, at gauge 1 were compared to the target wave conditions agreed with the Client's Technical Advisor. The resulting differences were evaluated and subsequently accepted by the Client's Technical Advisor.

The run duration ranged from about 20 to 30 minutes (approximately 1000 waves). Run repetitions were carried out when requested by the Client's Technical Advisor.

3.5 Stability of Rock Armour

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:

- Alternative 1: 487 rocks
- Alternative 2: 487 rocks
- Alternative 3: 429 rocks
- Alternative 4: 429 rocks
- Alternative 5: 584 rocks
- Alternative 6: 547 rocks
- Alternative 7: 580 rocks
- Alternative 8: 490 rocks

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 (test repetitions were not filmed). After each test (a group of test runs characterised by the same target water level and wave period), the cumulative damage for the test was evaluated and the damaged sections of the structure were rebuilt.

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 %.

The results are presented in Appendix B2. Before and after photographs of the state of the armour of the various alternatives are presented in Appendix D for the test runs in which rock displacements occurred (the displacements are indicated in the photographs).

3.6 Wave Overtopping

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 2 and Appendix D). 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. The computer collected and stored the data in digital format 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. The measurement of the water level variation inside the tank, together with the pump calibration curve, allowed the determination of the overtopping volume per test run. The mean overtopping discharges per metre length of structure were obtained by dividing the overtopping volume by the run duration and by the width of the chute. The precision of the measurements of Q was ±0.005 l/s/m.

The mean overtopping discharges per metre length of structure obtained for the test runs were compared with the acceptable overtopping discharge that had been agreed with the Client’s Technical Advisor as being appropriate for this frontage, based on current guidance (Pullen *et al.*, 2007): that is, Q≤0.1 l/s/m.

A visual classification of the type of overtopping was also carried out according to LNEC’s overtopping criteria for tests carried out with irregular waves (presented in Appendix C).

The mean overtopping discharges per metre length of structure obtained for the test runs, as well as the corresponding visual classification of overtopping, are presented in Appendix B2.

Note that the effect of the wind was not reproduced in the model. For the wind effect on wave overtopping, the EurOtop manual makes suggestions based on limited laboratory data. The manual suggests that the wind effect may be significant for low wave overtopping rates. Given the relevance of the proposed crest berm and wave walls in reducing overtopping and the acceptable criterion of 0.1 l/s/m, the wind effect is not considered important for this project.

4 TEST RESULTS

4.1 Wave Conditions

The wave conditions measured in the model for all tested alternatives are shown in Appendix B1. In general, the wave conditions at gauge 1 agreed well with the target conditions requested by the Client's Technical Advisor, with a maximum relative error of 10 % (the relative error is determined by dividing the difference between measured and target conditions by the target conditions). However, for the lower water levels of +3.5 m ODN and +4.0 m ODN, most of the wave conditions were somewhat lower than the target values (with a maximum relative error of about 15 %), as much wave energy was lost due to depth limited conditions. The peak wave periods, Top, deviated from the target periods by a maximum relative error of 4 %. The differences were accepted by the Client's Technical Advisor during the course of the tests.

4.2 Stability

Alternatives 1 to 8 are very stable: the percentage of rock displacements was always smaller than 1 % (less than the maximum acceptable value of 5 % referred to as the *no damage* condition by CIRIA/CUR/CETMEF, 2007). When damage did occur it was only in the upper armour layer and no gaps down to the filter layer were visible (see Appendices B2 and D). The maximum number of displacements per test (4 test runs) was 5 (0.9 %) and occurred for Alternative 7 for a prototype water level of +4.8 m ODN. The results suggest that the observed displacements were mainly due to rock adjustments during the tests and that the rock armour in the different alternatives provides adequate protection and a stable structure. Furthermore, throughout the course of the tests, RH/LNEC felt there might be room for decreasing rock size without compromising the structure's stability.

4.3 Overtopping

4.3.1 Alternative 1

The performance of Alternative 1 was assessed for four water levels: +3.50 m ODN, +4.00 m ODN, +4.80 m ODN and +5.80 m ODN (see Appendix B2). The mean overtopping discharges obtained for the lowest water levels of +3.5 m ODN and +4.0 m ODN do not exceeded the overtopping criterion agreed with the Client's Technical Advisor (0.1 l/s/m), except for test run 9. The mean overtopping discharges obtained for the highest water levels of +4.8 m ODN and +5.8 m ODN exceeded the criterion for most test runs (11 to 13 and 15 to 17), with Q reaching maximum values of 7.5 l/s/m (test run 13) and 3.7 l/s/m (test run 17), respectively.

Based on the results of 17 tests, it was estimated that Alternative 1 would provide less than 1 in 50 year standard of service without consideration of future sea level rise, reducing to about 1 in 10 year standard of service with consideration of sea level rise in the next 75 years, based on the current DEFRA guidance.

4.3.2 Alternative 2

The performance of Alternative 2 was assessed for water levels of +4.80 m ODN and +5.80 m ODN (see Appendix B2). The mean overtopping discharges obtained for Alternative 2 for test runs 19 to 21 and 25, exceeded the overtopping criterion agreed with the Client’s Technical Advisor, with Q reaching a maximum value of 2.2 l/s/m for a water level of +4.8 m ODN (test run 21) and 0.5 l/s/m for +5.8 m ODN (test run 25).

As expected, given the higher crest level of the wall, Alternative 2 showed lower discharges compared with Alternative 1 for all wave conditions tested (especially for the highest values of significant wave heights), with a reduction that varied between 25 % and 90 % (the relative difference in discharges between any two alternatives A and B, reduction or increase, is determined by dividing the difference between the discharges for alternatives A and B by the discharge for alternative A).

Based on the results of 8 tests, it was estimated that Alternative 2 would provide approximately 1 in 50-100 year standard of service without consideration of future sea level rise, reducing to above 1 in 20-50 year standard of service with consideration of sea level rise in the next 75 years, based on the current DEFRA guidance.

4.3.3 Alternative 3

The performance of Alternative 3 was assessed for +3.50 m ODN, +4.00 m ODN, +4.80 m ODN and +5.80 m ODN (see Appendix B2). Unlike for the lowest water levels of +3.5 m ODN and +4.0 m ODN, the mean overtopping discharges obtained for the highest water levels of +4.8 m ODN and +5.8 m ODN exceeded the Client overtopping criterion for most test runs (34 to 37 and 39 to 41), with Q reaching maximum values of 7.5 l/s/m (test run 37) and 4.1 l/s/m (test run 41), respectively.

For the lowest water levels, the mean discharges for Alternatives 1 and 3 are of the same order of magnitude, with Q reaching a maximum value of 0.06 l/s/m for a water level of +4.00 m ODN (test run 33). For the highest water levels, there is generally an increase in overtopping from Alternative 1 to Alternative 3, which ranged between 11 % and 41 %. This increase in overtopping is due to a steeper armour slope and a reduction in the total permeability of the structure (part of the rock armour was replaced by a wider impermeable crest). However, the maximum mean overtopping discharge measured in both alternatives was 7.5 l/s/m for a water level of +4.8 m ODN.

Based on the results of 16 tests, it was estimated that Alternative 3 would provide 1 in 20-50 year standard of service without consideration of future sea level rise, reducing to about 1 in 10 year standard of service with consideration of sea level rise in the next 75 years based on the current DEFRA guidance.

4.3.4 Alternative 4

The performance of Alternative 4 was assessed for the water level of +4.8 m ODN only (see Appendix B2). The mean overtopping discharges obtained for Alternative 4 for test runs 43 to 45 exceeded the overtopping criterion agreed with the Client’s Technical Advisor, with Q reaching a maximum value of 3.5 l/s/m (test run 45).

As expected, given the higher crest level of the wall, Alternative 4 showed lower discharges than Alternative 3 for all wave conditions tested (the reduction varied between 11 % and 53 %).

Based on the results of 4 tests, it was estimated that Alternative 4 would provide 1 in 20-50 year standard of service without consideration of future sea level rise, reducing to < 1 in 20 year standard of service with consideration of sea level rise in the next 75 years based on the current DEFRA guidance.

4.3.5 Alternative 5

The performance of Alternative 5 was assessed for water levels of +4.80 m ODN and +5.80 m ODN (see Appendix B2). The mean overtopping discharges obtained for Alternative 5 for test runs 47 to 49 and 53 exceeded the overtopping criterion agreed with the Client's Technical Advisor, with Q reaching a maximum value of 0.59 l/s/m for a water level of +4.8 m ODN (test run 49) and 0.13 l/s/m for +5.8 m ODN (test run 53).

As expected, given the wider permeable crest, Alternative 5 showed lower discharges than Alternative 4 for a water level of +4.8 m ODN (the reduction varied between 80 % and 85 %). Comparison of Alternatives 3 and 5 for a water level of +4.8 m ODN shows a reduction in overtopping that varied between 82 % and 92 %, which suggests that the impact of the wider permeable crest is greater than the impact of the higher crest level of the wall.

Alternative 5 also showed lower discharges than Alternative 3 for a water level of +5.8 m ODN (the reduction varied between 94 % and 100 %).

Based on the results of 8 tests, it was estimated that Alternative 5 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 based on the current DEFRA guidance.

4.3.6 Alternative 6

The performance of Alternative 6 was assessed for water levels of +4.80 m ODN and +5.80 m ODN (see Appendix B2). The mean overtopping discharges obtained for Alternative 6 for test runs 55 to 57 and 60 to 61 exceeded the overtopping criterion agreed with the Client's Technical Advisor, with Q reaching a maximum value of 1.5 l/s/m for a water level of +4.8 m ODN (test run 57) and 0.42 l/s/m for +5.8 m ODN (test run 61).

Alternative 6 showed higher discharges than Alternative 5 for both water levels (an increase between 49 % and 300 %) but showed discharges equal to or lower than Alternative 2 for most test conditions (a reduction between 16 % and 51 %).

Based on the results of 8 tests, it was estimated that Alternative 6 would provide above 1 in 50 year standard of service without consideration of future sea level rise, reducing to about 1 in 20 year standard of service with consideration of sea level rise in the next 75 years based on the current DEFRA guidance.

4.3.7 Alternative 7

The performance of Alternative 7 was assessed for +4.80 m ODN and +5.80 m ODN (see Appendix B2). The mean overtopping discharges obtained for Alternative 7 for test runs 65 and 69 exceeded slightly the overtopping criterion agreed with the Client's Technical Advisor, with Q reaching a maximum value of 0.17 l/s/m for a water level of +4.8 m ODN (test run 65) and 0.11 l/s/m for +5.8 m ODN (test run 69).

As expected, given the higher levels of the crest berm and of the recurve wall, Alternative 7 showed lower discharges than Alternative 6 for all wave conditions tested (the reduction varied between 75 % and 100 %). It also showed lower discharges than Alternative 5 (a reduction between 20 % and 100 %).

Based on the results of 8 tests, it was estimated that Alternative 7 would provide generally 1 in 100 year standard of service without consideration of future sea level rise, reducing to 1 in 50 year standard of service with consideration of sea level rise in the next 75 years based on the current DEFRA guidance.

4.3.8 Alternative 8

The performance of Alternative 8 was assessed for +4.30 m ODN, +4.80 m ODN, +5.30 m ODN and +5.80 m ODN (see Appendix B2). The mean overtopping discharges obtained for Alternative 8 for test runs 72, 73, 75, 79, 81, 82, 85, 87 and 88 exceeded the overtopping criterion agreed with the Client's Technical Advisor, with Q reaching a maximum value of 0.17 l/s/m for a water level of +4.3 m ODN (test run 85), 0.44 l/s/m for +4.8 m ODN (test run 73), 0.59 l/s/m for +5.3 m ODN (test run 88) and 5.3 l/s/m for +5.8 m ODN (test run 82).

Alternative 8 showed higher discharges than Alternative 7 for all tested conditions (an increase between 37 % and 220 %) but showed discharges equal to or lower than Alternative 5 for most test conditions (a reduction between 25 % and 57 %). There is also a decrease in overtopping from Alternative 1 to Alternative 8, which ranged between 75 % and 100 %.

Based on the results of 21 tests, it was estimated that Alternative 1 would provide generally 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 based on the current DEFRA guidance.

4.3.9 Comparison of Alternatives

The mean overtopping discharges per metre length of structure, Q (l/s/m), for all the alternatives tested are compared in Figures 3 to 6 for the different test conditions shown in Appendix B1. As the figures show, Alternative 7 was the least overtopped structure, followed by Alternatives 8 and 5. Nevertheless, for TC12 and TC16, the values of Q for Alternative 7 (0.17 l/s/m and 0.11 l/s/m, respectively) were still slightly greater than the overtopping criterion agreed with the Client's Technical Advisor (0.1l/s/m). For Alternative 8, the values of Q were greater than the overtopping criterion for TC11, TC12, TC16, TC18, TC20, TC21, TC24, TC26 and TC27 (0.30 l/s/m, 0.44 l/s/m, 0.15 l/s/m, 0.26 l/s/m, 1.3 l/s/m, 5.3 l/s/m, 0.17 l/s/m, 0.20 l/s/m and 0.59 l/s/m, respectively) and for Alternative 5 for TC10 to TC12 and TC16 (0.13 l/s/m, 0.68 l/s/m, 0.59 l/s/m and 0.13 l/s/m, respectively).

The greatest overtopping discharges were obtained for Alternatives 1 and 3 for TC12 (maximum value of 7.5 l/s/m).

The assessment of standard of service can only be considered as indicative due to the wide range of performance for different combinations of waves and water levels with the same Joint Probability of occurrence, with usually one combination providing much worse performance than the others. Assessments are based on evaluation of the majority of conditions applying.

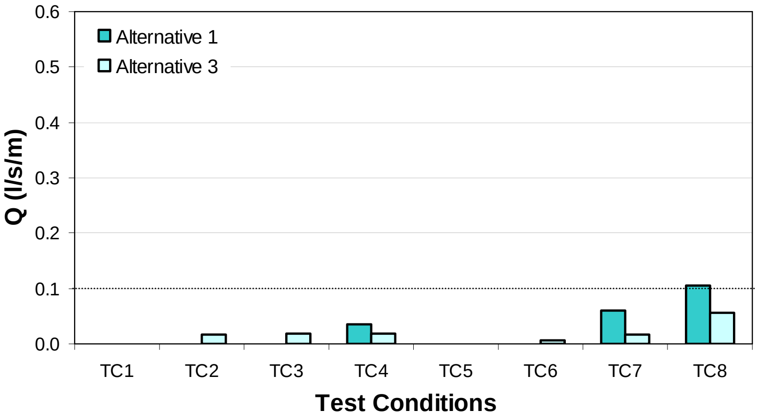


Figure 3 – Mean overtopping discharges obtained for the still-water-levels of +3.5 m ODN (TC1 to TC4) and +4.0 m ODN (TC5 to TC8).

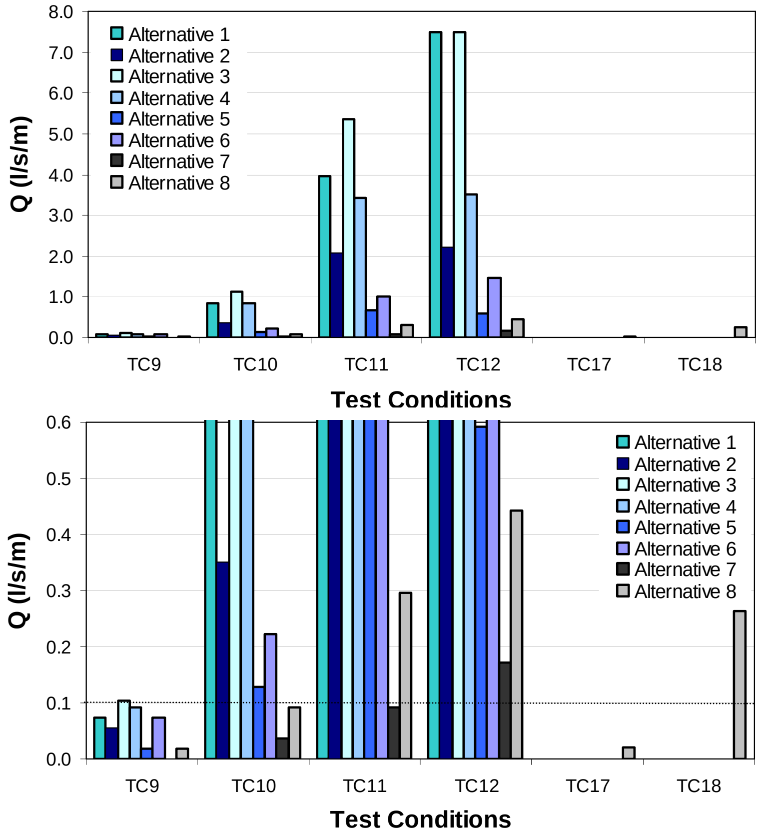


Figure 4 – Mean overtopping discharges obtained for the still-water-level of +4.8 m ODN (TC9 to TC12, TC17 and TC18).

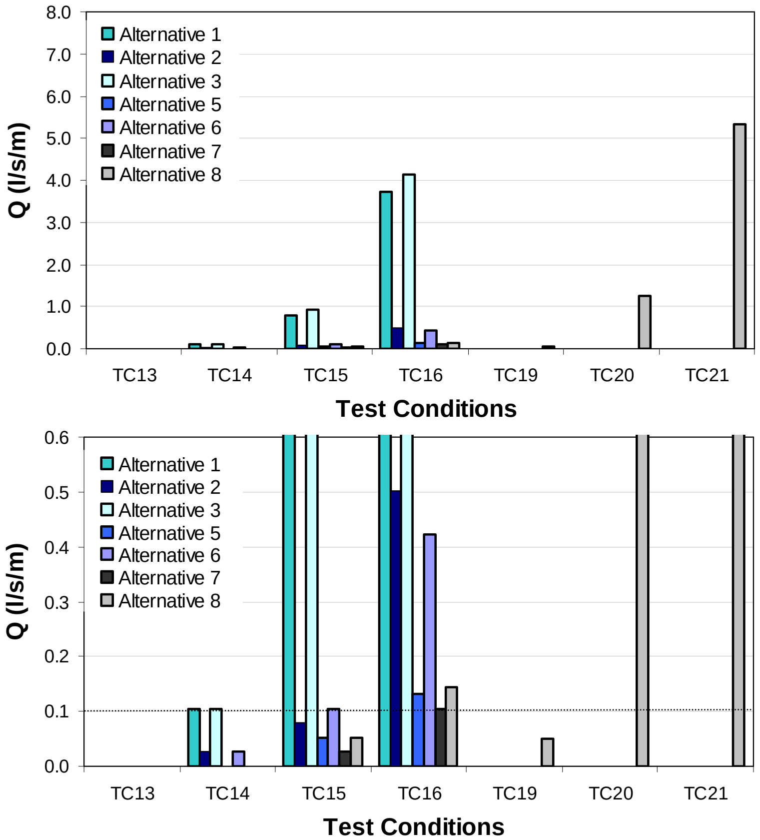


Figure 5 – Mean overtopping discharges obtained for the still-water-level of +5.8 m ODN (TC13 to TC16 and TC19 to TC21).

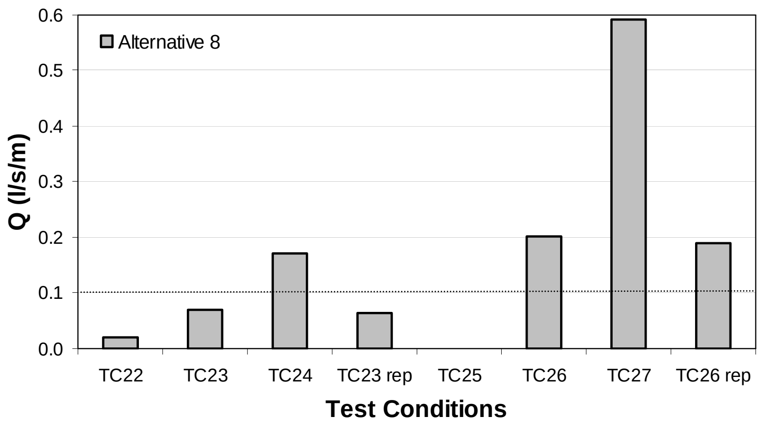


Figure 6 – Mean overtopping discharges obtained for the still-water-levels of +4.3 m ODN (TC22 to TC24 and repetition of TC23) and +5.3 m ODN (TC25 to TC27 and repetition of TC26).

5 DISCUSSIONS, CONCLUSIONS AND RECOMMENDATIONS

Eight alternative cross-sections considered for the new linear defences were studied:

- Alternative 1: a rock revetment with a concrete slab maintenance access roadway arrangement 4.50 m wide, at about +7.00 m ODN, and a concrete wave wall on the landward side, with its crest at +8.00 m ODN. The primary armour has 2 layers of 3 to 6 tonne rock, in a 1 : 3 slope, with a crest berm of approximately 3.50 m (3 rocks) at +7.50 m ODN, and a 300 kg to 1 tonne rock filter layer;
- Alternative 2: similar to Alternative 1 with an increased elevation of the crest wall (+8.50 m ODN);
- Alternative 3: similar to Alternative 1 with an increased steepness of the armour slope (1 : 2.5) and a wider concrete maintenance access roadway (7.97 m);
- Alternative 4: similar to Alternative 3 with an increased elevation of the crest wall (+8.50 m ODN);
- Alternative 5: similar to Alternative 4 with an increased width of the rock berm (approximately 11.47 m wide, 10 rocks) and removal of the concrete maintenance access roadway;
- Alternative 6: similar to Alternative 2 with an increased elevation of the crest berm (+8.00 m ODN), which is narrower (no concrete maintenance access roadway and an approximately 4.70 m wide rock berm, 4 rocks), and a recurve concrete wall with its crest at +9.00 m ODN;
- Alternative 7: similar to Alternative 6 with an increased elevation of the crest berm and of the recurve wall (+9.00 m ODN and +10.00 m ODN, respectively); different toe detail from previous alternatives;
- Alternative 8: similar to Alternative 1 with a recurve concrete wall with its crest at +8.15 m ODN, an increased width of the rock berm (approximately 4.70 m wide, 4 rocks); toe detail similar to Alternative 7.

The outcome of the physical modelling of the eight alternatives can be summarised as follows:

Rock Stability

The results of tests on Alternatives 1 to 8 show 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 adequate protection and a stable structure. Throughout the course of the tests, RH/LNEC felt there might be room for decreasing rock size without compromising the structure’s stability.

Wave Overtopping

The first alternative modelled was developed from the preliminary section identified in the Colwyn Bay Coastal Defence Strategy Plan (Conwy County Borough Council, 2007) and empirical evaluation of overtopping performance. The subsequent arrangements were tested to evaluate the impact of changing, specifically, crest arrangements, in order to provide data on scheme performance and examine how interaction with the existing hinterland could most appropriately be effected.

When considering the test results of Alternatives 1 and 2, they show that overtopping of Alternative 1 exceeds the acceptable criterion (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 test runs 19 to 21 and 25, i.e. TC10 to TC12 and TC16 (see Appendix B2). Increaasing the crest level of the rear wall improved the standard of service from below 1 in 50 year 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 demonstrate that a steeper slope 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 show that a wider rock berm was effective in reducing overtopping. Alternative 5 would provide similar performance to option 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 based on the current DEFRA guidance.

In Alternatives 6 and 7, the concrete berm was removed compared with Alternative 2, leaving only a rock berm 4.7 m wide. With a narrower overall berm width, i.e. having removed the concrete element of the berm, to control overtopping rates within the required criterion, the rock berm needs to be at +9 m ODN with 1 m high recurve crest wall on top (crest level +10 m ODN). Although the overtopping rates were above the criterion in test runs 65 and 69 (TC12 and TC16, respectively) in Alternative 7 and the wind was not reproduced in the model, they may be considered acceptable, bearing in mind that they only exceed the target rate by a small margin.

Among all tested profiles, Alternative 7 produced the best performance with respect to overtopping. It would provide 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 based on the current DEFRA guidance.

These two profiles however require significantly higher crest levels, 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 based on the current DEFRA guidance.

Conclusions & Recommendations

The physical modelling has tested a wide range of conditions in respect of both wave and water level combinations, typically ranging from 0.5 % - 99.9 % annual probability of occurrence (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).

The modelling has also shown that different combinations of waves and water levels with the same joint probability of occurrence can produce dramatically different overtopping performance.

We believe that the acceptance of risk plays a significant part in determining the appropriate level of protection to be adopted.

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.

On the basis of the modelling carried out – Alternatives 1, 2 and 8 are considered to represent the preferred form of Works that will meet specific hinterland and regeneration objectives whilst providing appropriate standard of coastal defence, subject to them meeting required economic criteria for investment. The modelling results provide the necessary data from which a preferred cross section can be developed.

6 REFERENCES

Capitão, R. (2002). Modelação estocástica numérica e física da agitação maritime. PhD Thesis, Technical University of Lisbon. (In Portuguese)

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

Conwy County Borough Council (2007). Colwyn Bay Coastal Defence Strategy Plan. Stage 2: Strategic Assessment and Proposals – Draft for Consultation.

Goda, Y. (2000). Random Seas and Design of Maritime Structures. Advanced Series on Ocean Engineering, Volume 15, World Scientific.

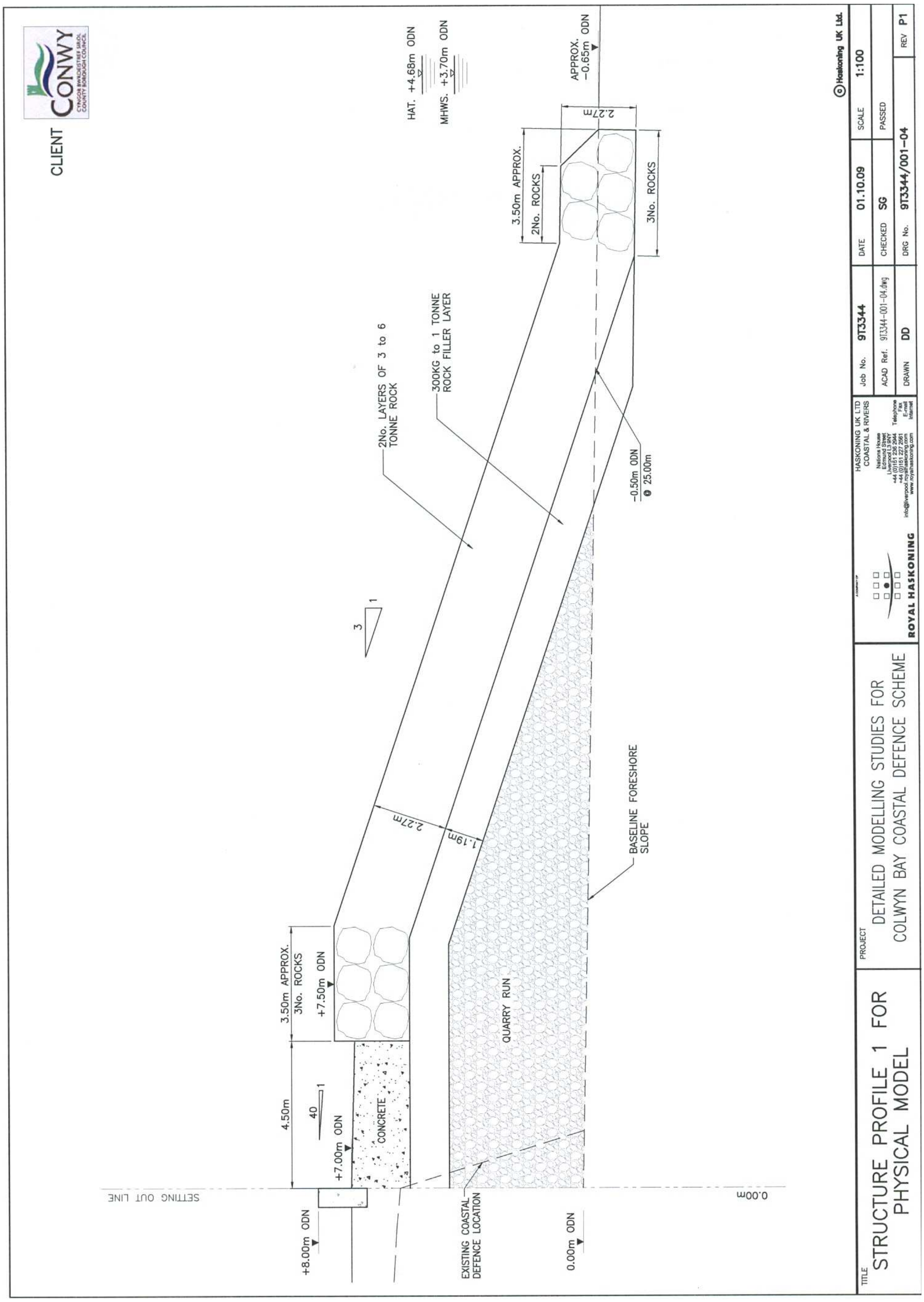
Pullen, T.; Allsop, N.W.H.; Bruce, T.; Kortenhaus, A.; Schuttrumpf, H.; Van der Meer, J.W. (2007). EurOtop: Wave Overtopping of Sea Defences and Related Structures: Assessment Manual. Environment Agency, UK, Expertise Netwerk Waterkeren, NL, and Kuratorium fur Forschung im Kusteningenieurwesen, DE, August.

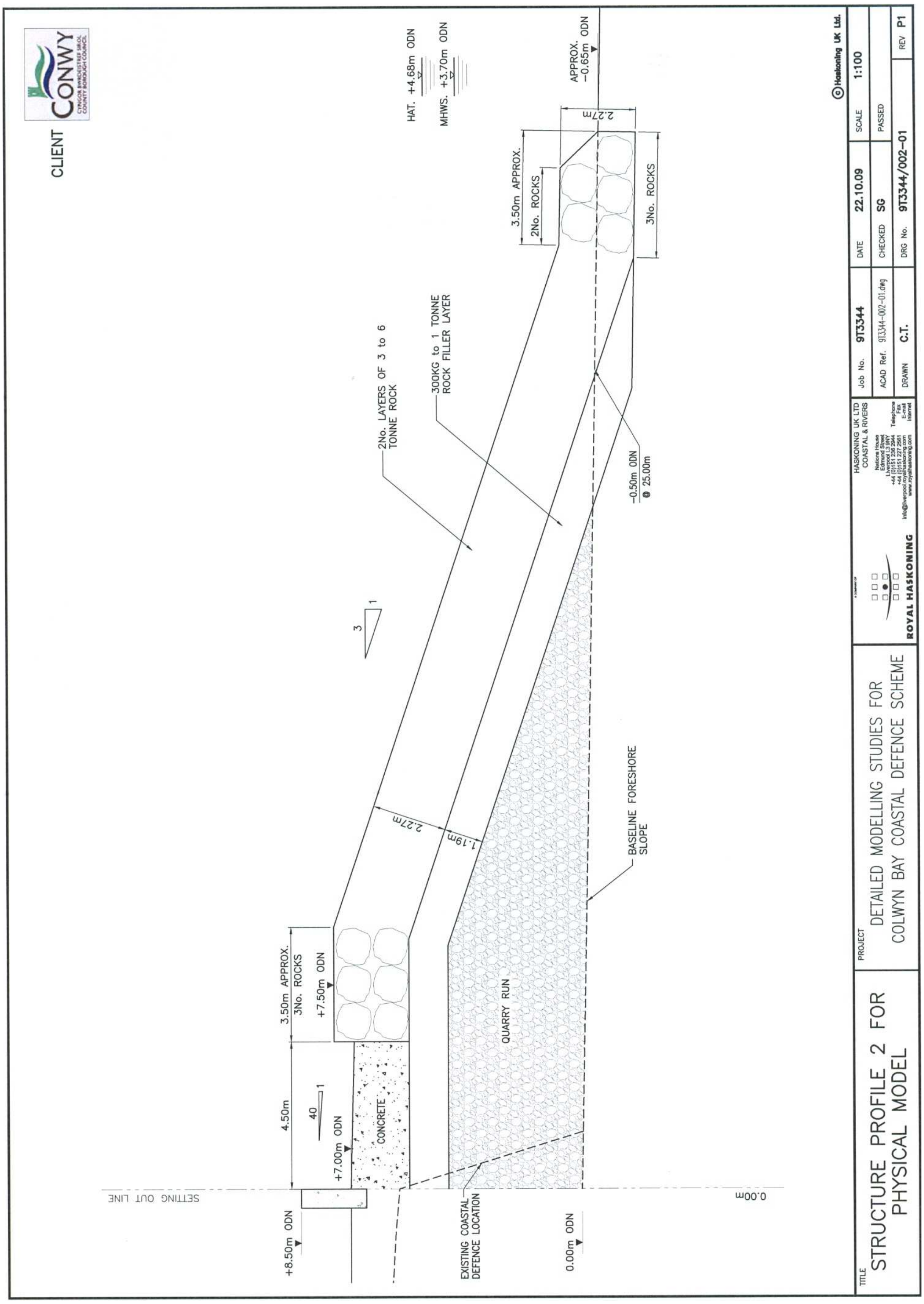
Troch, P. (2005). User Manual: Active Wave Absorption System, Gent University, Department of Civil Engineering, Denmark.

APPENDIX A

Alternative Cross-Sections and Rock Gradings

A.1. Alternative Cross-Sections

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3No. ROCKS
2No. LAYERS OF 3 to 6 TONNE ROCK
300KG to 1 TONNE ROCK FILLER LAYER
−0.50m ODN @ 25.00m
3
1
2.27m
1.19m
BASELINE FORESHORE SLOPE
3.50m APPROX.
3No. ROCKS
+7.50m ODN
QUARRY RUN
4.50m
40
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+7.00m ODN
CONCRETE
SETTING OUT LINE
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0.00m ODN
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ROYAL HASKONING
HASKONING UK LTD COASTAL & RIVERS
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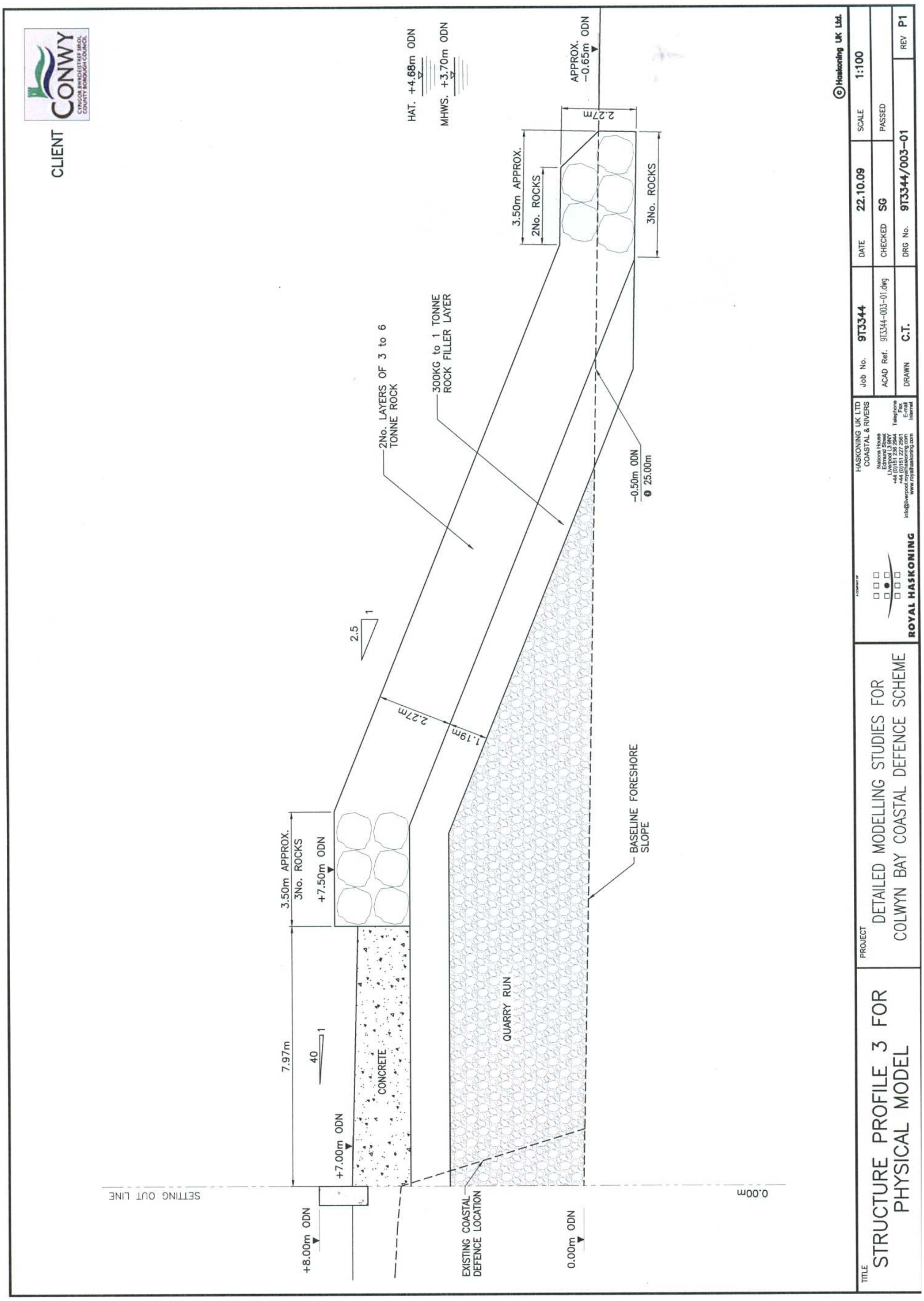
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300KG to 1 TONNE ROCK FILLER LAYER
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@ 25.00m
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1
2.27m
1.19m
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3No. ROCKS
+7.50m ODN
7.97m
40
1
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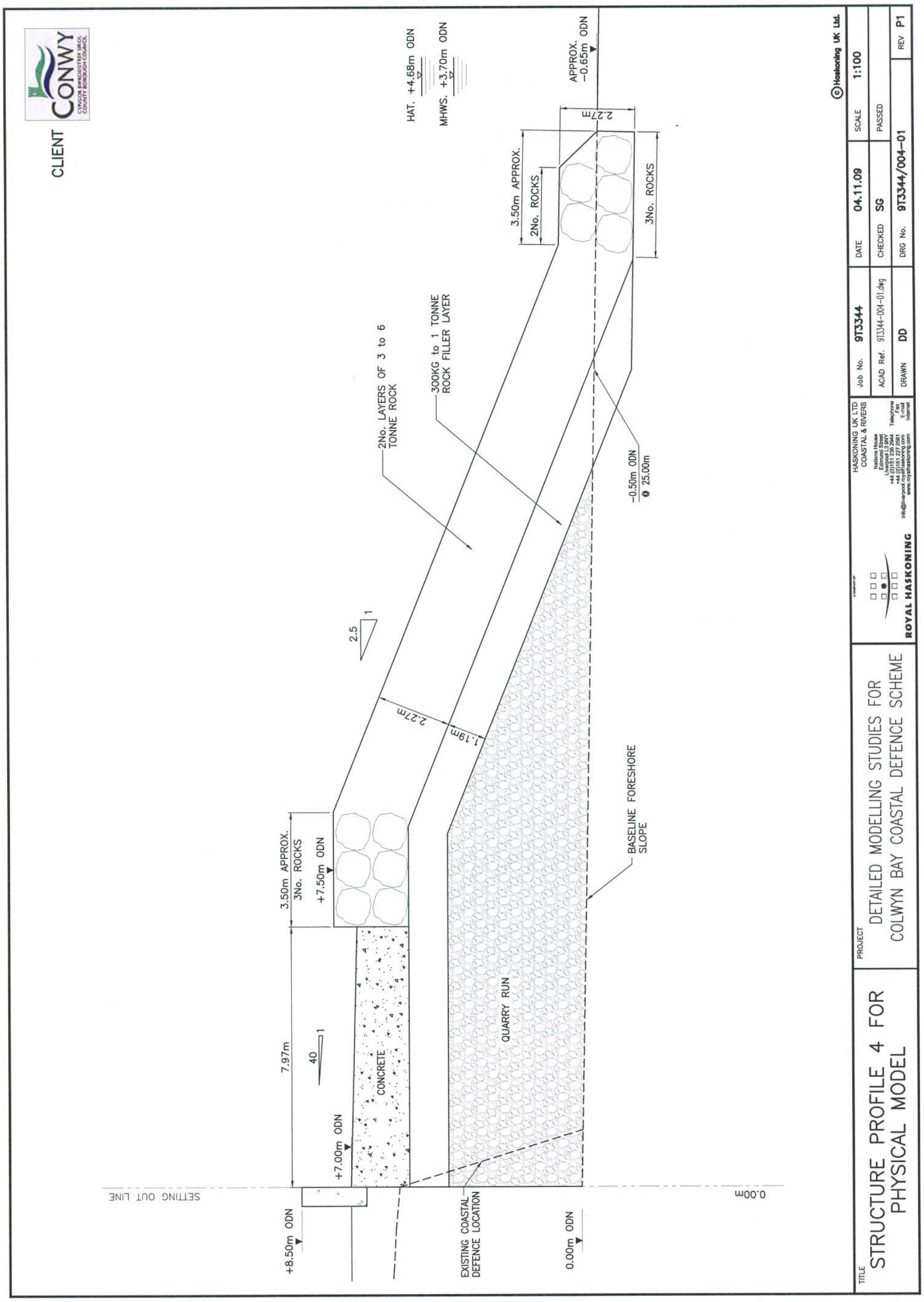
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1.19m
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CONCRETE
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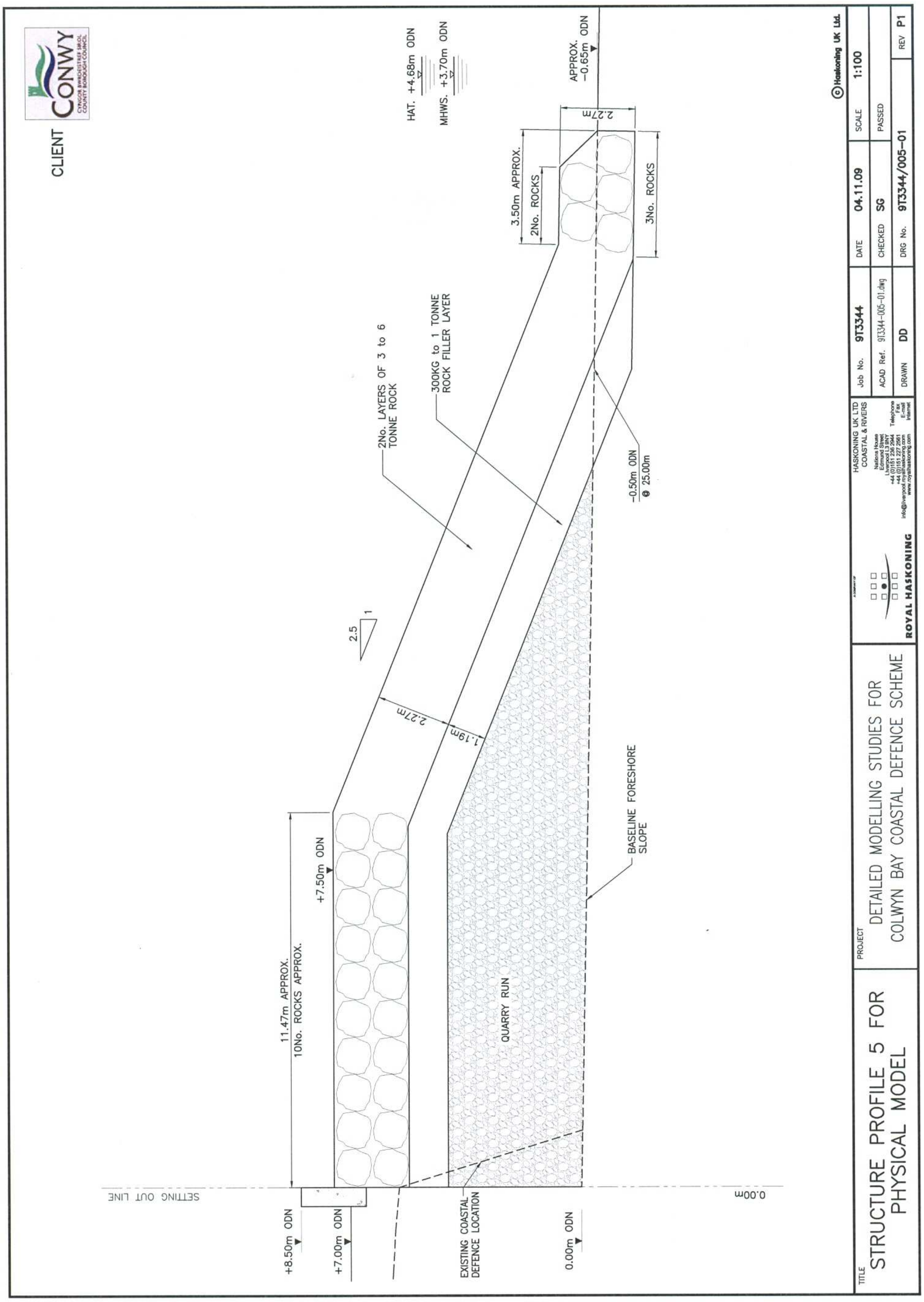
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2.27m
1.19m
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QUARRY RUN
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+7.50m ODN
+8.50m ODN
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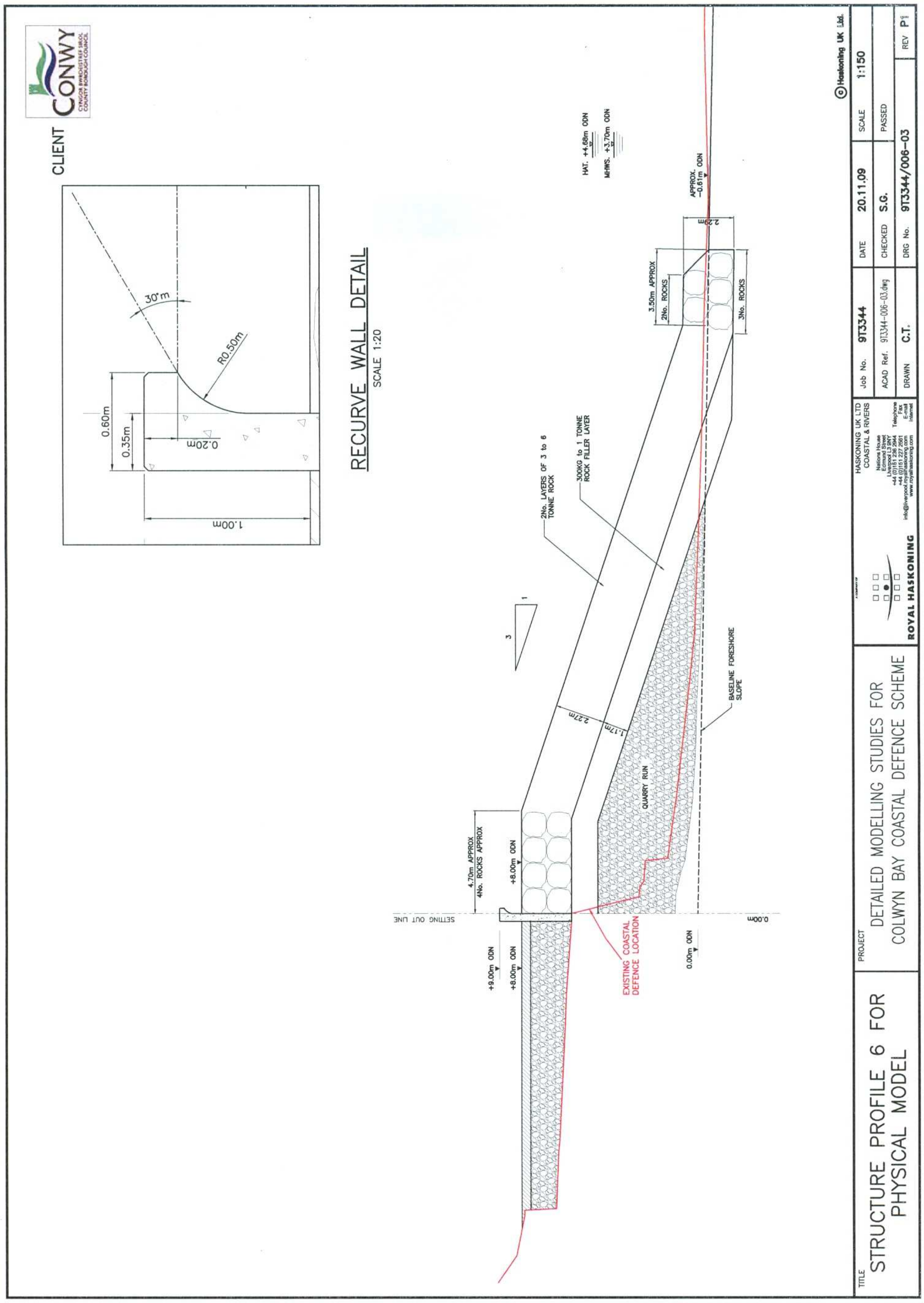
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3No. ROCKS
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300KG to 1 TONNE ROCK FILLER LAYER
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1.17m
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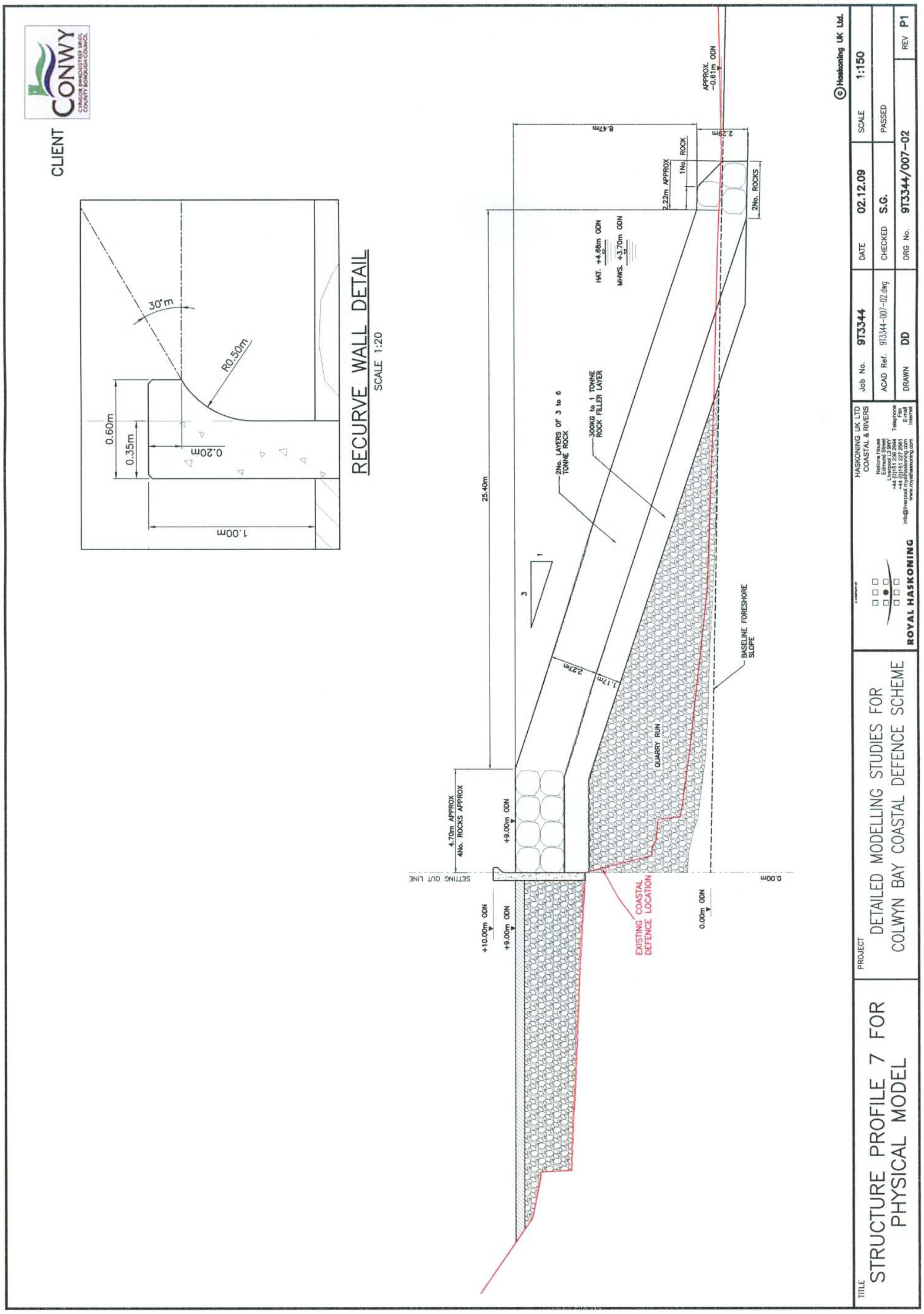
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+10.00m ODN
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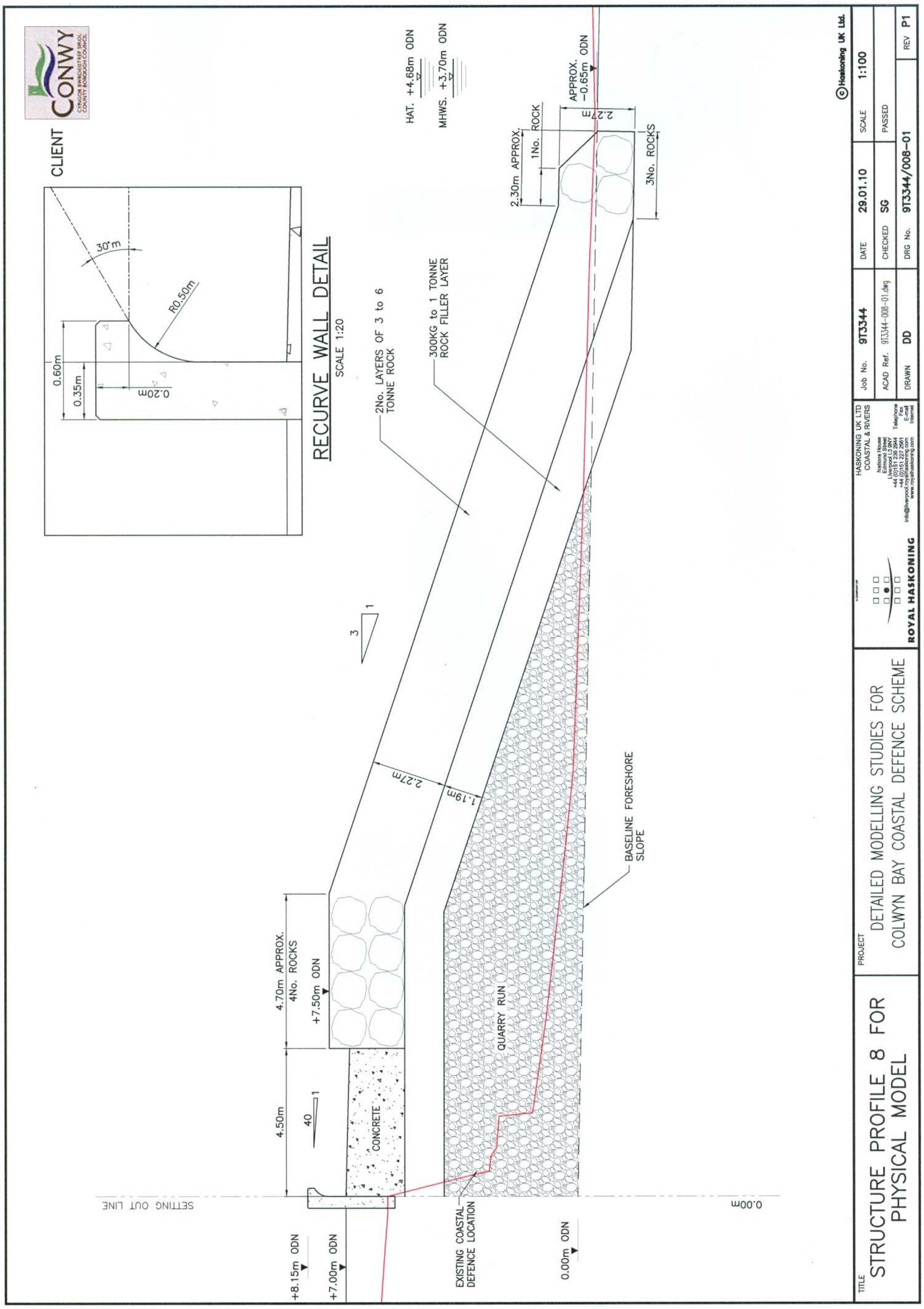
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1.19m
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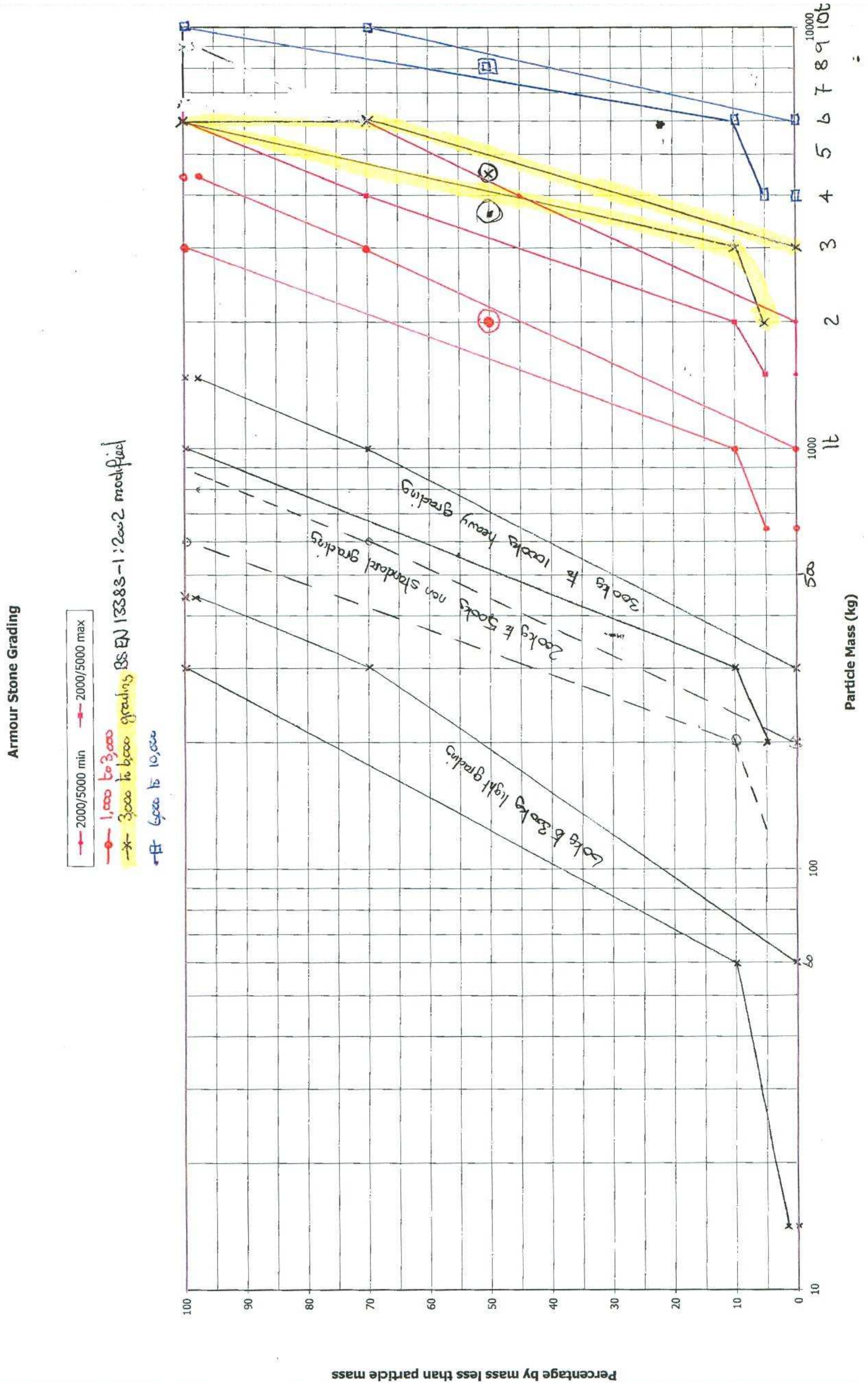
A.2. Rock Gradings (Prototype Values)



APPENDIX B

Tables of Test Conditions and Results

B.1. Test Conditions

Target Conditions at -4.1m ODN

Water Level (m ODN)	Tom (s)	Top (s)	Hos (m)	Target Condition No
3.5	8.5	10.6	2.5	TC1
			3.0	TC2
			3.5	TC3
			4.0	TC4
4.0	7.5	9.4	2.5	TC5
			3.0	TC6
			3.5	TC7
			4.0	TC8
4.8	8.0	10.0	2.5	TC9
			3.0	TC10
			3.5	TC11
			4.0	TC12
5.8	5.6	7.0	1.8	TC13
			2.0	TC14
			2.5	TC15
			3.0	TC16
4.8	7.0	8.8	2.7	TC17
	8.4	10.5	3.8	TC18
5.8	7.0	8.8	2.7	TC19
	7.5	9.4	3.0	TC20
	8.0	10.0	3.4	TC21
4.3	7.4	9.3	2.9	TC22
	8.0	10.0	3.4	TC23
	8.6	10.8	3.9	TC24
5.3	5.6	7.0	1.8	TC25
	7.4	9.3	2.9	TC26
	8.0	10.0	3.4	TC27

Alternative 1

Test Run		Water Level (m ODN)	Gauge 1		Gauge 2		Gauge 3		Gauge 4	
			Hos (m)	Top (s)	Hos (m)	Top (s)	Hs (m)	Tp (s)	Hs (m)	Tp (s)
1	---	3.5	1.9	11.0	1.8	10.7	1.7	11.1	1.7	12.4
2	TC1		2.3	10.6	2.3	10.6	2.1	10.6	2.1	12.5
3	TC2		2.6	10.7	2.6	10.7	2.4	10.7	2.4	13.8
4	TC3		3.0	10.5	2.9	10.6	2.7	12.6	2.7	12.6
5	TC4		3.4	10.4	3.3	10.4	3.0	10.9	3.0	13.6
5'	---	4.0	1.9	9.5	1.9	9.5	1.9	9.5	1.9	9.5
6	TC5		2.3	9.3	2.2	9.4	2.2	9.3	2.2	9.3
7	TC6		2.6	9.4	2.6	9.4	2.5	9.6	2.5	8.7
8	TC7		3.0	9.6	2.9	9.6	2.8	9.5	2.8	12.4
9	TC8		3.6	9.4	3.5	9.4	3.2	10.2	3.2	12.1
10	TC9	4.8	2.5	9.9	2.5	10.1	2.3	10.4	2.3	12.1
11	TC10		3.0	10.3	3.0	10.1	2.7	10.3	2.7	11.5
12	TC11		3.6	10.3	3.6	10.4	3.2	10.3	3.2	11.5
13	TC12		3.8	10.2	3.8	10.2	3.3	10.5	3.3	12.3
14	TC13	5.8	1.8	7.1	1.8	7.0	1.5	7.1	1.5	7.0
15	TC14		2.0	7.1	2.0	6.8	1.7	7.1	1.7	6.8
16	TC15		2.6	7.1	2.5	7.0	2.2	7.1	2.1	6.8
17	TC16		2.9	6.9	2.9	7.0	2.5	7.1	2.4	6.9

Alternative 2

Test Run		Water Level (m ODN)	Gauge 1		Gauge 2		Gauge 3		Gauge 4	
			Hos (m)	Top (s)	Hos (m)	Top (s)	Hs (m)	Tp (s)	Hs (m)	Tp (s)
18	TC9	4.8	2.5	9.9	2.5	10.1	2.3	10.3	2.3	11.6
19	TC10		3.0	10.1	3.0	10.1	2.8	10.3	2.8	11.3
20	TC11		3.6	9.9	3.6	10.4	3.2	10.3	3.2	12.5
21	TC12		3.8	10.2	3.8	10.2	3.3	12.3	3.3	12.3
22	TC13	5.8	1.8	7.1	1.8	7.0	1.6	7.1	1.5	6.8
23	TC14		2.0	7.1	2.0	7.1	1.8	7.1	1.7	6.6
24	TC15		2.6	7.1	2.6	7.0	2.3	7.0	2.2	6.8
25	TC16		3.0	6.9	2.9	7.0	2.6	7.1	2.5	6.9

Alternative 3

Test Run		Water Level (m ODN)	Gauge 1		Gauge 2		Gauge 3		Gauge 4	
			Hos (m)	Top (s)	Hos (m)	Top (s)	Hs (m)	Tp (s)	Hs (m)	Tp (s)
26	TC1	3.5	2.3	11.0	2.4	11.1	2.1	11.0	2.2	12.3
27	TC2		2.6	10.7	2.7	10.7	2.5	10.7	2.5	13.2
28	TC3		3.0	10.5	3.0	10.6	2.7	10.7	2.7	12.6
29	TC4		3.5	10.4	3.4	10.5	2.9	10.9	2.9	13.5
30	TC5	4	2.4	9.3	2.4	9.4	2.2	9.4	2.1	9.4
31	TC6		2.7	9.4	2.6	9.1	2.4	10.6	2.4	12.5
32	TC7		3.0	9.6	3.0	9.5	2.7	9.5	2.7	12.0
33	TC8		3.6	9.2	3.5	9.4	3.1	10.1	3.1	12.1
34	TC9	4.8	2.6	9.8	2.6	10.1	2.4	10.4	2.4	12.1
35	TC10		3.1	10.3	3.0	10.1	2.8	10.3	2.8	11.3
36	TC11		3.7	10.3	3.7	10.4	3.3	10.3	3.3	12.9
37	TC12		3.9	10.2	3.9	10.2	3.3	10.1	3.3	12.1
38	TC13	5.8	1.8	6.9	1.8	7.0	1.6	7.0	1.5	6.6
39	TC14		2.1	7.1	2.0	6.8	1.8	7.1	1.8	6.6
40	TC15		2.6	7.0	2.6	7.0	2.3	7.0	2.2	6.8
41	TC16		3.0	6.8	3.0	7.0	2.6	7.1	2.5	6.6

Alternative 4

Test Run		Water Level (m ODN)	Gauge 1		Gauge 2		Gauge 3		Gauge 4	
			Hos (m)	Top (s)	Hos (m)	Top (s)	Hs (m)	Tp (s)	Hs (m)	Tp (s)
42	TC9	4.8	2.6	10.2	2.6	10.1	2.6	10.3	2.4	12.1
43	TC10		3.1	10.1	3.1	10.1	3.1	10.3	2.8	11.5
44	TC11		3.7	9.9	3.7	10.4	3.7	10.3	3.2	11.5
45	TC12		3.9	10.2	3.9	10.2	3.9	10.4	3.3	12.4

Alternative 5

Test Run		Water Level (m ODN)	Gauge 1		Gauge 2		Gauge 3		Gauge 4	
			Hos (m)	Top (s)	Hos (m)	Top (s)	Hs (m)	Tp (s)	Hs (m)	Tp (s)
46	TC9	4.8	2.6	10.3	2.6	10.1	2.4	10.4	2.4	12.1
47	TC10		3.1	10.1	3.1	10.1	2.7	10.3	2.9	11.6
48	TC11		3.7	10.4	3.7	10.4	3.2	10.3	3.3	12.7
49	TC12		3.9	10.2	3.9	10.2	3.3	12.0	3.4	12.0
50	TC13	5.8	1.8	6.9	1.8	7.0	1.6	7.1	1.6	6.7
51	TC14		2.1	7.1	2.0	6.8	1.8	6.6	1.7	6.6
52	TC15		2.6	7.0	2.6	7.0	2.3	7.0	2.2	6.7
53	TC16		3.0	7.1	3.0	7.0	2.6	7.0	2.5	7.5

Alternative 6

Test Run		Water Level (m ODN)	Gauge 1		Gauge 2		Gauge 3		Gauge 4	
			Hos (m)	Top (s)	Hos (m)	Top (s)	Hs (m)	Tp (s)	Hs (m)	Tp (s)
54	TC9	4.8	2.6	9.9	2.6	10.2	2.4	10.4	2.4	12.1
55	TC10		3.1	10.3	3.1	10.1	2.8	10.3	2.8	11.5
56	TC11		3.8	9.9	3.7	10.4	3.3	10.5	3.3	13.0
57	TC12		4.0	10.2	3.9	10.2	3.5	10.4	3.5	12.2
58	TC13	5.8	1.8	7.1	1.8	7.0	1.7	7.1	1.6	7.1
59	TC14		2.1	7.1	2.1	6.8	1.9	7.1	1.9	6.9
60	TC15		2.6	6.8	2.6	7.0	2.4	7.0	2.3	6.8
61	TC16		3.0	6.9	3.0	6.9	2.7	7.2	2.6	6.9

Alternative 7

Test Run		Water Level (m ODN)	Gauge 1		Gauge 2		Gauge 3		Gauge 4	
			Hos (m)	Top (s)	Hos (m)	Top (s)	Hs (m)	Tp (s)	Hs (m)	Tp (s)
62	TC9	4.8	2.7	10.3	2.6	10.2	2.5	10.4	2.6	12.1
63	TC10		3.2	10.3	3.1	10.1	2.9	10.3	3.1	11.5
64	TC11		3.9	9.9	3.8	10.4	3.4	10.3	3.6	11.5
65	TC12		4.0	10.2	3.9	10.2	3.5	10.2	3.7	12.3
66	TC13	5.8	1.9	7.1	1.8	7.0	1.7	7.1	1.7	7.1
67	TC14		2.1	7.1	2.1	6.8	2.0	7.1	2.0	6.8
68	TC15		2.7	7.1	2.6	7.0	2.5	7.1	2.5	6.8
69	TC16		3.1	7.1	3.0	7.0	2.8	7.1	2.8	7.1

Alternative 8

Test Run		Water Level (m ODN)	Gauge 1		Gauge 2		Gauge 3		Gauge 4	
			Hos (m)	Top (s)	Hos (m)	Top (s)	Hs (m)	Tp (s)	Hs (m)	Tp (s)
70	TC9	4.8	2.6	10.2	2.6	10.1	2.4	10.4	2.4	12.1
71	TC10		3.1	9.9	3.1	10.1	2.8	10.3	2.8	11.6
72	TC11		3.8	10.3	3.7	10.4	3.3	10.3	3.3	11.5
73	TC12		4.0	10.2	3.9	10.2	3.4	12.0	3.4	12.0
74	TC17	4.8	2.7	8.7	2.7	8.7	2.4	8.7	2.4	8.6
75	TC18	4.8	3.7	10.5	3.6	10.5	3.3	12.0	3.2	12.8
76	TC13	5.8	1.8	7.1	1.8	7.0	1.6	7.1	1.6	6.6
77	TC14		2.1	7.1	2.1	6.8	1.9	7.1	1.8	6.6
78	TC15		2.6	7.1	2.6	7.0	2.4	7.0	2.3	6.8
79	TC16		3.0	6.9	3.0	7.0	2.7	7.1	2.6	6.6
80	TC19	5.8	2.7	8.8	2.6	8.5	2.4	8.8	2.4	8.5
81	TC20	5.8	3.0	9.4	2.9	9.4	2.7	10.0	2.6	11.0
82	TC21	5.8	3.5	10.2	3.5	10.3	3.2	11.8	3.2	11.8
83	TC22	4.3	2.8	9.2	2.8	8.8	2.5	10.6	2.5	8.9
84	TC23	4.3	3.3	10.0	3.3	10.0	3.0	12.3	3.0	12.3
85	TC24	4.3	4.0	11.1	3.9	11.1	3.4	12.1	3.4	12.1
84 repet	TC23	4.3	3.3	10.0	3.3	10.1	3.0	10.5	3.0	12.2
86	TC25	5.3	1.8	7.0	1.7	7.1	1.6	7.1	1.5	7.0
87	TC26	5.3	3.0	9.3	3.0	9.3	2.7	10.1	2.7	11.3
88	TC27	5.3	3.5	10.1	3.5	10.1	3.2	10.2	3.1	12.1
87 repet	TC26	5.3	3.0	9.3	2.9	9.3	2.7	10.1	2.6	11.3

B.2. Stability and Mean Overtopping Discharges

Alternative 1

Test Run		Water Level (m ODN)	Gauge 1		3t - 6t Rock Displacement		Overtopping Class (according to LNEC's Criteria)	Prototype Discharge (l/s/m)
			Hos (m)	Top (s)	Nr.	%		
1	---	3.5	1.9	11.0	0	0.0%	1	0.000
2	TC1		2.3	10.6	0	0.0%	1	0.000
3	TC2		2.6	10.7	0	0.0%	1-2	0.000
4	TC3		3.0	10.5	1	0.2%	2	0.000
5	TC4		3.4	10.4	2	0.4%	3	0.035
5'	---	4.0	1.9	9.5	0	0.0%	1	0.000
6	TC5		2.3	9.3	0	0.0%	1-2	0.000
7	TC6		2.6	9.4	0	0.0%	2	0.000
8	TC7		3.0	9.6	0	0.0%	3	0.059
9	TC8		3.6	9.4	0	0.0%	3-4	0.106
10	TC9	4.8	2.5	9.9	0	0.0%	3	0.074
11	TC10		3.0	10.3	1	0.2%	4	0.850
12	TC11		3.6	10.3	3	0.6%	5	3.953
13	TC12		3.8	10.2	3	0.6%	5	7.499
14	TC13	5.8	1.8	7.1	0	0.0%	2	0.000
15	TC14		2.0	7.1	0	0.0%	3-4	0.106
16	TC15		2.6	7.1	0	0.0%	4	0.792
17	TC16		2.9	6.9	0	0.0%	5	3.720

Alternative 2

Test Run		Water Level (m ODN)	Gauge 1		3t - 6t Rock Displacement		Overtopping Class (according to LNEC's Criteria)	Prototype Discharge (l/s/m)
			Hos (m)	Top (s)	Nr.	%		
18	TC9	4.8	2.5	9.9	0	0.0%	3	0.055
19	TC10		3.0	10.1	0	0.0%	3-4	0.351
20	TC11		3.6	9.9	0	0.0%	5	2.069
21	TC12		3.8	10.2	2	0.4%	5	2.216
22	TC13	5.8	1.8	7.1	0	0.0%	2	0.000
23	TC14		2.0	7.1	0	0.0%	3	0.026
24	TC15		2.6	7.1	0	0.0%	3	0.079
25	TC16		3.0	6.9	0	0.0%	4	0.501

Alternative 3

Test Run		Water Level (m ODN)	Gauge 1		3t - 6t Rock Displacement		Overtopping Class (according to LNEC's Criteria)	Prototype Discharge (l/s/m)
			Hos (m)	Top (s)	Nr.	%		
26	TC1	3.5	2.3	11.0	3	0.7%	1	0.000
27	TC2		2.6	10.7	3	0.7%	3	0.017
28	TC3		3.0	10.5	3	0.7%	3	0.020
29	TC4		3.5	10.4	3	0.7%	3	0.020
30	TC5	4.0	2.4	9.3	0	0.0%	1-2	0.000
31	TC6		2.7	9.4	0	0.0%	3	0.007
32	TC7		3.0	9.6	0	0.0%	3	0.017
33	TC8		3.6	9.2	1	0.2%	3	0.056
34	TC9	4.8	2.6	9.8	0	0.0%	3-4	0.104
35	TC10		3.1	10.3	0	0.0%	4-5	1.134
36	TC11		3.7	10.3	2	0.5%	5	5.375
37	TC12		3.9	10.2	2	0.5%	5	7.499
38	TC13	5.8	1.8	6.9	0	0.0%	2	0.000
39	TC14		2.1	7.1	0	0.0%	3-4	0.106
40	TC15		2.6	7.0	0	0.0%	4	0.923
41	TC16		3.0	6.8	0	0.0%	5	4.143

Alternative 4

Test Run		Water Level (m ODN)	Gauge 1		3t - 6t Rock Displacement		Overtopping Class (according to LNEC's Criteria)	Prototype Discharge (l/s/m)
			Hos (m)	Top (s)	Nr.	%		
42	TC9	4.8	2.6	10.2	0	0.0%	3	0.092
43	TC10		3.1	10.1	0	0.0%	4	0.850
44	TC11		3.7	9.9	3	0.7%	5	3.417
45	TC12		3.9	10.2	3	0.7%	5	3.509

Alternative 5

Test Run		Water Level (m ODN)	Gauge 1		3t - 6t Rock Displacement		Overtopping Class (according to LNEC's Criteria)	Prototype Discharge (l/s/m)
			Hos (m)	Top (s)	Nr.	%		
46	TC9	4.8	2.6	10.3	1	0.2%	3	0.018
47	TC10		3.1	10.1	2	0.3%	3-4	0.129
48	TC11		3.7	10.4	3	0.5%	4	0.683
49	TC12		3.9	10.2	3	0.5%	4	0.591
50	TC13	5.8	1.8	6.9	0	0.0%	1-2	0.000
51	TC14		2.1	7.1	0	0.0%	2	0.000
52	TC15		2.6	7.0	0	0.0%	3	0.053
53	TC16		3.0	7.1	1	0.2%	3-4	0.132

Alternative 6

Test Run		Water Level (m ODN)	Gauge 1		3t - 6t Rock Displacement		Overtopping Class (according to LNEC's Criteria)	Prototype Discharge (l/s/m)
			Hos (m)	Top (s)	Nr.	%		
54	TC9	4.8	2.6	9.9	0	0.0%	3	0.074
55	TC10		3.1	10.3	0	0.0%	3-4	0.222
56	TC11		3.8	9.9	0	0.0%	4-5	1.016
57	TC12		4.0	10.2	0	0.0%	4-5	1.459
58	TC13	5.8	1.8	7.1	0	0.0%	2	0.000
59	TC14		2.1	7.1	0	0.0%	3	0.026
60	TC15		2.6	6.8	0	0.0%	3-4	0.106
61	TC16		3.0	6.9	0	0.0%	3-4	0.422

Alternative 7

Test Run		Water Level (m ODN)	Gauge 1		3t - 6t Rock Displacement		Overtopping Class (according to LNEC's Criteria)	Prototype Discharge (l/s/m)
			Hos (m)	Top (s)	Nr.	%		
62	TC9	4.8	2.7	10.3	0	0.0%	2	0.000
63	TC10		3.2	10.3	1	0.2%	3	0.037
64	TC11		3.9	9.9	3	0.5%	3	0.092
65	TC12		4.0	10.2	5	0.9%	3-4	0.172
66	TC13	5.8	1.9	7.1	0	0.0%	1-2	0.000
67	TC14		2.1	7.1	0	0.0%	2	0.000
68	TC15		2.7	7.1	0	0.0%	3	0.026
69	TC16		3.1	7.1	0	0.0%	3-4	0.106

Alternative 8

Test Run		Water Level (m ODN)	Gauge 1		3t - 6t Rock Displacement		Overtopping Class (according to LNEC's Criteria)	Prototype Discharge (l/s/m)
			Hos (m)	Top (s)	Nr.	%		
70	TC9	4.8	2.6	10.2	0	0.0%	3	0.018
71	TC10		3.1	9.9	0	0.0%	3	0.092
72	TC11		3.8	10.3	0	0.0%	3-4	0.296
73	TC12		4.0	10.2	1	0.2%	3-4	0.443
74	TC17	4.8	2.7	8.7	0	0.0%	3	0.021
75	TC18	4.8	3.7	10.5	1	0.2%	3-4	0.264
76	TC13	5.8	1.8	7.1	0	0.0%	1-2	0.000
77	TC14		2.1	7.1	0	0.0%	2	0.000
78	TC15		2.6	7.1	0	0.0%	3	0.053
79	TC16		3.0	6.9	1	0.2%	3-4	0.145
80	TC19	5.8	2.7	8.8	0	0.0%	3	0.051
81	TC20	5.8	3.0	9.4	0	0.0%	4-5	1.264
82	TC21	5.8	3.5	10.2	0	0.0%	5	5.338
83	TC22	4.3	2.8	9.2	0	0.0%	3	0.020
84	TC23	4.3	3.3	10.0	1	0.2%	3	0.070
85	TC24	4.3	4.0	11.1	0	0.0%	3-4	0.172
84 repet	TC23	4.3	3.3	10.0	---	---	3	0.065
86	TC25	5.3	1.8	7.0	0	0.0%	2	0.000
87	TC26	5.3	3.0	9.3	0	0.0%	3-4	0.202
88	TC27	5.3	3.5	10.1	0	0.0%	4	0.591
87 repet	TC26	5.3	3.0	9.3	---	---	3-4	0.190

APPENDIX C

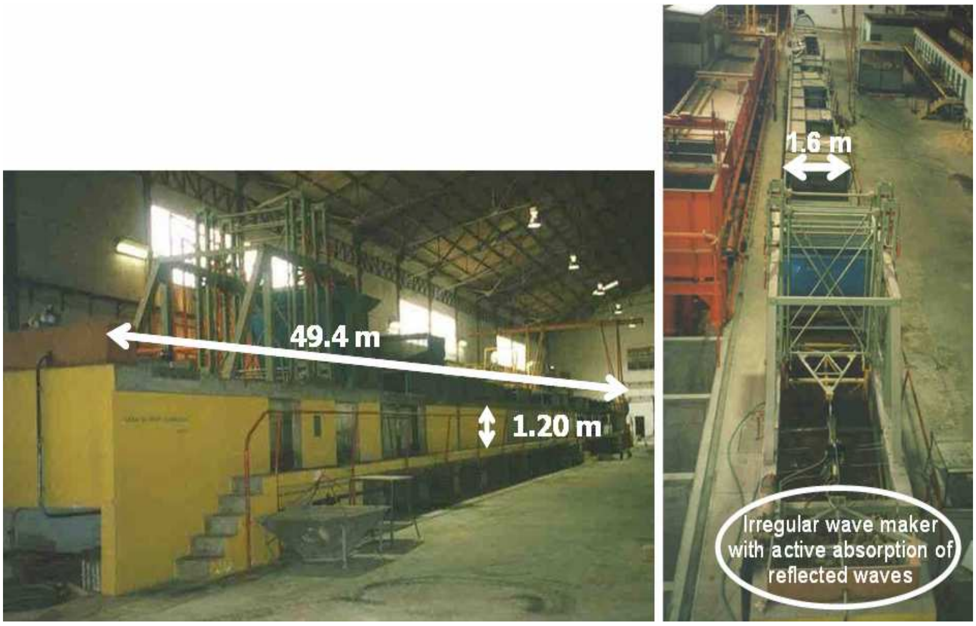
LNEC's Overtopping Criteria

LNEC's Overtopping Criteria for Tests with Irregular Waves

Class	Classification	Description
0	Nonexistent	No overtopping
1	Slight	Only the highest waves cause drops of water to overtop the structure
2	Small	Drops of water frequently overtop the structure
3	Moderate	The highest waves cause sheets of water to overtop the structure
4	Important	Sheets of water frequently overtop the structure; the highest waves may cause masses of water to overtop the structure
5	Serious	Masses of water frequently overtop the structure

APPENDIX D

Photographs of Experimental Facilities and Equipment, Modelled Structures and Rock Displacements



Irregular wave flume of LNEC

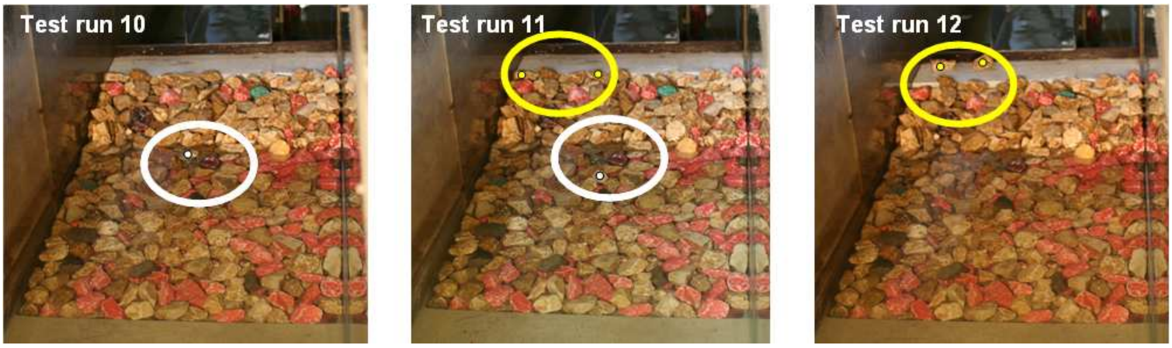
Experimental equipment for free surface elevation: wave gauges

Experimental equipment for overtopping: chute, overtopping tank, water level gauge and pump

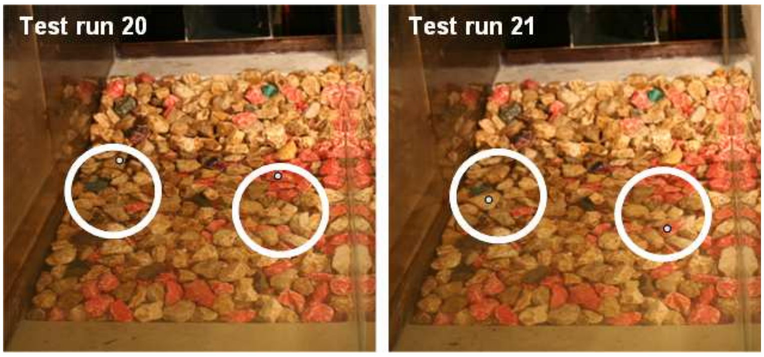
Constructed alternatives

Constructed alternatives (continued)

Alternative 1: identification of rock displacements for test runs 4 and 5 (initial and final rock positions)



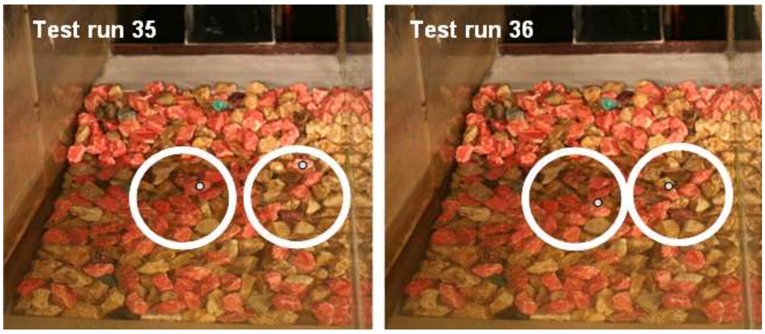
Alternative 1: identification of rock displacements for test runs 11 and 12 (initial and final rock positions)



Alternative 2: identification of rock displacements for test run 21 (initial and final rock positions)

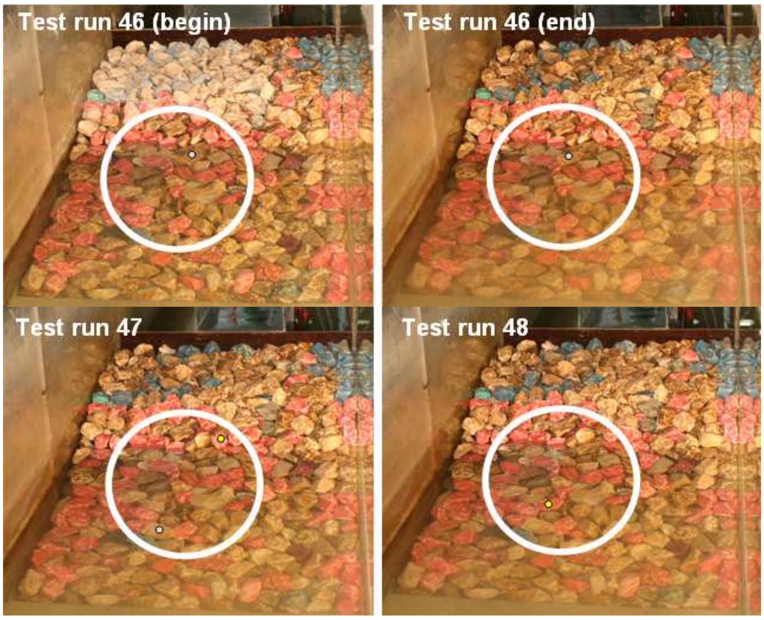
Alternative 3: identification of rock displacements for test run 26 (initial and final rock positions)

Alternative 3: identification of rock displacements for test run 33 (initial and final rock positions)

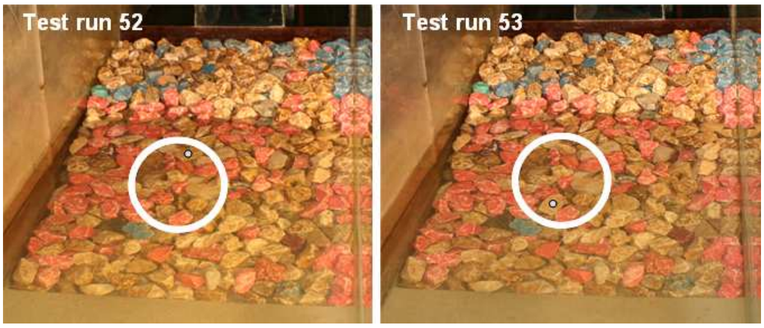


Alternative 3: identification of rock displacements for test run 36 (initial and final rock positions)

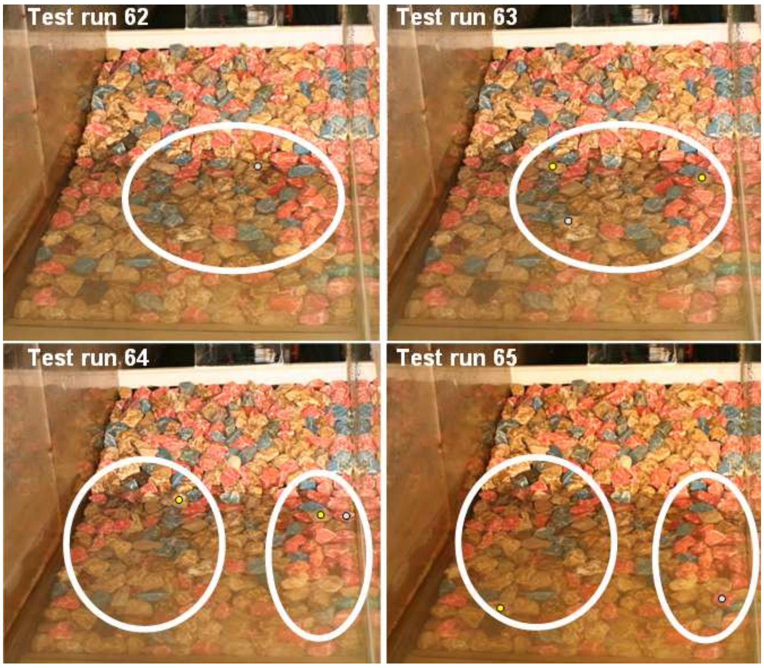
Alternative 4: identification of rock displacements for test run 44 (initial and final rock positions)



Alternative 5: identification of rock displacements for test runs 46, 47 and 48 (initial and final rock positions)

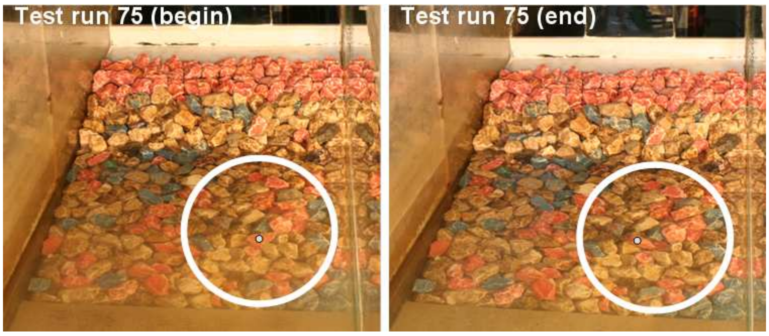


Alternative 5: identification of rock displacements for test run 53 (initial and final rock positions)



Alternative 7: identification of rock displacements for test runs 63, 64 and 65 (initial and final rock positions)

Alternative 8: identification of rock displacements for test run 73 (initial and final rock positions)



Alternative 8: identification of rock displacements for test run 75 (initial and rock positions)

Alternative 8: identification of rock displacements for test run 79 (initial and final rock positions)

Alternative 8: identification of rock displacements for test run 84 (initial and final rock positions)

APPENDIX E

Deriving the Design Wave and Water Level Conditions

Memo

ROYAL HASKONING

HASKONING UK LTD.
COASTAL & RIVERS

To	:	Alan Williams
From	:	Keming Hu
Date	:	23 September 2009
Copy	:	Paul Winfield, Steve Graham
Our reference	:	9T3344/KH
Subject	**:**	Test water level and wave conditions at 0m CD for linear defence

Joint Probability at 0m CD

This section is to address the problem that the proposed linear defence is some distance away from the inshore wave data Points C, D and E (see Figure 1) where the joint probability of water level and wave conditions were available.

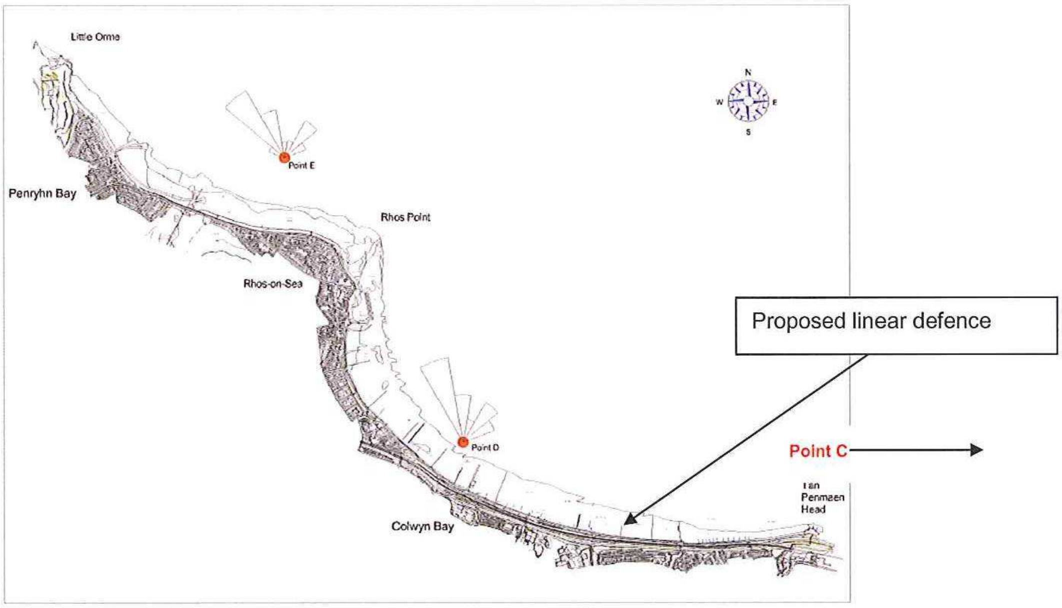


Figure 1: Locations of joint probability data points

Table 1 presents the extreme wave conditions at these 3 points based on a marginal probability analysis. Table 1 shows that the extreme wave condition along Colwyn Bay is less severe than the neighbouring coasts in the east and west, which may be associated with the sheltering effect of Rhos Point

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Table 1: Extreme wave at Point D and E (without wave breaking)

	Point C	Point D	Point E
1:1 year	3.51	2.04	3.25
1:10 year	4.13	2.27	3.84
1:50 year	4.57	2.43	4.24
1:100 year	4.75	2.50	4.40
1: 200 year	4.94	2.57	4.57
1: 1000 year	5.37	2.73	4.97

Note: provided by Alan Williams

From a separate project, the client (Conwy Council) obtained the modelled time series wave data at 5 inshore locations (see Figure 2). The time series data starts from 01/01/1987 and ends at 31/12/2005 with an interval of 60 minutes. In total, the modelled wave data covers a period 19 years. We understand that the modelled inshore wave data was transformed by HR Wallingford from forecasted offshore wave data provided by the UK Met Office. Point 4 is the closet point to the proposed site for the linear defence.

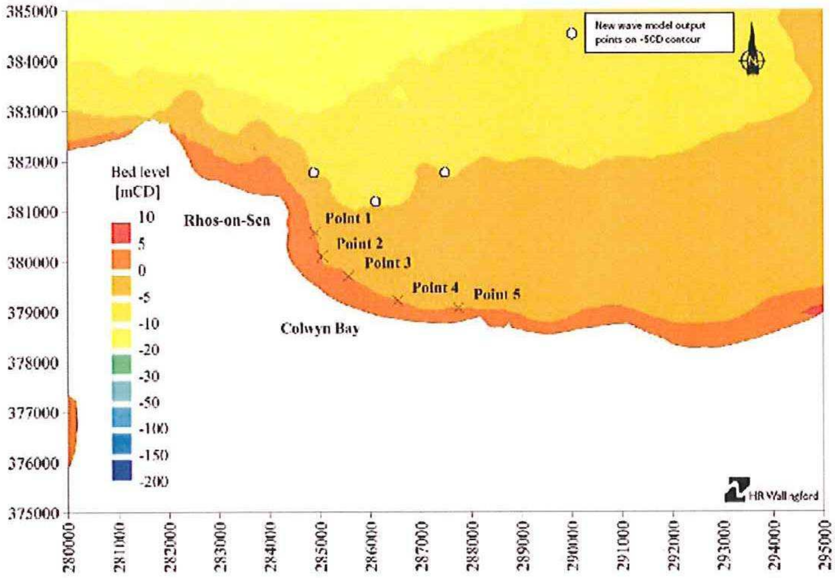

Figure 2: Location of time series inshore wave data points

We have carried out marginal probability analysis for inshore waves at 4 of 5 points based on the modelled time series data using Gumbel Distribution method. The results of our extreme values analysis are present in Table 2.

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Table 2: Results of marginal probability analysis for extreme wave conditions

	Point 1	Point 3	Point 4	Point 5
1:1 year	3.15	3.18	2.56	2.70
1:10 year	3.97	3.98	3.15	3.35
1:50 year	4.54	4.55	3.56	3.80
1:100 year	4.79	4.79	3.73	4.00
1: 200 year	5.04	5.03	3.91	4.19
1: 1000 year	5.61	5.59	4.32	4.64

The results show that extreme wave conditions are similar between Point 1 and Point 3 but extreme wave conditions at Point 4 and Point 5 were less severe than Points 1 and 3.

We notice that the extreme wave conditions we derived at Points 1 and 3 are similar to those at Point C. Therefore, the joint probability at Point 4, the closet point to the linear defence, may be obtained by interpolation between Points C and D. Table 3 presents results of the interpolation by simple average and weighted average. The weighted average was to match 1:100 year waves at Point 4.

Table 3: Results of interpolation between Points C and D

	Point 4	Average between Points C and D	Weighted average between Points C and D (54.5%:45.5%)
1:1 year	2.56	2.78	2.84
1:10 year	3.15	3.20	3.28
1:50 year	3.56	3.50	3.60
1:100 year	**3.73**	**3.63**	**3.73**
1: 200 year	3.91	3.76	3.86
1: 1000 year	4.32	4.05	4.17

It should be pointed out that the marginal probability analysis at inshore locations may under-estimate wave conditions for lower return periods while overestimate extreme wave conditions for higher return periods, because shallow water process is not considered. For this reason, we think it is safe to use the weighted average wave conditions for the proposed linear defence.

Design Return Period and Sea Level Rise

Based on our telephone conference on 11th September 2009, we have agreed on the use of 1:50 year joint probability of water levels and waves for the rock size/slope calculation and initial laboratory tests of the proposed linear defence. For sea level rise, we also agreed to consider 25 year sea level in initial tests but also consider 75 year sea level rise at later stage to understand the extent of upgrading potentially required on defence profile (e.g. higher berm level or crest level of wave return wall).

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Conclusions

Based on the above analysis and discussions with Alan Williams by telephone, we recommend the following water levels and wave conditions for rock size/slope calculation, numerical overtopping modelling and initial laboratory tests for the proposed linear sea defence:

- Use 1:50 year joint probability of water levels and waves
- Consider 25 year sea level rise
- Use weighted average joint probability between Point C and D to match 1:100 year wave conditions at Point 4 based on 19 year time series modelled wave data.

We understand that 1:200 year joint probability of water levels and waves and 75 year sea level rise may be tested at the later stage to understand the extent of upgrading potentially required on defence profile.

Sea Level Rise for Physical Modelling

The calculation of sea level rise was shown below extracted from spreadsheet "JP conditions (+25 years - No Wave increase)r1.xls" provided by Alan Williams.

REGIONAL NET SEA LEVEL RISE ALLOWANCES					
	Assumed vertical land	Net Sea Level Rise (mm/year) 1990-2025	2025-2055	2055-2085	2085-2115
Wales	-0.5	3.5	8	11.5	14.5
NW England	0.8	2.5	7	10	13

Base Year		2004				
Current Year		2009				
Area	Years in future	21	9	0	0	**Total**
Wales	**25**	0.0735	0.072	0	0	**0.1455**

Weighted Average Joint Probability of Water Levels and Waves for Physical Modelling

The weighted average joint probability is calculated in a spreadsheet "JP conditions (+25 years - No Wave increase)r1_WeightedAverage.xls" and the results are shown below.

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Weighted average conditions (between C and D)

Return periods	WL (mOD	Sop	Tm (s)	Hs (m)	Beta	Nw
1	2.84	0.03	6.35	2.18	0	5000
	3.29	0.03	6.01	1.96	0	5000
	3.75	0.03	5.58	1.71	0	5000
	4.21	0.03	4.95	1.37	0	5000
	4.67	0.03	2.89	0.54	0	5000
10	2.84	0.03	7.70	3.15	0	5000
	3.29	0.03	7.70	3.15	0	5000
	3.75	0.03	7.61	3.08	0	5000
	4.21	0.03	7.38	2.91	0	5000
	4.67	0.03	7.02	2.65	0	5000
	5.13	0.03	5.63	1.81	0	5000
50	3.29	0.03	8.21	3.56	0	5000
	3.75	0.03	8.17	3.54	0	5000
	4.21	0.03	8.03	3.42	0	5000
	4.67	0.03	7.82	3.25	0	5000
	5.13	0.03	7.35	2.88	0	5000
	5.58	0.03	5.58	1.71	0	5000
100	3.29	0.03	8.42	3.74	0	5000
	3.75	0.03	8.34	3.68	0	5000
	4.21	0.03	8.25	3.60	0	5000
	4.67	0.03	8.03	3.42	0	5000
	5.13	0.03	7.70	3.15	0	5000
	5.58	0.03	7.04	2.65	0	5000
200	3.29	0.03	8.68	3.97	0	5000
	3.75	0.03	8.67	3.96	0	5000
	4.21	0.03	8.60	3.90	0	5000
	4.67	0.03	8.41	3.74	0	5000
	5.13	0.03	8.04	3.42	0	5000
	5.58	0.03	7.57	3.05	0	5000
500	3.75	0.03	8.95	4.22	0	5000
	4.21	0.03	8.92	4.19	0	5000
	4.67	0.03	8.77	4.05	0	5000
	5.13	0.03	8.51	3.82	0	5000
	5.58	0.03	8.17	3.54	0	5000
	6.04	0.03	7.24	2.79	0	5000
1000	4.21	0.03	9.20	4.45	0	5000
	4.67	0.03	9.09	4.35	0	5000
	5.13	0.03	8.83	4.11	0	5000
	5.58	0.03	8.45	3.76	0	5000
	6.04	0.03	7.79	3.22	0	5000
1	2.98	0.03	6.35	2.18	0	5000
	3.44	0.03	6.01	1.96	0	5000
	3.90	0.03	5.58	1.71	0	5000
	4.36	0.03	4.95	1.37	0	5000
	4.81	0.03	2.89	0.54	0	5000
10	2.98	0.03	7.70	3.15	0	5000
	3.44	0.03	7.70	3.15	0	5000
	3.90	0.03	7.61	3.08	0	5000
	4.36	0.03	7.38	2.91	0	5000
	4.81	0.03	7.02	2.65	0	5000
	5.27	0.03	5.63	1.81	0	5000
50	3.44	0.03	8.21	3.56	0	5000
	3.90	0.03	8.17	3.54	0	5000
	4.36	0.03	8.03	3.42	0	5000
	4.81	0.03	7.82	3.25	0	5000
	5.27	0.03	7.35	2.88	0	5000
	5.73	0.03	5.58	1.71	0	5000
100	3.44	0.03	8.42	3.74	0	5000
	3.90	0.03	8.34	3.68	0	5000
	4.36	0.03	8.25	3.60	0	5000
	4.81	0.03	8.03	3.42	0	5000
	5.27	0.03	7.70	3.15	0	5000
	5.73	0.03	7.04	2.65	0	5000
200	3.44	0.03	8.68	3.97	0	5000
	3.90	0.03	8.67	3.96	0	5000
	4.36	0.03	8.60	3.90	0	5000
	4.81	0.03	8.41	3.74	0	5000
	5.27	0.03	8.04	3.42	0	5000
	5.73	0.03	7.57	3.05	0	5000
500	3.90	0.03	8.95	4.22	0	5000
	4.36	0.03	8.92	4.19	0	5000
	4.81	0.03	8.77	4.05	0	5000
	5.27	0.03	8.51	3.82	0	5000
	5.73	0.03	8.17	3.54	0	5000
	6.19	0.03	7.24	2.79	0	5000
1000	4.36	0.03	9.20	4.45	0	5000
	4.81	0.03	9.09	4.35	0	5000
	5.27	0.03	8.83	4.11	0	5000
	5.73	0.03	8.45	3.76	0	5000
	6.19	0.03	7.79	3.22	0	5000

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