Impact of Bi-modal Seas on Beaches and Control Structures

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Summary

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A sea state is made up of either wind-sea, swell-sea or a combination of the two. Wind-seas are generated by local winds; their impacts at the coast in terms of overtopping, beach movement, armour damage, etc, are relatively well understood for many simple configurations. Swell-seas result from the transfer of energy to lower frequencies as wind-seas decay; their magnitudes and impacts at the coast are more difficult to predict than wind-seas.

Often during the transition from wind-sea to swell, the wind speed will rise and begin generating new wind-sea. This situation is characterised by a bi-modal wave spectrum, in which the lower frequency peak is the swell-sea component and the higher frequency peak is the wind-sea component. These types of conditions, and the responses occurring at the coast, are probably the hardest of all to predict accurately. Very few physical model tests of sea defences, and no established empirical methods, deal specifically with the impacts of bi-modal sea conditions.

The Ministry of Agriculture, Fisheries and Food has funded a series of studies at HR Wallingford on swell and bi-modal seas and their impacts. This report describes physical model tests on beaches and structures under swell and bi-modal sea conditions. The first project involved the impact on shingle beaches in terms of crest elevation and crest roll-back. The second project involved measurement of overtopping on different types of seawall. In both cases, results were analysed in detail and guidelines were developed for future assessment of the effects of bi-modal seas. In an opportunistic third project, additional measurements were taken of run-up, run-down, armour movement and reflection; these are presented in the present report but are not analysed in detail.

The report concludes by listing physical situations and locations in which swell and bi-modal seas should be considered as potential design conditions. It also summarises the knowledge gathered from the test programme and the areas where further research is needed. A companion technical report gives full details of the structures, sea states and measurements taken.

For further information on this report, please contact Dr Peter Hawkes or Mr Tom Coates of the Coastal Group at HR Wallingford.

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1 Introduction

1.1 Background

It is well known that wave period, as well as wave height, can be important in determining the action of waves on beaches and coastal structures. In the common situation of waves being depth-limited at the toe of a sea defence after breaking over a gently sloping foreshore, wave period and water level may both be more important than wave height. Swell waves, produced as wind waves decay after a storm, have longer periods than locally generated storm waves (wind-sea), although usually lower wave heights. Because of the effect of wave period in determining coastal response, it is possible that extreme swell wave conditions (or a mixture of wind-sea and swell) represent a worst case sea state for some aspects of design. However, little is known about the effect of bi-modal sea conditions on sea defences or beaches, and swell is rarely considered explicitly in the design or assessment of shoreline management operations.

This lack of knowledge provoked a series of research projects on swell and bi-modal seas funded by the Ministry of Agriculture, Fisheries and Food (MAFF). One product of the earlier studies was a swell wave atlas for England and Wales (Hawkes *et al*, Reference 1). As well as looking at commonly occurring and extreme swells all around the coast of England and Wales, the atlas divided the coast into 25 sections and provided more detailed offshore swell wave climate data for each one. The information included a distribution of swell wave height, period and direction for each area, and extreme swells and bi-modal seas for a range of return periods. The bi-modal sea conditions are specified in terms of a significant wave height and a mean wave period for both the wind-sea component and the swell-sea component (i.e. four parameters in all); if necessary, separate mean wave directions can also be specified for each component, giving up to six parameters in all. A method for generating the corresponding bi-modal spectrum is also given in the swell atlas.

It was intended that the swell wave information would provide input to nearshore transformation models, physical models, and design calculations, in the same way that deep water wind-sea data would be used. A potential difficulty is that applications and design methods developed using wind-sea conditions may not work well with swell wave conditions.

A more obvious difficulty with the bi-modal sea data arises from the separate parameters needed to specify the separate wind-sea and swell components (Figure 1.1). These could be used within a random wave numerical or physical model in which a spectrum can be fully specified by the user. However, most design methods use a single wave height and period (or equivalent) and it is far from obvious how to represent bi-modal sea data in this form before it can be used in such methods. This problem has been addressed intuitively in different ways in the past, but without relevant physical model or field data for validation, it is difficult to say which (if any) of those approaches were correct. The simplest approach, for example, is just to calculate the significant wave height and mean wave period (and mean wave direction if required) in the usual way by spectral analysis, without regard for the possibility that the sea may be bi-modal. This has on occasion been demonstrated to be incorrect, because the mean wave period (or direction), at which there may be little wave energy, produces a different response to the combined effect due to the two very different modes of the spectrum. An example of beach response to bi-modal seas was observed during a MAFF-funded field study in Christchurch Bay; these observations are discussed in Chapter 2.

1.2 Project description

HR Wallingford was commissioned by MAFF to conduct a series of investigations relating to the impacts of bi-modal seas on coastal defences and beaches. The objectives were:

- to investigate the influence of bi-modal seas on shingle beach profiles, produce guidance for beach management and, if possible, to derive generic prediction methods compatible with existing beach profile response models
- to investigate the influence of bi-modal seas on wave run-up, wave overtopping and the structural stability of coastal structures, and to develop practical guidelines for modifying existing design approaches.

The work was undertaken in a wave flume at HR Wallingford as part of a rolling programme of MAFF research. Details of the methods and results for all of the flume studies are presented in the companion Technical Report (Reference 2).

1.3 Report outline

This report presents analysis and discussion of the flume investigations described in the Technical Report (Reference 2), and presents guidance on the application of the results. Results for shingle beaches and structure overtopping are presented in Chapters 2 and 3. Results for other parameters (run-up, structural stability) are presented in Chapter 4. Chapter 5 sets out the practical applications, while Chapters 6 and 7 present the conclusions and recommendations for further research.

All dimensions used in this report are prototype unless otherwise specified.

2 Effects of bi-modal seas on shingle beach response

2.1 Field observations

The response of shingle beaches to long period wave conditions has been noted and observed many times, but quantitative data regarding sea conditions and beach profiles are not generally available. An exception to this is the data set collected under a MAFF field research commission undertaken in Christchurch Bay, as reported in Coates & Bona (1997 - Reference 3).

Profiles were collected from sites at Highcliffe and Hordle beaches in Christchurch Bay between January 1993 and April 1995, while wave conditions and water levels were recorded continuously at a nearshore location. A number of the profile surveys were carried out following identifiable storm events, two of which can be compared to illustrate the potential impact of bi-modal seas.

Christchurch Bay is protected from severe wave attack by the Dolphin Bank and the Isle of Wight to the east, and by Christchurch Ledge and Hengistbury Head to the southwest. Much of the coastline of Christchurch Bay, and the neighbouring Poole Bay, is undergoing long term erosion as equilibrium bay plan shapes between the major headlands are still developing. Wave induced nett longshore transport is from west to east and increases in rate in the same direction. The area has a low tidal range relative to much of the UK, with a spring tide range of about 1.4m, and experiences an extended high water stand or double high water.

The lengths of shoreline investigated were:

- the eastern end of the recharged and groyned beach at Highcliffe
- the natural, open beach at Hordle.

The beach at Highcliffe is discussed in some detail in Reference 3. Briefly, the beach comprises lower sand, upper shingle in front of formerly unstable cliffs of flint, sand and clay strata. It has undergone recharges in 1984 and 1992 and is controlled by large rock groynes, built over earlier timber groynes, which prevent any significant longshore transport. The 1992 recharge was designed by HR Wallingford, making use of a parametric model developed from a wave flume research programme (Powell, Reference 4) which is discussed later in this report. The eastern section of the beach built up into a relatively stable beach following the recharge of 1984, and received very little new material in 1992. The beach was considered to be nearing maturity in terms of its profile, shape and dimensions. The shingle has a D_{50} of about 25mm and a grading range from 10mm to 100mm. The interface between the lower sand beach and the shingle comprises a zone of constantly changing layers of sand and shingle that can be eroded down to form a deep scour channel or built up to form a sandy toe.

The beach at Hordle is natural. It comprises a wide shingle bank with a steep face running down to about –2m OD where the gently sloping sand nearshore zone begins. The shingle grading ranges from 10mm to 50mm and has a D_{50} of about 16mm. The section of beach investigated has no groynes and can be considered to have reached a state of dynamic equilibrium with the wave and tidal climate over many years.

During December 1994 two major storms affected Christchurch Bay. The first ran from the 3rd to the 8th, while the second ran from the 25th to the 30th. Beach profiles were surveyed on the 19th and the 31st. Figures 2.1 and 2.2 present the profiles.

The profiles for Highcliffe show that the beach crest increased in elevation from 4.0m OD to 4.3m OD between the two surveys, while the Hordle profile crests remained unchanged. As the Highcliffe crest had remained relatively static at the design level of 3.9m OD (± 0.1m) over the 2½ years since the 1992 recharge, then this change in elevation was considered noteworthy.

Initial assessment of the wave and water level records indicated that the peak energy between the two surveys occurred at high water on the morning of 30 December, when water levels reached 1.35m OD (07:22) and wave heights (H_s) were between 2.88m (06:00) and 3.14m (09:00). These levels were high for Christchurch Bay, but a review of conditions during the previous storm event earlier in December indicated that very similar conditions had already been experienced: on the morning of 8 December the water level reached 1.31m OD (04:22) and wave heights were between 2.93m (03:00) and 3.17m (06:00).

As the basic sea state conditions were so similar then it was considered that other factors must be involved. Previous observations at other sites, such as Chesil Beach, had suggested that swell waves might be important in beach profile response, so a more detailed analysis of the wave record was undertaken. The wave records were analysed spectrally for each high water period throughout the two storm events. The results for the two highest water levels are presented in Figures 2.3 and 2.4.

During the early December event the wave energy spectra were distinctly peaked within the wind-sea frequencies (0.11-0.14Hz or 7-9s). The later event showed significantly different spectra, with much higher proportions of total energy in the swell frequency band. During the high water of 29 December the spectra separated into two distinct peaks, with the swell peak at 0.05Hz (20s) and the wind-sea peak between 0.11-0.14Hz (7-9s).

A simplified method of separating swell from wind-sea was applied to those spectra based on the Pierson-Moskowitz (Reference 5) approach:

$$f\ (\text{wind-sea}) = 0.13g/U$$

where f (wind-sea) is the lowest frequency (Hz) for waves generated by local winds
g is acceleration due to gravity in m/s^2
U is the local wind speed in m/s.

This approach was derived for fully developed seas, but provides a convenient separation for this application. As local winds were around 17m/s then the separation frequency is taken as 0.075Hz. Calculations for the two storm events are presented in Figures 2.3 and 2.4. Swell energy for the first storm was around 10% of total energy, while the second storm had much more obvious swell accounting for about 20% of total wave energy.

The apparent response of the beach to wave conditions with a large low frequency component or a bi-modal distribution suggested that the existing method of predicting beach response could be inadequate. This conclusion led to the present flume study on shingle beaches.

2.2 Flume study

2.2.1 Objectives

A wave flume study was proposed to investigate the field observations discussed above, and to improve existing beach management tools for use by coastal engineers.

Existing methods for predicting shingle beach profiles are based on an extensive wave flume study reported by Powell (1990, Reference 4). Powell recorded the responses of a physical model beach to a range of wave conditions based on JONSWAP spectra. The parametric model developed from the results uses a single wave height (H_s) and a single wave period (T_m) as the input wave parameters. The model has been used in the design of many beach recharge and beach management schemes around the UK. Although the model is widely acknowledged as a successful design tool, the field observations discussed above indicate that the results may under-predict beach crest development under sea conditions with distinct bi-modal spectra.

The intention of the present study was to build on Powell's work by extending some of his original test conditions to consider bi-modal seas. Several sequences of tests were run in which the total wave energy level was kept constant, but the energy levels within the swell frequencies were varied from near zero (as in the original work) through several levels to near 100% swell energy. In addition, a sequence of tests was run using a set of wave conditions with an assumed constant return period, based on the 2:1 year return period conditions for Great Yarmouth. This sequence included total wind-sea, total swell and a range of intermediate conditions; total energy levels diminished from wind-sea to swell.

2.2.2 Methods

The study methods are described in the companion Technical Report (Reference 2). The following is a brief summary.

Tests were undertaken in the Wave Absorbing Flume at HR Wallingford at a notional scale of 1:20. The required wave conditions were calibrated at the outset of the study, and were monitored throughout each test to ensure correct generation. The shingle beach was modelled using crushed and graded anthracite coal, scaled according to well established procedures that correctly simulate threshold of motion, onshore-offshore transport and permeability. Beach response was recorded on video, photographs and by an automated beach profiler. Test sequences included:

- initial verification against the work of Powell
- six sequences of equal energy conditions, with three energy levels (H_s = 2.12m, 2.83m and 3.53m) and three peak swell frequencies (T_p = 11s, 14s and 19s)
- one sequence of constant return period conditions.

Table 2.1 summarises the test conditions, whilst Section 3.1 describes the background to their selection.

2.2.3 Results

The full set of beach response profiles are presented in the Technical Report. The results discussed in this report are changes in crest elevation and crest position, presented in Figures 2.5–2.10. Changes of crest elevation and position for each bi-modal or swell wave condition are non-dimensionalised as percentage variations relative to the crest response for the appropriate wind-sea only condition. Elevations are relative to the still water level and crest positions are relative to the position of still water on the wind-sea only profiles.

2.3 Discussion

2.3.1 Comparison with Powell (1990)

A major objective of the beach study was to extend the work of Powell (Reference 4) by investigating bi-modal wave conditions. The first step to achieving this objective was to ensure compatibility of flume model results by repeating a number of tests from the original research programme. The model facility, mobile bed and wave/water level conditions were all set up to replicate Powell's work. The scale of the new model was slightly smaller, at 1:20 relative to the original 1:17. In addition, the new work was run with a piston type paddle rather than the previous wedge paddle. Theoretically these two differences should not have affected the results as the scale change was carried through all aspects of the model and the performance capabilities of the paddles are equal.

Comparison of the measured profiles under three different conditions showed a substantial difference in post-test beach profiles. Crest elevations measured in the new series were between 0.3m and 0.7m lower than those recorded by Powell. Despite numerous repetitions and sensitivity tests the differences remained too large to conclude that the two sets of results were compatible.

Differences may be attributed to differences in wave generation or in sediment distribution. The original spectral analyses of individual wave conditions are no longer available, and nor are the details of the original sediment distributions. Although there are no apparent reasons for differences, these two factors are the only variables that could not be thoroughly checked.

Rather than abandon the project, it was concluded that data analysis should concentrate on the relative differences between beach profiles under different wave conditions, rather than on the absolute values. Powell's work has been validated against site information on a number of occasions and is considered to be a reliable predictive tool. Therefore the relative influences of different wave spectra on beach response derived from this study can be applied to Powell's model despite the lack of compatibility.

2.3.2 Equal energy tests

Six test sequences (Tests 6-32) were run to investigate the influence of altering the distribution of wave energy from wind-sea frequencies, through bi-modal spectra to swell frequencies. The tests included a representative range of energy levels and frequencies; consideration was not given to the probabilities of occurrence at this stage. Test sequences comprised an initial test of wind-sea only followed by three combinations of wind-sea and swell, ending with a test of swell only. The wind-sea only tests are the base against which each of the others is compared.

The key parameters selected for analysis were the profile crest elevation and position. These are two of the critical parameters in beach recharge design as they are important to both volume and in assessing risk to the backshore. Figures 2.5-2.10 present the results.

Figures 2.5, 2.7 and 2.9 show the importance of long period energy on crest elevation. In all sequences there is a trend of increasing elevation with percentage swell and with increasing swell period. The rate of elevation increase reduces after 50% swell energy relative to total energy. The influence of swell period is greatest between 11 seconds and 14 seconds; the rate of increase reduces between 14 seconds and 19 seconds.

A similar pattern is seen for the change in crest position in Figures 2.6, 2.8 and 2.10. The influence of swell is greatest up to about 20%, with change beyond that level being variable and, in some cases, showing a reversal in trend. The 19 second period swell appears to have a much greater influence than the 11 second and 14 second conditions.

The conclusions from these sequences are simple and agree with the field measurements from the Highcliffe study (Reference 3). Sea conditions with between 20% and 50% swell energy at periods of 14 seconds or greater may be important design conditions for beach management schemes, depending on the probability of occurrence.

2.3.3 Equal return period tests

The importance of swell at a specific location is dependent on the probability of occurrence of swell in conjunction with large wind-sea and high water levels, plus the duration and sequencing of the storm events. An example of the potential importance of a probabilistic approach was considered in a single test sequence (Tests 1-5). Wave conditions of equal return period were derived for a specific site (Great Yarmouth) so the results are not of generic value and are not presented in this report (a full set of results are presented in Technical Report TR 24, Reference 2). The tests are not conclusive, except for the specific site, but illustrate the importance of considering bi-modal conditions for all sites.

3 Effect of bi-modal seas on overtopping of sloping seawalls

3.1 Introduction

3.1.1 Purposes of the overtopping tests and analysis

The main purposes of the swell and bi-modal sea overtopping tests are:

1. To observe in general terms how overtopping changes as sea conditions change from wind-sea, through bi-modal sea, to swell. This is to help determine whether or not bi-modal and swell seas are potential worst cases for UK sea defences, and hence whether or not they should be considered in design.

2. To demonstrate a desk study application of the bi-modal sea data provided by the swell atlas (Reference 1). The hope was to produce a workable modification of a uni-modal design formula for predicting one aspect of coastal response under bi-modal sea conditions.

The tests focused on mean overtopping rate on plain seawalls, since this is a commonly used design parameter with well established methods for measurement and prediction, which depends strongly on wave period. Associated measurements of run-up, run-down, reflection coefficient and armour damage, are analysed in less detail in Chapter 4.

The results presented in Chapters 3 and 4 should be applied with caution. They are not intended to provide definitive guidance on structural responses under swell and bi-modal seas, but they do have relevance to design criteria covered by the scope of the tests, namely:

- overtopping, run-up, run-down, reflection coefficient and armour movement;
- on plain slopes in the range 1:2 to 1:4;
- for breaking and non-breaking waves arriving on a foreshore slope of about 1:50.

3.1.2 The flume tests and measurements

The overtopping tests were designed to include cases easily represented by the SWALLOW (Sea WALL Overtopping under Waves) empirical prediction model, namely storm waves on a plain steep slope fronted by a plain shallow foreshore slope. Additional tests would cover related bi-modal and swell-sea conditions. The SWALLOW model applies a simplification for wave shoaling and breaking on a foreshore slope before applying Owen's (1980) formula for mean overtopping rate (Reference 6).

<u>The six series of wave conditions</u>
The individual tests within each of six series followed a similar format to assist in interpreting the results. Each series included:

- a pure wind-sea with given significant wave height (H_{s1}) and peak period (T_{p1});

- a pure swell-sea with a given significant wave height (H_{s2}) and peak period (T_{p2});

- three (six for Series 2) intermediate or bi-modal sea conditions in which the peak periods of the two separate modes were T_{p1} and T_{p2}.

Typically T_{p2} (for swell) was two to three times higher than T_{p1} (for wind-sea) to give well separated modes and to cover a wide range of conditions: the values of the H_s and T_p parameters for each of the six series are listed in Table 3.1.

Table 3.1 Summary of wave conditions used in the flume tests

Series No	Relationship between tests	No of tests	Wind-sea		Swell-sea	
			H_{s1}	T_{p1}	H_{s2}	T_{p2}
1	Equal energy	5	3.53	7	3.53	11
2	Equal energy	8	2.83	7	2.83	14
3	Equal energy	5	2.12	7	2.12	19
4	Equal return period	5	4.40	8	2.76	11½
5	Equal return period	5	4.40	8	1.50	15
6	Equal return period	5	4.40	8	0.70	21

The "equal energy" series
The wave spectra for each test within Series 1 (Figure 3.1) were arranged to have *equal energy* (i.e. $H_{s1} = H_{s2}$):

Test 1a - pure wind-sea
Test 1b - 20% swell
Test 1c - 50% swell
Test 1d - 80% swell
Test 1e - pure swell.

Series 2 and 3 were arranged the same way. Series 2 included three additional tests:

Test 2a1 - 2% swell
Test 2a2 - 8% swell
Test 2b3 - 32% swell.

The "equal return period" series
The wave heights for each test within Series 4 (T_{p2} = 11.5s) were arranged to have *equal return period*, based on expectations for the east coast of England (in practice $H_{s1} > H_{s2}$):

Test 4a - extreme wind-sea alone
Test 4b - more extreme wind-sea than swell
Test 4c - equally extreme wind-sea and swell
Test 4d - more extreme swell than wind-sea
Test 4e - extreme swell-sea alone.

Series 5 (T_{p2} = 15s) and 6 (T_{p2} = 21s) were arranged the same way. However, since Tests 4a, 5a and 6a would be identical (H_{s1} = 4.4m, T_{p1} = 8.0s), Tests 5a and 6a were not actually run in the flume tests: instead (where necessary) results for Test 4a serve for all three.

The flume tests
Nearly all of the 33 test conditions were run for each of:

two seawall slopes - 1:2 and 1:4
two offshore water depths - 14m (high freeboard) and 16m (low freeboard)
one foreshore slope - 1:50.

Series 1 was re-run for some additional foreshore slopes of 1:7, 1:10 and 1:20. About 150 tests were run altogether.

The wave spectra and measurements
The wave spectra needed to drive the wavemaker were based on two superimposed JONSWAP spectra (Reference 1), one with the H_s and T_p of the wind-sea component and one with the H_s and T_p of the swell component (Figure 3.1). Wave conditions were measured in the

approaches to the foreshore and at the toe of the wall; the mean overtopping rate was calculated from a measurement of the total volume of overtopping during the test.

Full details of the tests and measurements are given in the companion Technical Report (Reference 2).

3.2 Analysis of the results

3.2.1 Outline of the analysis

Existing prediction methods (Owen, 1980 – Reference 6; van der Meer & Janssen, 1995 – Reference 7) for overtopping were shown to match the measurements under wind-sea conditions reasonably well. Section 3.2 describes the attempt to find a workable empirical method for prediction of the mean overtopping rate under swell and bi-modal sea conditions. The hope was to demonstrate a possible use for bi-modal sea data within a desk study approach. The analysis fell into two main stages:

1. Stage 1 involved the application of existing prediction methods to the swell wave tests, and when they proved to be inadequate, the development of rule-of-thumb adjustments to produce better agreement (prediction of swell wave overtopping was not the primary purpose of this project, and it may be worth returning to this topic in further research at a later date).

2. Stage 2 started from the position (at the end of Stage 1) of having adequate predictions of the mean overtopping rate under both wind-sea and swell-sea conditions. It attempted then to predict the overtopping for the intermediate bi-modal conditions, based on the separate predictions for wind-sea and swell and a knowledge of the relative amounts of energy in the two modes.

3.2.2 Presentation of the overtopping measurements

For convenience and consistency of presentation throughout this chapter, the tests were split into five sets as described in Table 3.2, with about 30 tests in each one.

Table 3.2 Division of the tests into five sets for presentation purposes

Set No	Wave series	Foreshore	Seawall	Water depth
Set 1	Series 1-6	1:50	1:2	14m
Set 2	Series 1-6	1:50	1:2	16m
Set 3	Series 1-6	1:50	1:4	14m
Set 4	Series 1-6	1:50	1:4	16m
Set 5	Series 1	1:7, 10, 20, 50	1:2, 1:4	14m

The measured mean overtopping rates are plotted in Figures 3.2-3.6; natural dimensional scales are used so as to highlight relatively small differences within each group of tests. Tests within each group of about five are related either by having equal energy (Series 1-3) or equal return period (Series 4-6), label "a" indicating wind-sea through to label "e" indicating swell.

Where related tests had equal energy, overtopping clearly increased with the move from wind-sea, through bi-modal sea, towards swell. However, the rate of increase with wave period, particularly for the steeper wall slope, was rather less than expected based on tentative predictions using Owen's formula. For the series with equal return period, the results were mixed: for the 1:2 slope, mean overtopping in the wind-sea conditions was higher than for the swell conditions; for the 1:4 slope, there was little difference in overtopping within some of the series, but often the wind-sea was the worst case.

There was clearly an effect of wave period, but rather less than had been expected, and very much less in the case of the 1:2 wall slope. Figure 3.7 focuses on this by contrasting results for Series 1 and 4, for each of the two structure slopes, at the same water level.

The effects of foreshore slope can be seen in Figure 3.6. Overtopping increases slightly as the foreshore slope increases (so allowing a slightly higher depth-limited wave height at the toe of the wall) whilst at the same time the dependence on wave period reduces slightly.

3.2.3 Analysis of wind-sea and swell-sea overtopping

Two commonly used empirical methods were used as the basis for a prediction scheme for mean overtopping in the wind-sea and swell-sea tests (excluding the bi-modal tests at this stage). These are the Owen (Reference 6) and van der Meer and Janssen (Reference 7) formulae, which are described and discussed in Besley (1997 - Reference 8))[1].

The two methods were applied to all of the non-bi-modal test cases, and the resulting predictions were compared with measured rates of overtopping. The results for the 1:50 foreshore slope are shown in Figures 3.8 to 3.11; measurements for steeper foreshore slopes (Set 5) are given in Figure 3.12, in this case compared only with predictions by Owen's method as van der Meer is inappropriate here. The *mean* wave period (T_m) required by SWALLOW was taken to be $0.8T_p$, a slightly lower value than theory would suggest, but using a ratio between T_m and T_p fairly typical of coastal seas.

There seems little to choose between the two methods. As would be expected, both work well for the wind-sea conditions for which they were developed. Owen's method, which assumes that overtopping will increase indefinitely as wave period increases, over-predicts the mean overtopping rate in swell, by a factor of up to about four. Conversely, van der Meer's method, which includes much less dependence upon wave period, under-predicts by up to a factor of one hundred.

Neither method is adequate for predicting overtopping under swell wave conditions without significant modification. To take the analysis forward (and guided partly by a recommendation in Reference 8 to continue using the Owen formula) the next section describes the search for a simple modification of the SWALLOW method to produce workable overtopping predictions for swell-seas.

3.2.4 Use of the Owen formula for swell-sea overtopping

Owen's method was developed from basin tests representing storm wave conditions in the Severn Estuary, reported by Owen (Reference 6). The range of wave steepness ($2\pi H_s/gT_m^2$) tested was 0.035-0.055 (compared to 0.005-0.020 for the present swell wave tests). The method is driven by the deep water significant wave height and mean period, although the height is modified for wave breaking on the foreshore before being used in the Owen formula. Owen's "breaking wave height" (at the toe of the structure) was not specifically intended to be a best estimate of the real breaking wave height, but rather a realistic value designed to give the correct answer when used in the overtopping formula. The same is true to some extent even where breaking does not occur, in that the deep water wave height is used directly in the formula, even though shoaling may have occurred.

The search for an adjustment to Owen's method to produce a workable method for swell seas involved three alternative approaches, all of them having some physical basis rather than simply using arbitrary correction factors.

<u>The first approach</u> considered the use of the *measured* wave height at the toe of the structure (available from the present measurements, but not recorded during Owen's tests) as input to the Owen formula. For these tests, the difference between the measured wave height, and the wave height used by Owen's method was relatively small, and the generally higher measured values would lead to increased over-prediction. This approach was not pursued further, and the

[1] Subsequent to publication of SR 507, the formulae of van der Meer and Janssen (Reference 7) have been changed (Reference 14). The new formulae alter the results of some of the tests but the conclusions drawn here remain valid.

results are not reproduced here. It was, however, concluded that Owen's over-predictions were primarily due to the Owen formula being inappropriate for swell seas, rather than to any error in the wave height at the toe.

The second approach notes the implication of Owen's formula that overtopping will continue to increase indefinitely as a function of wave period. It seems more likely that beyond some limit, the rate of increase will gradually reduce to a point at which no further increase occurs. With this in mind, the wave periods for all of the individual swell wave tests were reduced to the values needed for Owen's method to match the measured mean overtopping rates. This approach was quite promising in that there did appear to be a limiting wave period (in the range 6-10 seconds) for each group of about five tests, beyond which measured overtopping did not increase further. Although it was noted that the limiting wave period was consistently higher for the 1:4 slope than for the 1:2 slope, its value seemed otherwise unpredictable without resort to model tests. This approach was therefore not taken further.

The third approach, and the one eventually adopted, notes that Owen's formula over-predicts for long wave periods, especially on steeper slopes. It also notes that the surf similarity parameter or Iribarren Number (ξ), often used as a guide to the way in which waves behave in the surf zone, combines these two parameters in a convenient way. For each of the wind-sea and swell-sea tests, the degree of over (or under) prediction (F) and ξ were calculated from:

F = (predicted - measured overtopping) / (measured overtopping)

$$\xi = \tan\alpha / \sqrt{s_m}$$

where $\tan\alpha$ = structure slope,
and s_m = mean wave steepness $2\pi H_s/gT_m^2$

Values of F and ξ for all the tests (arranged in order of increasing ξ) are plotted against each other in Figure 3.13. The physical implications of Figure 3.13 are that the Owen's predictions of discharge are reasonably close to target values for plunging waves ($\xi < 2.3$), but increasingly above measured values for higher values of ξ corresponding to collapsing and surging waves. (This effect is incorporated in van der Meer's method by the use of two separate formulae for the two ranges.) The empirical adjustment factors for Owen's predictions of mean overtopping rate listed in the table below were derived as a function of Iribarren Number.

Table 3.3 Factors to be applied to SWALLOW predictions

Range of Iribarren Number	Adjustment factor (F)
$0.0 < \xi < 2.5$	~~1 / 1.0~~ 0
$2.5 < \xi < 3.0$	1 / 2.5
$3.0 < \xi < 4.3$	1 / 4.5
$4.3 < \xi$	1 / 8.0

See Figure 3.13

The promising possibility of using *inshore* wave length, as opposed to the *offshore* wave length implicit in the use of the normal Iribarren Number, was also considered briefly as a way of limiting the effect of the largest wave periods. Thorough analysis of swell wave overtopping, and the development of new types of method, are beyond the scope of the present project, but may repay further research in the future. At present however, the approach summarised in Table 3.3 is sufficient to be able to move on to the main thrust of the analysis described in the next section.

3.2.5 Analysis of bi-modal sea overtopping

Several approaches to the prediction of overtopping discharge under bi-modal sea conditions were tested. The following "weighted average" method was the most successful, although it does rely on the existence of adequate methods for wind-sea alone and swell-sea alone.

Step 1
Start with values of H_{s1} and T_{m1} for the wind-sea component of a bi-modal sea, and values of H_{s2} and T_{m2} for the swell-sea component. These can be obtained, for example, with reference to the swell atlas (Reference 1).

Step 2
Calculate the overall significant wave height $H_s = \sqrt{(H^2_{s1} + H^2_{s2})}$ by summing the energies of the two components.

Step 3
Run SWALLOW using T_{m1} (wind-sea period) and the overall H_s as input, to obtain mean overtopping rate Q_1.

Step 4
Run SWALLOW using T_{m2} (swell-sea period) and the overall H_s as input, to obtain a mean overtopping rate; calculate an Iribarren Number, based on H_s, T_{m2} and the wall slope, and estimate a SWALLOW reduction factor F from Table 3.3; hence calculate mean overtopping rate Q_2 as the product of these two values.

Step 5
Estimate an overall mean overtopping rate (Q), calculated as an energy-weighted average of the two separate predictions Q_1 and Q_2:

$$Q = (Q_1H^2_{s1} + Q_2H^2_{s2}) / (H^2_{s1} + H^2_{s2})$$

Figures 3.14-3.18 show the measured overtopping rates for all tests (the same values already presented in Figures 3.2-3.6 and 3.8-3.12) together with the corresponding predictions from the modified Owen method. Predictions for the wind-sea tests are taken directly from Owen's formula; predictions for swell are based on Owen's method with a reduction factor based on Table 3.3; predictions for bi-modal seas are derived as described in Steps 1-5 above.

The comparisons between measurements and predictions are adequate throughout the range of tests (perhaps not surprising, given the way that the empirical adjustment factors listed in Table 3.3 were derived). The predictions for bi-modal seas can (at best) only be as good as the individual predictions for wind-sea and swell, which were themselves subject to some uncertainty. The larger part of the difference between measurements and predictions for the bi-modal sea tests is carried through from the separate preliminary predictions for wind-sea and swell. The main purpose of the analysis and comparisons was to demonstrate a prediction method *for bi-modal seas*, and the part of the method specifically relating to bi-modal seas worked well.

3.3 Observations and ways forward

3.3.1 Observations on measurements and predictions of swell-sea overtopping

Overtopping increases, for a given wave height, with the increase in wave period from wind-sea, through bi-modal sea to swell. The measurements showed the degree to which the overtopping increased, and the approximate proportion of swell needed to cause a significant increase.

Generally the overtopping did not increase with wave period as much as had been expected. Above a mean wave period of about six to ten seconds (the exact value being dependent on wall slope and water level) there was little further increase in overtopping. Owen's formula, which assumes that overtopping increases indefinitely with increasing wave period, significantly over-predicted swell wave overtopping. Conversely, van der Meer & Janssen's method, which has no effect of wave period once wave breaking has entered the surging range, significantly under-predicted swell wave overtopping. Neither method, as it stands, provided an adequate comparison with measurements in swell-sea conditions.

Various modifications of Owen's method were investigated, and a number of promising possibilities were identified as described in Section 3.2.4. In order to take the present project forwards, an empirical adjustment factor was derived, as a function of Iribarren Number (and hence implicitly the form of wave breaking) to be applied to the direct SWALLOW predictions. In practice, no adjustment was made in the range of plunging wave breaking ($\xi < 2.3$) but reductions were made at values of ξ above 2.5. There is no theoretical basis for this adjustment, and it may not be generally applicable, but it did provide reasonably good agreement between SWALLOW and the present swell-sea model tests.

3.3.2 Observations on measurements and predictions of bi-modal sea overtopping

Simple visual inspection of the measurements shown in Figures 3.2-3.6 shows that the rate of overtopping in bi-modal seas is usually somewhere between the rates for wind-sea and swell-sea. The rate of overtopping (at least for the equal energy test series) increases roughly in proportion to the percentage of swell in the spectrum, even for quite small amounts of swell. This in turn suggests that a simple function of the two separate overtopping rates for wind-sea and swell-sea might provide an adequate prediction of overtopping in a bi-modal sea condition.

A reliable and physically plausible approach was developed for prediction of overtopping rate in bi-modal seas. All of the wave energy in the spectrum was applied firstly at the mean wave period for the wind-sea component and secondly at the mean wave period for the swell-sea component; a simple energy-weighted average of the two separate rates was then calculated.

In practice, prediction of overtopping under bi-modal seas is therefore a two-stage process: prediction (or measurement) of overtopping for wind-sea and for swell-sea, and then the weighted averaging of the two rates. There appears to be far more uncertainty associated with the first stage than with the second, but a workable method giving fair agreement with the present measurements is detailed in Section 3.2.5.

It should be stressed that the simple methods of Sections 3.2.4 and 3.2.5 apply only to overtopping rate (and strictly only to the range of situations studied in the physical model). They would not necessarily work well for other coastal response parameters, and do not obviate the need for physical model tests of bi-modal and other sea states. Nevertheless, they do provide a workable first estimate of overtopping rate under swell and bi-modal sea conditions, where these are considered to be important in design.

3.3.3 Are bi-modal and swell seas important in design?

As might have been expected, the answer to this question is a qualified "Yes". The mean overtopping rate for a swell-sea is significantly higher than for a wind-sea *with the same wave height*, although not as much higher as might have been expected based on the commonly used Owen formula. However, for the test series based on an equal frequency of occurrence on the east coast of England (Test Series 4-6), the wind-sea often produced higher overtopping than equivalent bi-modal and swell-sea tests.

These observations tend to support the present intuitive approach to the use of swell beginning to be used in HR's coastal engineering consultancy studies. In geographical areas where swell is considered to be important, it may be treated as a separate design case. In areas where the highest swell significant wave heights are less than half that of the highest wind-seas, it may not need specific consideration. As a rough guide we would suggest that coastal engineering studies in exposed locations in the south-west part of the UK, south of Holyhead and west of the Isle of Wight, should consider swell as a potential worst case for design. In other areas, it is probably not the worst case in terms of overtopping rate, but see below.

A less obvious, but quite common situation in which swell may be important, is where wave heights are depth-limited at the toe of a seawall. If wave heights are limited to some fairly low value (say one metre or so) by the maximum water depth at the structure, then the fact that wind-sea wave heights may be much higher offshore is largely irrelevant. A metre or so of swell may be a significantly worse case than a metre or so of wind-sea, as shown by the results for Test Series 1-3. Therefore, where waves are severely depth-limited at a structure, swell is quite likely to be a worst case for design in terms of overtopping, even in geographical areas not particularly exposed to swell.

The difficulty in making reliable predictions of overtopping rate for swell-seas confirms the continuing need for physical model tests of these conditions.

The flume test results (both for the equal energy and the equal return period series) show that bi-modal seas do not cause higher overtopping than whichever is the higher of the equivalent wind-sea and swell-sea conditions. This implies that for calculation of mean overtopping rate, bi-modal seas are not likely to be a potential worst case for design. This does not necessarily mean, of course, that they may not be more important in other aspects of design.

The one situation when bi-modal seas may be important for overtopping is where a small amount of swell is concealed within what appears to be a severe wind-sea. Tests 1b, 2b and 3b (with 20% swell) at the lower water depth on the less steep wall slope give roughly twice as high overtopping as equivalent Tests 1a, 2a and 3a (with no swell). (Admittedly, pure swell with the same spectral energy would be an even worse case, but this may not be considered in design as it may not be a realistic sea condition at most sites.)

4 Effect of bi-modal seas on other parameters

4.1 Introduction

In addition to wave overtopping and beach responses, other hydraulic parameters were also studied during this research. Measurements of wave induced pressures, armour movements, wave run-up/run-down exceedance levels, wave transmission and wave reflection were obtained. A description and presentation of these research measurements was given previously by Coates, Jones & Bona (Reference 2).

The purpose of this chapter of the report is to consolidate the current understanding of the influence of bi-modal seas upon these other hydraulic responses. Wave induced pressures are not reported here, but will be reported separately by McConnell & Allsop (Reference 9).

The results presented in Chapter 3 and in the present chapter should be applied with caution. They are not intended to provide definitive guidance on structural responses under swell and bi-modal seas, but they do have relevance to design criteria covered by the scope of the tests, namely:

- overtopping, run-up, run-down, reflection coefficient and armour movement;
- on plain slopes in the range 1:2 to 1:4;
- for breaking and non-breaking waves arriving on a foreshore slope of about 1:50.

4.2 Armour movements

Armour movements were determined for a simple 1:2 non-overtopping seawall structure using uni-modal and bi-modal wave conditions. Approach seabed slopes of 1:50 and 1:20 were studied. The nominal offshore wave steepness, $s_{mo,}$ of the uni-modal wave conditions was either 0.02 or 0.04. The percentage of armour displacements was determined using a detailed photographic method described by Coates, Jones & Bona (Reference 2). Corresponding armour damage levels, S, were estimated according to a method described by van der Meer (Reference 10).

Van der Meer devised formulae to calculate armour damage which include the effects of random waves, storm duration, a wide range of core / underlayer permeabilities, and distinguish between plunging and surging wave conditions. For plunging waves:

$$H_s/\Delta D_{n50} = 6.2\ P^{0.18}(S/\sqrt{N_z})^{0.2}\ \xi_m^{-0.5}$$

and for surging waves:

$$H_s/\Delta D_{n50} = 1.0\ P^{-0.13}(S/\sqrt{N_z})^{0.2}\ \sqrt{\cot\alpha}\ \xi_m^{P}$$

Where the parameters are defined as:

Δ	relative buoyant density of material considered, $(\rho_r/\rho_w) - 1$
ρ_r	mass density of rock
ρ_w	mass density of sea water
D_{n50}	nominal diameter of armour
P	notional permeability factor
S	design damage number = A_e/D_{n50}^2
A_e	erosion area from profile
N_z	number of waves
ξ_m	Iribarren number = $\tan\alpha/s_m^{1/2}$
s_m	wave steepness for mean period = $2\pi H_s/gT_m^2$ (H_s and T_m were derived using a statistical analysis)

and the transition from plunging to surging waves is calculated using a critical value of ξ_m:

$$\xi_m = (6.2\ P^{0.31}\ (\tan\alpha)^{0.5}\ )^{1/(P+0.5)}$$

Before considering the influence of bi-modal seas upon armour movement, it is necessary to consider the armour movements caused by uni-modal wave action. The percentage of full displacements observed using uni-modal wave conditions is shown in Figure 4.1a. Data has been re-presented in terms of armour damage level, S, in Figure 4.1b. Three groups of uni-modal wave conditions were considered in the analysis, two shallow water groups (measured at the seawall toe) described by s_{mo}=0.02 & 0.04; and one moderate water group, s_{mo}=0.04 (where s_{mo} is the offshore wave steepness). The data has been normalised by the offshore significant wave height, thus tests with the same identifier had the same nominal offshore wave height (while period varied with sea steepnesses). When considering Figures 4.1a & b it should be noted that an armour damage of 30%, or damage level S=25, represents failure of the seawall slope.

Two conclusions can be drawn from Figures 4.1a and 4.1b. More armour movements were seen with a 1:20 approach slope when compared with the 1:50 approach slope. Less damage was observed on both the 1:50 and 1:20 approach slopes at s_{mo}=0.04 for the moderate water depth case. An analysis of the inshore wave height statistics indicates that consistently larger wave heights were present in the test with the lower static water level, due to greater wave shoaling, leading to greater levels of armour displacements as demonstrated in Figure 4.2a. The example figure gives the significant wave height against the distance from the offshore wave probe for Test Ob4/3A in water depths of 12m and 14m, using a foreshore slope of 1:50. The structure was built 580m from the offshore probe. Wave heights with the shallow water are consistently higher than those with moderate water level in the area of the structure.

The armour damage levels predicted by van der Meer for approach slopes of 1:50 and 1:20 have been presented in Figure 4.2b. The inshore significant wave height, H_{si}, and the offshore mean wave period, T_{mo}, and an assumed permeability factor, P=0.1, were used to predict armour damage, S, given in the figure. Figure 4.2b can be compared with Figure 4.1b. Predicted values of S were greater for the 1:20 slope compared with the 1:50 approach slope, but predicted differences would not account for the large differences seen earlier in Figure 4.1b.

Armour damage, S, predicted using both the offshore and inshore significant wave heights, has been compared with movements recorded during the tests in Figures 4.3a & b. Movements recorded with an approach slope of 1:50 are shown in Figure 4.3a, while those recorded with a 1:20 approach slope are shown in Figure 4.3b. Van der Meer's tests were completed for shallow approach slopes: good agreement between the measured and predicted damage levels would normally be anticipated for a 1:50 approach slope. Throughout the tests using the 1:50 approach slope, measured damage was less than predicted damage, indicating that the research model was more stable than those of van der Meer. It would be feasible to adjust the armour damage levels in accordance with the prediction method, but, it would then be necessary to make the same adjustment to both the 1:20 and bi-modal wave tests. That adjustment has not been made. On the whole, measured damage on the steeper 1:20 slope was greater than predicted damage and an adjustment would further increase the difference.

The increased levels of armour damage observed on the 1:20 approach slope were also observed in an earlier research study conducted by Allsop & Jones (Reference 11). In their research Allsop & Jones suggest a modification to the van der Meer formulae to take account of increased levels of armour damage with steep approach slopes.

For plunging waves:

$$H_s/\Delta D_{n50} = 4.8\ P^{0.18}(S/\sqrt{N_z})^{0.2}\ \xi_m^{-0.5}$$

and for surging waves:

$$H_s/\Delta D_{n50} = 0.77\ P^{-0.13}(S/\sqrt{N_z})^{0.2}\ \sqrt{\cot\alpha}\ \xi_m^{P}$$

These new formulae have been used to predict armour damage, S, for the 1:20 approach slope data presented in Figures 4.4a and b. The data presented in Figure 4.4b have been expressed in non-dimensional terms. The data points plotted at $S/N^{0.5} = 0.80$ were described as slope failure and may well result in higher values than 0.80. In practice a seawall designer might perhaps design using an armour damage S=2, and might allow for a storm duration of 2000 waves, resulting in $S/N^{0.5}=0.05$. The van der Meer prediction equation given in Figure 4.4b offers an upper bound (limit) to the data from the 1:50 approach slope, while the modification suggested by Allsop & Jones offers a similar upper bound for the 1:20 approach slope data leading to the conclusion that the armour movements produced by uni-modal waves follow previously published criteria.

Having considered the influence of uni-modal waves upon armour movements, it is now possible to consider the influence of bi-modal waves. The percentage of full displacements observed for tests conducted using the bi-modal wave conditions are shown in Figure 4.5a. The data has been re-presented in terms of armour damage level, S, in Figure 4.5b. Two groups of bi-modal waves were studied. The first group studied was defined as the "equal energy" test series, while the second group studied was classified as the "equal return period" series (see grouping explanation section 3.1.2).

It is possible to draw three conclusions from Figures 4.5a and b. The steepness of the approach slope influenced the number of armour movements. As was seen in the uni-modal wave test series, there were more movements with the 1:20 approach slope than with the 1:50 approach slope. The amount of armour damage increased with an increase in the percentage of swell-sea during the "equal energy" tests. During the "equal return" period tests the level of armour damage sustained by the seawall remained relatively constant despite the range of different swell-sea combinations tested.

The measured values of S have been compared to predicted values of S for both the 1:50 and 1:20 approach slopes in Figure 4.6a. The same data have been presented in non-dimensional terms in Figure 4.6b. Again van der Meer's prediction method has been applied to the 1:50 approach slope data, while the modification of Allsop & Jones has been applied to the 1:20 data. In all cases the measured damage was less than that obtained from the prediction methods. Test identifiers have been included for the data from individual tests on Figure 4.6b. The "equal energy" test series data gives a clear trend of increased damage level, S, with an increased proportion of swell-sea component. The "equal return period" series data was grouped together and showed no such relationship.

It is possible to compare the results of the uni-modal and bi-modal wave tests by comparing the non-dimensional plots given in Figures 4.4b and 4.6b. There is evidence to suggest that bi-modal waves produce no worse armour movements than comparable uni-modal wave conditions, i.e. those with the same significant wave height and mean wave period.

In summary, these tests have demonstrated that:

- More armour movements were observed with the 1:20 approach slope compared with the 1:50 approach slope;

- More armour movements were observed with the shallow water depth compared with the moderate water depth;
- Armour movement observations were within limits defined previously by van de Meer (for the 1:50 approach slope) and modified by Allsop & Jones for steep approach slopes (1:20 approach slope);
- Bi-modal seas did not influence armour stability beyond that which would be given by existing prediction methods.

4.3 Run-up/run-down

Wave run-up/run-down measurements were studied on 1:2 and 1:4 simple seawalls using an approach slope of 1:50. Two different static water levels were studied, 14 and 16m, (annotated as 7 or 8 respectively in the following presentation). Statistics of wave excursions were determined using software written at HR Wallingford. A summary of the results was presented previously by Coates, Jones & Bona (Reference 2); and summarised in this chapter in Figures 4.7 and 4.8. The level exceeded by only 2% of wave run-up/run-down events for both bi-modal and uni-modal waves has been plotted separately in the figures.

Investigations by Ahrens and Allsop reported in the CIRIA/CUR "Rock Manual" (Reference 12) used simple empirical relationships between the 2% run-up levels and the Iribarren number in the form:

$0 < \xi_m < 2.18$	$R_{u2\%} / H_s = 1.84\ \xi_m$	Ref 12 Eq 5.12, from Ahrens
$\xi_m > 2.18$	$R_{u2\%} / H_s = 4.5 - 0.23\xi_m$	Ref 12 Eq 5.13, from Ahrens
$2.44 < \xi_m < 5.22$	$R_{u2\%} / H_s = 3.39 - 0.24\xi_m$	Ref 12 Eq 5.16, from Allsop et al

These empirical relationships for run-up levels are given on Figure 4.7. A comparison of the run-up/run-down results presented in Figures 4.7 and 4.8 indicates that bi-modal wave conditions produce higher levels than corresponding uni-modal wave conditions.

Two groups of bi-modal waves were studied, an "equal energy" group and an "equal return period" group (section 3.1.2). The analysis has been continued considering the bi-modal wave groups separately, in non-dimensional terms, in Figures 4.9 to 4.12. The graphs obtained for run-up/run-down under the influence of the "equal energy" bi-modal wave group are given in Figures 4.9 and 4.10. As expected, run-up/run-down levels increased with increased wave period (increased Iribarren number) for both structure slopes studied. This may be seen by comparing tests identified on the figures with "a", wind sea only, through to swell-sea only, "e". The bi-modal wave tests "b", "c" and "d" falling between these 2 extremes follow the same trend, but there is evidence to suggest that the bi-modal wave conditions provided slightly higher levels of run-up/run-down. A similar trend was identified in tests of "equal return period" bi-modal waves, Figures 4.11 and 4.12. Again, run-up/run-down levels increased with increased wave period.

4.4 Wave transmission

Wave transmission measurements were studied using bi-modal waves for a 1:2 non-porous breakwater and a 1:2 bermed structure built with a 1:50 approach slope. Static water levels of 14 and 16m were studied. These simple breakwater structures incorporated a small crest and a simple 1:2 rear slope. A summary of the wave transmission data was presented previously by Coates, Jones & Bona (Reference 2).

The bi-modal wave data has been considered in either "equal energy" or "equal return period" groups. A presentation of the wave transmission data is given in Figure 4.13 for the "equal energy" group and Figure 4.14 for the "equal return period" group. (The simple 1:2 breakwater incorporating a small crest has been annotated with a "c" on the figures, while the bermed structure has been annotated "b".) The dimensionless freeboard, R_c/H_{si}, has been plotted in the figures against the wave transmission coefficient, C_t. The bermed structure suffered less wave transmission than the simple seawall. Tests conducted using both water levels suffered higher levels of transmission under the higher water level. An examination of both figures reveals that for the two bi-modal wave groupings a reduction in dimensionless freeboard usually resulted in a corresponding increase in wave transmission. There is, however, no evidence to suggest that bi-modal waves provide more wave transmission than corresponding uni-modal waves.

4.5 Wave reflection

The reflection performances of 1:2 and 1:4 simple seawalls were studied for both uni-modal and bi-modal wave conditions. A summary of the wave reflection data was presented previously by Coates, Jones & Bona (Reference 2).

Uni-modal and bi-modal wave reflections obtained for tests using an approach slope of 1:50 have been summarised in Figure 4.15. Investigations by Allsop (Reference 13) used a simple empirical relationship between the reflection coefficient C_r, and the mean Iribarren number ξ_m, in the form:

$$C_r = a\,\xi_m^2 / b+\xi_m^2$$

where a and b are empirically derived constants.

The results of tests at HR Wallingford suggest values for coefficients a = 0.96 and b = 4.8 for a smooth structure. The empirical relationship is shown as a prediction line in Figure 4.15. Both uni-modal and bi-modal waves possessed similar reflection performances, although there appeared to be less data scatter with the bi-modal waves, and they appeared to provide a better fit to the empirical relationship.

The reflection analysis has been taken a stage further in Figures 4.16 and 4.17, where the bi-modal wave data has been presented in groups of "equal energy" in Figure 4.16 or "equal return period" in Figure 4.17. Like the run-up/run-down analysis wave reflections increased with increased wave period (increased Iribarren number) for both structure slopes investigated. There is evidence to indicate that for some bi-modal wave conditions, wave reflection increased by approximately 10% compared with corresponding uni-modal waves (see Figure 4.16).

5 Practical applications of research results

In this chapter of the report, the practical applications of the research findings are described for the two main areas under consideration, shingle beach response and wave overtopping performance under the influence of bi-modal seas.

5.1 Shingle beach profiles

A generic method of predicting the impact of swell energy on shingle beach response has not been achieved due to the lack of compatibility between the results from this study and those of earlier work by Powell (Reference 4). However, important conclusions can be drawn that provide guidance for the design of shingle beaches.

For sites that experience significant swell the worst case design conditions may be those with a bi-modal spectrum or a spectrum with a broad range including a significant (20% or greater) proportion of low frequency energy. The graphs presented in Figures 2.5-2.10 can be used in conjunction with Powell's original work to give a first estimate of the required crest elevation and width to prevent failure of the beach as a part of a coast defence scheme. It should be noted that wave conditions used in this study are unbroken waves at the toe of the shingle beach. The swell wave atlas (Reference 1) provides offshore conditions for the coasts of England and Wales which can be transformed inshore.

Preliminary trials of the method using wave conditions with a constant return period suggest that 20% swell events can be important in crest level and cut-back predictions for sites where swell waves occur frequently. Sensitivity tests for specific sites will be required to establish the most promising designs, which can then be tested fully in a physical model.

Limitations with this approach are:

- Powell's work did not include wave steepnesses of less than 0.01 and is therefore not valid for long period swell;

- the present study did not investigate shallow nearshore conditions or low wave heights, so some extrapolation will be needed to predict beach response where waves are depth limited by the nearshore bathymetry.

5.2 Overtopping

5.2.1 Overtopping in swell-seas

Design situation 1: Geographical

In geographical areas where swell is considered to be important, it should probably be considered as a separate design case; in areas where the highest swell significant wave heights are less than half that of the highest wind-seas, it may not need specific consideration. As a rough guide we would recommend that coastal engineering studies in exposed locations in the south-west part of the UK, south of Holyhead and west of the Isle of Wight, should consider swell as a potential worst case for design. This is broadly in line with current practice, but we would suggest that it be made a more formal requirement, since its neglect may lead to an underestimate of the risk due to overtopping.

Swell is also important in other exposed areas such as south and west Ireland and Scotland, but as yet there is no swell wave atlas outside England and Wales.

Design situation 2: Depth limitation

Swell may be important where wave heights are severely depth-limited at the toe of a seawall: the fact that wind-sea wave heights may be much higher than swell-sea wave heights offshore is largely irrelevant. A metre or so of swell may be a significantly worse case than a (depth-limited) metre or so of wind-sea. Therefore, where waves are strongly depth-limited at a structure, we would recommend that swell be considered as a potential worst case for design in terms of overtopping, even in geographical areas not particularly exposed to swell. This tends not to be done at present, and results for Test Series 1-3 suggest that the risk due to overtopping may be underestimated as a result.

Overtopping rate for swell

Overtopping increases with wave period, but not by as much as had been expected. Above a mean wave period of about six to ten seconds (the exact value being dependent on wall slope and water level) there was little further increase in overtopping. The SWALLOW (Owen, 1980, Reference 6) empirical formula, which assumes that overtopping increases indefinitely with increasing wave period, significantly over-predicted swell wave overtopping. Conversely, van der Meer and Janssen's (1995, Reference 7) method, which has no effect of wave period once wave breaking has entered the surging range, significantly under-predicted swell wave overtopping.

Workable empirical prediction method

An empirical adjustment factor was derived, as a function of Iribarren Number (and hence implicitly the form of wave breaking), to be applied to the direct Owen's predictions. In practice, no adjustment was needed in the range of plunging wave breaking ($\xi < 2.3$) but reductions were made at higher values of ξ. There is no theoretical basis for this adjustment, and it may not be generally applicable, but it did provide reasonably good agreement between SWALLOW and the present swell-sea model tests. The difficulty in making reliable predictions of overtopping rate for swell-seas confirms the continuing need for research and physical model tests of these conditions. (A possible alternative to the empirical methods usually used would be to apply ANEMONE-OTT which can numerically model swell wave overtopping using a empirical wave flume approach.)

5.2.2 Overtopping in bi-modal seas

Overtopping rate for bi-modal seas

The rate of overtopping in bi-modal seas is usually somewhere between the rates for wind-sea and swell-sea. The rate of overtopping (at least for the equal energy test series) increases roughly in proportion to the percentage of swell in the spectrum, even for quite small amounts of swell. This implies that for calculation of mean overtopping rate, where pure wind-sea and pure swell have already been considered, bi-modal seas are not likely to be a potential worst case for design. This does not necessarily mean, of course, that they may not be more important in other aspects of design.

Design situation for bi-modal seas
One situation in which bi-modal seas may be important for overtopping is where a small amount of swell is concealed within what appears to be a severe wind-sea. Tests 1b, 2b and 3b (with 20% swell) give up to twice as much overtopping as equivalent Tests 1a, 2a and 3a (with no swell). A probabilistic approach to assessing wave conditions would be required to determine the importance of these particular bi-modal conditions.

Workable empirical prediction method
A reliable and physically plausible approach was demonstrated for prediction of overtopping rate in bi-modal seas. All of the wave energy in the spectrum was applied firstly at the mean wave period for the wind-sea component and secondly at the mean wave period for the swell-sea component; a simple energy-weighted average of the two separate rates was then calculated. There appears to be far less uncertainty here than in prediction of overtopping due to swell wave energy alone.

Warning about the empirical method
It should be stressed that the simple method for swell and bi-modal seas detailed in Sections 3.2.4 and 3.2.5 applies only to overtopping rate (and strictly only to the range of situations studied in the flume model). It would not necessarily work well for other coastal response parameters, and does not obviate the need for physical model tests of bi-modal and other sea states.

6 Conclusions

6.1 Hydraulic processes

Beach response to bi-modal seas
Flume model results confirmed field observations of the importance of long period energy as a significant influence on beach response. A generic predictive method has not been developed, but graphs are presented that provide guidance for preliminary beach design. Design that does not consider this work may result in beach failure.

Overtopping rate in swell-seas
Overtopping increases with the increase in wave period from wind-sea, through bi-modal sea, to swell, although not by as much as would be predicted by Owen's method (Reference 6). An empirical adjustment factor was derived (as a function of Iribarren Number) to be applied to predictions using Owen's equations of swell-sea overtopping. The difficulty in making reliable predictions of overtopping rate for swell-seas confirms the continuing need for research and physical model tests of these conditions.

Overtopping rate in bi-modal seas
The rate of overtopping in bi-modal seas is usually somewhere between the rates for wind-sea and swell-sea. The rate of overtopping (at least for the equal energy test series) increases roughly in proportion to the percentage of swell in the spectrum, even for quite small amounts of swell. A workable approach was developed for prediction of overtopping rate in bi-modal seas. All of the wave energy in the spectrum is applied firstly at the mean wave period for the wind-sea component and secondly at the mean wave period for the swell-sea component; a simple energy-weighted average of the two separate rates is then calculated. There appears to be far less uncertainty here than in prediction of overtopping due to swell wave energy alone.

Armour movements
Armour movements were studied using a 1:2 seawall constructed with approach slopes of either 1:50 or 1:20. The seawall was tested using both uni-modal and bi-modal wave conditions. Under uni-modal wave action more movements were detected with the 1:20 approach slope than with the 1:50 approach slope. A standard prediction method, developed

by van der Meer, provided an upper limit to armour movements under tests on the seawall with a 1:50 approach slope. A second prediction method, developed by Allsop and Jones allowing for steeper approach slopes, provided an upper limit to armour movements for tests conducted on the seawall with the steeper 1:20 approach slope. The "equal energy" group of bi-modal waves suggested a trend of increasing armour displacements associated with increased proportion of swell-sea.

Wave run-up/run-down
As expected, wave run-up/run-down exceedance levels increased with increasing wave period. There was evidence that bi-modal seas produced slightly higher exceedance levels than corresponding uni-modal wave conditions.

Wave transmission
Wave transmission was studied using a simple seawall structure and a structure incorporating a berm. Both structures were constructed with 1:2 front slopes. The level of wave transmission was reduced behind the bermed structure compared with the transmission behind the simple seawall structure. A reduction in the dimensionless freeboard resulted in a corresponding increase in wave transmission. There was, however, no evidence that bi-modal wave conditions produced different levels of wave transmission compared with uni-modal wave conditions.

Wave reflections
Wave reflection performances were studied using 1:2 and 1:4 simple seawalls. Both bi-modal and uni-modal waves possessed similar reflection performances, although there appeared to be less data scatter with the bi-modal wave conditions studied. There was evidence indicating that under some bi-modal wave conditions the reflection was increased by about 10% compared with corresponding uni-modal wave conditions.

6.2 Design significance

Design situations where swell may be important
In geographical areas where swell is considered to be important, it should probably be considered as a separate design case. As a rough guide we would recommend that coastal engineering studies in locations exposed to the Atlantic should consider swell as a potential worst case for design.

Depth limitations
Swell may be important where wave heights are severely depth-limited at the toe of a seawall: the fact that wind-sea wave heights may be much higher than swell-sea wave heights offshore is largely irrelevant. A metre or so of swell may be a significantly worse case than a (depth-limited) metre or so of wind-sea. Therefore, where waves are strongly depth-limited at a structure, we would recommend that swell be considered as a potential worst case for design in terms of overtopping, even in geographical areas not particularly exposed to swell.

Design situation where bi-modal seas may be important
Bi-modal seas may be important for overtopping where a small amount of swell is concealed within what appears to be a severe wind-sea. In some tests, conversion of 20% of the wind-sea energy to swell produced up to twice as much overtopping. A probabilistic approach to assessing the occurrence of bi-modal conditions, is required to determine the worst case conditions.

Which hydraulic processes?
In bi-modal sea conditions containing 20% swell, typical of exposed oceanic shorelines, the following checklist shows which processes would be affected by the presence of the swell component.

- Shingle beach profile - yes
- Overtopping - probably
- Run-up - probably
- Armour - probably
- Transmission - no
- Reflection - yes

7 Identification of further research

The swell wave atlas (Reference 1) provides guidance on the prediction of bi-modal waves around England and Wales. However, little work has been devoted to analysing wave records to verify the predictions. As bi-modal waves have been shown to be important to beach and seawall design, then it is recommended that work be undertaken to address this gap.

The present study on beach response considers a very limited range of wave conditions and does not address depth limited waves. Work is needed to improve prediction of wave spectra at the beach face and, if necessary, to revisit the flume study to determine the effect of shallow nearshore bed slopes.

The inability of existing formulae to predict overtopping in swell-sea conditions came as a surprise. Although not directly relevant within the present research project, this was perhaps the single most interesting conclusion to come from the overtopping tests. The reasons for this were briefly explored in Sections 3.2.3 and 3.2.4, but there is much more that could be done with the present data. The empirical adjustment of SWALLOW for use with swell-seas described in Section 3.2.4 was enough to be able to complete the subsequent bi-modal sea analysis, but is rather unsatisfactory for wider use. A number of promising possibilities were identified in Section 3.2.4 for improved predictions of overtopping at swell wave periods. This appears to be a priority for further research based on continuing analysis of the existing data, preferably as soon as possible whilst the ideas are still fresh.

Further analysis and development of empirical methods for the "other" parameters, i.e. run-up, run-down, armour damage etc, considered in Chapter 4 would be useful. However, it seems more efficient to delay this until discussion of the present report is complete and until the recommended further research (if any) on swell wave overtopping is complete. Further work on empirical prediction methods for overtopping under bi-modal seas should also be delayed until very much better methods are available for swell alone.

8 Acknowledgements

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This study was supervised by Dr P J Hawkes, Mr T T Coates and Professor N W H Allsop. The flume work and data processing was conducted by Mr R J Jones, Mr B Gouldby and Mr P F D Bona. The authors are pleased to acknowledge the support and assistance in testing and analysis by Mr M Cavanna, Ms J Gill and Mr N P Keshwala.

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Tables

Table 2.1 Wave conditions used in the shingle beach tests

Test number	Condition reference	Wave conditions				Depth of water (m)
		Wind-sea		Swell		
		H_{s1} (m)	T_{p1} (s)	H_{s2} (m)	T_{p2} (s)	
Tests for comparison with SR219						
0a	K02	0.75	5	----	----	14
0b	K04	1.5	5	----	----	14
0c	K06	2.4	5	----	----	14
0a1	K02	0.75	5	----	----	14
0a2	K02	0.75	5	----	----	14
0b1	K04	1.5	5	----	----	14
Tests with constant return period (1 in 6 months)						
1	0g6	4.4	8	----	----	14
2	5b	4	7.63	1.08	15	14
3	5c	3.4	7.03	1.27	15	14
4	5d	2.6	6.15	1.39	15	14
5	5e	----	----	1.5	15	14
1a	0g6	4.4	8	----	----	14
Tests with constant total spectral energy H_s = 3.53m and T_{p2} = 11s						
6	0e6	3.53	7	----	----	14
7	1b	3.12	7	1.67	11	14
8	1c	2.5	7	2.5	11	14
9	1d	1.67	7	3.12	11	14
10	1e	----	----	3.53	11	14
6 repeat	0e6	3.53	7	----	----	14
7 repeat	1b	3.12	7	1.67	11	14
9 repeat	1d	1.67	7	3.12	11	14
Tests with constant total spectral energy H_s = 2.12m and T_{p2} = 19s						
11	0b4	2.12	7	----	----	14
12	3b	1.91	7	0.94	19	14
13	3c	1.5	7	1.5	19	14
14	3d	0.94	7	1.91	19	14
15	3e	----	----	2.12	19	14
14 repeat	3d	0.94	7	1.91	19	14

Table 2.1 continued

Tests with constant total spectral energy H_s = 2.83m and T_{p2} = 14s						
16	0d6	2.83	7	----	----	14
17	2b	2.56	7	1.2	14	14
18	2c	2	7	2	14	14
19	2d	1.2	7	2.56	14	14
20	2e	----	----	2.83	14	14
16 repeat	0d6	2.83	7	----	----	14
Tests with constant total spectral energy H_s = 2.83m and T_{p2} = 11s						
21	7b	2.56	7	1.2	11	14
22	7c	2	7	2	11	14
23	7d	1.2	7	2.56	11	14
24	7e	----	----	2.83	11	14
Tests with constant total spectral energy H_s = 2.83m and T_{p2} = 19s						
25	8b	2.56	7	1.2	19	14
26	8c	2	7	2	19	14
27	8d	1.2	7	2.56	19	14
28	8e	----	----	2.83	19	14
Tests with constant total spectral energy H_s = 2.12m and T_{p2} = 11s						
29	9b	1.91	7	0.94	11	14
30	9c	1.5	7	1.5	11	14
31	9d	0.94	7	1.91	11	14
32	9e	----	----	2.12	11	14

Figures

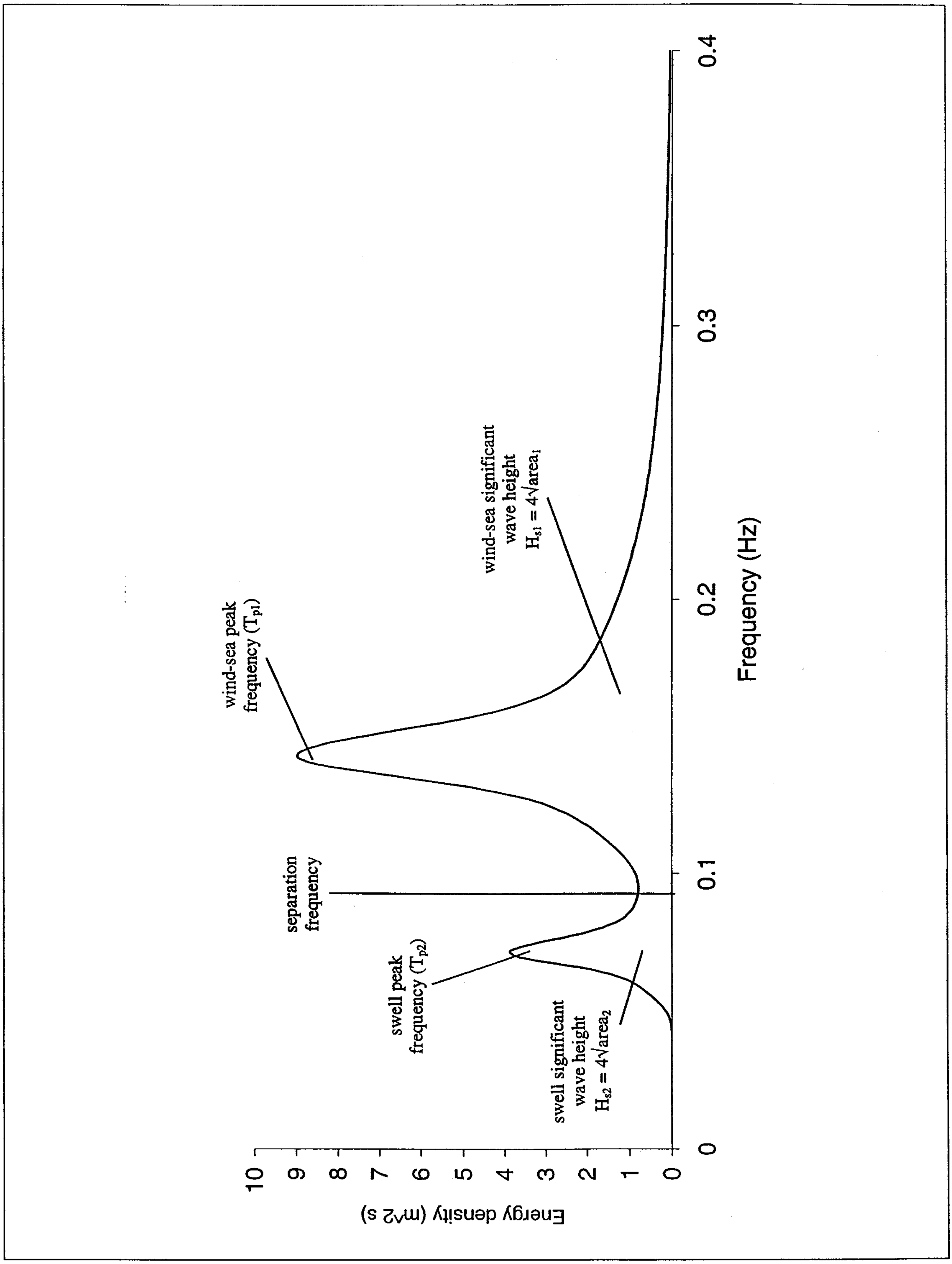


Figure 1.1 Spectral definition of wind-sea and swell

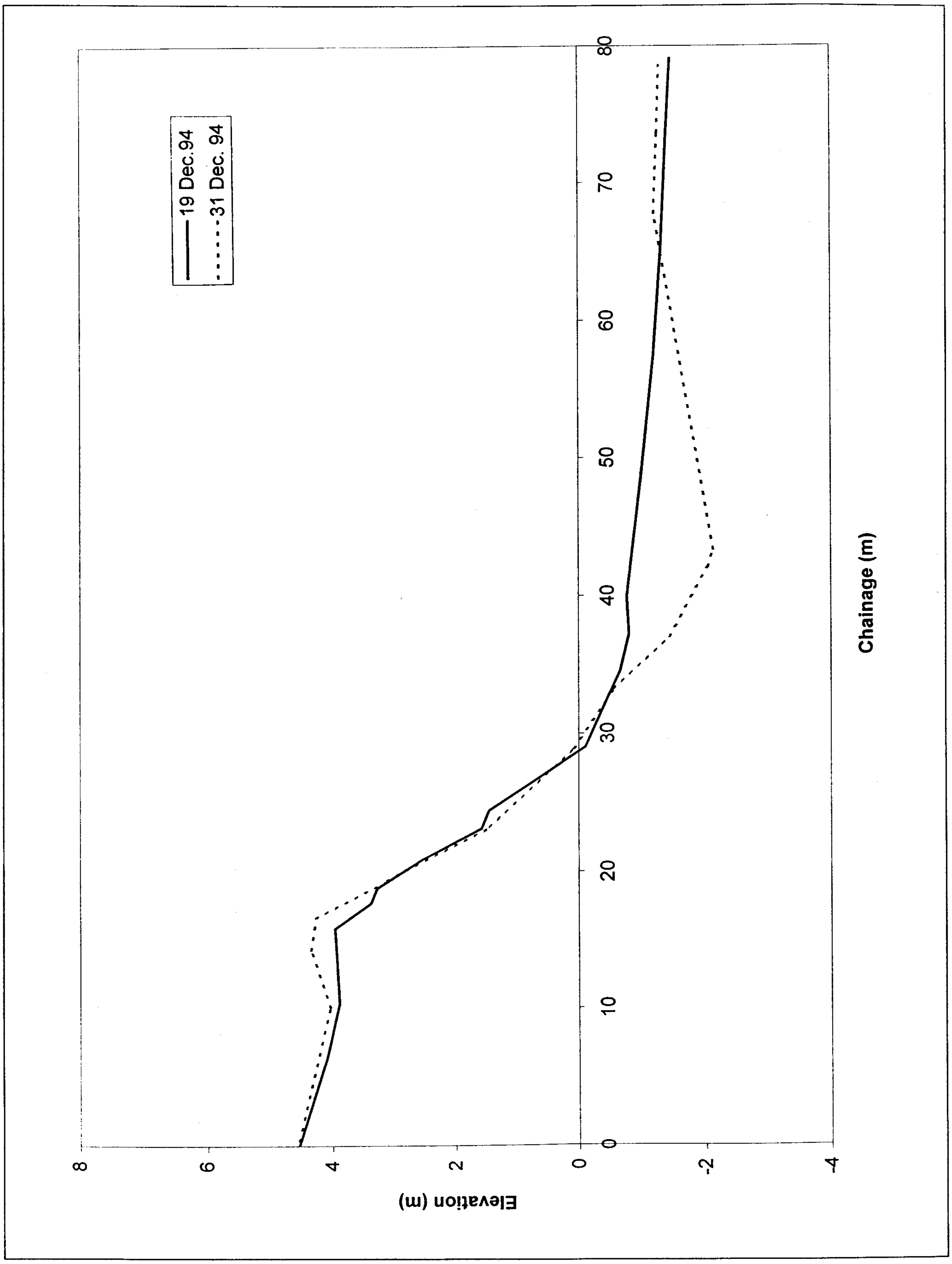


Figure 2.1 Beach profiles – Highcliffe, December 1994

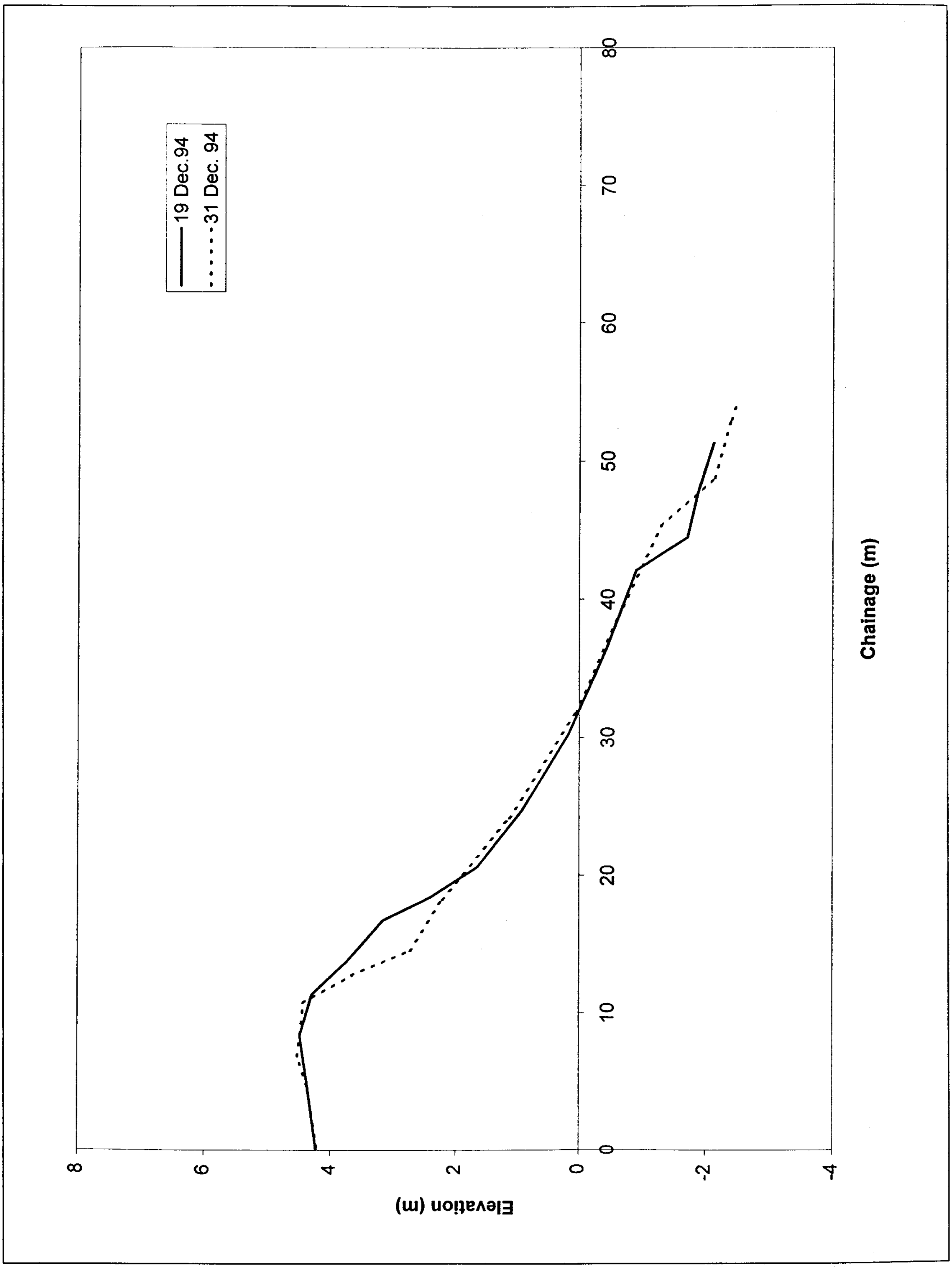
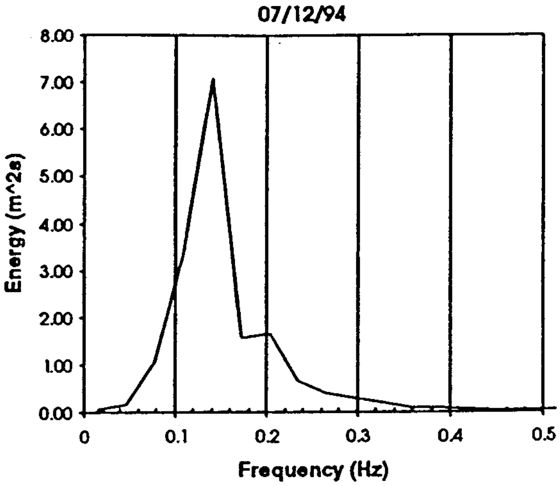
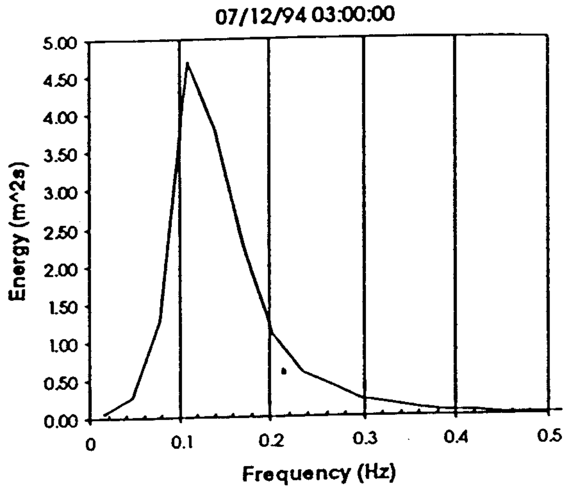


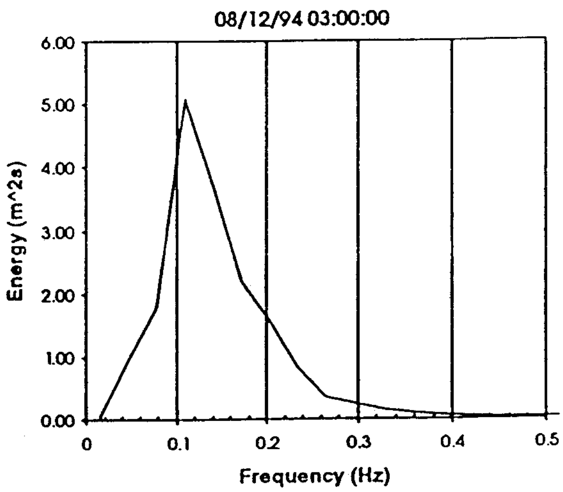
Figure 2.2 Beach profiles – Hordle, December 1994



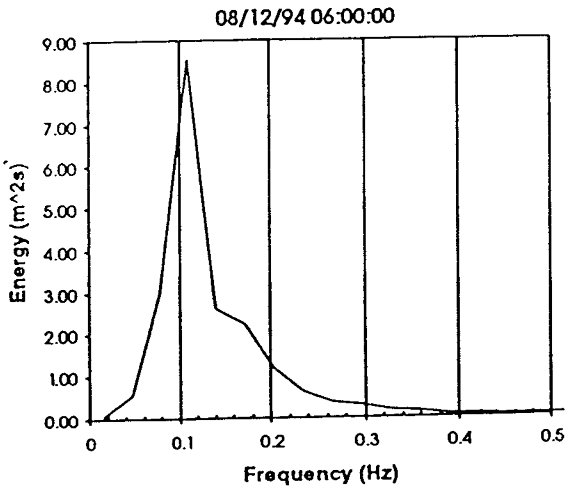
$H_{mo\ (total)}$ = 2.91m



$H_{mo\ (total)}$ = 2.75m
High water =1.48mOD at 02.22hrs
$H_{mo\ (swell)}$ =0.69m
$H_{mo\ (wind)}$ =2.66m
Swell energy ≈ 6% total energy

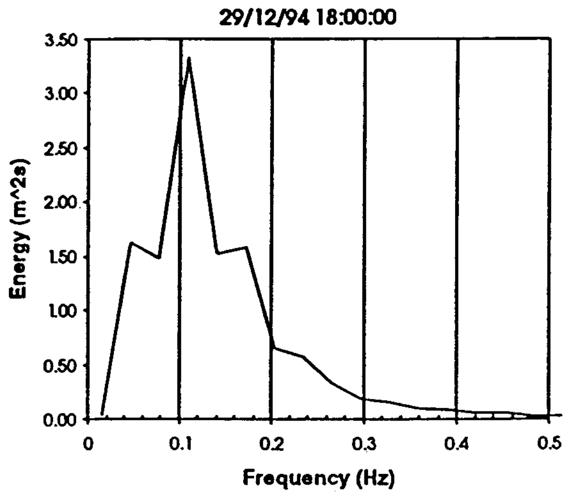


$H_{mo\ (total)}$ = 2.93m

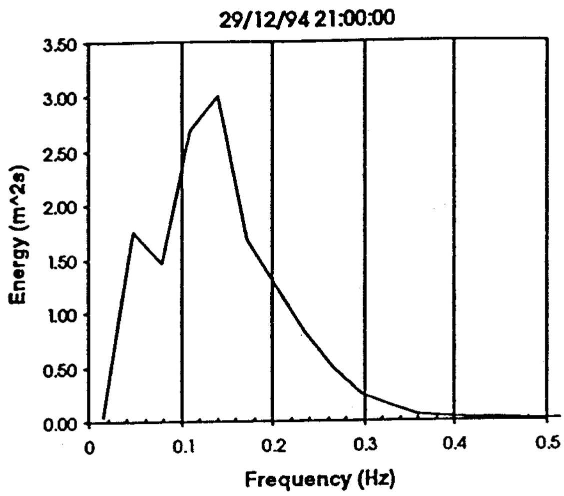


$H_{mo\ (total)}$ = 3.17m
High water = 1.31mOD at 04.22 hrs
$H_{mo\ (swell)}$ =1.03m
$H_{mo\ (wind)}$ =3.00m
Swell energy ≈ 11% total energy

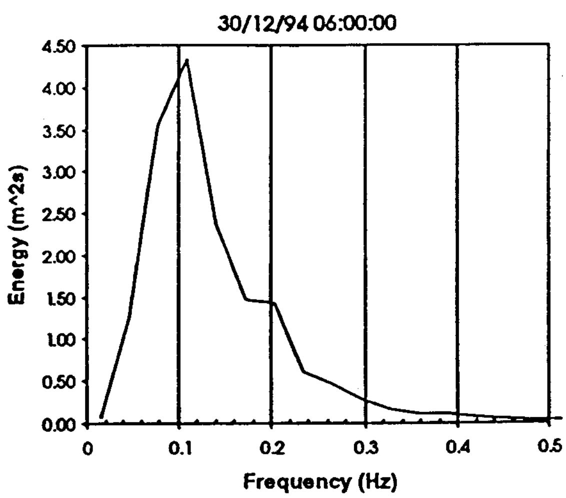
Figure 2.3 Wave spectra at peak water levels during storm event of 7-8 December 1994



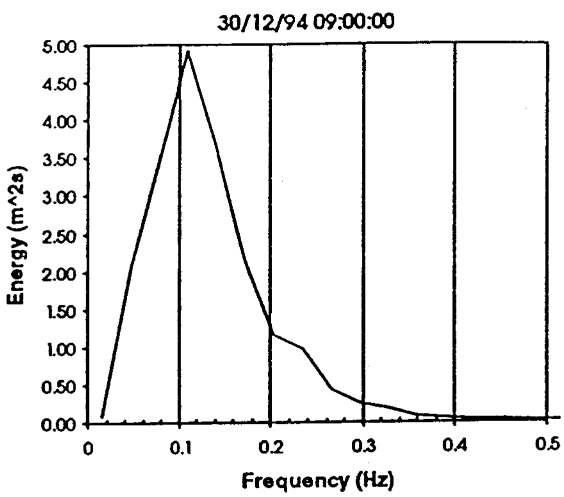
$H_{mo\ (total)}$ = 2.44m



$H_{mo\ (total)}$ = 2.63m
High water = 1.21mOD at 19.52 hrs
$H_{mo\ (swell)}$ =1.13m
$H_{mo\ (wind)}$ =2.37m
Swell energy ≈ 20% total energy



$H_{mo\ (total)}$ = 2.88m



$H_{mo\ (total)}$ = 3.14m
High water = 1.35mOD at 07.22 hrs
$H_{mo\ (swell)}$ =1.42m
$H_{mo\ (wind)}$ =2.80m
Swell energy ≈ 20% total energy

Figure 2.4 Wave spectra at peak water levels during storm event of 29-30 December 1994

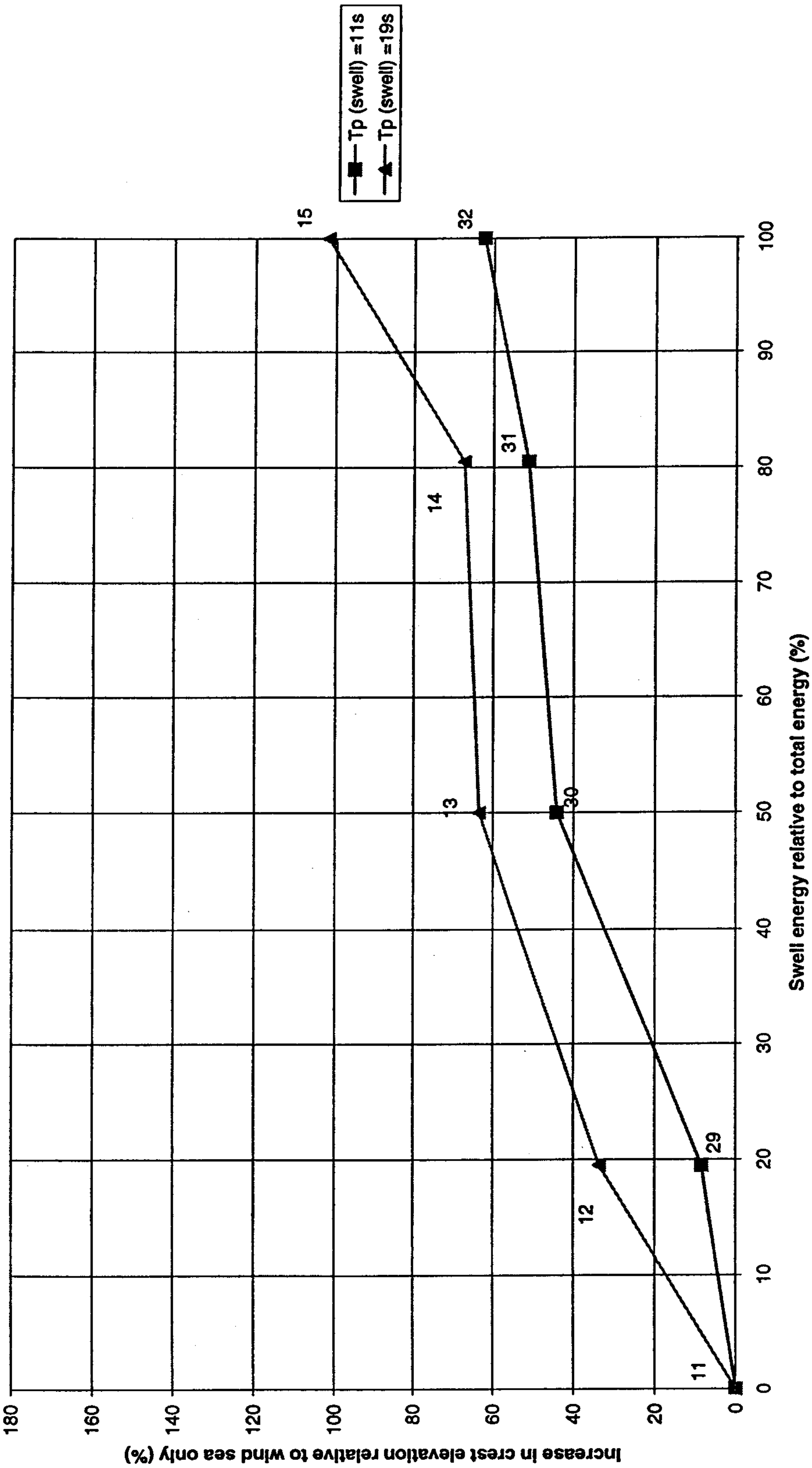


Figure 2.5 Swell influence on crest elevation: Hs = 2.12m

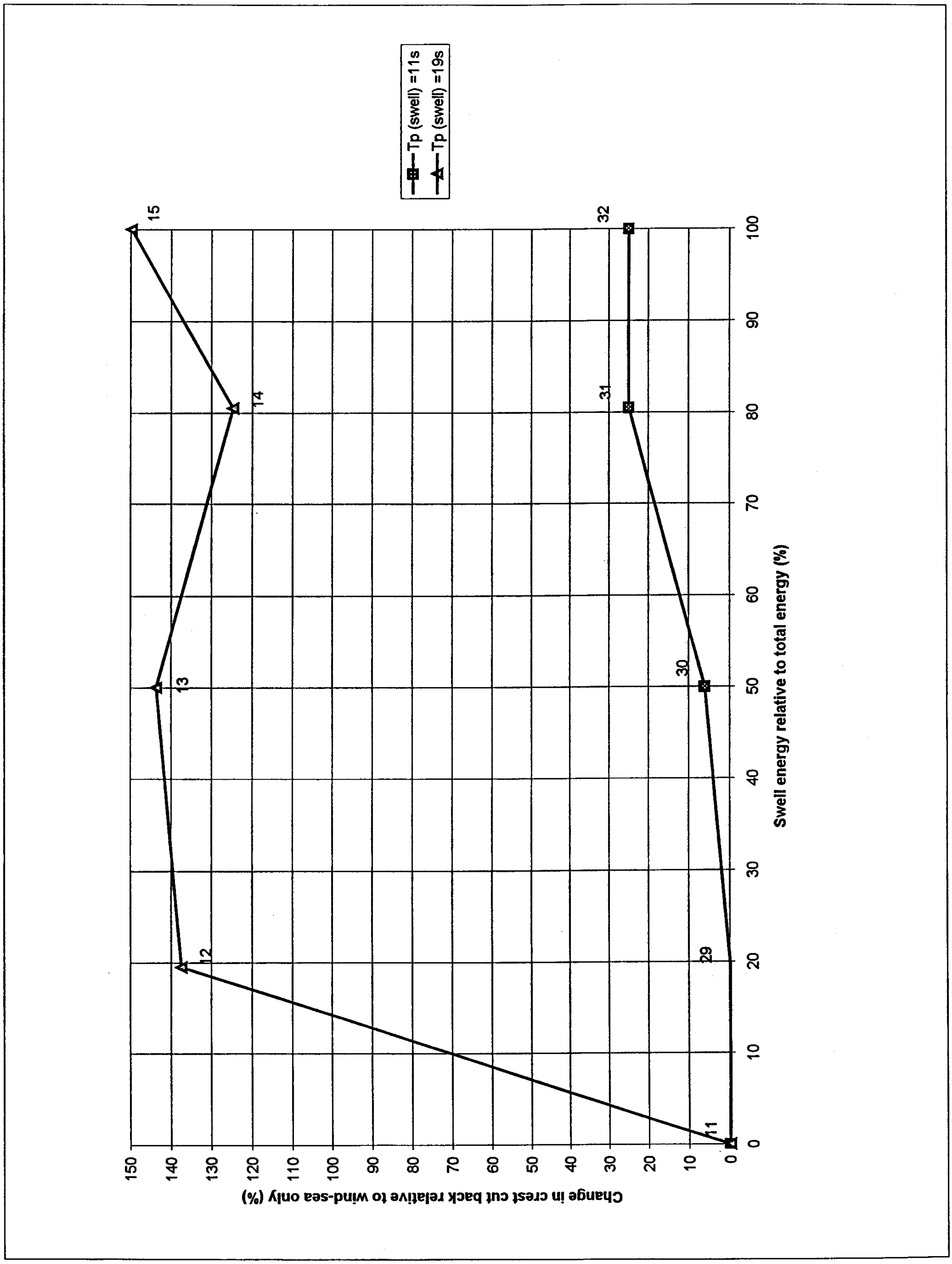


Figure 2.6 Swell influence on crest cut back: Hs = 2.12m

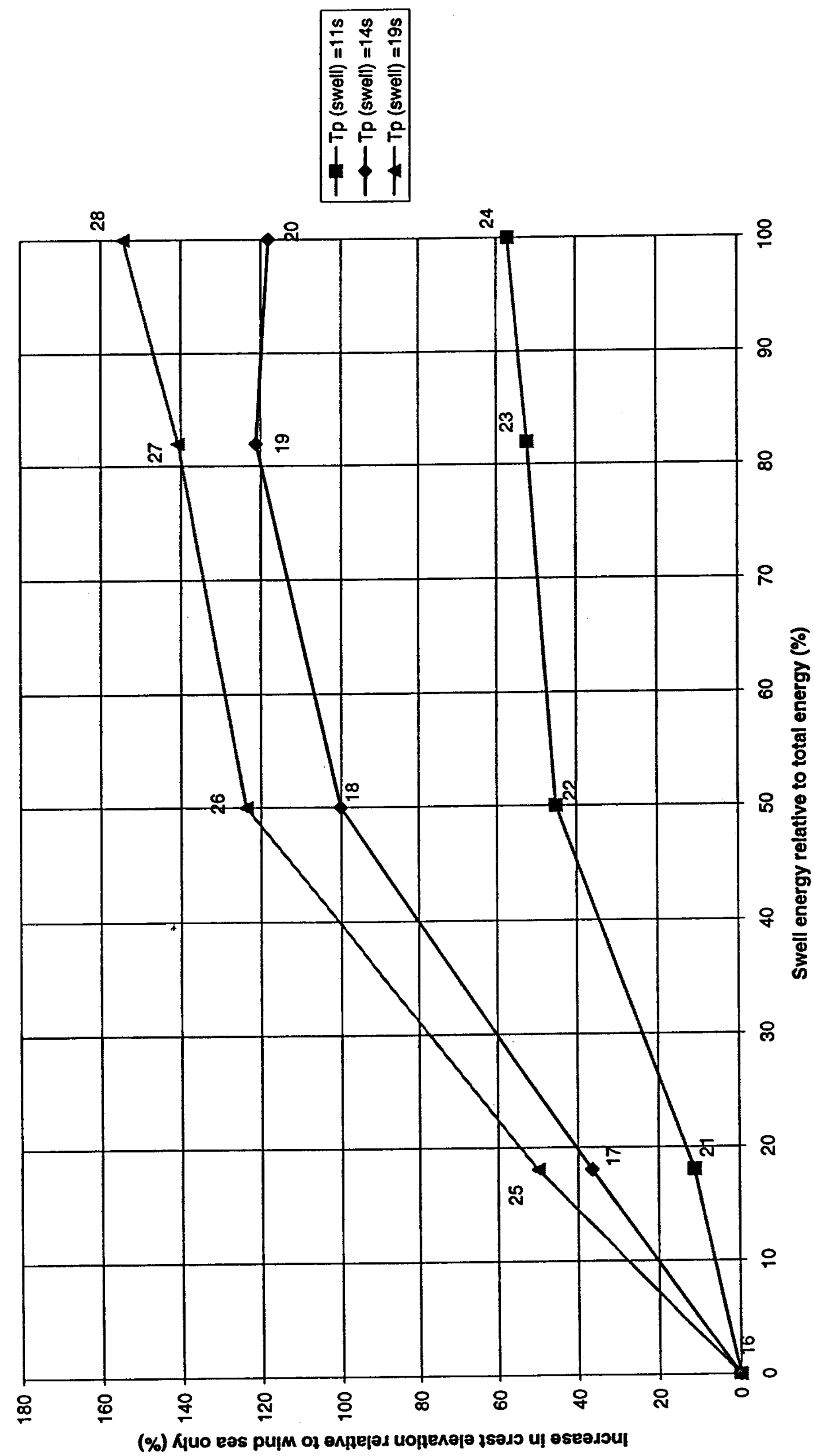


Figure 2.7 Swell influence on crest elevation: Hs = 2.83m

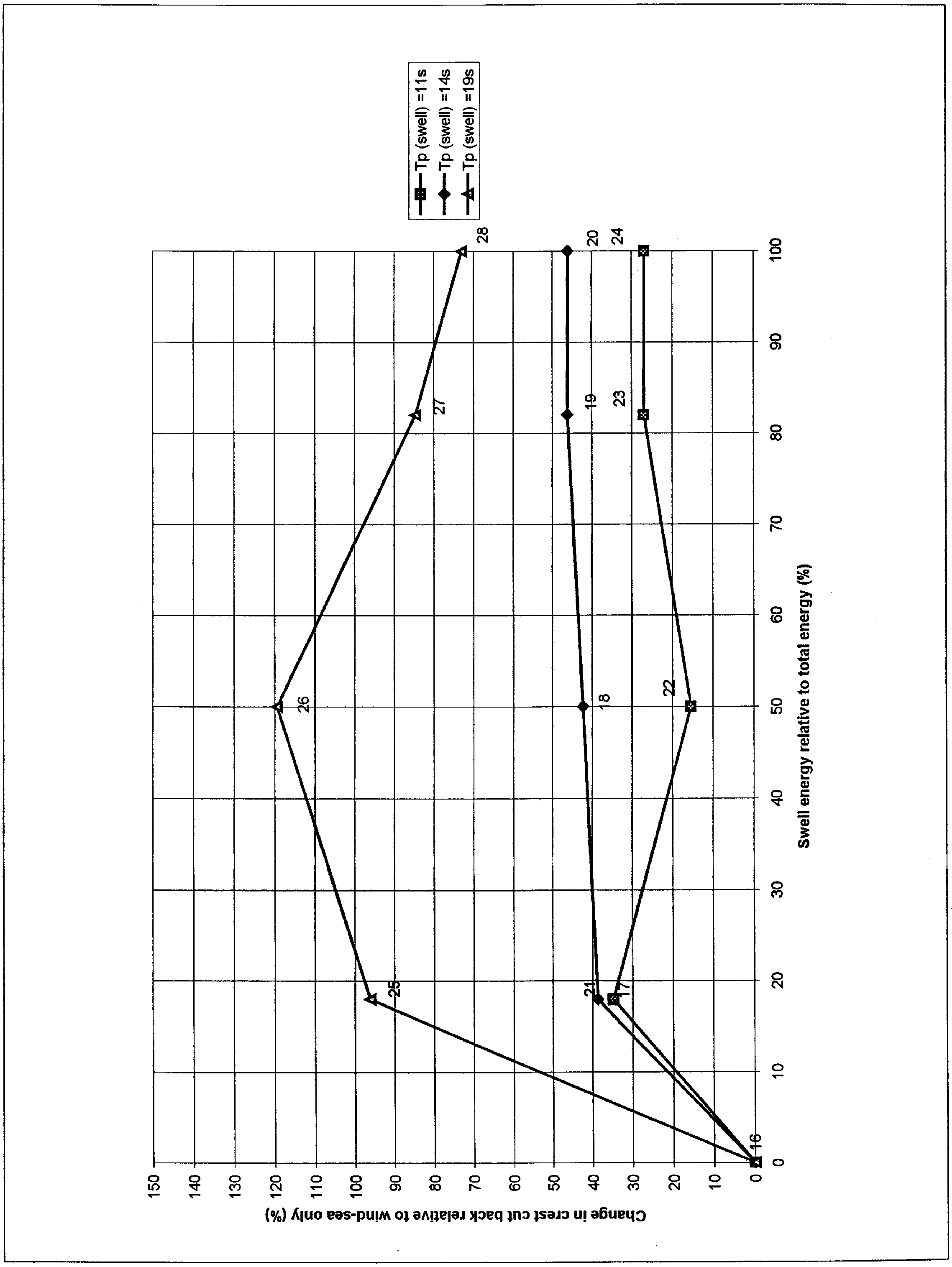


Figure 2.8 Swell influence on crest cut back: Hs = 2.83m

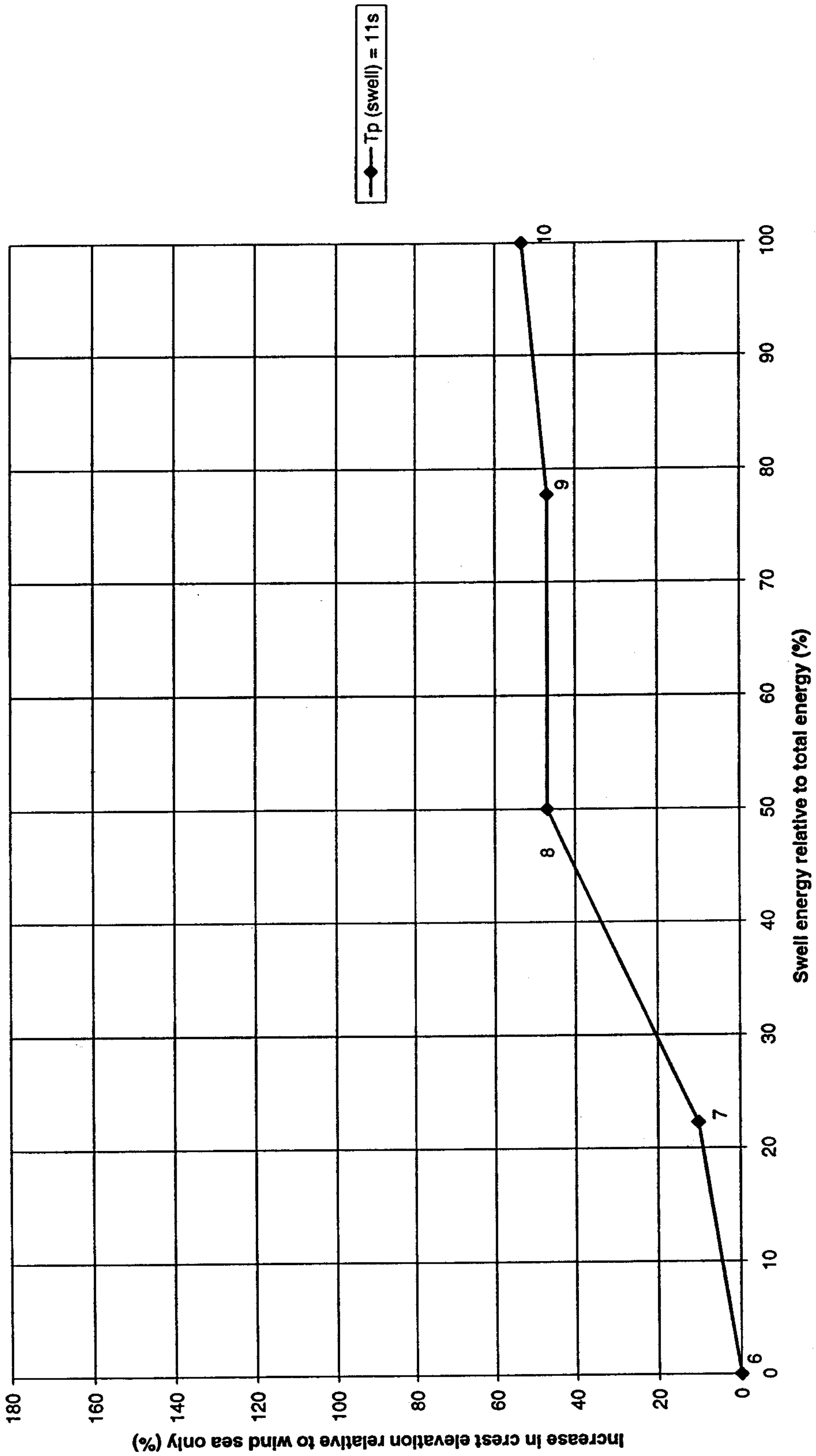


Figure 2.9 Swell influence on crest elevation: Hs = 3.53m

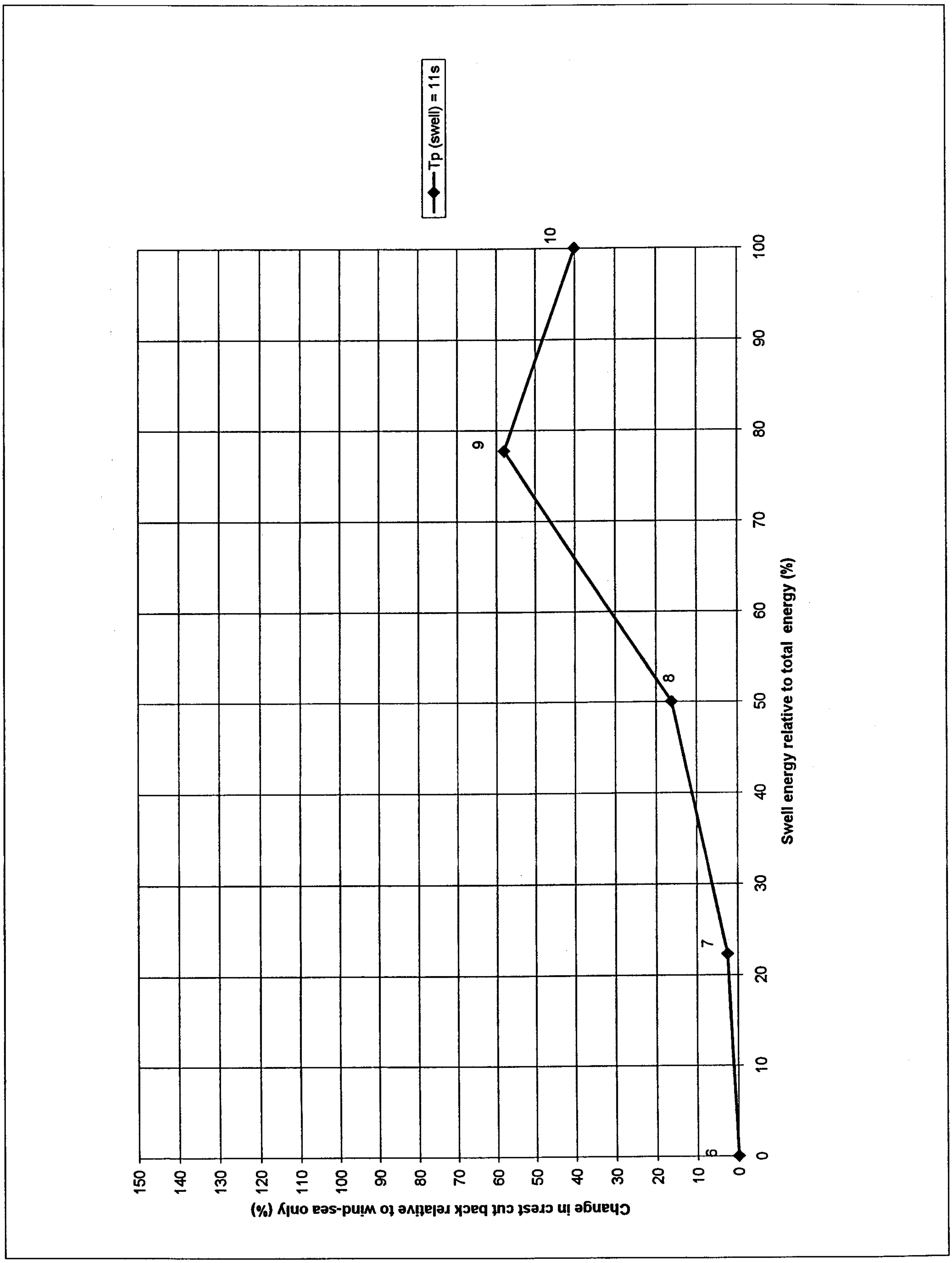


Figure 2.10 Swell influence on crest cut back: Hs = 3.53m

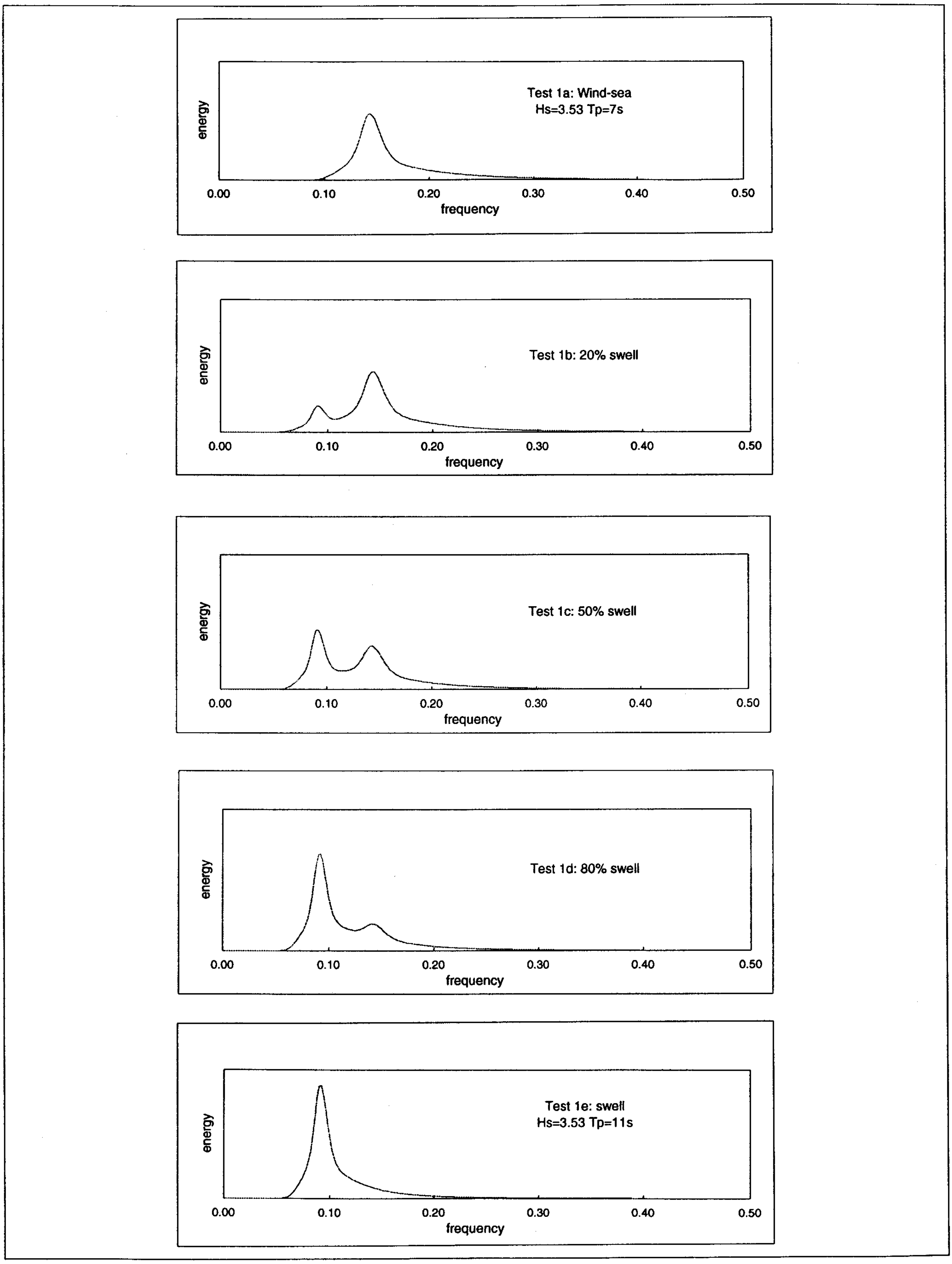


Figure 3.1 Spectra for Test Series 1

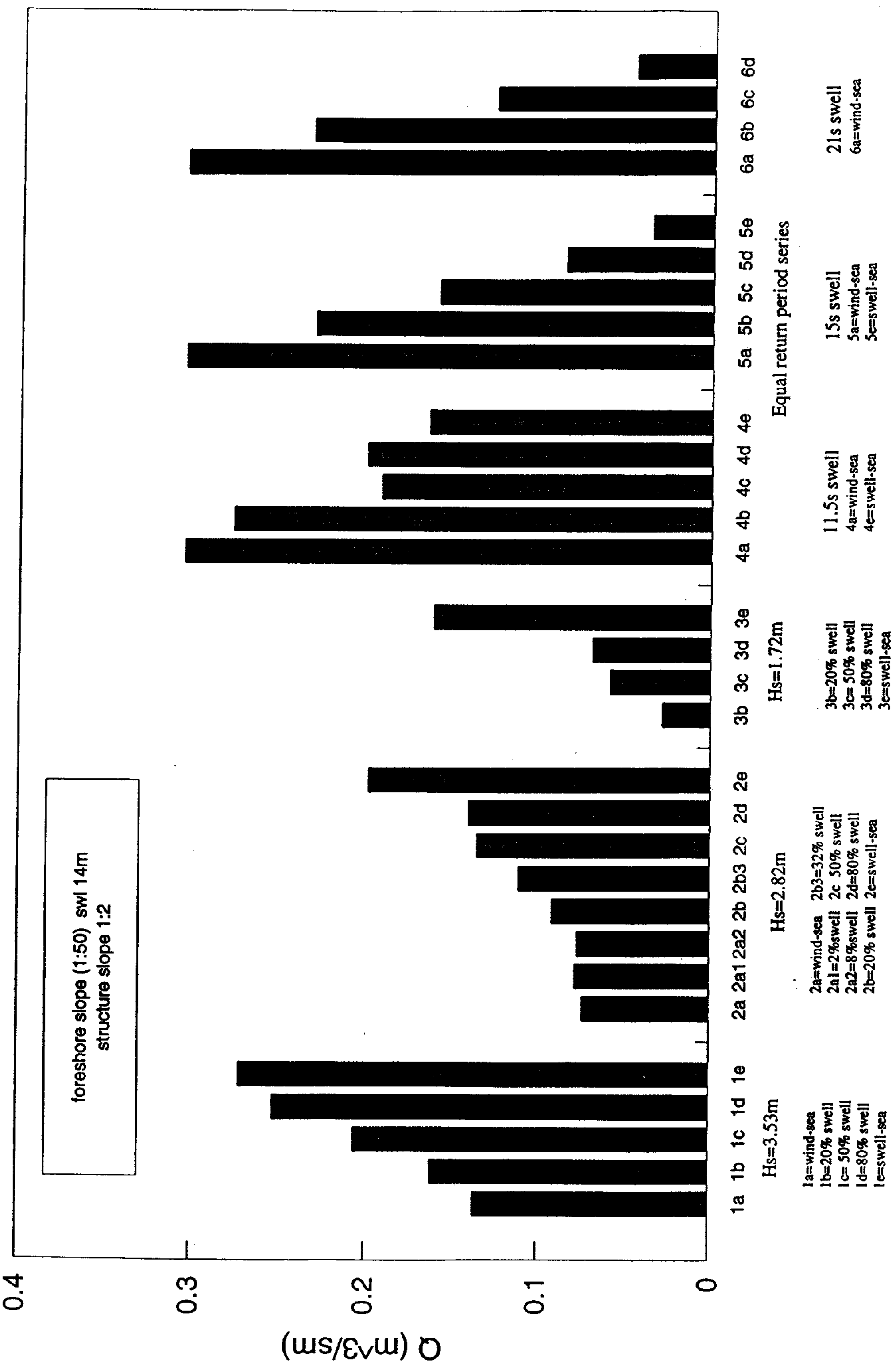


Figure 3.2 Measured mean overtopping rate – set 1

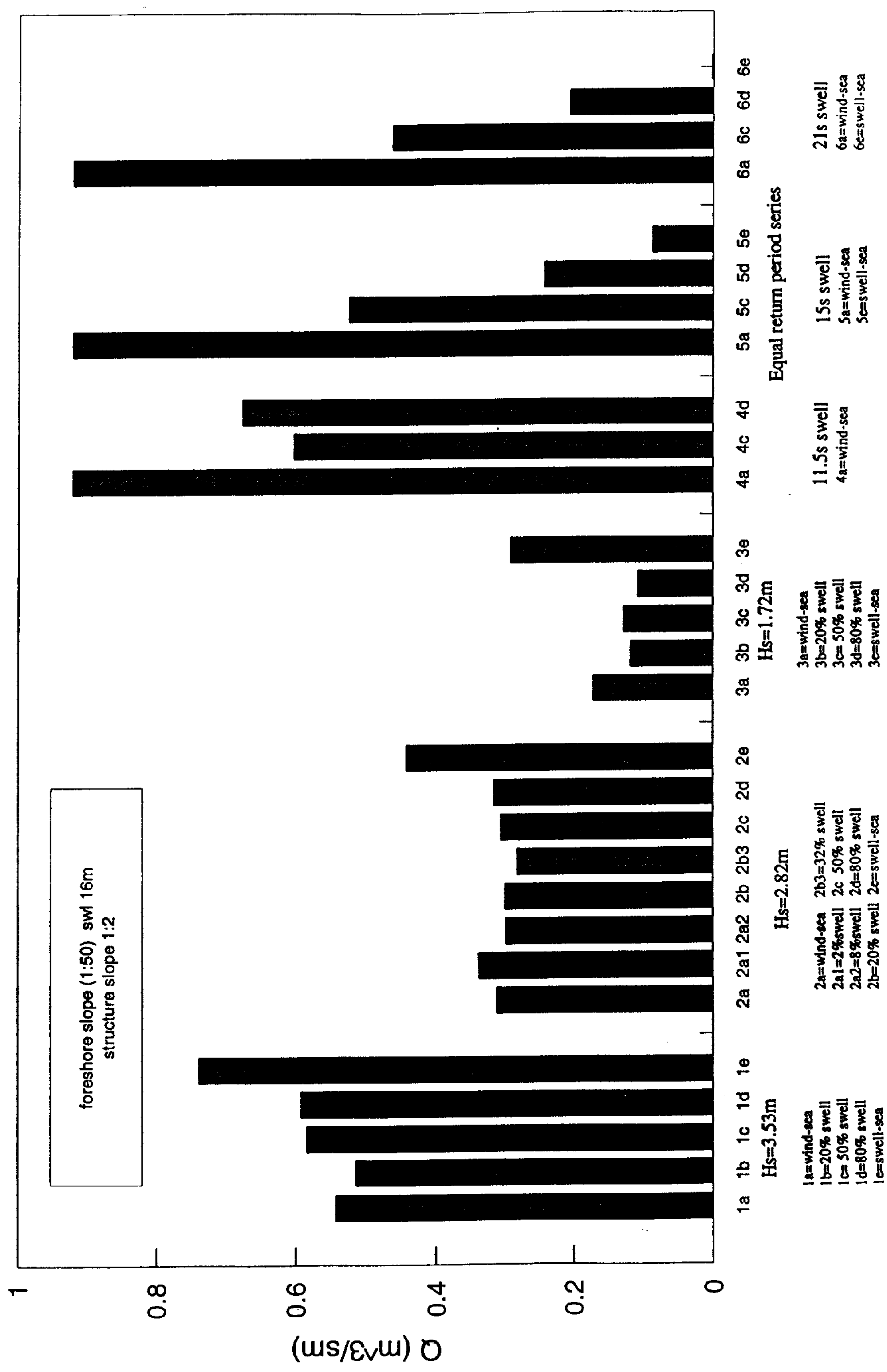


Figure 3.3 Measured mean overtopping rate – set 2

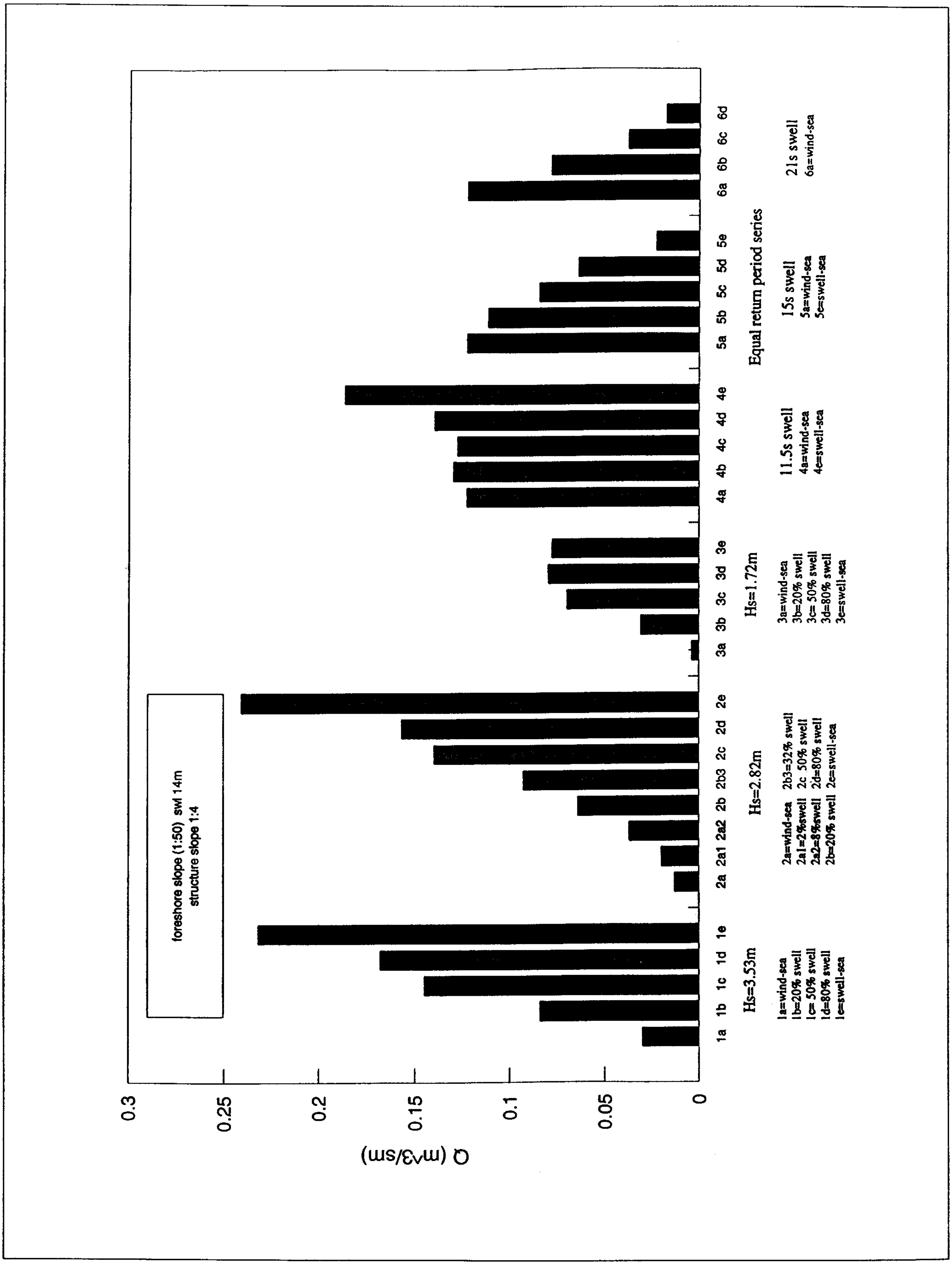


Figure 3.4 Measured mean overtopping rate – set 3

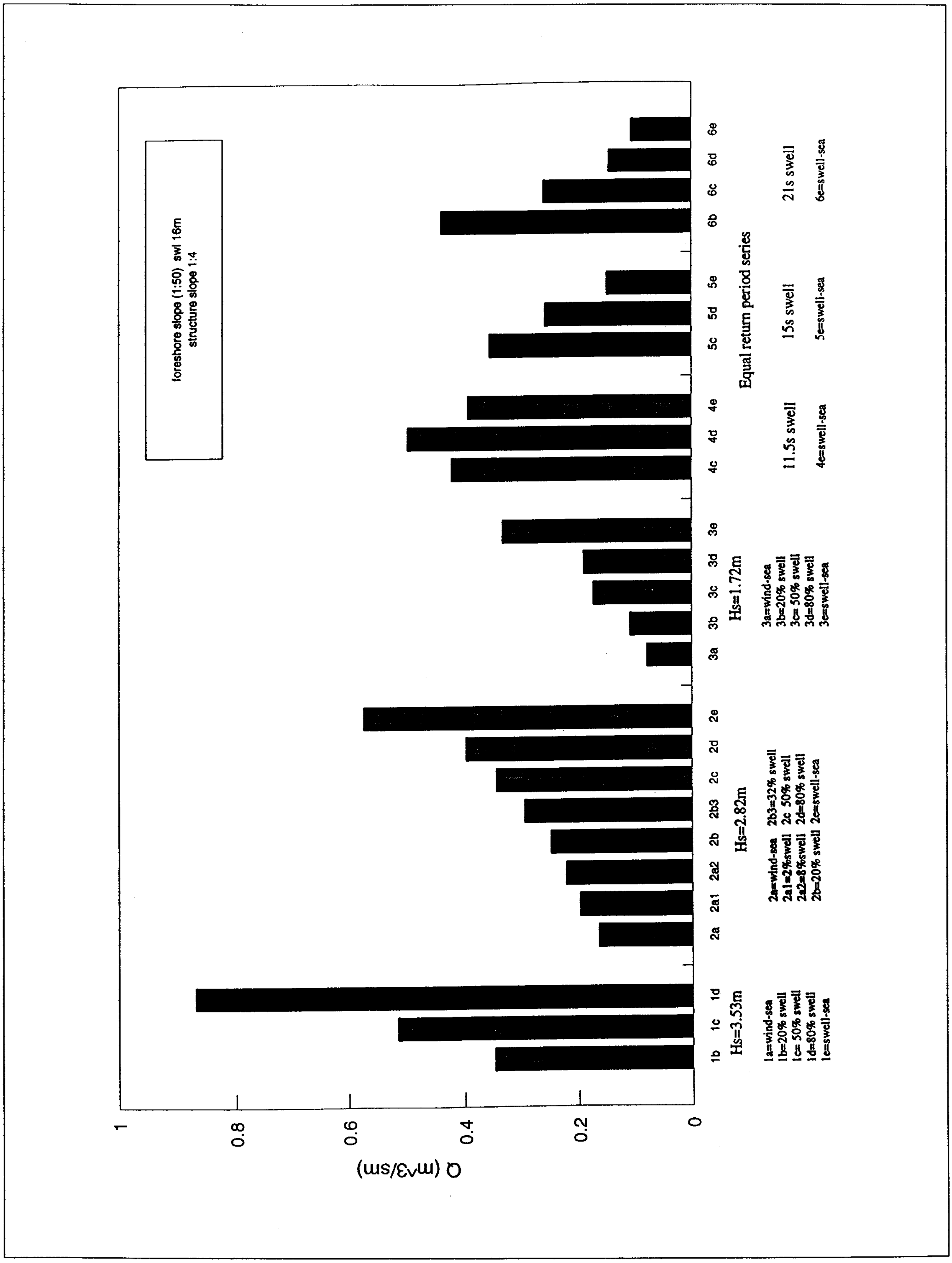


Figure 3.5 Measured mean overtopping rate – set 4

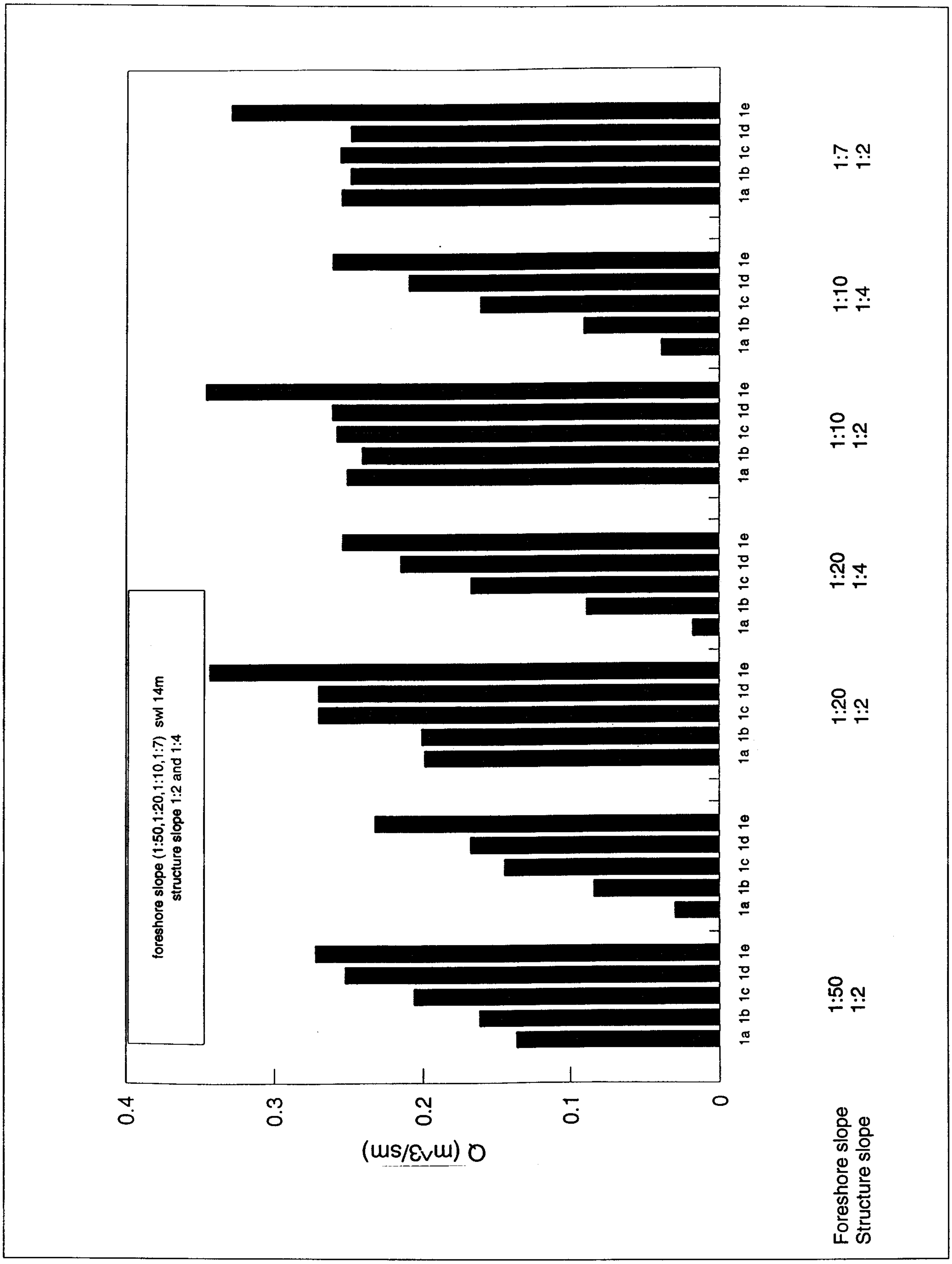


Figure 3.6 Measured mean overtopping rate – set 5

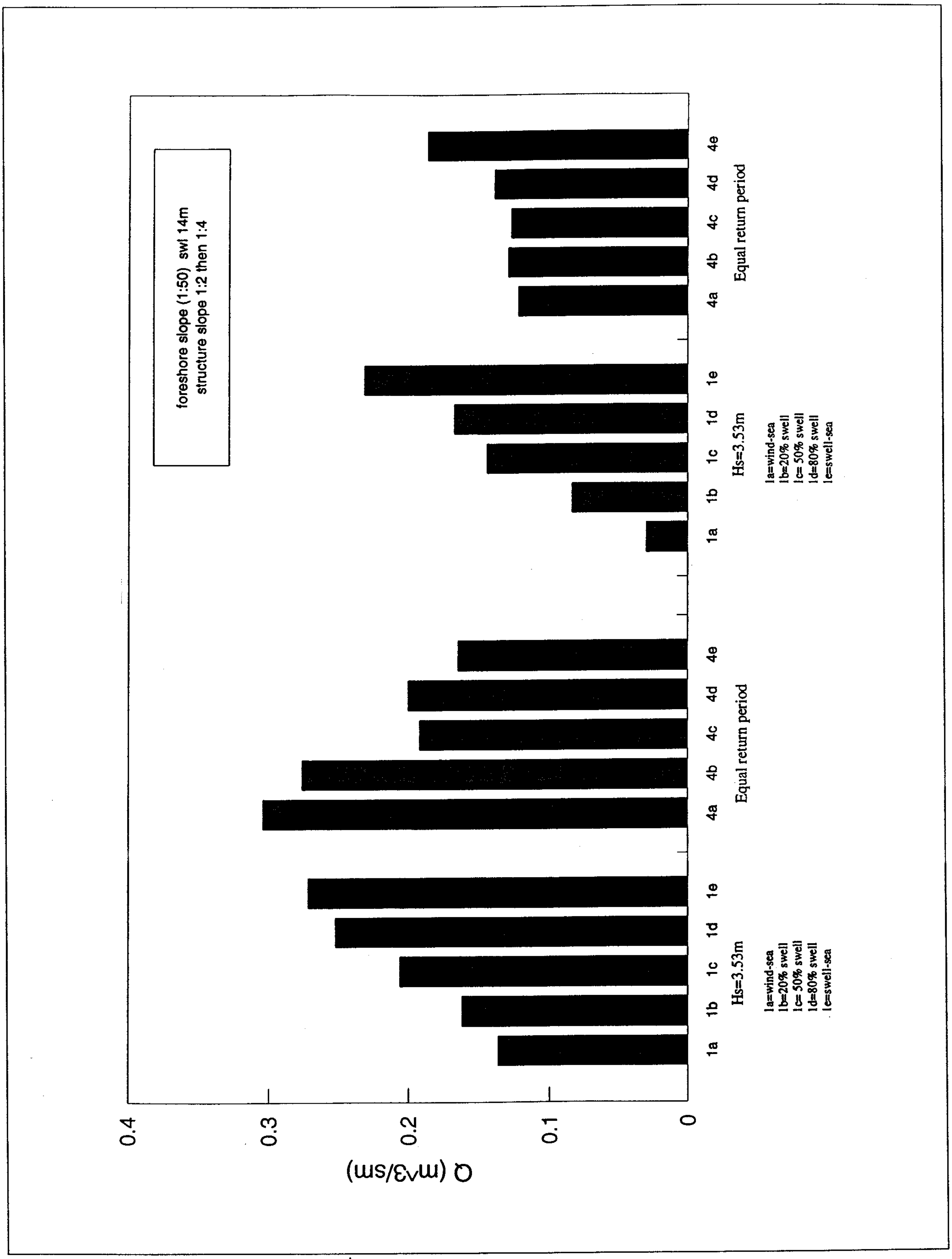


Figure 3.7 Measured mean overtopping rate – test series 1 and 4, water level 14m, two structure slopes

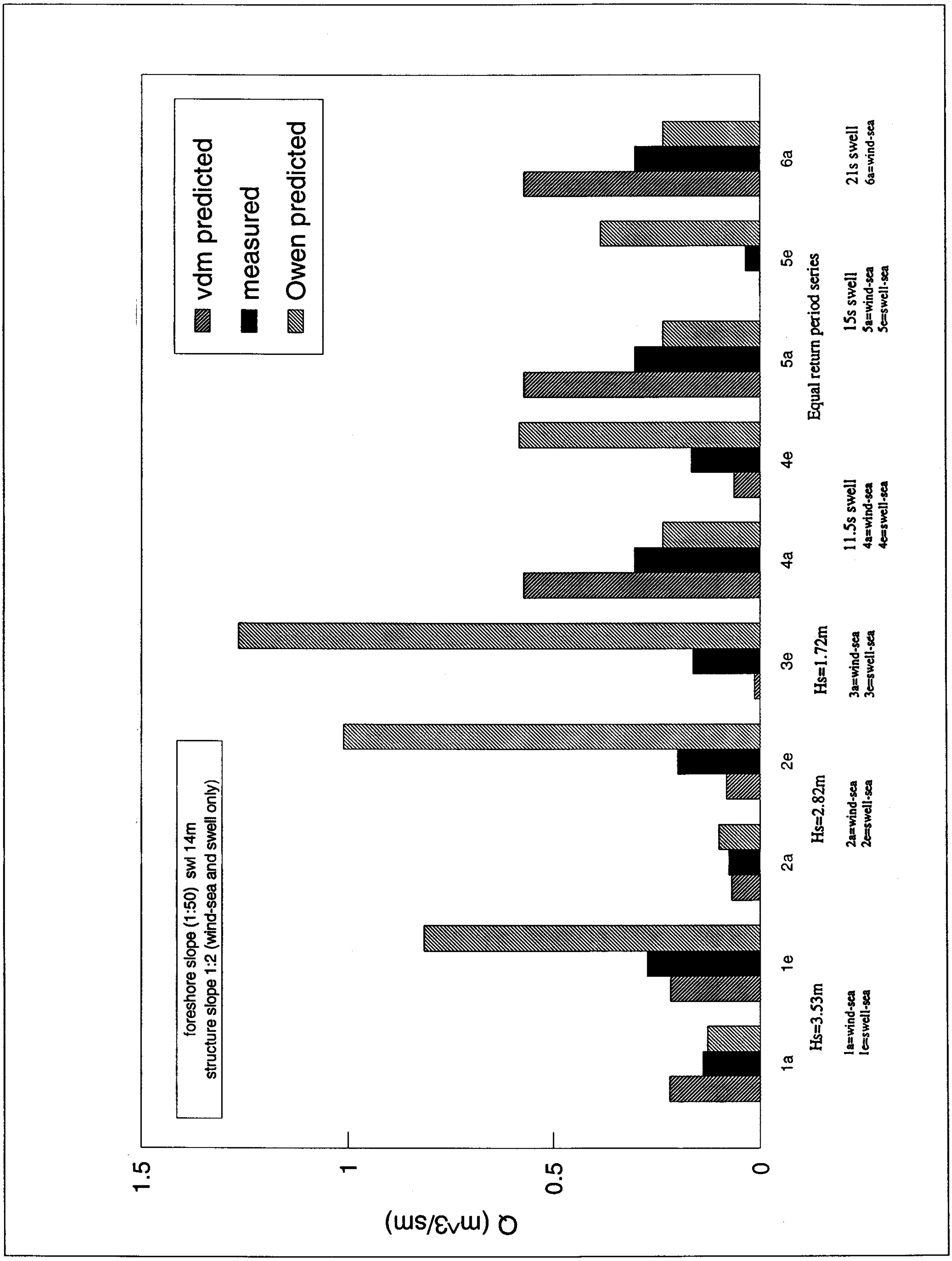


Figure 3.8 Comparison between measurements, Owen and van der Meer – set 1

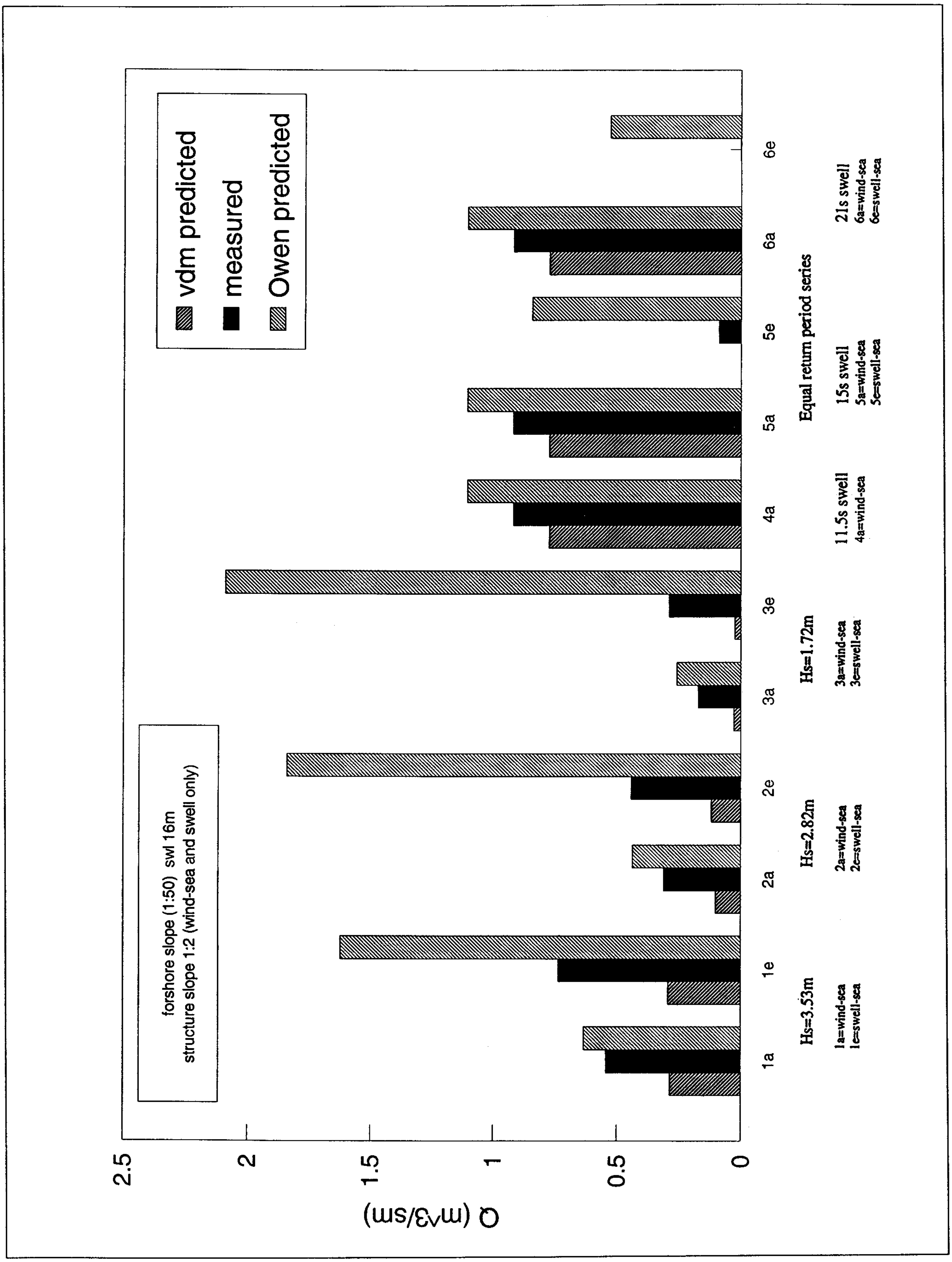


Figure 3.9 Comparison between measurements, Owen and van der Meer – set 2

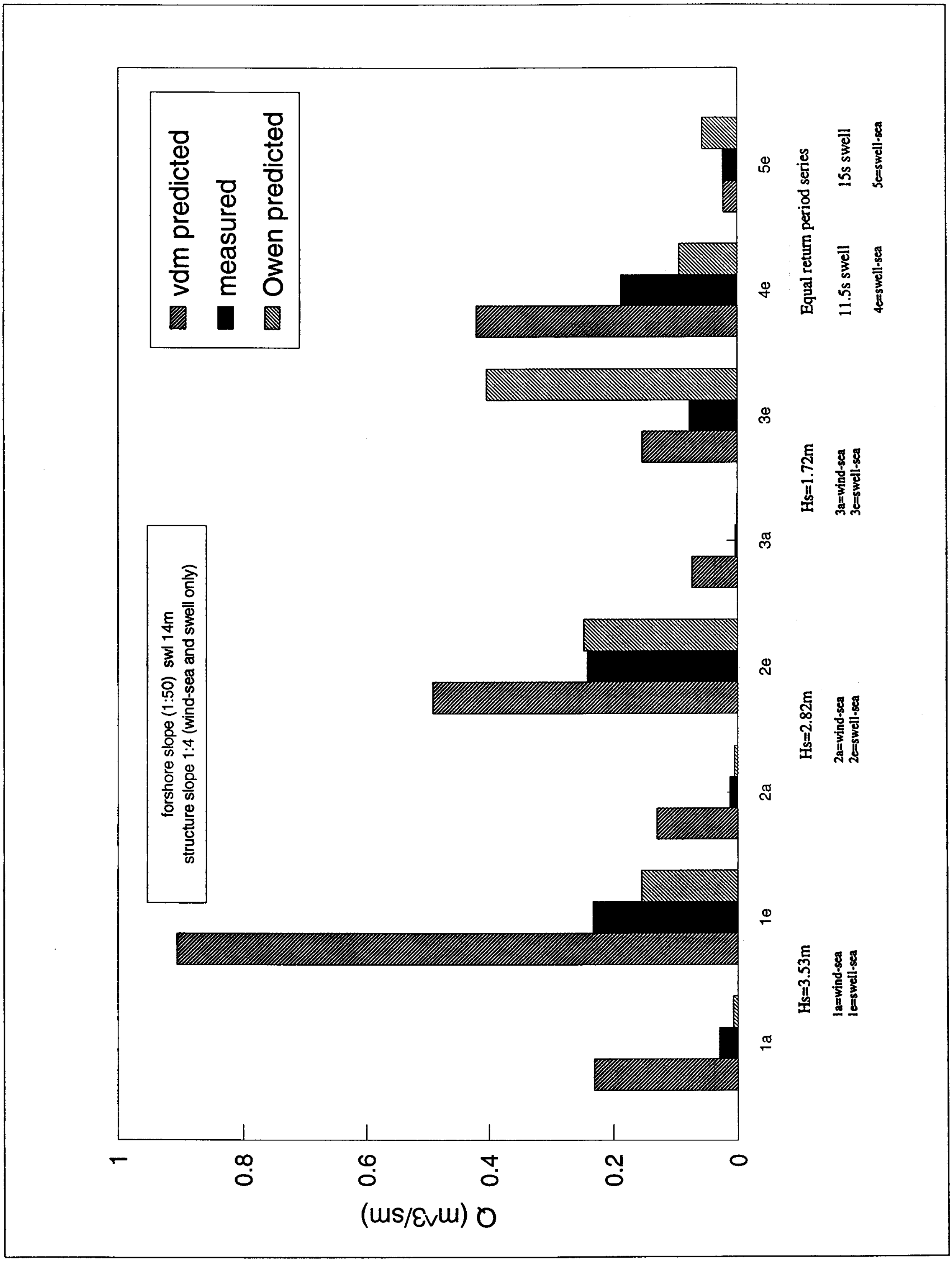


Figure 3.10 Comparison between measurements, Owen and van der Meer – set 3

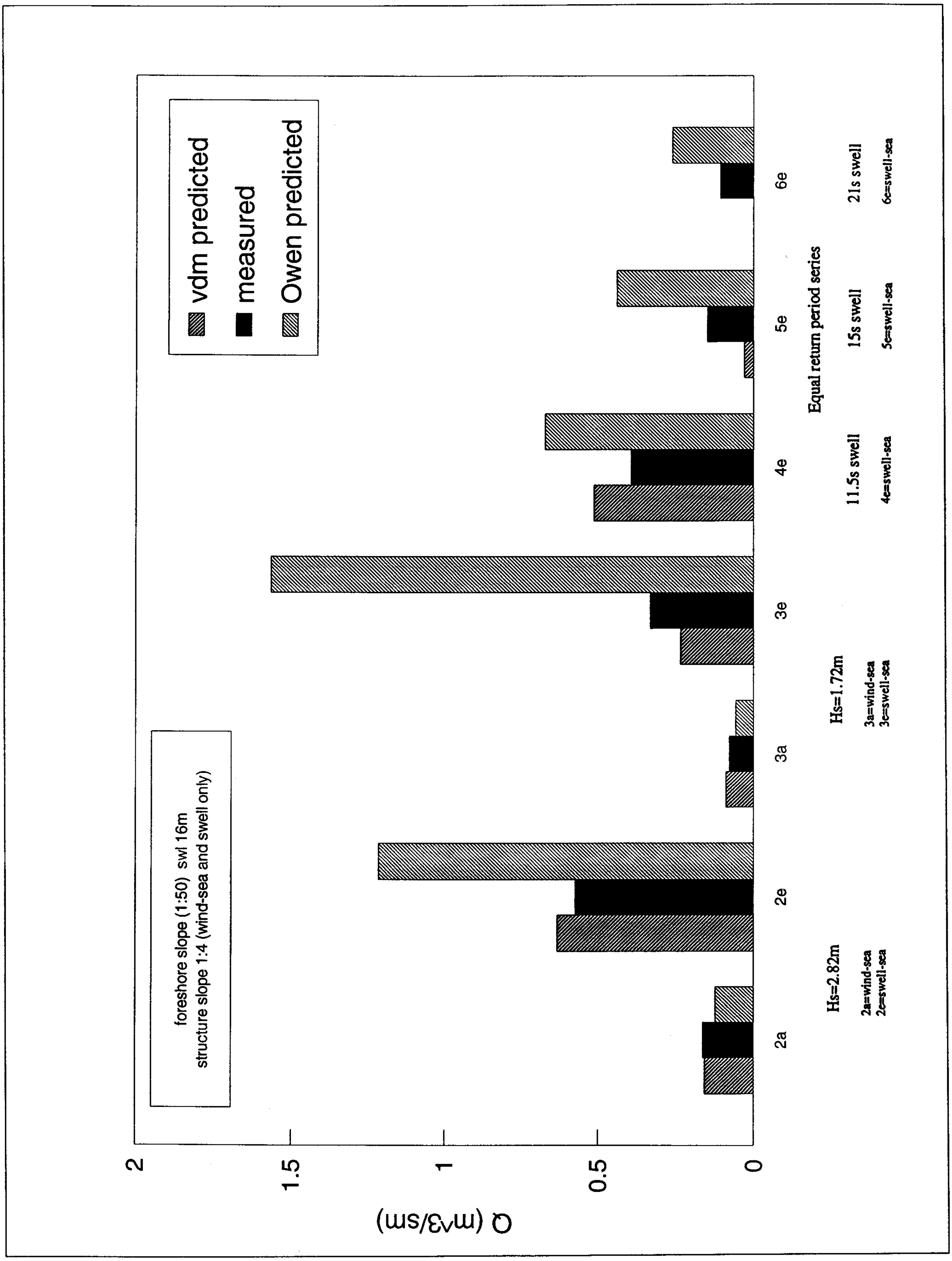


Figure 3.11 Comparison between measurements, Owen and van der Meer – set 4

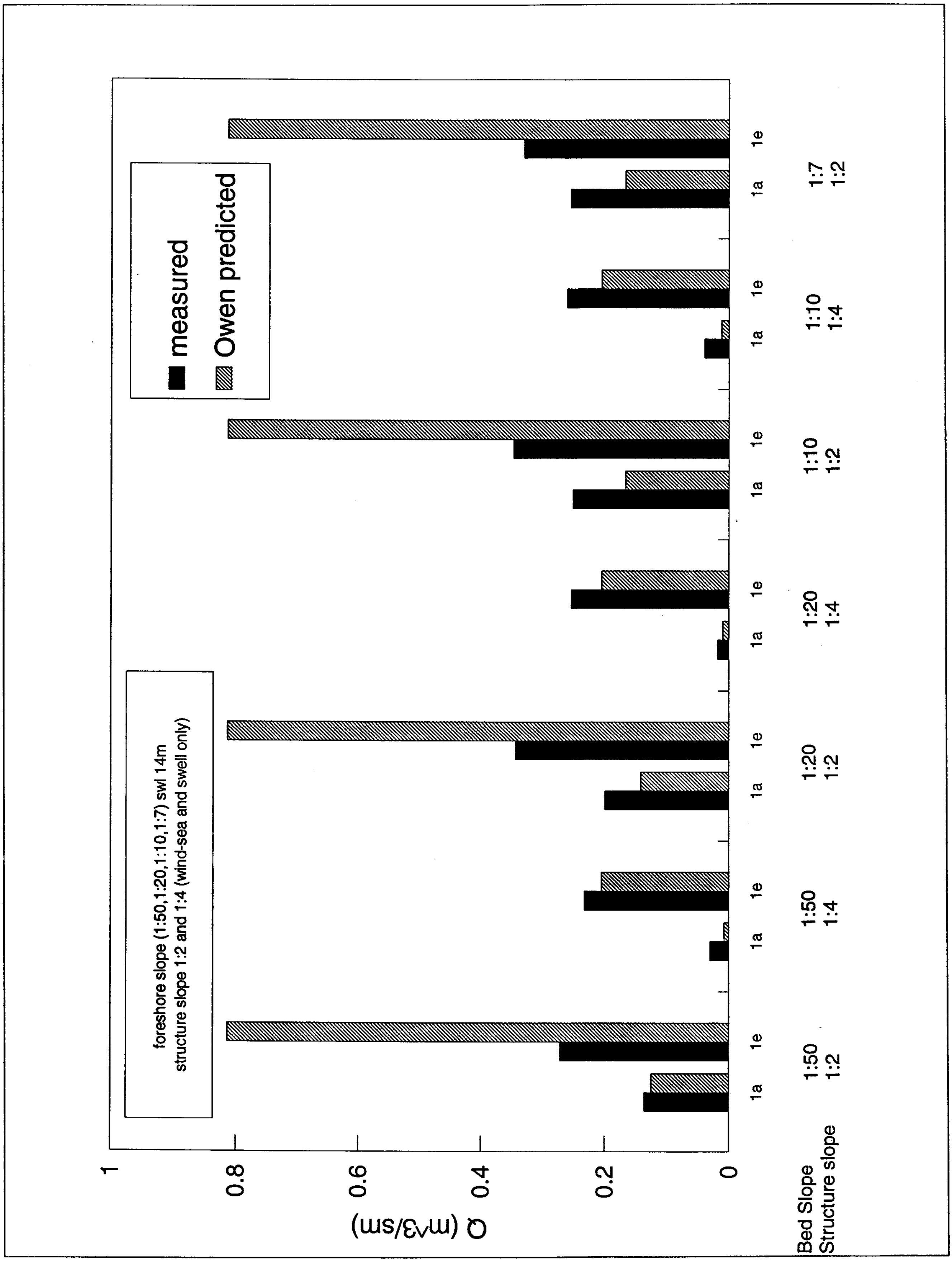


Figure 3.12 Comparison between measurements and Owen – set 5

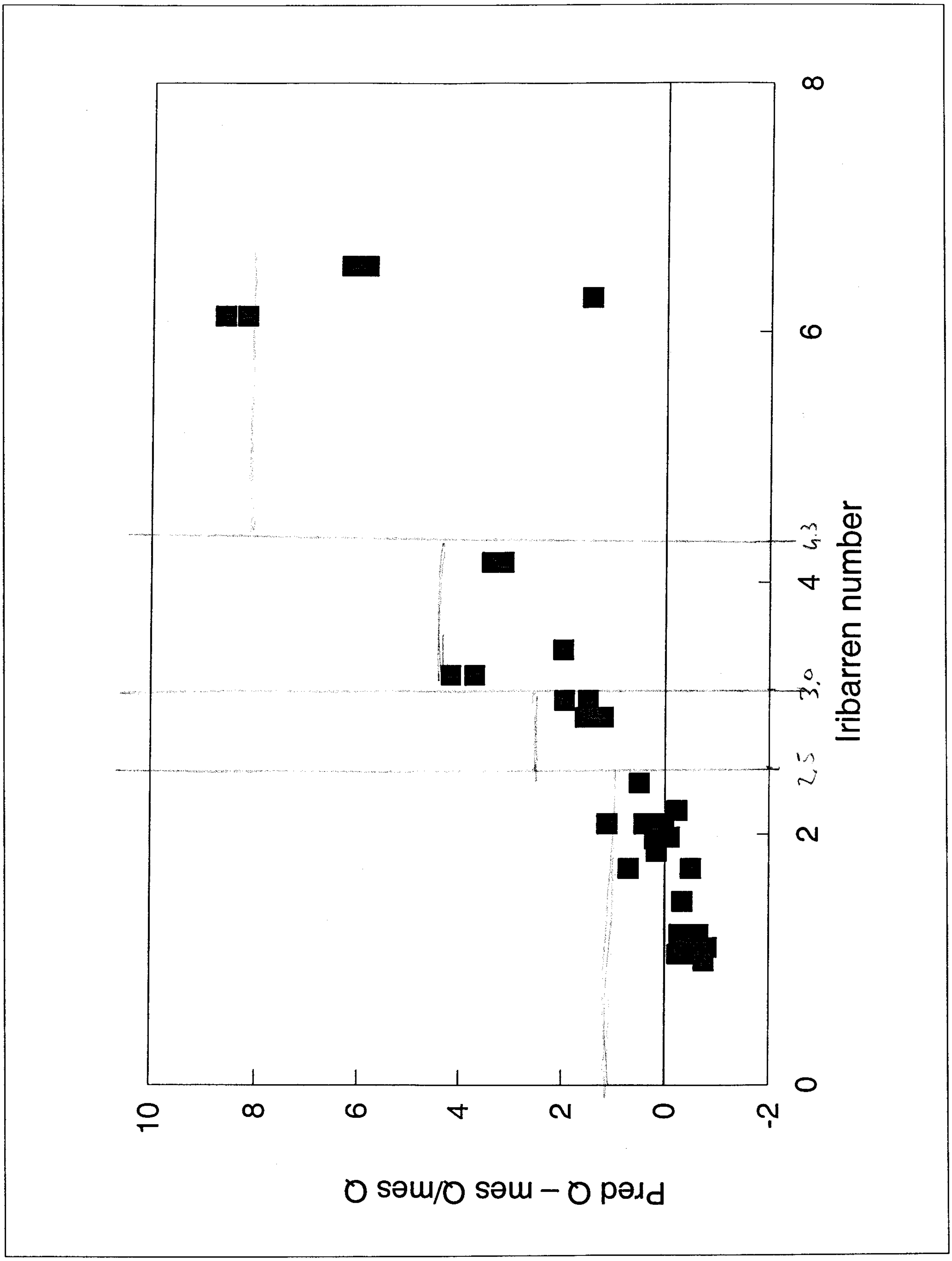


Figure 3.13 Discrepancy in Owen formula as a function of Iribarren Number (1:50 foreshore slope; wind-sea and swell only)

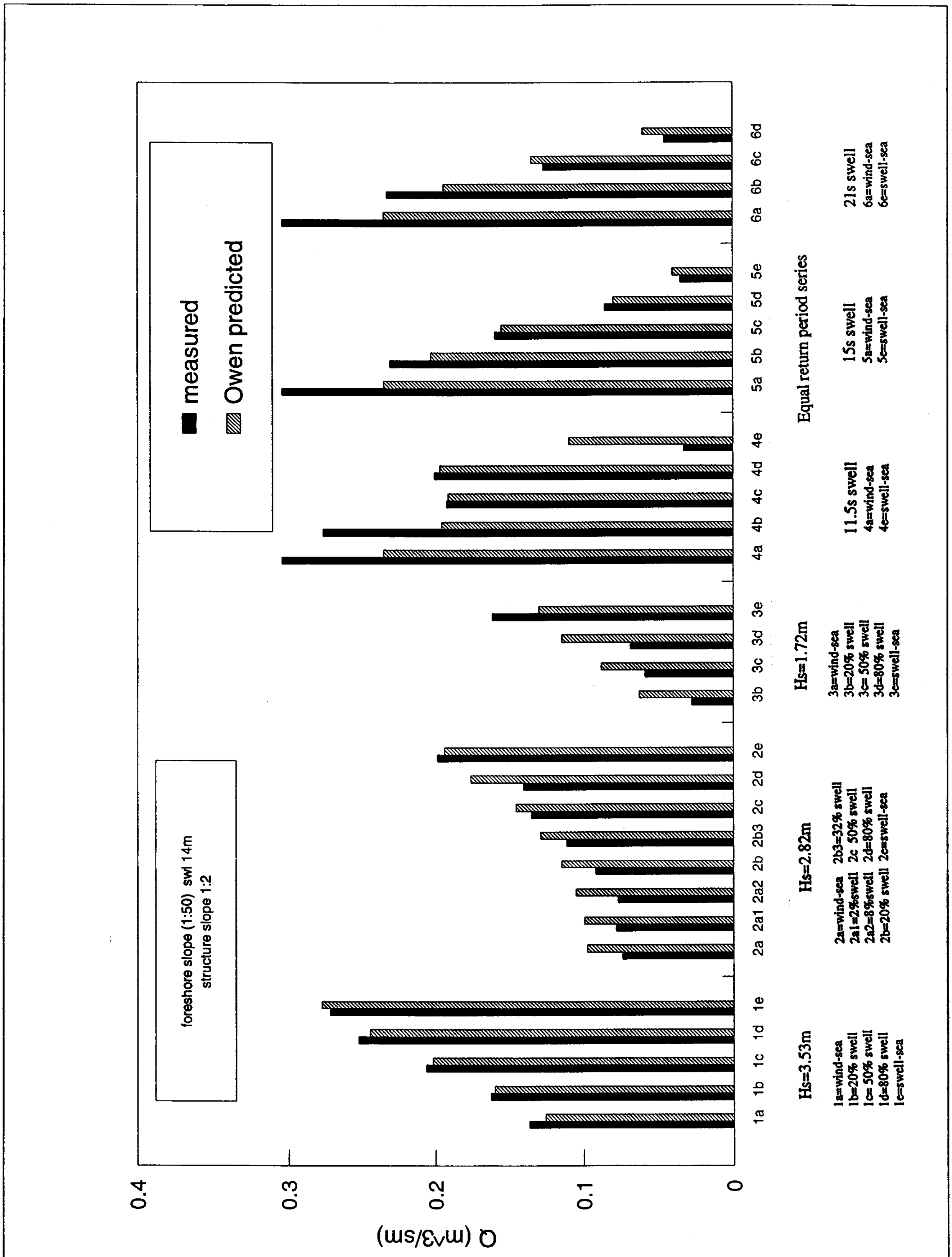


Figure 3.14 Comparison between measurements and modified Owen – set 1

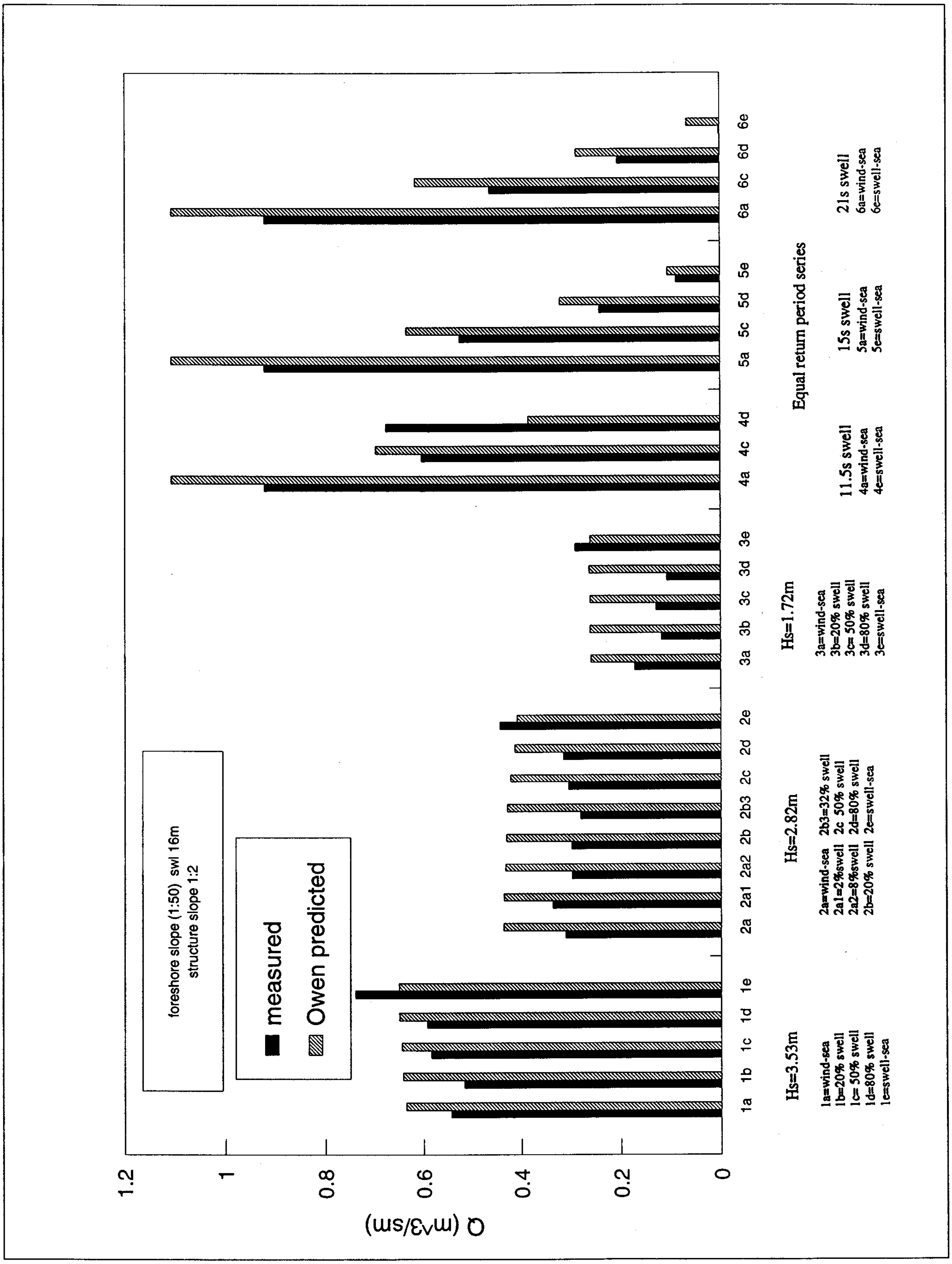


Figure 3.15 Comparison between measurements and modified Owen – set 2

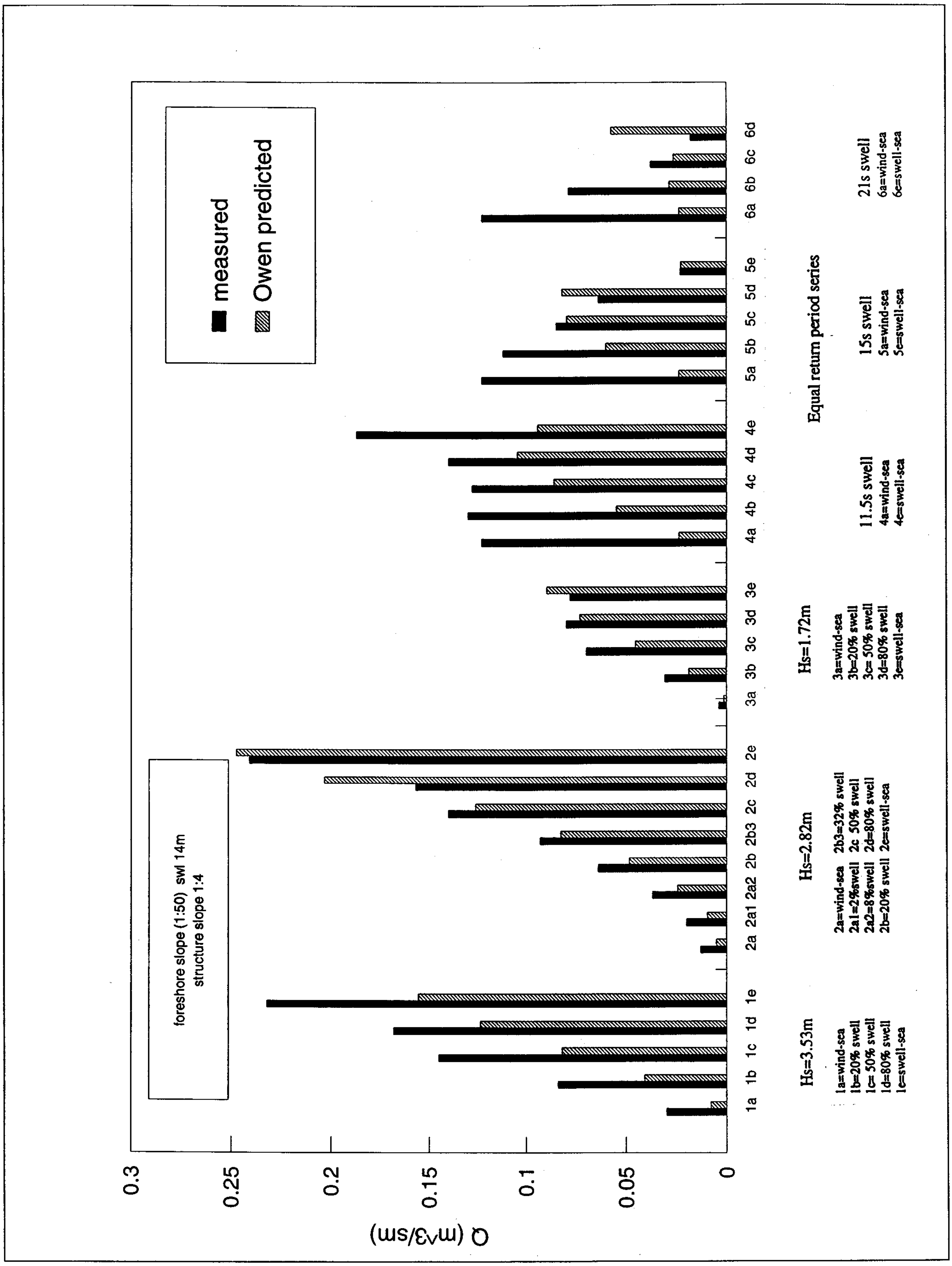


Figure 3.16 Comparison between measurements and modified Owen – set 3

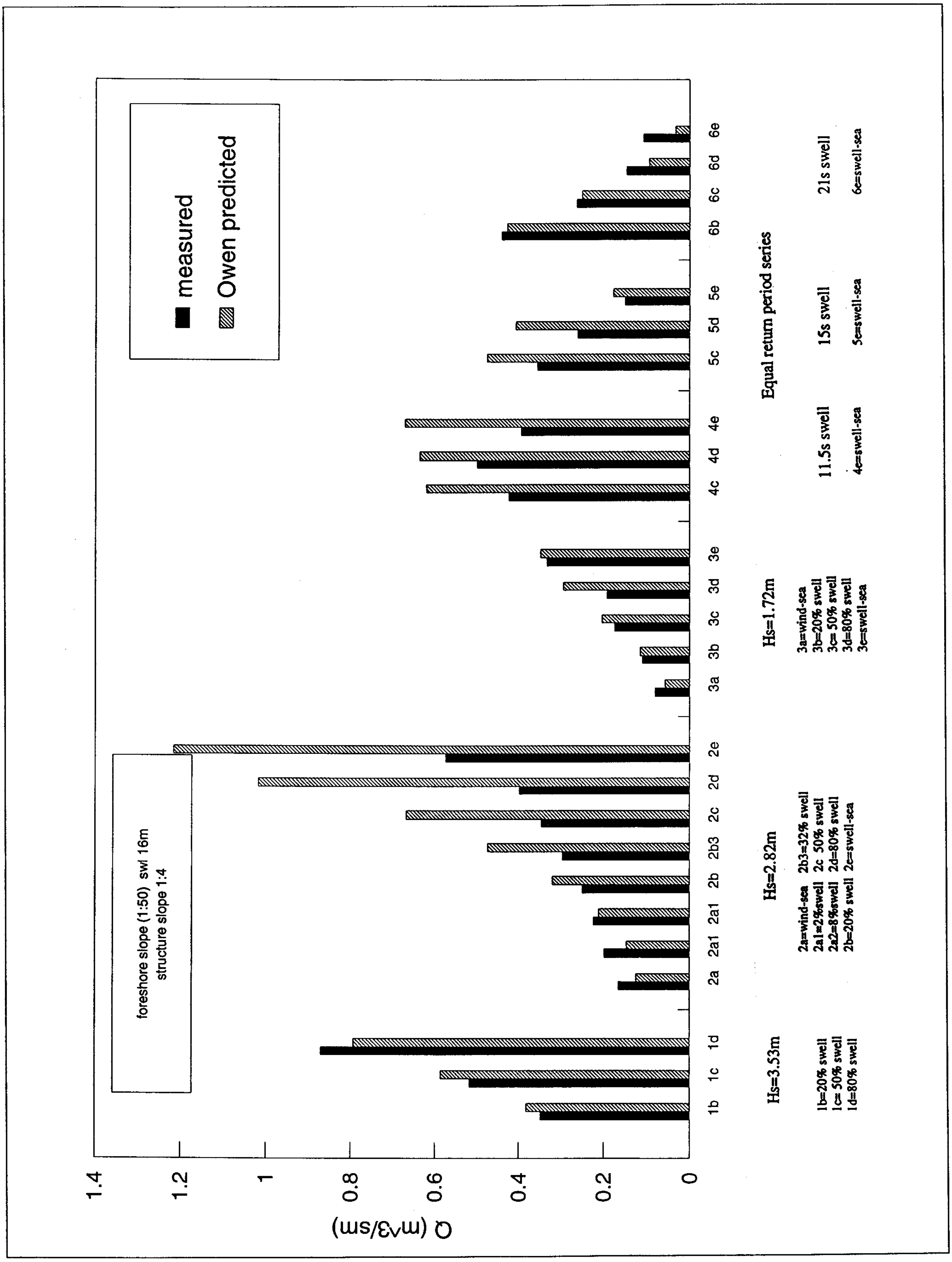


Figure 3.17 Comparison between measurements and modified Owen – set 4

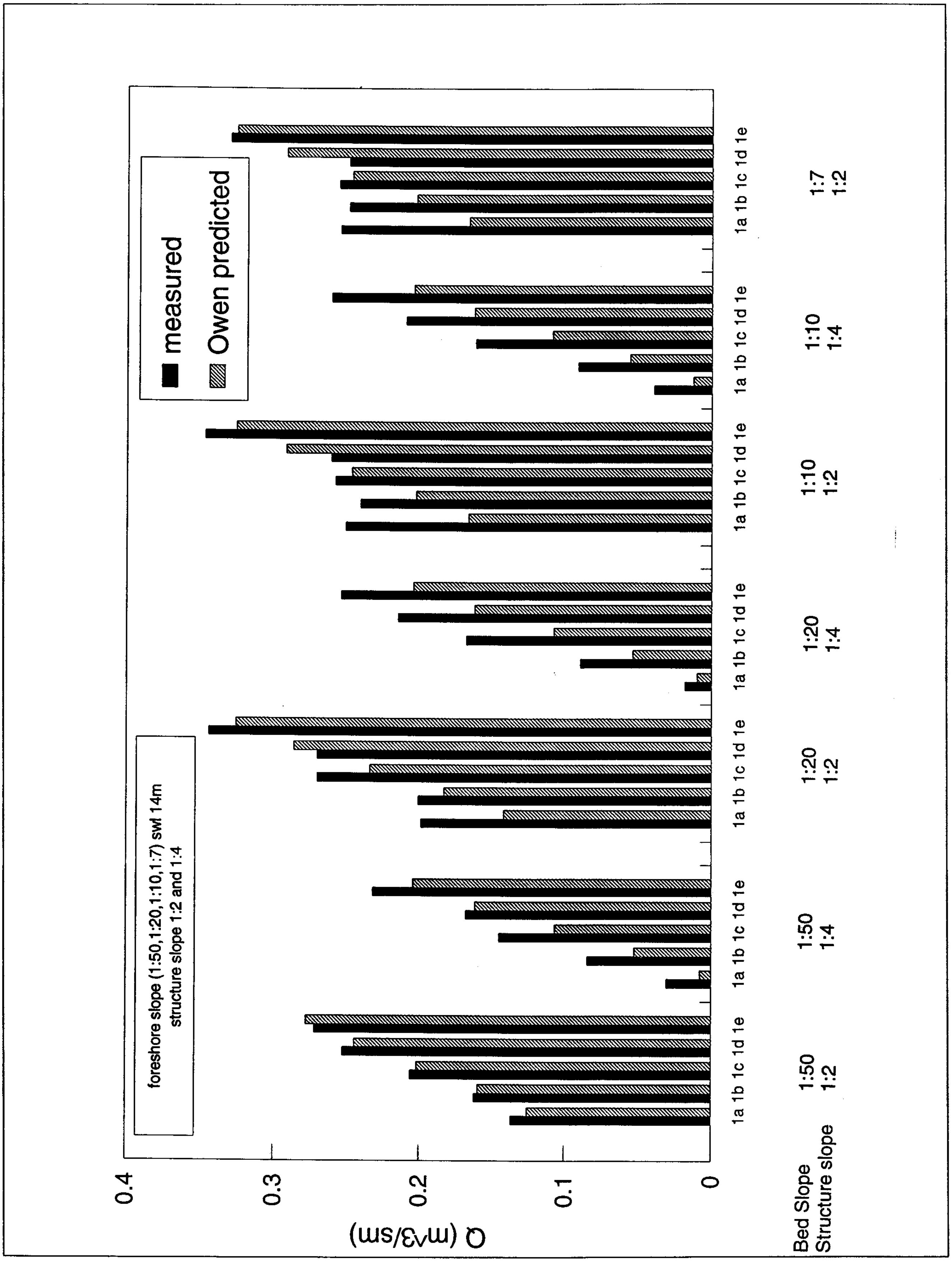


Figure 3.18 Comparison between measurements and modified Owen – set 5

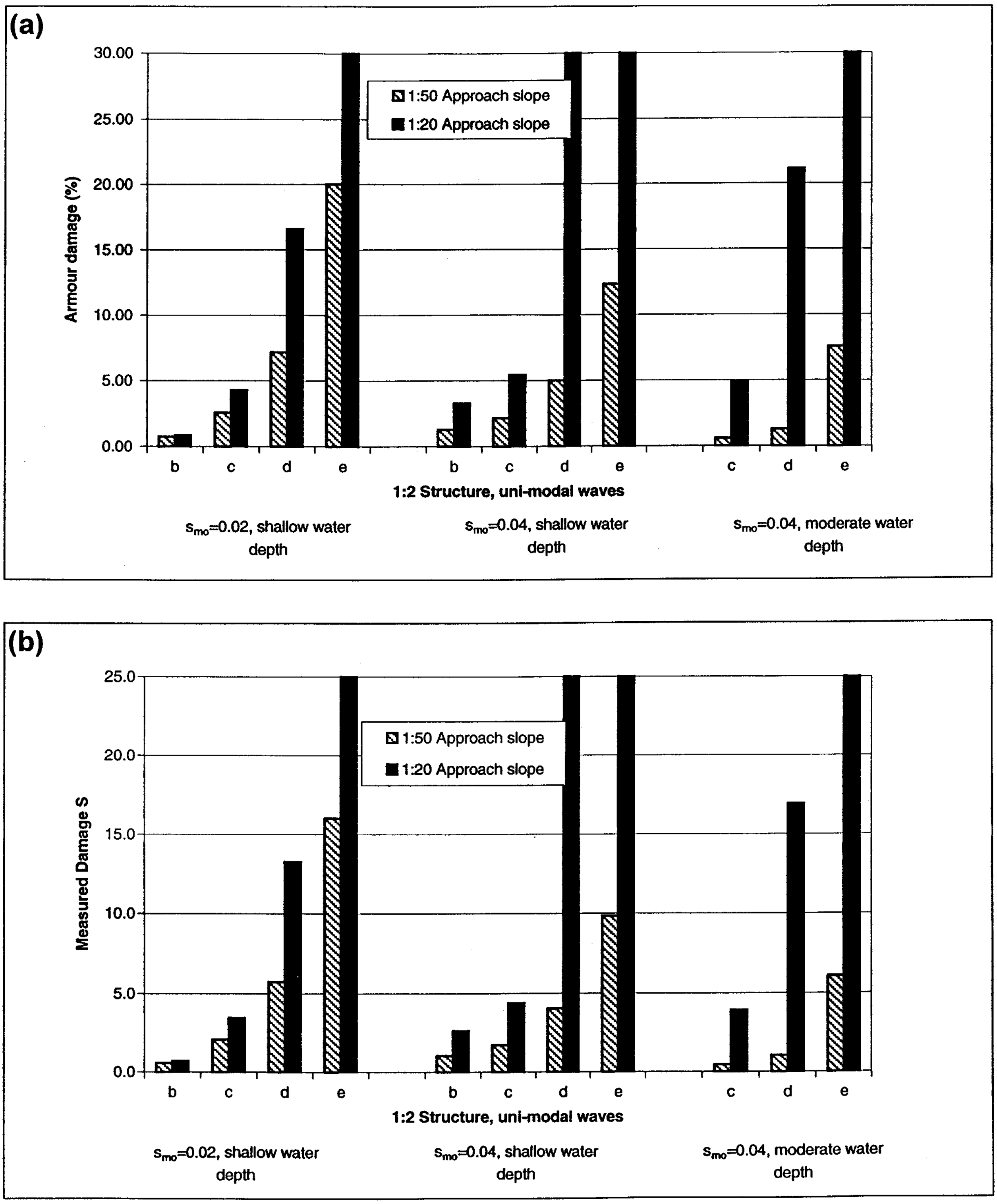


Figure 4.1 Uni-modal waves, armour displacements

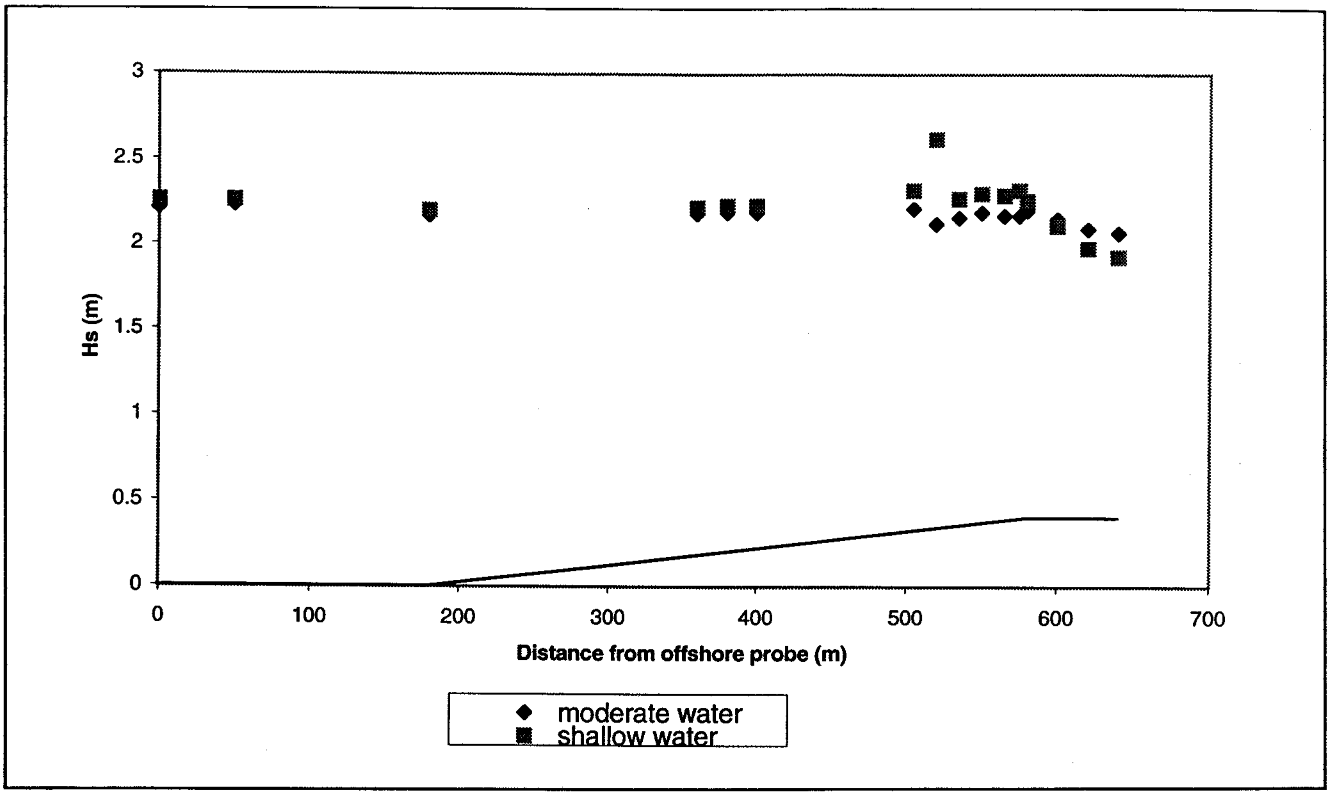


Figure 4.2(a) Example of wave shoaling and breaking

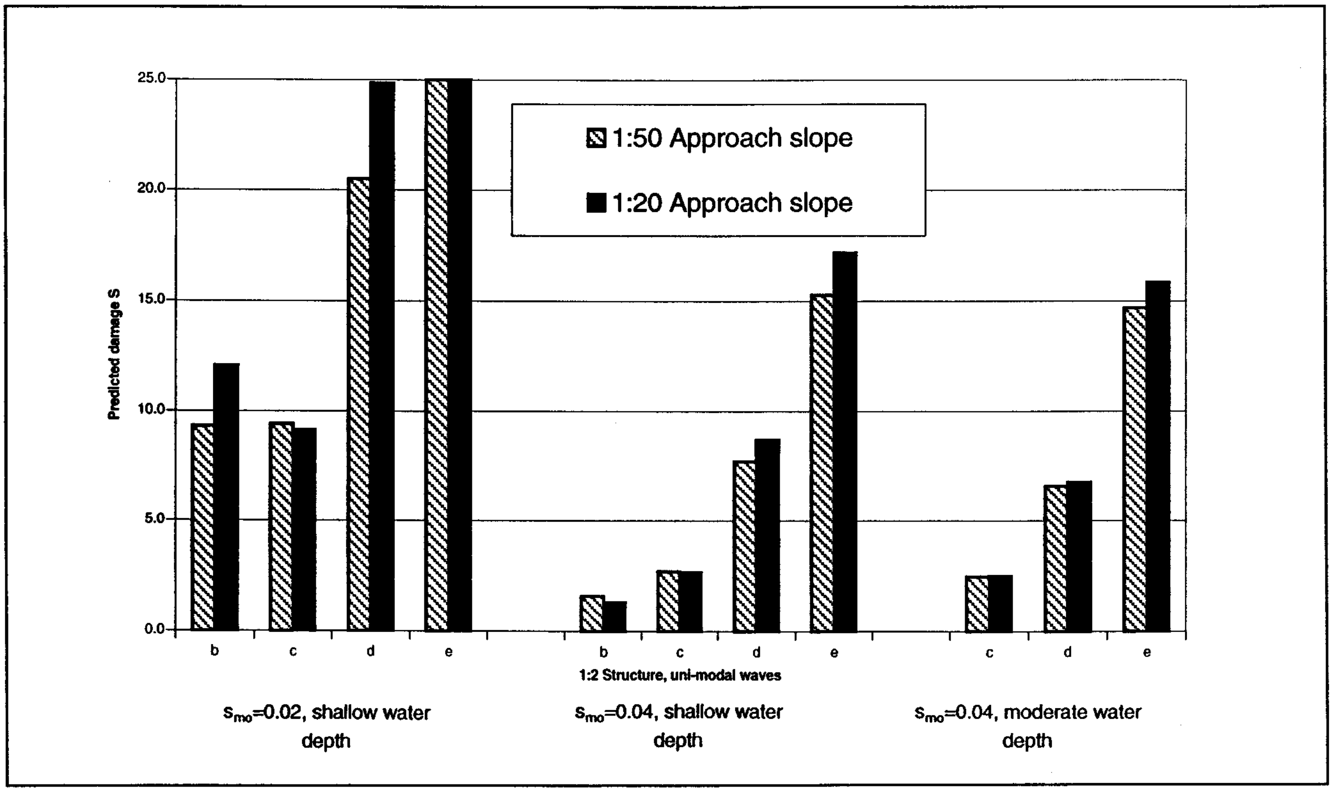


Figure 4.2(b) Uni-modal waves, predicted armour damage S

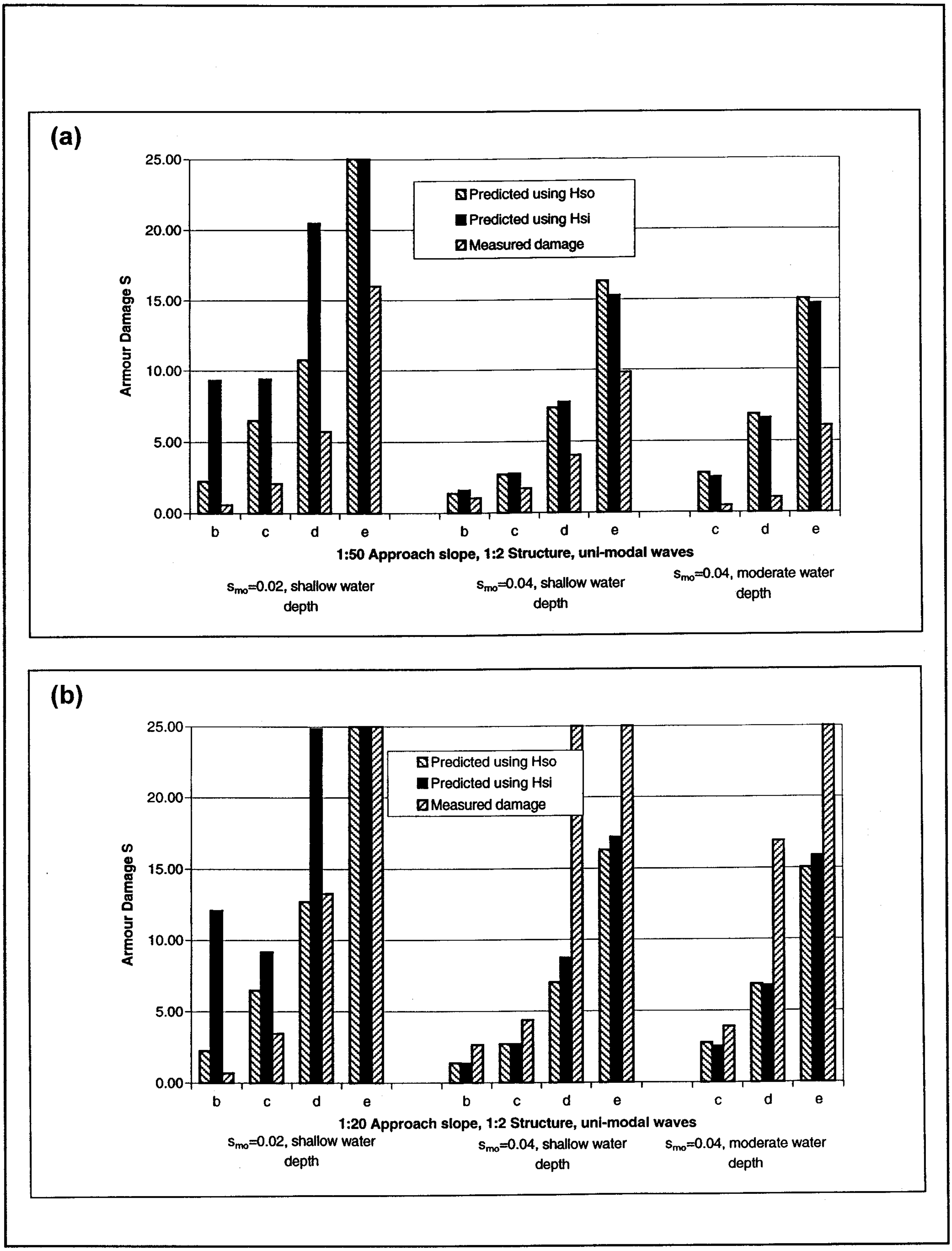
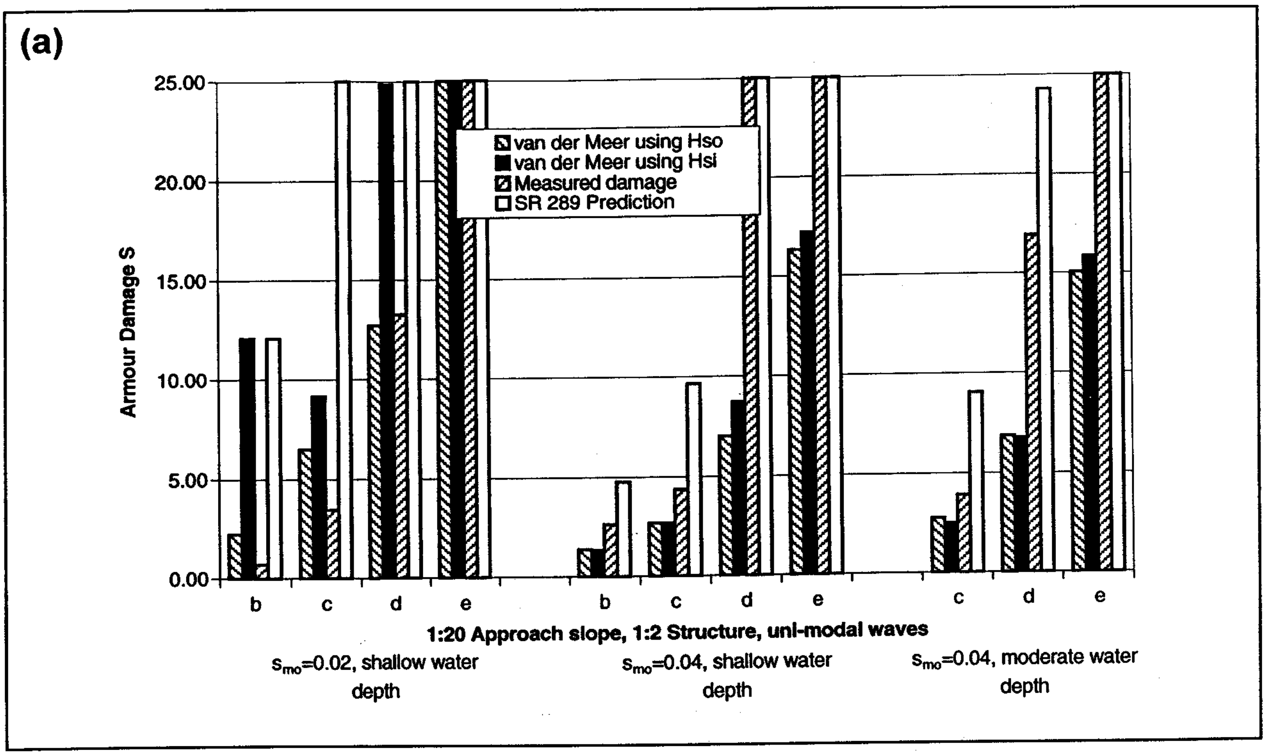


Figure 4.3 Uni-modal waves, comparison of measured and predicted displacements



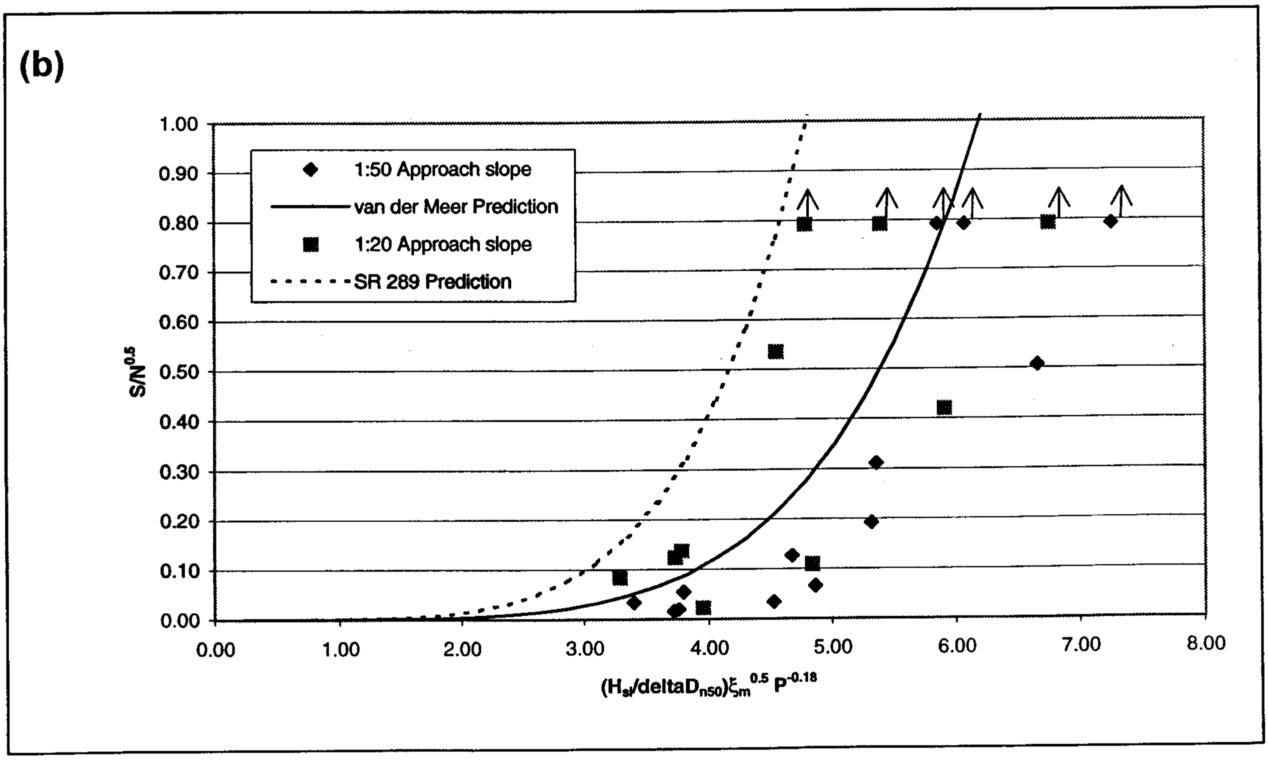
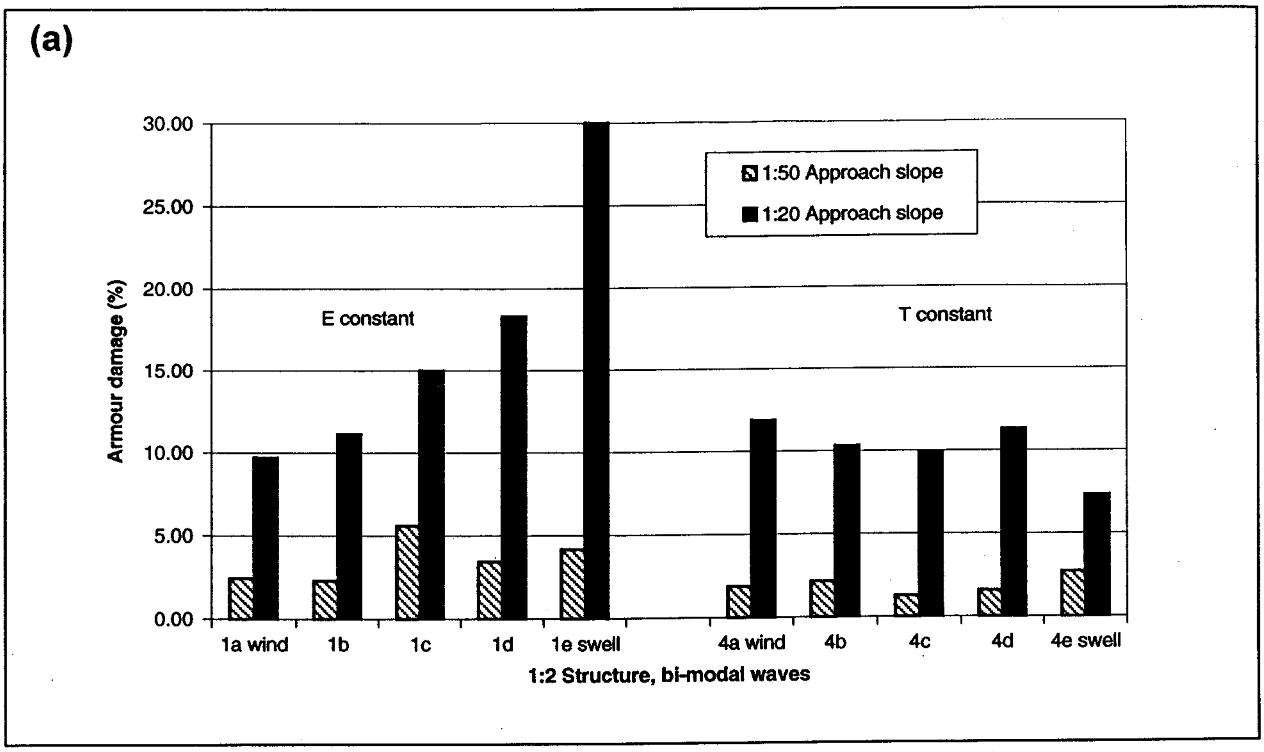


Figure 4.4 Uni-modal waves, comparison of measured and predicted displacements allowing for steep approach slopes



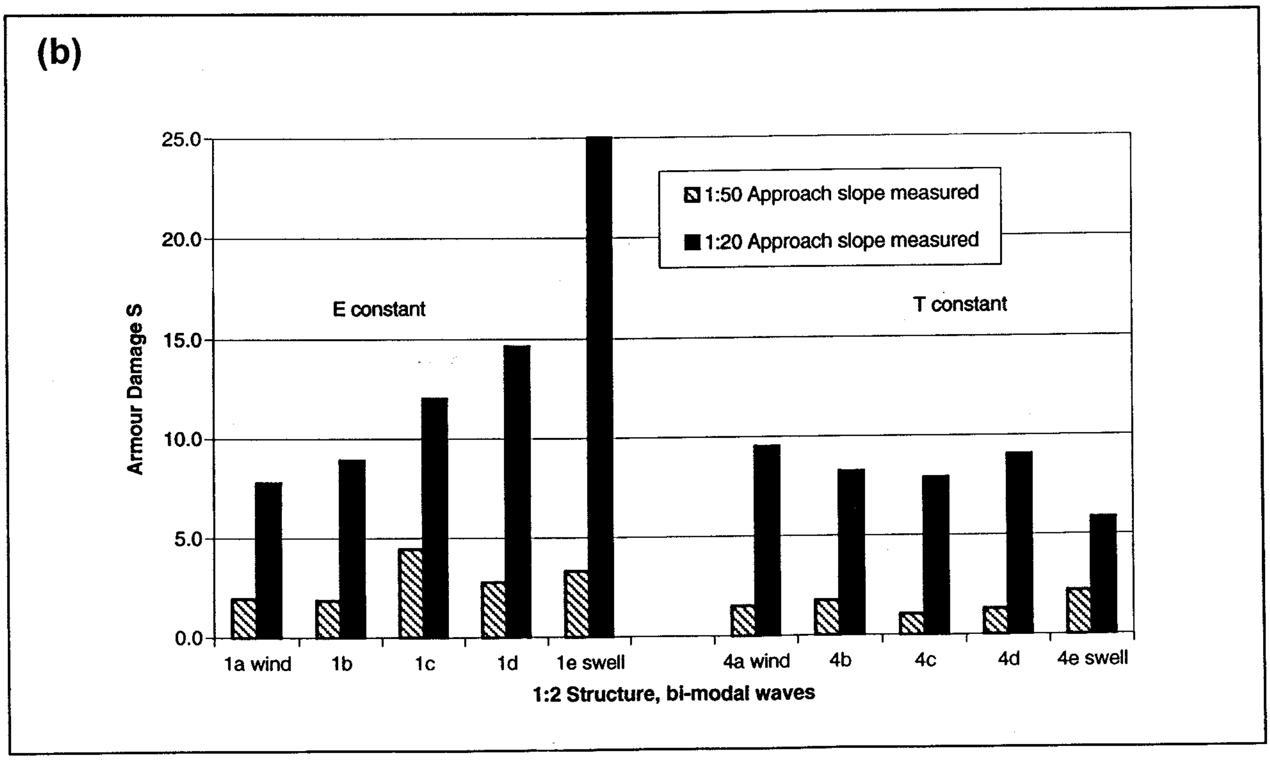
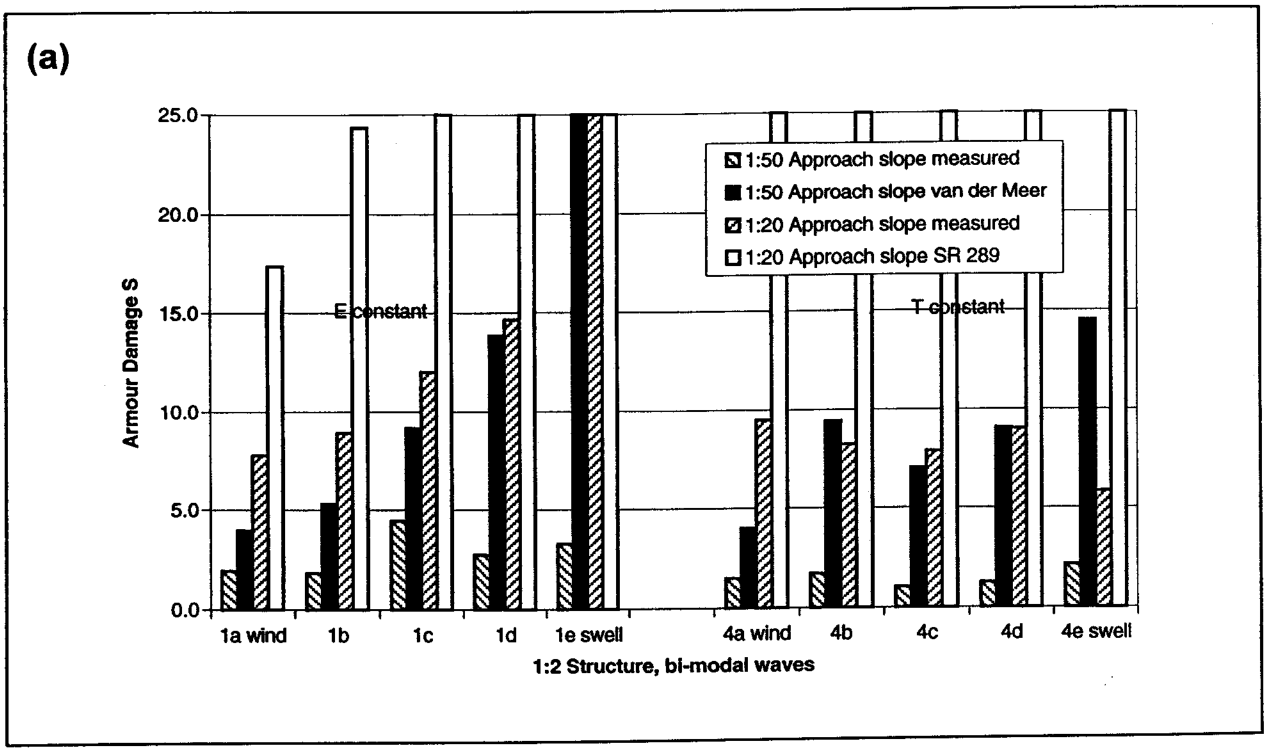


Figure 4.5 Bi-modal waves, armour displacements



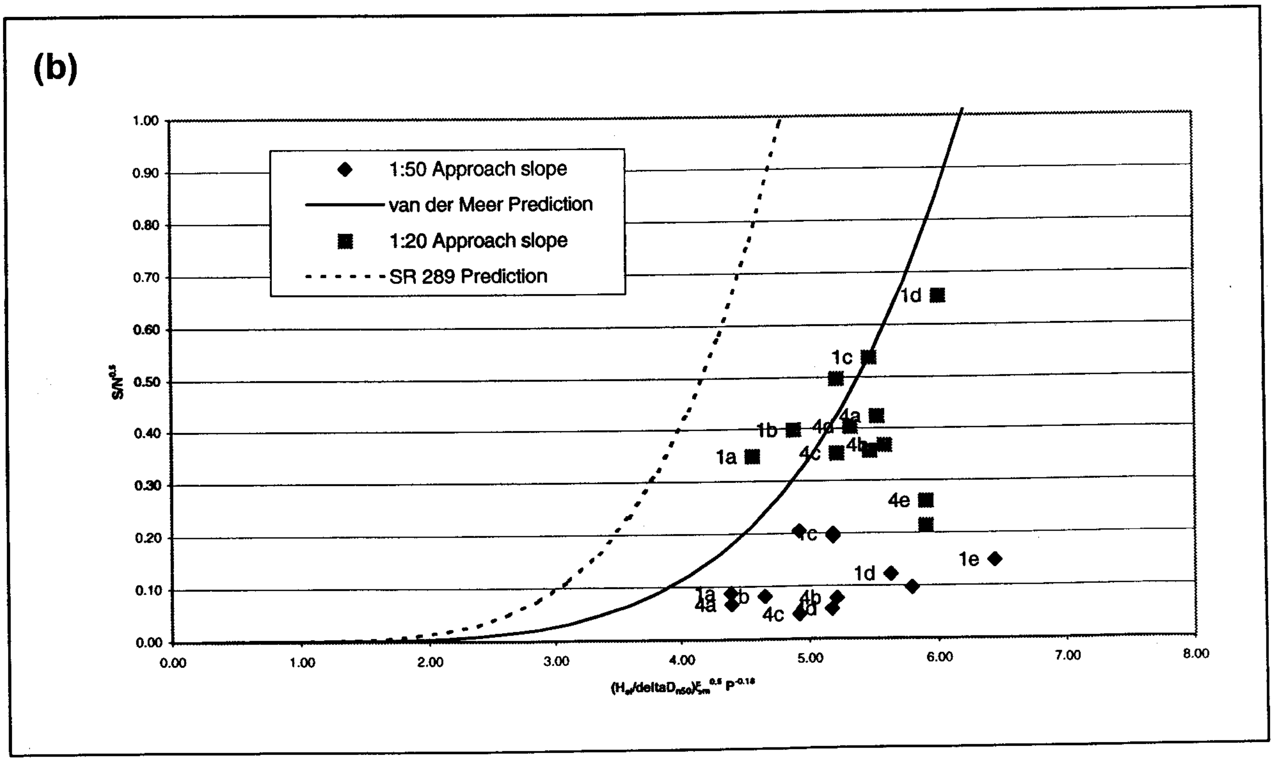


Figure 4.6 Bi-modal waves, comparison of measured and predicted displacements

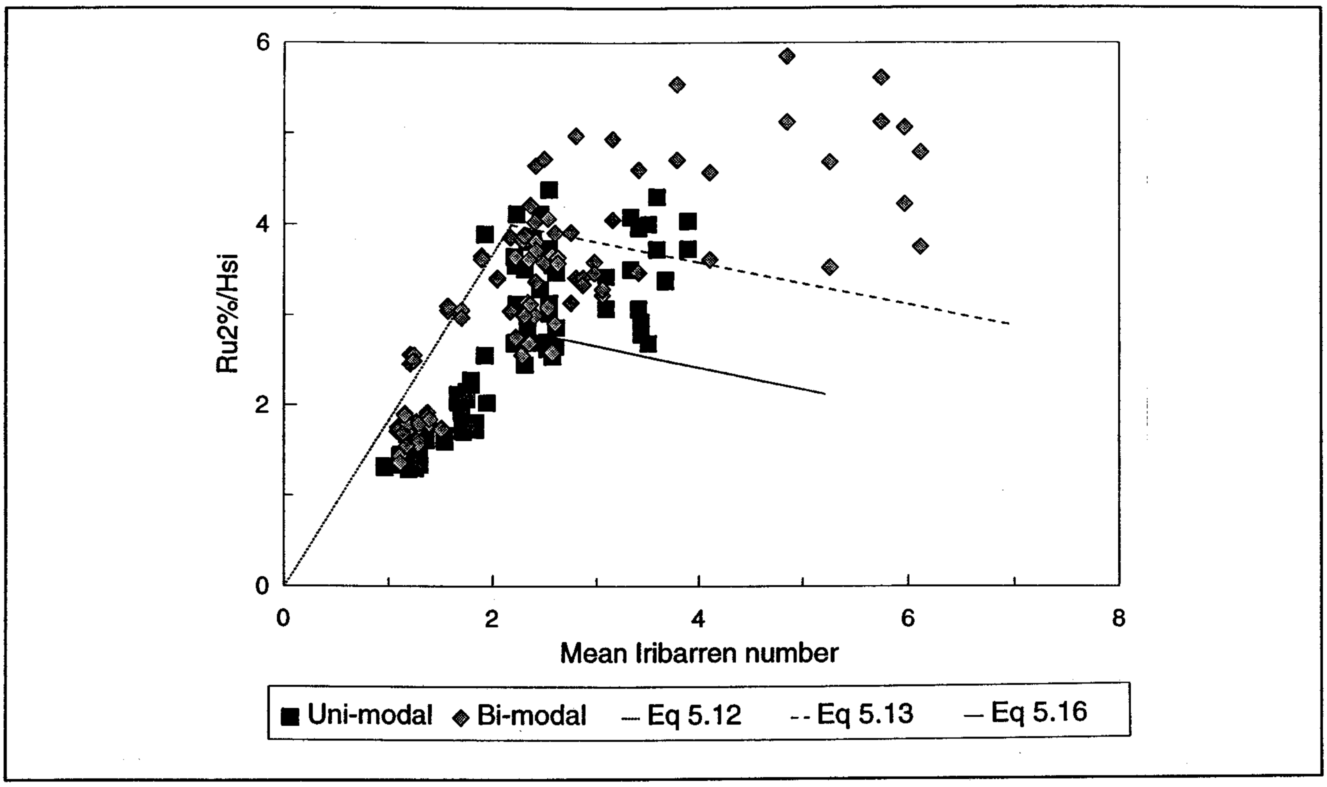


Figure 4.7 Run-up performance

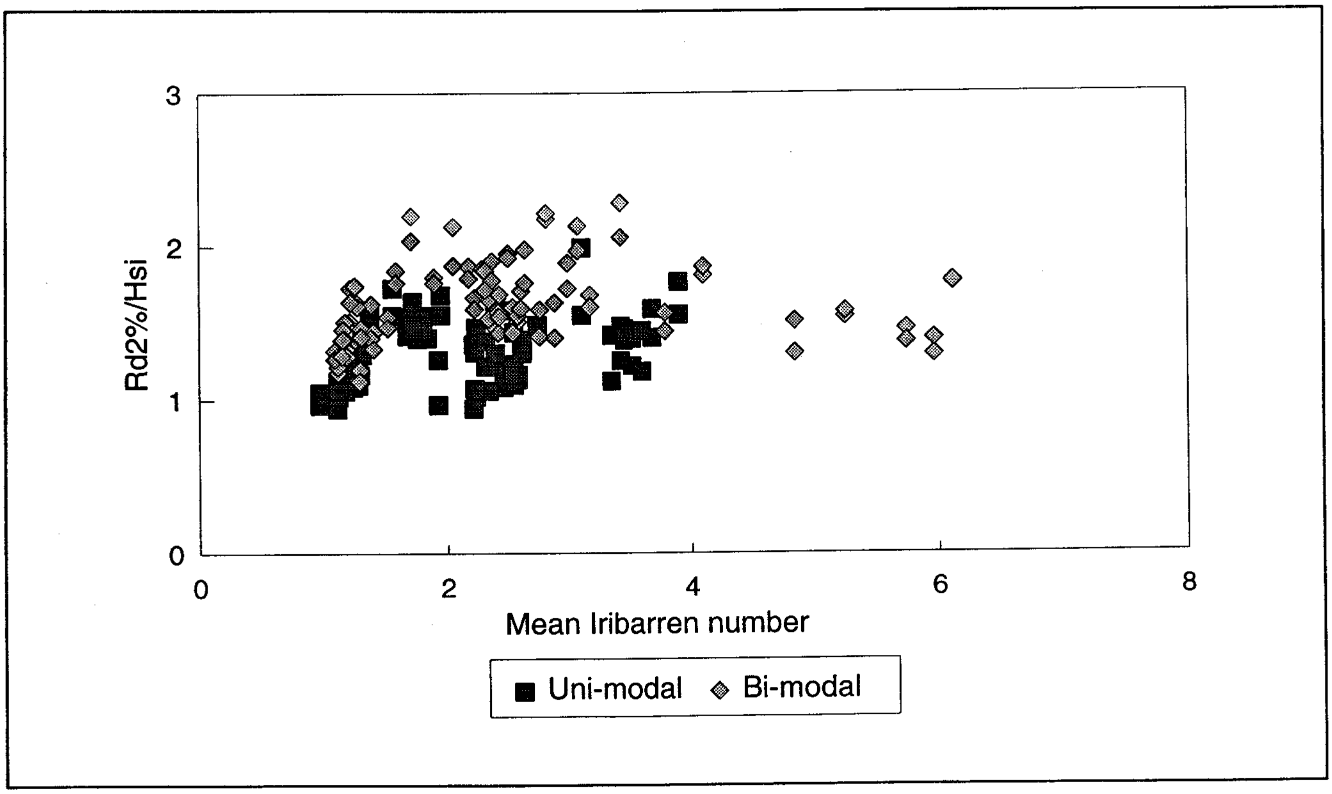


Figure 4.8 Run-down performance

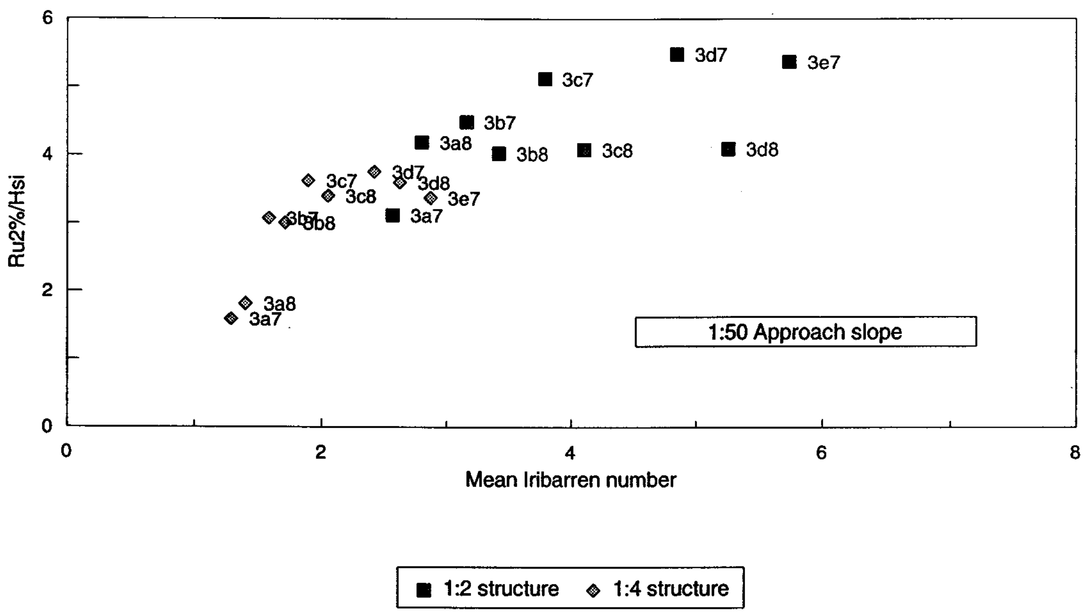


Figure 4.9 Bi-modal "equal energy" waves, run-up performance

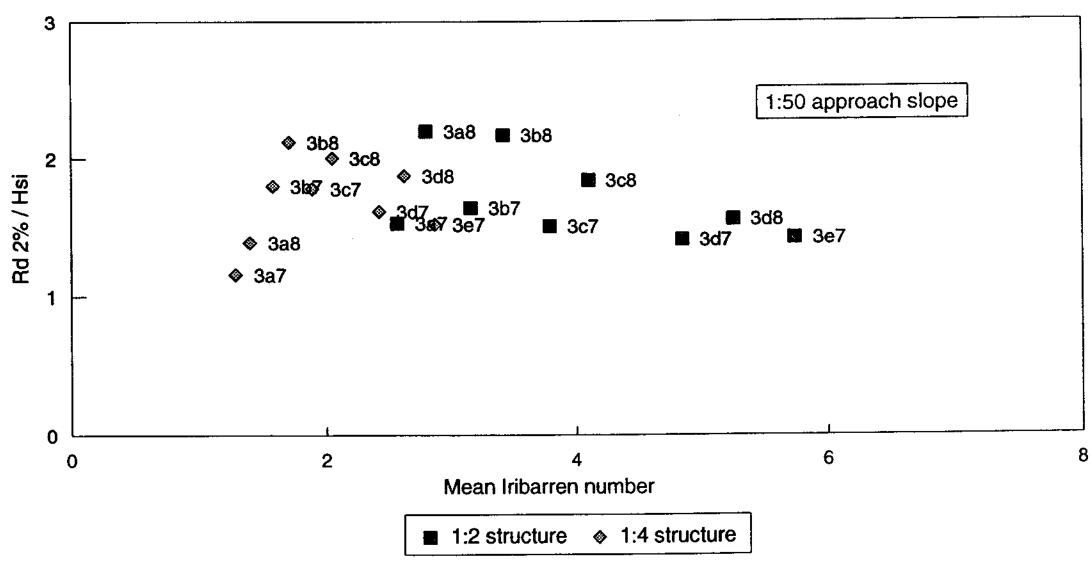


Figure 4.10 Bi-modal "equal energy" waves, run-down perfomance

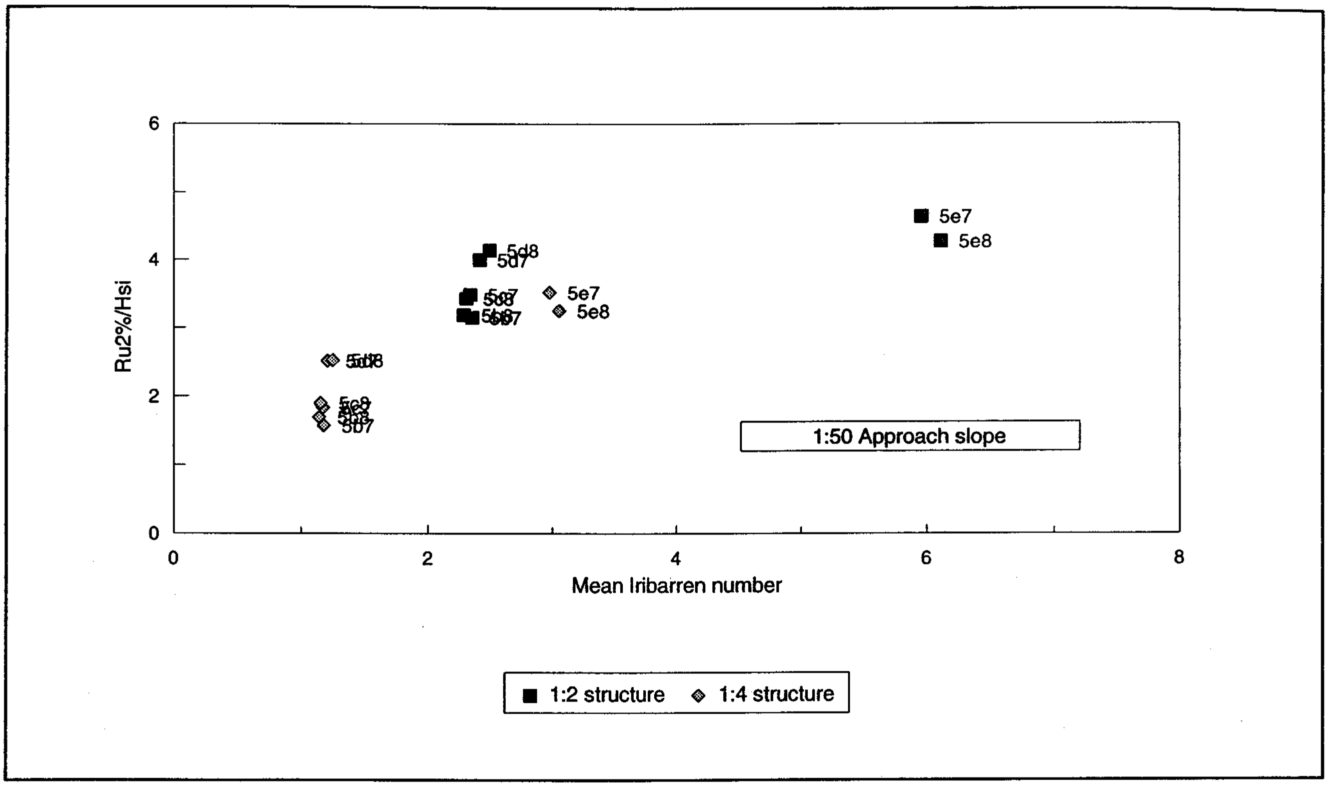


Figure 4.11 Bi-modal "equal return period" waves, run-up performance

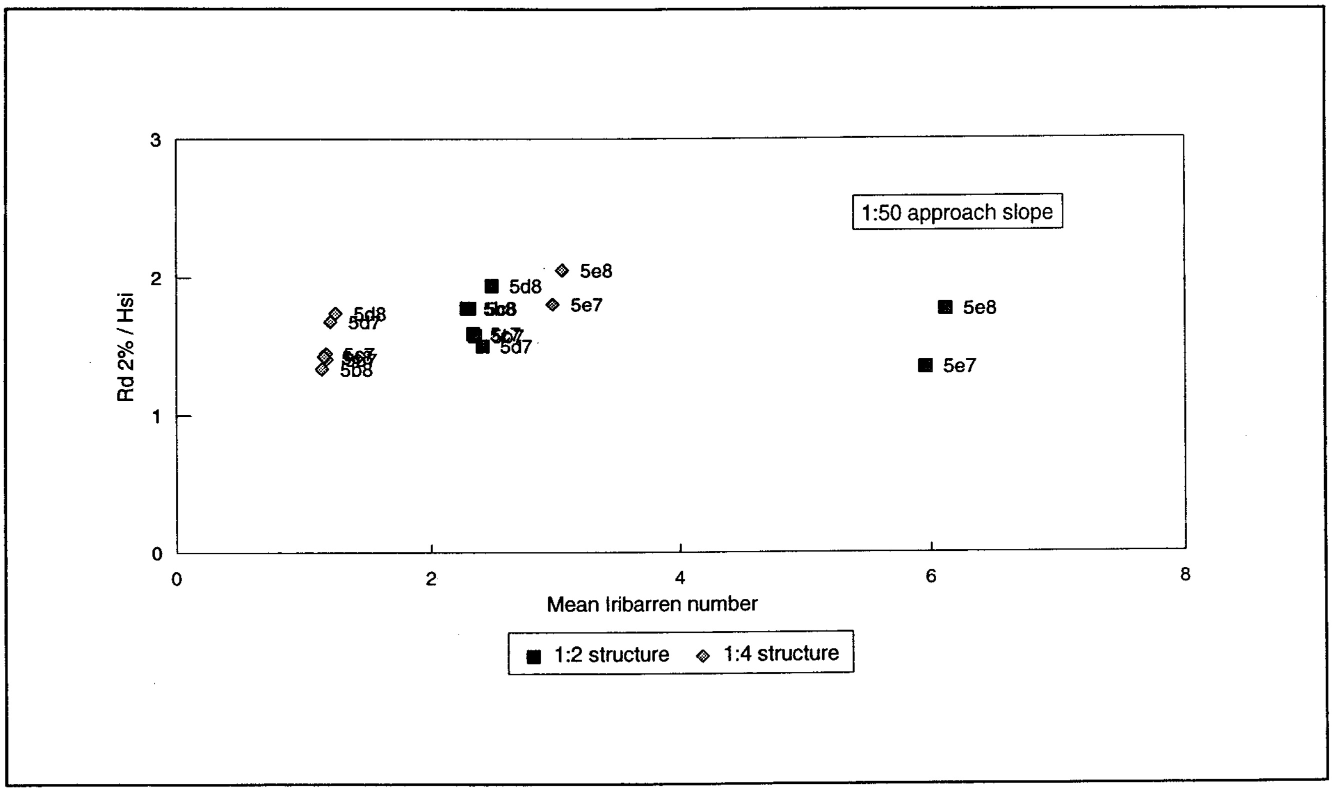


Figure 4.12 Bi-modal "equal return period" waves, run-down performance

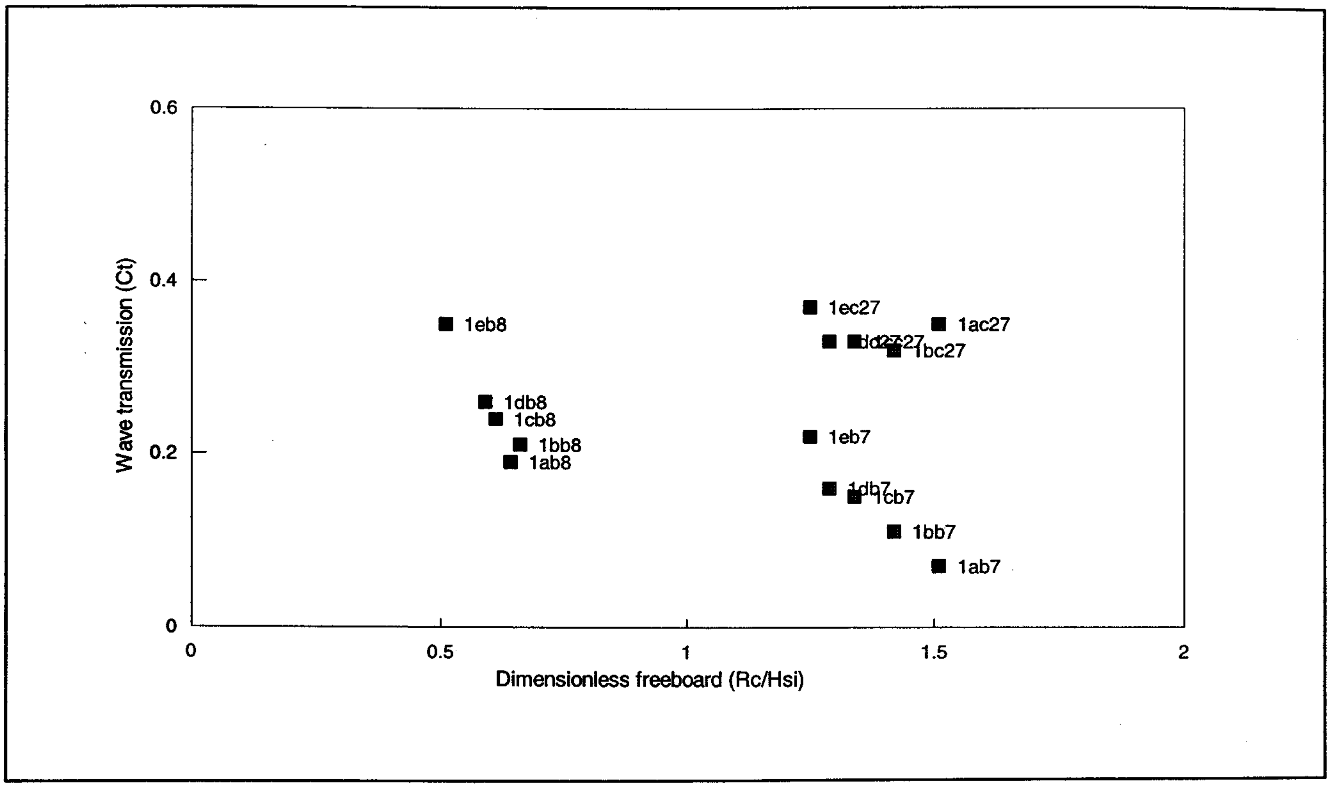


Figure 4.13 Bi-modal "equal energy" waves, transmission performance

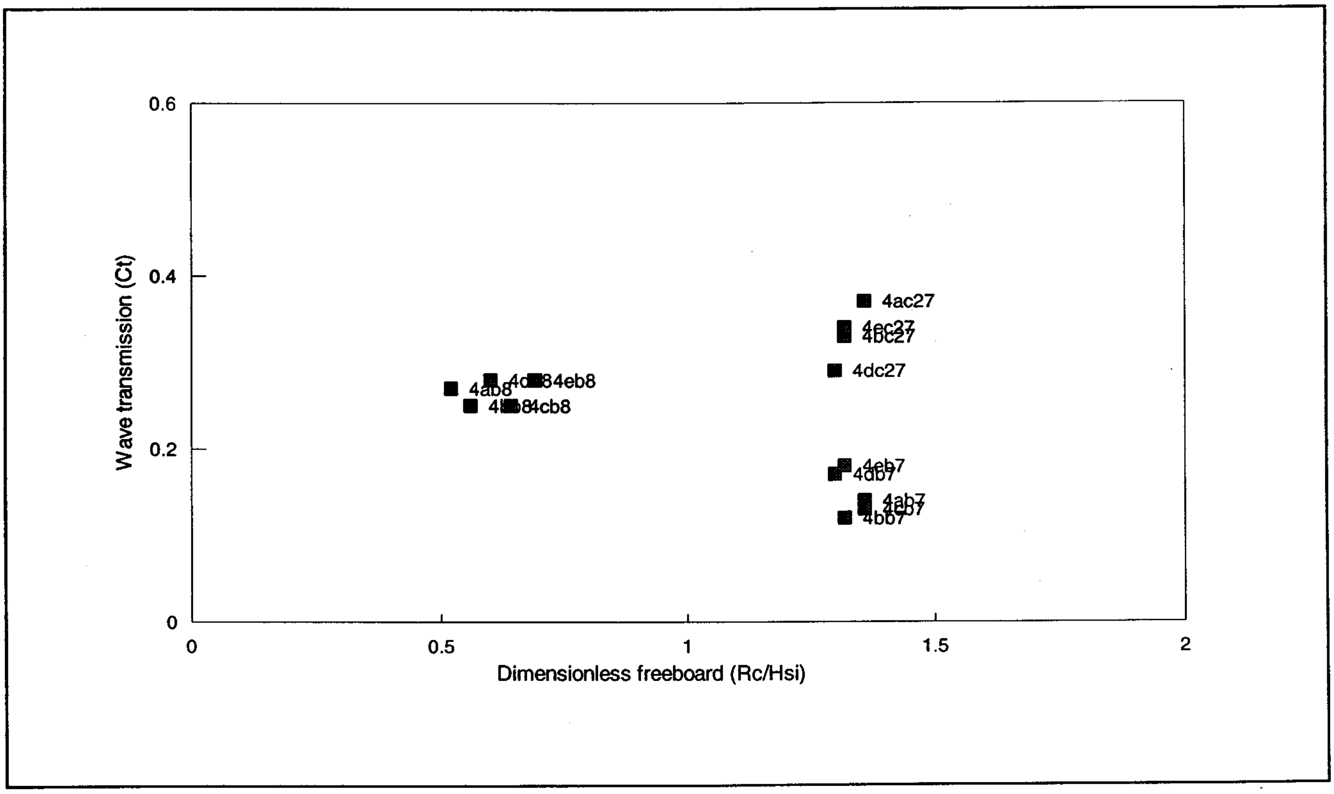


Figure 4.14 Bi-modal "equal return period" waves, transmission performance

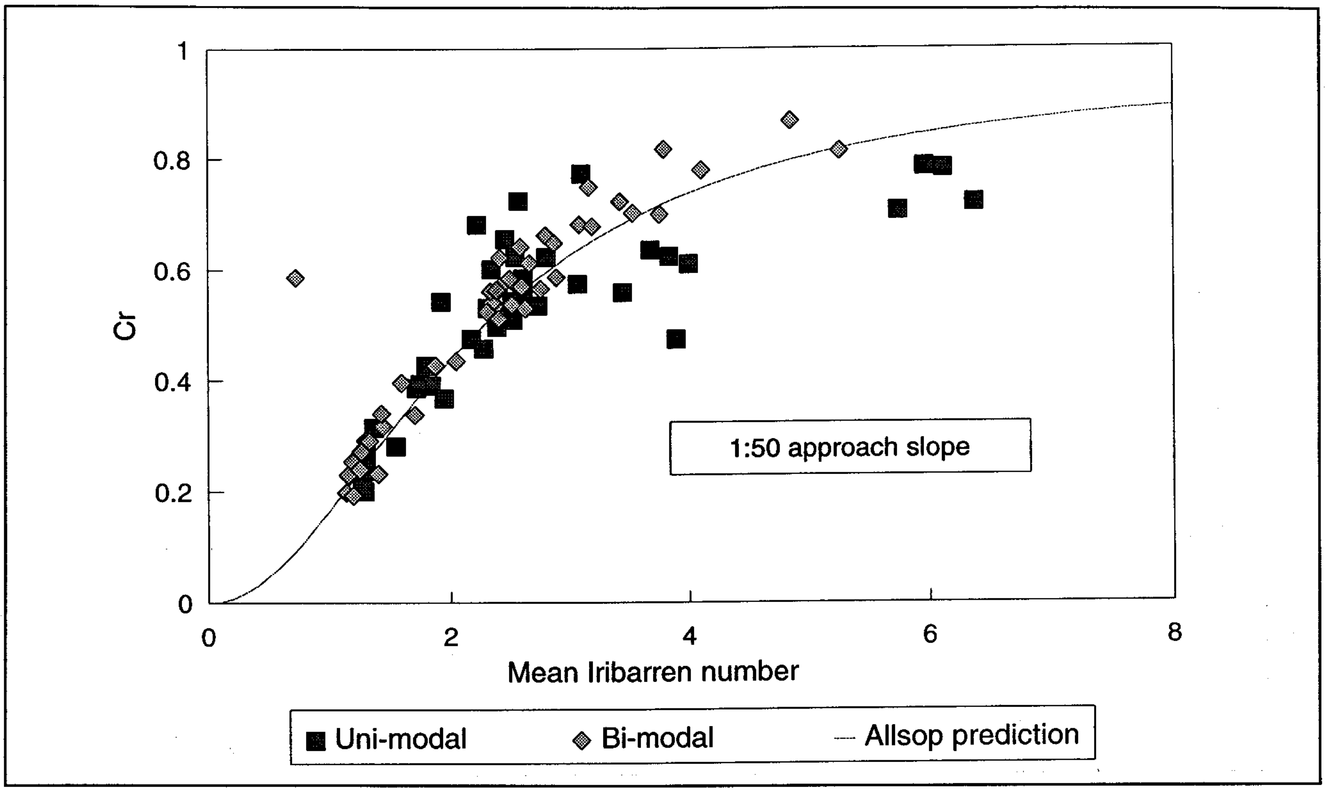


Figure 4.15 Reflection performance of uni-modal and bi-modal spectra

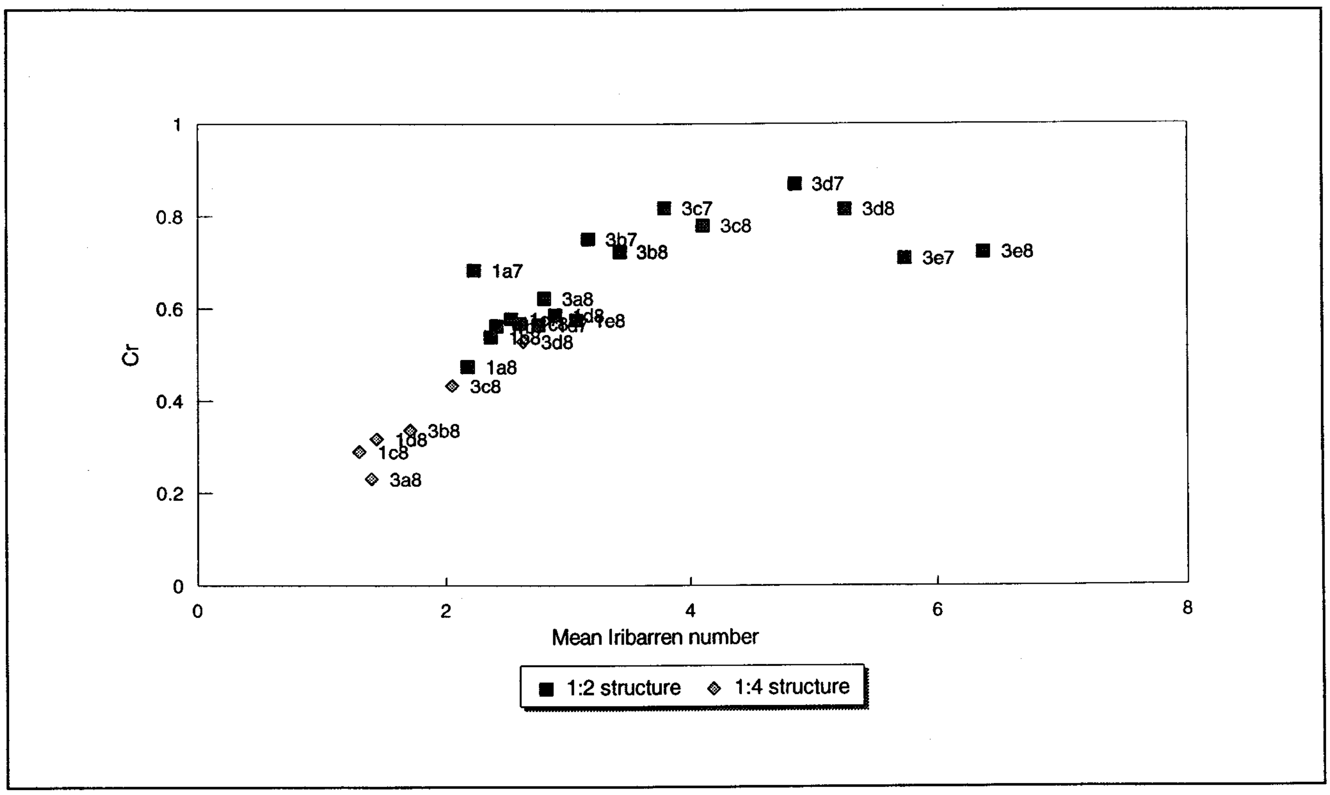


Figure 4.16 Bi-modal “equal energy” waves, reflection performance

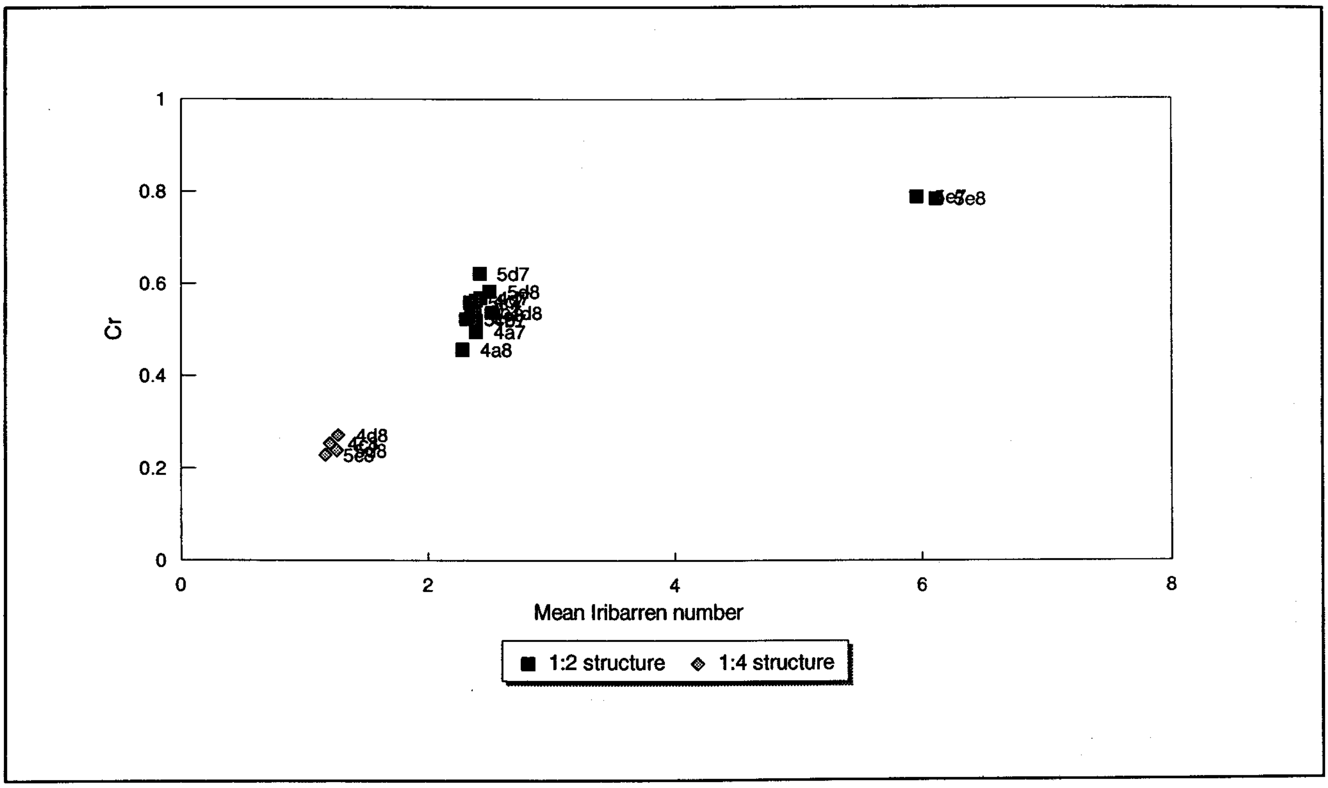


Figure 4.17 Bi-modal "equal return period" waves, reflection performance