Effectiveness of Control Structures on shingle beaches

Physical Model Studies

T T Coates

Report SR 387
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HR Wallingford

Registered Office: **HR Wallingford Ltd.** Howbery Park, Wallingford, Oxfordshire, OX10 8BA, UK
Telephone: 0491 835381 International + 44 491 835381 Telex: 848552 HRSWAL G.
Facsimile: 0491 832233 International + 44 491 832233 Registered in England No. 2562099
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Prepared by ..Ian Cruickshank.................. Project Manager............

Approved by ..K A Powell.................. Group Manager......

Date ..12. Jan. 1995..........

Summary

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Shingle beach response in the presence of groynes and detached breakwaters was investigated using a 1:50 mobile bed physical model. The study is part of an ongoing coastal research programme at HR Wallingford.

The groynes study investigated a number of structural variables for both timber and rock groynes under a range of sea conditions. Groyne effectiveness is determined from measurements of longshore transport, cross-shore distribution of transport, beach profiles, plan shapes and volumes. The conclusions drawn are not directly applicable to site situations, but indicate a number of factors that should be considered during groyne design for recharged beaches. Field verification of the model results, followed by further modelling are necessary before general design guidelines can be specified.

The breakwater study concentrated on single rubble mound structures, but concluded with a brief series of tests on pairs of structures. Structural variables include length, freeboard, distance offshore and gap width. The study is more conclusive than the groynes work, though it is also limited by the variables tested and the lack of field verification. A design approach is proposed which relates the dominant structural variables to potential and actual sediment drift rates. Further work will allow the proposed approach to be developed into a general design method.

A final chapter is included which attempts to place the research results into a practical engineering context. Users of this report must be aware of the limitations of the available experimental and field data.

Notation

h_{cg}	Crest elevation of groyne head
h_{cs}	Crest elevation of breakwaters
d_b	Water depth at wave breaking point
D	Sediment grain size
G_s	Gap length between breakwaters
H_s	Significant wave height (offshore)
L_m	Mean wave length (offshore)
L_g	Groyne length
L_s	Breakwater length
θ	Offshore wave angle
Q	Longshore transport rate
Q_i	Available updrift input
Q_c	Controlled drift
Q_o	Downdrift output
Q_p	Potential updrift input
R_c	Freeboard (crest elevation - SWL)
S	Wave steepness (H_s/L_m)
S_g	Groyne spacing
SWL	Still water level
T_m	Mean wave period
X	Cross shore chainage
X_{pp}	Cross-shore chainage of pinch point
X_s	Cross-shore chainage of breakwater centre line
η	Structure efficiency

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1 Introduction

1.1 Background

An understanding of the potential response of beaches to the local wave and water level regime is fundamental to successful coastal management. This is particularly so in situations in which the beaches are influenced by cross-shore or nearshore control structures. Much of the existing research and experience of beach response relates to sand beaches in micro (<2m) or mesotidal (2m-4m) environments. A number of morphodynamic models are available which go some way towards predicting sand beach response. Unfortunately these models are not applicable to the shingle and sand/shingle beaches found along much of the UK coastline. Recent research at HR Wallingford has provided parametric models to predict shingle beach profile response to normal or angled wave attack (References 1 and 2) and to predict the longshore planshape development of beaches with simple groynes (References 3 and 4). However, as yet, prediction of the impact of groynes is based on simple assumptions derived from limited physical model data that have not been verified by field measurement. No models are available to predict the impact of more complex groynes or other control structures.

Control structures include shore-connected breakwaters, detached breakwaters and sills, as well as groynes. They are used to retain a satisfactory beach in areas where the natural beach is not sufficiently stable to provide an adequate level of shoreline protection, or where the rate of longshore transport is too high to permit the cost-effective implementation of beach management techniques such as periodic replenishment or recycling. Generally the control structures are designed as part of a system of similar structures and are often combined with a beach replenishment.

The effectiveness of a system of beach control structures is determined by:

- the maximum recession of the beach, under the design storm conditions and over the design life of the structures, at any point within the area of concern, including the downdrift frontage;

- the rate of longshore transport through the area of concern relative to the open beach situation;

- the potential variation in beach planshape in response to changes in the availability of input drift;

- the capital and maintenance costs;

- the public and environmental acceptability.

The present study utilized a mobile bed physical model to investigate the first three of these factors. It is part of an ongoing programme of shingle beach research at HR Wallingford, which has so far investigated cross-shore beach response (Reference 1), longshore transport on open beaches and the response of a beach within a single groyne bay (References 2 and 3). This work extends the programme to look at multiple groyne systems, single detached breakwaters and, briefly, pairs of detached breakwaters.

The programme is supported by the Ministry of Agriculture, Fisheries and Food under the Coastal R & D Commission FD07.

1.2 Scope and purpose of the research

The original aim of this research was to utilize the results from a 3-D physical model to quantify the effectiveness of groynes and detached breakwaters under potential design sea conditions. This aim has been only partly achieved. Tests of different structural and sea state variables took much longer than originally expected and therefore the data set achieved is insufficient to develop a comprehensive method for beach response quantification. The groynes study provides some interesting results of use to designers, but part of the work is inconclusive and there is an urgent need for field work to verify a number of model processes. The breakwaters work is much more conclusive within the limits of the variables tested, and has resulted in a first step towards developing an approach to designing structures. Further modelling work is required to extend the results towards a full design method which can be incorporated into a numerical model. If the results of this study are used for design then care must be taken to understand the limitations of the work.

1.3 Outline of the report

This report describes the methods, presents the data and discusses the results of the research programme. Following this introductory chapter the physical model and the test methodology are described in Chapter 2. The results for the groyne and breakwater studies are presented and discussed in Chapters 3 and 4 respectively, while the conclusions and recommendations for further work are made in Chapters 5 and 6. A final chapter has been included which discusses the application of the research to actual design. It is hoped that this will be of use to coastal managers and engineers.

Details of the scaling of model sediment are presented in Appendix 1. Figures and photographs illustrating the test results are presented in Appendices 2-7.

All measurements in the report are in prototype terms unless otherwise indicated.

2 *The physical model*

2.1 The wave basin facility

The model test programme was conducted in a wave basin at HR Wallingford. The basin is 23m by 24m, with a maximum working water depth of 0.4m (Figure 2.1). It was equipped with:

- a wave generating system (random or regular waves) comprising a 15m long electro-hydraulically driven paddle controlled by a micro-computer. The paddle could be orientated at up to 45° relative to the beach;

- wave probes for calibrating and monitoring the required wave conditions;

- wave guides to prevent lateral loss of energy;

- a mobile test beach comprising crushed and graded anthracite coal scaled to simulate typical UK shingle upper beaches;

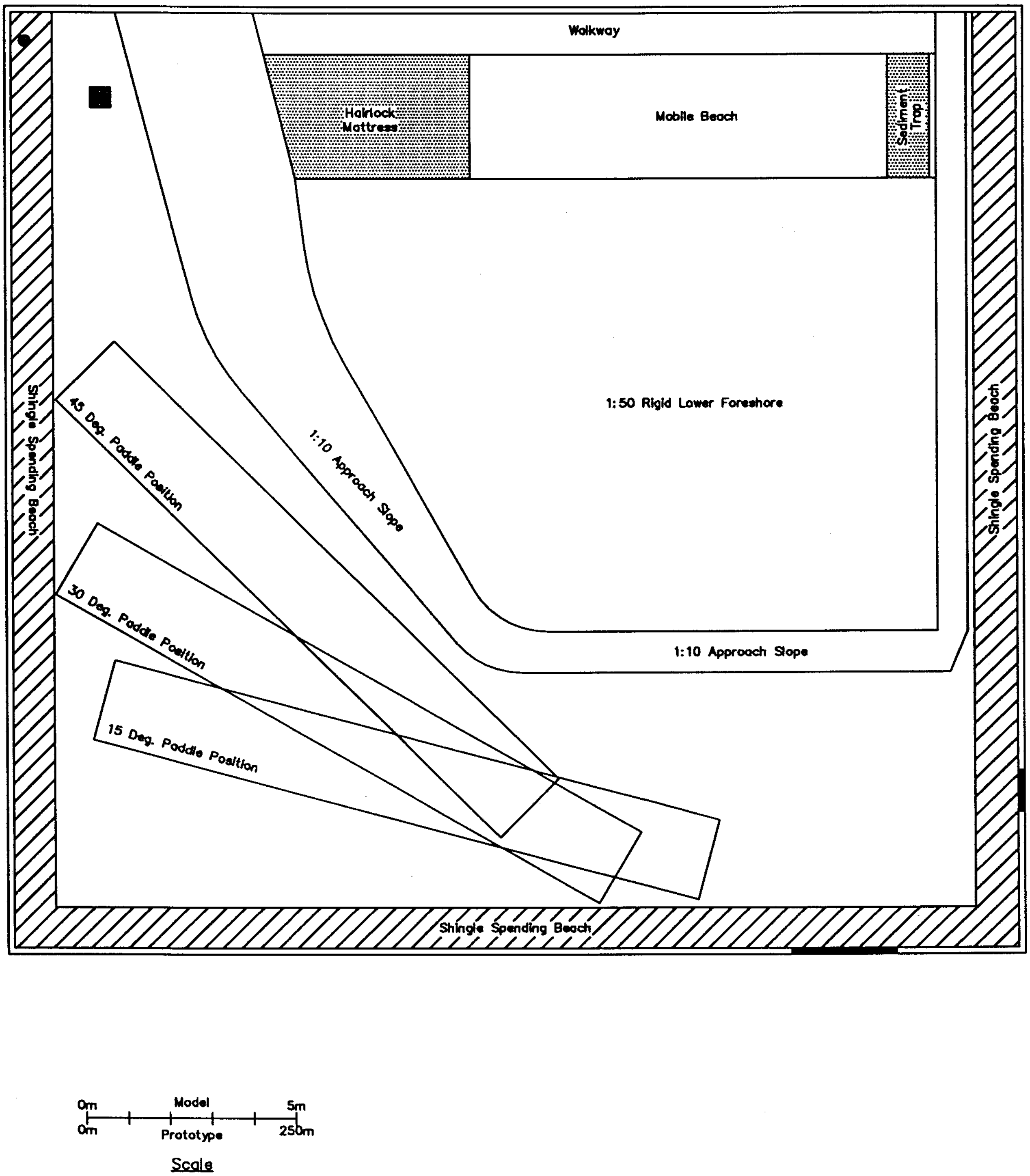


Figure 2.1 Wave basin layout

- a hard moulded nearshore bathymetry designed to simulate a gently sloping sand lower beach;

- a computer driven, semi-automatic, incremental bed profiler;

- a manual sediment input system;

- a downdrift sediment trap which could be compartmentalized to monitor the cross-shore distribution of longshore transport;

- oblique angle stills camera to provide overhead photographs as evidence of beach response.

2.2 The model beach

The model beach was designed to simulate a typical shingle upper-sand lower beach at a scale of 1:50. The mobile bed of crushed anthracite represented the shingle element and was moulded to an initial slope of 1:7½. The sand lower beach was represented by a rigid cement mortar moulding at a slope of 1:50. The model cross-section is shown in Figure 2.2.

The mobile bed was designed to be similar to those used in previous model studies (Reference 1 and 2) to allow direct comparison and to simulate typical UK beaches. The scaling relationships used to select the beach material are discussed in detail in the previous reports and in Appendix 1. The prototype and model grading curves are presented in Figure 2.3. The main sediment parameters are summarized below in prototype terms:

D_{10} (mm)	D_{50} (mm)	D_{90} (mm)	D_{85}/D_{15}
3	13	50	4.0

In contrast to the previous studies, the mobile bed did not extend down to "deep" water. The toe of the beach was fixed at 4m below the maximum water level; in order to simplify later discussions this toe level has been taken to be 0m.

A further difference with the previous groyne tests described in Reference 2 was the elevation of the mobile bed crest. At the outset of the tests in the previous study the bed was moulded with a crest elevation 2m above the working water level and was therefore below the maximum wave run-up level. In this study the crest was moulded at 6m above the maximum water level and was therefore well above the maximum run-up level.

2.3 Design of structures

A variety of groyne and breakwater layouts were tested during the study programme.

Groynes

The groyne layouts included both timber and rock structures with a range of head elevations, lengths and spacings; Table 2.1 provides a summary, while Figures 2.4-2.7 present profiles of each groyne type.

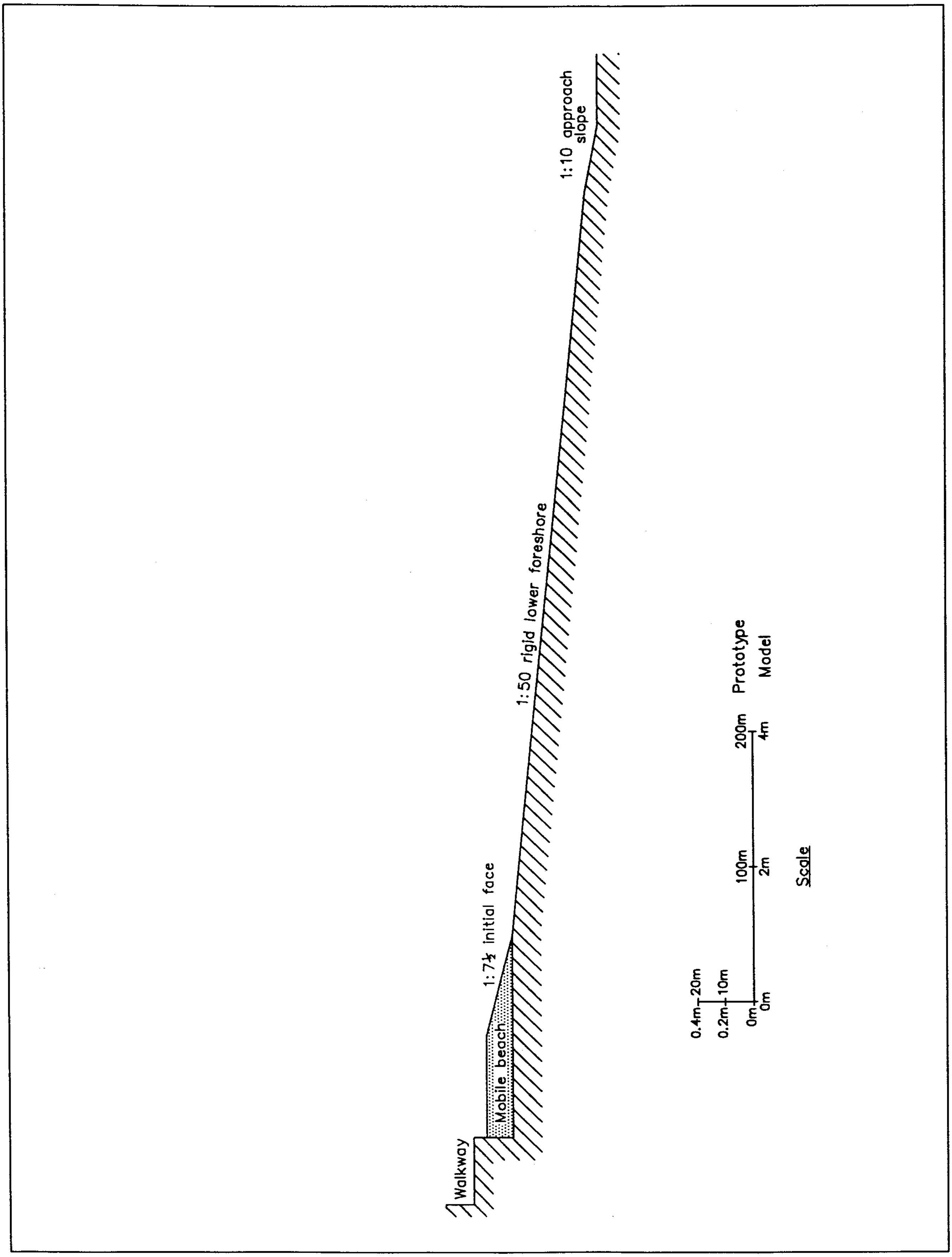


Figure 2.2 Model cross-section

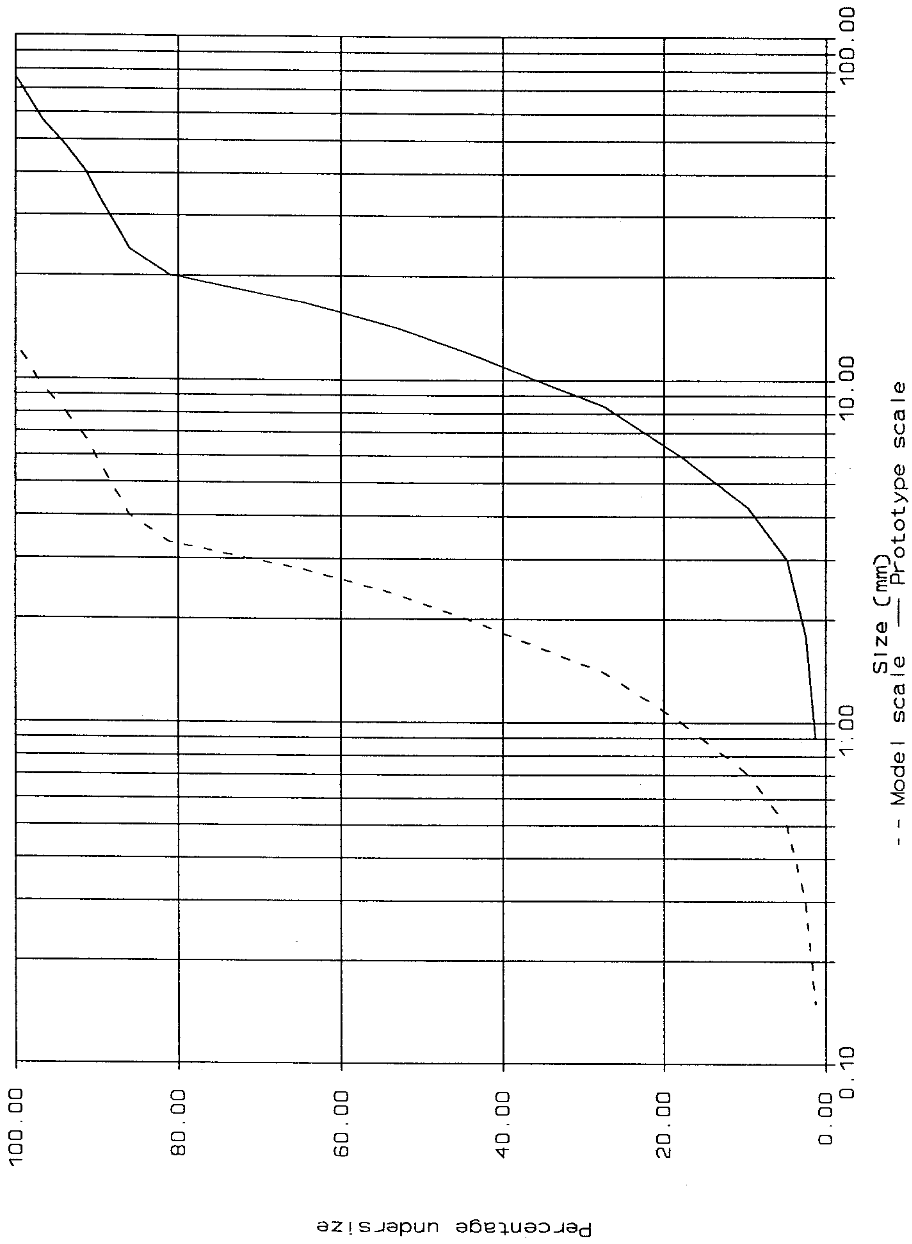


Figure 2.3 Beach grading curves - Prototype and model scales

Simple sloping timber groynes were designed to simulate the most common form of groyne around the UK coast. For the purposes of this report they are used as the base situation against which other configurations are compared.

Tests of the simple groynes included variations in wave height (H_s), wave steepness (S), water level (SWL), input transport volumes (Q_i) and groyne spacing (S_g). The groynes were 65m in length relative to the beach head. Figure 2.4 (Low Timber) illustrates the profile.

Two other timber groyne types were tested. The first was similar to the low groynes, but was raised by 1.5m along the full crest length (Figure 2.5a, High Timber)). This layout was used to investigate the effect of increased crest elevation, particularly at the groyne head (seaward end). The second type had the seaward end raised to a constant elevation at the maximum still water level of 4m (Figure 2.5b, High End Timber). The intention of this layout was to introduce a barrier to longshore transport which would have a variable effect dependent on the water level; at the maximum water level some sediment would be able to pass over the structure, while at lower water levels sediment would pass mainly around the head.

Five different rock groyne configurations were investigated. The first and second configurations retained the profiles of the simple sloping low timber groynes and the high end timber groynes, but included a 1:2 slope on the head (Figures 2.6a and 2.6b, Low Rock and High End Rock). The third had a similar profile to the high end groynes, but with the constant elevation section at the seaward end raised up by 2m, and shortened by 12m along the crest (Figure 2.7a, Short Rock Barrier). The fourth retained this crest elevation but extended the crest length back to 65m, which moved the toe out to 77m (Figure 2.7b, Long Rock Barrier). The fifth configuration comprised only the seaward portion of the groynes, which was modified to different lengths and elevations during the course of the test to determine whether a reduced structure could be effective in controlling beach response.

The rock groynes were designed to have nominal crest widths of 3m and side slopes of 1:2 running down to a toe elevation of about 1m below the mobile beach surface, or to the hard moulding. The rock grading replicated previous work (Reference 2) and was selected for minimal damage under the test wave conditions. The initial median rock weight was 6.55 tonne, with a linear grading from W_o = 4.0 tonne to W_{100} = 9.1 tonne. During construction larger rocks were preferentially placed in the areas of potential damage, based on the findings of earlier research (Reference 5).

Breakwaters
The detached breakwater layouts included variations in length, elevation, distance offshore and spacing. Most of the work concentrated on establishing a good data set for single breakwaters; spacing was only briefly investigated by installing a second structure for the final three tests of the programme. The basic structural design comprised a crest width of 4m, side slopes of 1:2 along the body of the structure and slopes of 1:3 at the ends. The rock sizes were the same as those used for the groynes tests, with larger rocks preferentially placed as an armour layer over the ends. Table 2.2 provides a summary of the structures tested; Figure 2.8 illustrates the basic structure used. Offshore distances were measured from the 3m water line of the initial 1:7.5 beach.

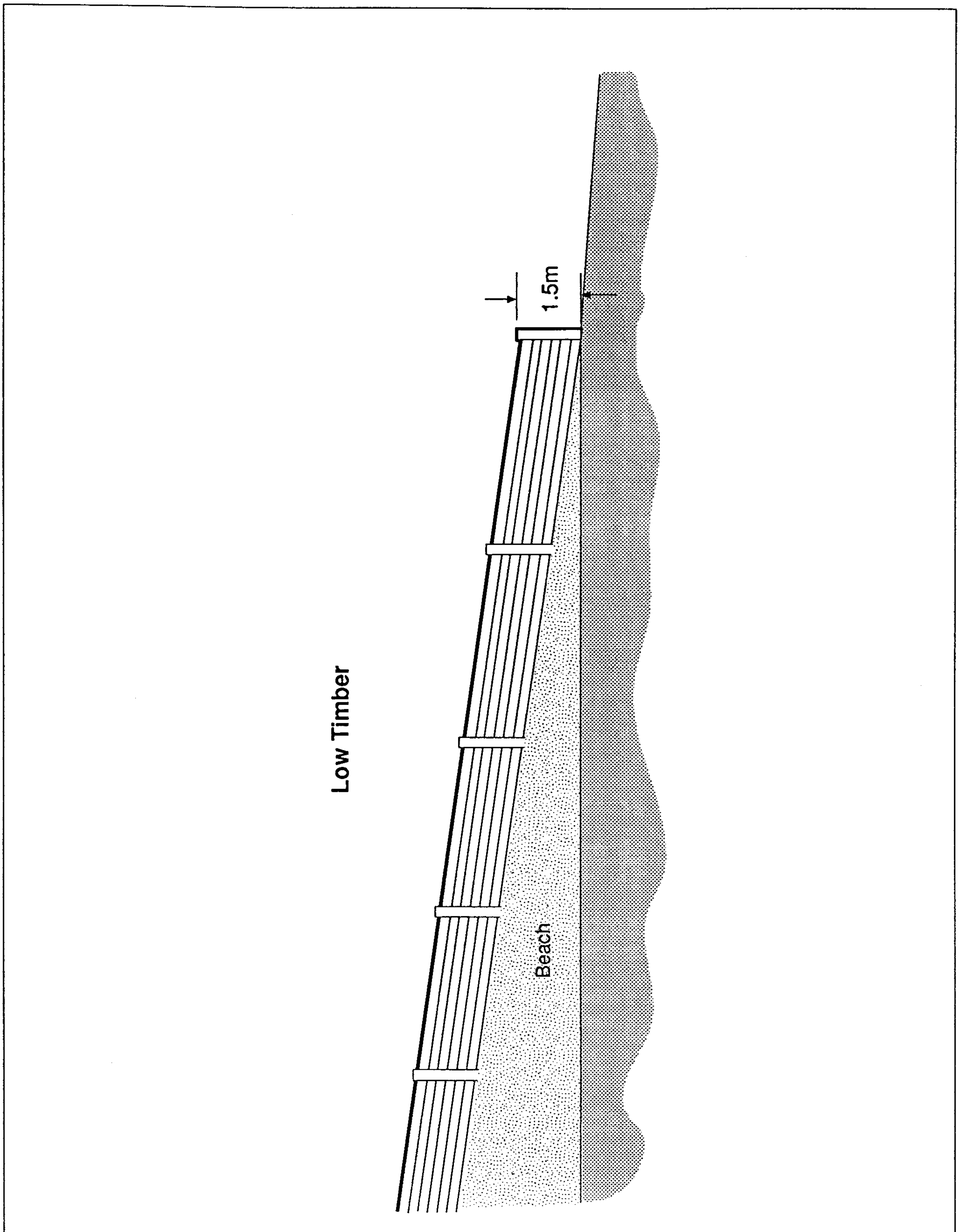


Figure 2.4 Groyne cross sections

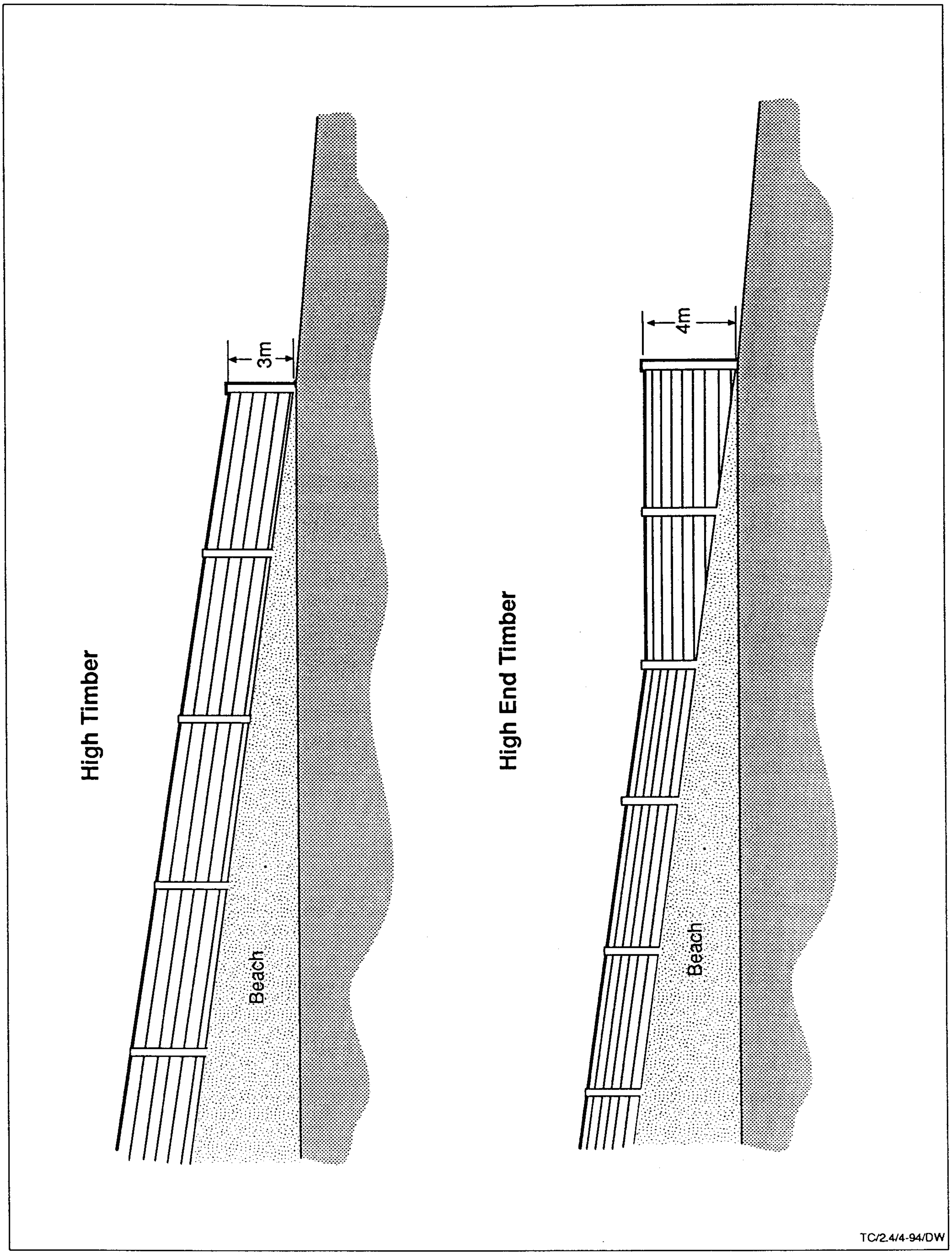


Figure 2.5 Groyne cross sections

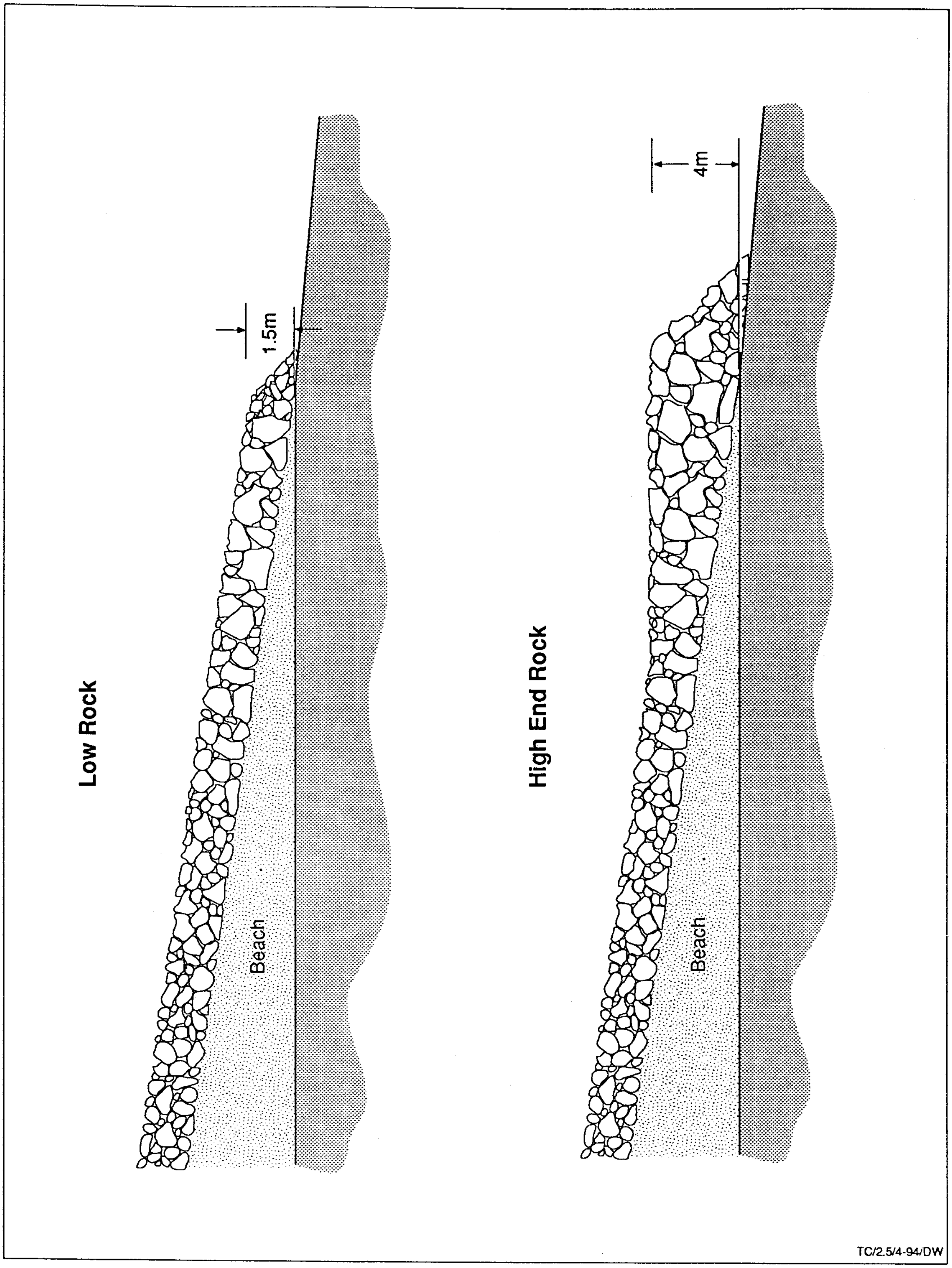


Figure 2.6 Groyne cross sections

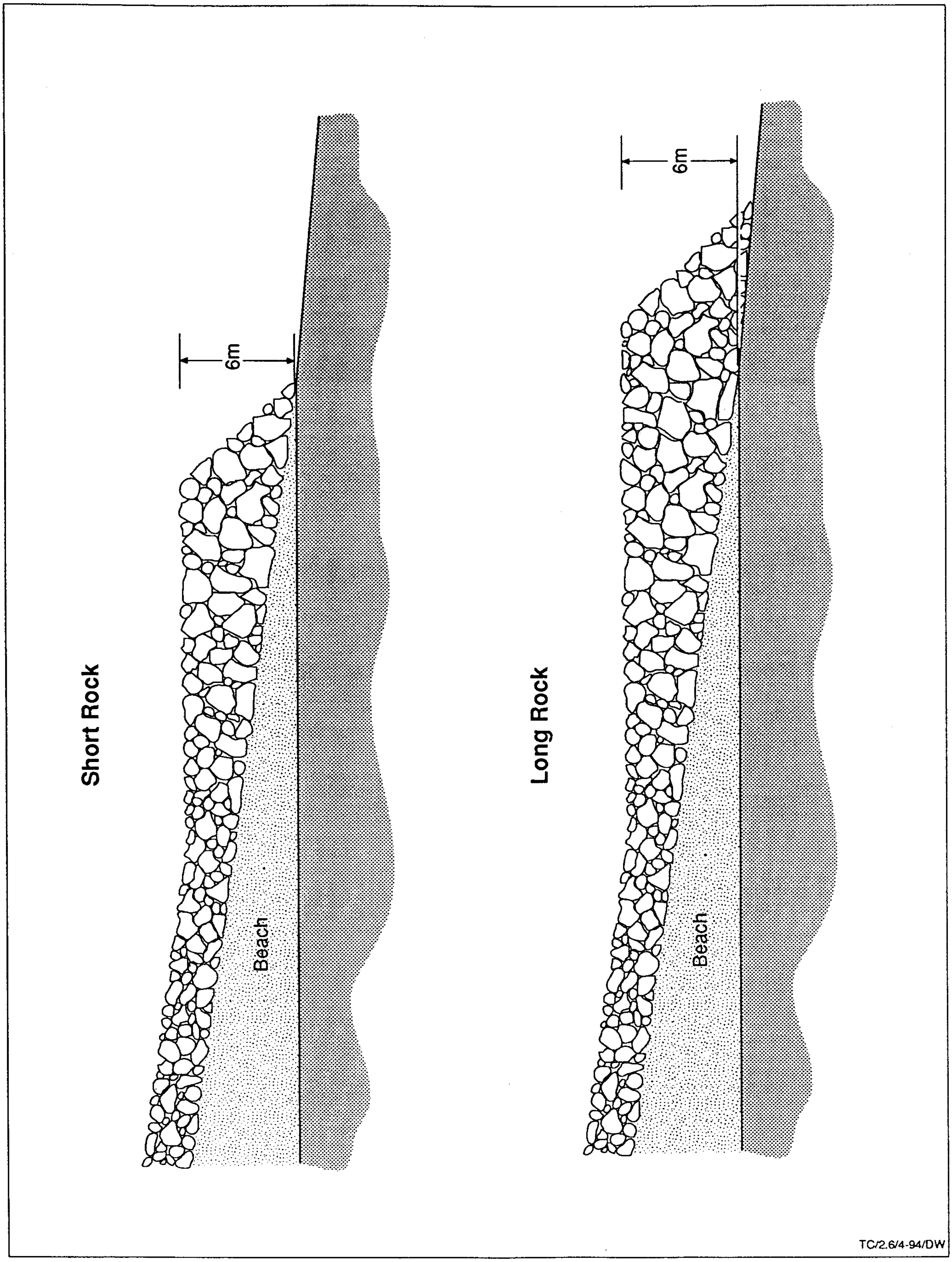


Figure 2.7 Groyne cross sections

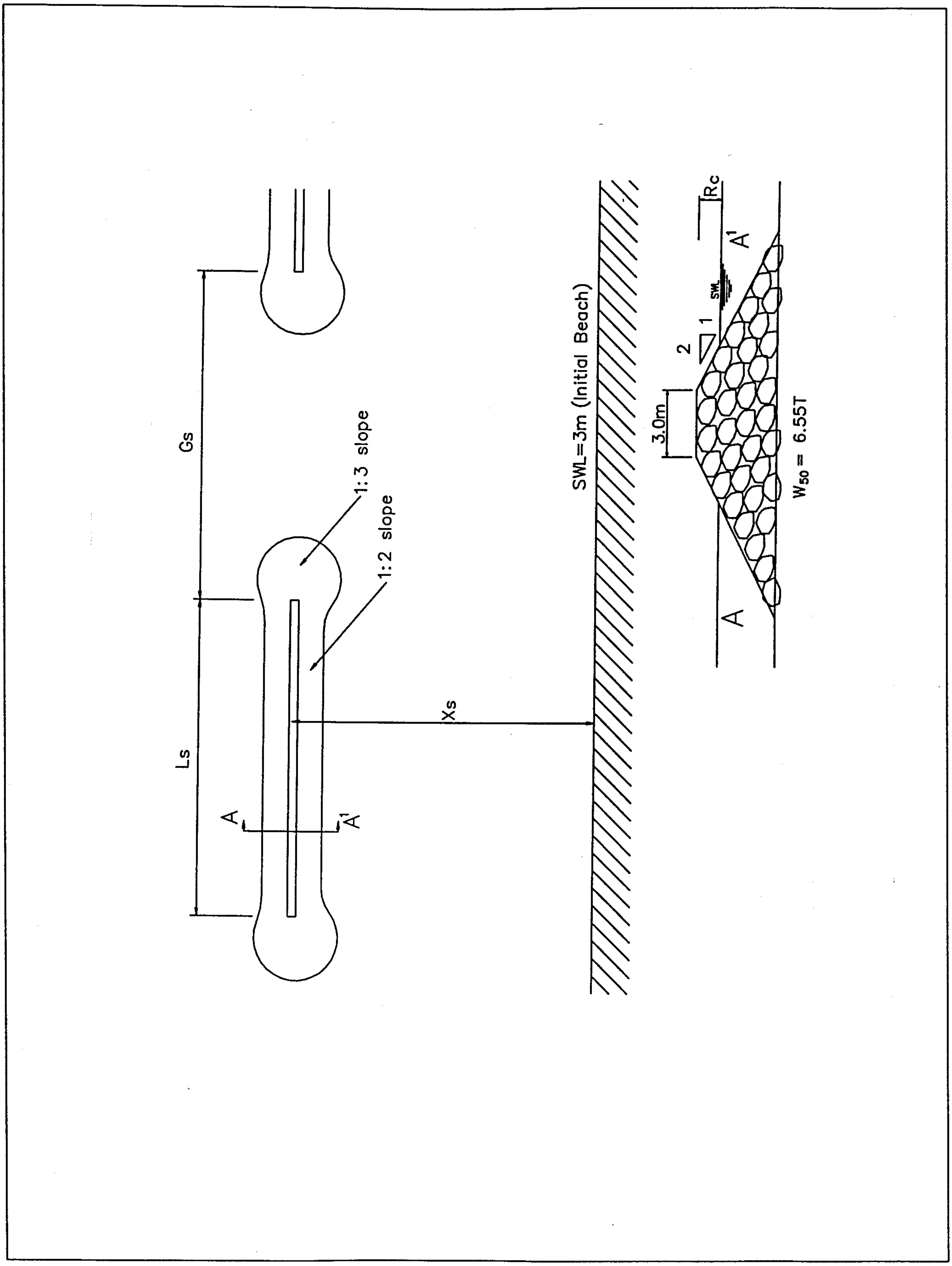


Figure 2.8 Basic breakwater layout

2.4 Data acquisition

2.4.1 *Wave conditions*

The waves were calibrated, and subsequently monitored, using eight twin wire wave probes which measure changes in water elevation by variations in conductivity. The probes were linked to a computer that was able to analyse the wave measurements spectrally or statistically. Three of the wave probes were placed in deep water in front of the paddle and the remaining five were placed 1m from the toe of the test beach (Figure 1). Measurements taken during testing were analysed statistically, while calibration measurements were analysed spectrally. Both methods determine the significant wave height (H_s) and mean wave period (T_m).

Inshore wave angles were measured along a shore parallel line on the outer edge of the surf zone. The simple manual method used proved to be consistent and gave the following results:

Offshore direction	Inshore direction
30°	23°
15°	13°

Cross-shore variations in wave height were also monitored during several tests by rearranging the inshore probes to lie along a shore normal line. Waves of H_s = 2m offshore shoaled up by about 135% at the seaward edge of the breaker zone under long period waves (S = 0.02) and decayed to about 85% under shorter period waves (S = 0.06).

2.4.2 *Beach profiles*

Beach profiles were measured at the end of each groyne test using a semi-automatic beach profiler which allowed the measurement of discrete elevations along up to 12 section lines within the test beach. The profiler has a vertically mounted probe attached to a potentiometer which records the vertical movement of the probe. The probe is automatically lowered until it rests lightly on the beach surface; the voltage of the vertical displacement potentiometer is then read and converted to a beach elevation relative to the 0m contour at the toe of the standard beach. The profiler probe is mounted on a computer controlled carriage which moves along a beam suspended above the beach. The system allows accurate and repeatable surveying of a model beach. The profile values are logged on the computer for subsequent analysis and plotting. The profile lines surveyed were always 7.5m (0.15m model) updrift or downdrift of the groynes and along the centre line of each groyne bay.

2.4.3 *Beach planshapes*

The planshape development of the beach was monitored at 2000 T_m, 4000 T_m and then at 4000 T_m intervals throughout each test. Measurements were taken of the cross-shore chainage of the beach crest, SWL and beach toe along up to 16 lines within the test beach. For the groyne tests the position of these lines was the same as the profile lines, but also included a variable line which allowed the position of greatest erosion within each groyne bay to be defined; this position is referred to as the pinch point (Figure 2.9) and the chainage is denoted as X_{pp}. For the breakwater tests lines were set out at 25m (0.5m model) intervals along the test beach.

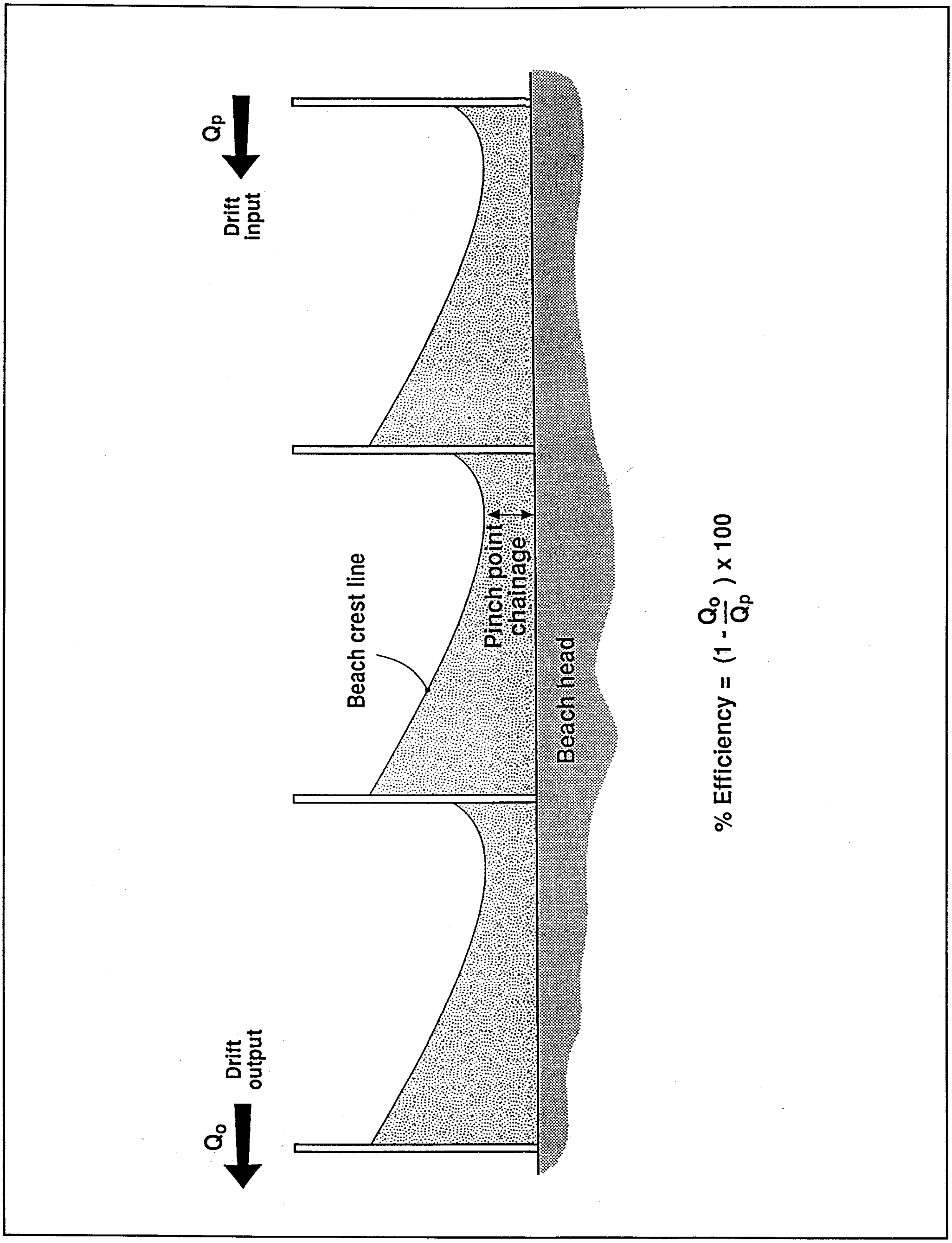


Figure 2.9 Pinch point and efficiency definition sketch

The beach planshape measurements were used to calculate and monitor the approximate volume of material within each groyne bay or to monitor the development of the salient in the lee of the breakwaters. Beach volumes for the groyne tests were calculated from the plan shapes using a simplified geometry for the profiles. The beach head was taken as a line 65m landward of the initial beach toe as this was the approximate position of the pinch point under the worst case conditions. Beach plan shapes for the breakwater tests were based on the position of the 3m still water line on initial beach; this line was 42.5m forward of the beach head used for the groynes and resulted in negative planshape values for eroded lengths of the model beach.

2.4.4 Photographs

Oblique angle photographs were taken from above the model at intervals throughout each test in order that the development of the test beach could be recorded. The photographs show the beach crest, SWL and toe.

Photographs were also used to record the final beach contours by lowering the SWL through 0.5m (prototype) increments and photographing each level. The SWL in each photograph of the groynes tests was digitized from a projected negative images with the intention of using the resulting contour plots during analysis of the different groyne types. However the 'fish-eye' distortions due to the camera lens could not be corrected with sufficient accuracy and therefore the photographs can only be used for subjective comparisons.

2.4.5 Sediment transport

Longshore transport through the model was monitored using a downdrift sediment trap. The trap was cleaned after every 1000 Tm throughout each test and the sediment weights were recorded.

The end point for the majority of the groyne tests was taken to be the point at which the measured output rate was in dynamic equilibrium with the input rate, indicating that the groynes were no longer influencing the gross sediment transport rate. The breakwater tests were stopped when a stable output rate was confirmed, after about $24000T_m$.

The term *structure efficiency* is used in this report to refer to the ratio of output drift to the potential open beach drift rate for the model beach under given sea conditions. It is denoted as η. When output is low then the structure has a high η value and vice versa. Figure 2.9 illustrates the concept for a groyned beach, but it is equally applicable to breakwaters.

The final cross-shore distribution of longshore transport over and around the groynes was also measured during some tests. Sediment was collected in a series of small, connected box traps. These traps were constructed from metal gauze with one open face. The open faces were placed along the line of one of the central groynes, from the beach crest to the beach toe, for periods of 2 minutes. The material collected was weighed and the position of each trap was noted. This process was repeated at least three times after the beach had reached stability in order to obtain a representative distribution under random wave conditions.

During tests of storm conditions a significant percentage of the beach material, particularly coarse material, was transported offshore from the beach. This material was collected and weighed at intervals throughout each relevant test.

The cross-shore distribution of sediment size was recorded at the conclusion of several tests. Surface samples from the crest, berm, and lower face, plus samples of offshore material, were taken for size analysis. In addition, random mixed samples of the beach material were collected and analysed during the course of the study to monitor any shift in the grading range due to degradation. The cross-shore distribution is discussed in Chapter 3. The mixed samples showed no significant change in the beach material over the test period.

2.4.6 Current tracking

Wave generated currents were monitored during some tests by injecting titanium dioxide dye. The dye paths give an indication of the potential transport paths for suspended sediments and the presence of rip currents.

2.5 Model calibration

2.5.1 Wave calibration

Waves were generated in the model facility using a 15m long electro-hydraulically driven paddle controlled by a micro-computer. The paddles were able to generate regular waves or random waves with a defined energy spectrum.

The generating system was calibrated at the outset of the programme to generate a range of random wave conditions at water levels of 2m, 3m and 4m. Several of these wave conditions were replicates of conditions used in the previous groynes research programme (Reference 2). Significant wave heights (H_s) included 1m, 2m and 3m. Wave steepnesses (S) included 0.02, 0.04 and 0.06; the 0.02 waves are referred to as "swell" conditions and short period 0.06 waves are referred to as "storm" conditions. Offshore wave directions (θ) included 15°, 30° and 45°. The full set of calibrated wave and water level conditions are presented in Table 2.3.

During calibration a short repeating sequence of waves, defined by spectral shape, was generated. The waves in the model were measured over the full sequence by the three offshore wave probes and were analysed spectrally over 16 frequency bands to determine H_s, T_m and the spectral shape. The output was compared with the required conditions. The generating system was adjusted and the analysis repeated until satisfactory wave conditions were achieved. Although the wave spectra were based on JONSWAP distribution of energy, the actual calibrated conditions were not adjusted to form a perfect match; earlier research (Reference 1) suggests that the details of spectral shape are less important to beach response than H_s and S.

Due to limitations on the test programme and practical problems encountered during testing only a limited number of the calibrated conditions were used. In particular, the 1m wave height and the 2m SWL were not used as drift rates were too low to provide meaningful results, and the 0.04 steepness was not used as the beaches appeared to remain unstable. Most of the tests were run with the 30° direction. No tests were run at 45° and only 5 groyne tests were run at 15°.

2.5.2 *Transport calibration*

One of the objectives of this research was to compare the rate and cross-shore distribution of longshore transport on a controlled beach with the transport on an open beach. Following calibration of the wave conditions, the rate and cross-shore distribution of longshore transport for each condition was measured under an open beach situation. The transport rates were subsequently used as the sediment input rate during tests of the various groyne and breakwater layouts.

The procedure for each calibration was as follows:

- mould the standard beach;
- run the calibrated wave condition for a period of $1500T_m$ to allow the cross-shore profile to reach equilibrium;
- add new material continuously to the updrift end to ensure that no points of erosion developed;
- stop the waves and clear the downdrift sediment trap (divided into 7.5m compartments to measure cross-shore distribution);
- run the waves for 3, or more, periods of $500T_m$ while adding new material updrift and measuring the downdrift output;
- take an average of the total output over the 3 periods to determine the potential open beach transport rate for each wave condition;
- take an average of the output within each of the trap compartments to determine the cross-shore distribution of transport.

During the test programme a hair lock mattress was laid over several metres of the beach at the updrift end to provide a constant drift input surface. It was separated from the test section of the beach by a low timber insert. The calibrated volumes of beach material that were fed into the model were placed onto the mattress at regular intervals throughout each test. Figure 2.1 illustrates the layout.

Midway through testing the operators became concerned that the calibrated rates were not high enough, and further calibration runs were completed. These runs used 3 test periods of $2000T_m$, and resulted in slightly higher transport rates. However the differences were not considered to be sufficient to alter the standard test procedures during the groyne tests. The new rates were used for the breakwaters tests and for part of the analysis of the groyne test results.

The calibrated transport rates for each wave condition are set out in Table 2.3 and are expressed as kg/$1000T_m$ (model). The rates for the 1m H_s conditions were negligible, while the rates for some of the 3m H_s conditions were too large for the model operators to measure.

2.6 Test programme

The test programmes for the groyne and breakwater studies are presented in Tables 2.4 to 2.8 and are described in the introductions to Chapters 3 and 4. The tables indicate the test numbers, the structure configurations, the seastates, the sediment input rates and the final durations. The actual wave heights (H_s) recorded in Tables 2.4 to 2.6 indicate the mean values recorded at intervals throughout the tests at the three offshore probe positions. Tests for which the actual heights differed significantly from the required ($\geq \pm 5\%$) are indicated.

2.7 Test procedures - Groynes

A standard procedure was developed for each test to ensure compatibility of results.

At the outset the appropriate groynes were placed along the test frontage, the standard beach was moulded and the wave basin was filled to the required SWL. The calibrated waves were then started and the calibrated sediment input was added. After a time period equivalent to 500 T_m the waves were stopped and an overhead photograph was taken to record the beach planshape development. The waves were then restarted. This procedure was then repeated after 1000T_m, 1500T_m and 2000T_m, and after every further 2000T_m to the end of the test. Planshape measurements were taken in conjunction with the photographs after 2000T_m and 4000T_m, and after every further 4000T_m. Drift material was collected from the sediment traps after 500T_m and 1000T_m, and after every further 1000T_m; the waves were not stopped for those periods which did not coincide with the taking of photographs. The cross-shore distribution of transport at one of the central groynes was measured during some of the tests; the measurements were taken after the appropriate groyne bays had reached their stable volumes. Titanium dioxide dye was injected into the surf zone during several of the early tests to observe wave induced currents. At the end of each test, profiles were measured along 12 section lines and the basin was drained down in 0.5m steps to allow photographs to be taken of the water line as a means of recording the beach contours; these photographs are not presented in this report.

Most of the groyne tests were run with an updrift input of beach material at the rate established during the open beach calibrations. The material was fed into the updrift end of the model beach at intervals of 1000T_m. The tests were stopped when the output drift rate equalled the input rate, indicating that the groynes were no longer influencing transport and the value of η was near zero.

Some of the groyne layouts were also tested with zero input. These tests commenced with the groyne bays full, as they were at the end of the normal tests. The beach was then allowed to erode until η had increased to over 80%. The rate of increase in η reduced at about this level; extending the tests to reach zero drift (η=100%) would have been very costly for little additional benefit.

2.8 Test procedures - Breakwaters

The procedure for the breakwater tests was similar to that used for the groynes. Photographs, planshapes and output drift collection were all taken in the same sequence. The cross-shore distribution of transport was not monitored and no profiles were taken at the conclusion of the tests.

The major procedural difference was the method of feeding the beach at the updrift end. Over the first 12-20,000 T_m beach material was input as for the groynes. During this initial period the output drift rate was monitored and a value for η for the structure and the sea condition was determined. Once η had stabilized then the hair lock mattress, used as the drift input surface, was extended to cover all of the updrift part of the beach that was not influenced by the breakwater. The input rate was then reduced to the level of the output and the test was resumed. Assuming that the input rate had been chosen correctly then the length of the beach directly affected by the structure was expected to remain stable. A further 12,000 T_m were run to confirm that stability had been achieved. No tests were run with a zero input.

At the conclusion of each test, contour photographs were taken using the method described for groynes.

3 Discussion of results - Groynes

3.1 Introduction

Groynes have been used extensively around the UK coast and in many other parts of the world. An extensive study carried out under the direction of CIRIA (References 6 & 7) reviewed the existing understanding of groyne design, undertook a physical model study and attempted to obtain field measurements. The study resulted in the publication of "Guide in the uses of groynes in coastal engineering" (Reference 7). Amongst the general conclusions of this study was the acceptance of short groynes as a means of controlling shingle beaches and the recognition that design methods were still based on experience and engineering judgement rather than on well established analytical methods.

Since the completion of the CIRIA study in 1990 some progress has been made in refining design methods for groynes on shingle beaches. Work undertaken by HR Wallingford has resulted in the development of numerical models which can be used to predict open beach response both in the cross-shore and longshore directions (References 1 and 2). A model has also been developed that predicts beach response in the presence of simple sloping groynes (References 3 and 4), however it is based on assumptions that have not been verified against field data and it is not capable of predicting beach response in the presence of more complex groynes.

The aim of the multiple groynes research programme presented in this report was to extend the existing guidelines for groyne design in conjunction with beach recharge schemes. The approach adopted was to investigate the influence of a number of structural variables on the performance of a standard beach when subjected to a range of wave and water level conditions. The structural variables included :

- spacing (S_g)
- length (L_g)
- head elevation (h_{cg})
- crest profile shape
- construction materials

(L_g is taken as the length from the toe to the line of maximum cut back, referred to as the beach head - Figure 2.9)

The sea condition variables included :

- wave height (H_s)
- wave steepness (S)
- wave angle (θ)
- water level (SWL)

This section presents and discusses the data obtained and its limitations with respect to design. Conclusions, recommendations for further work and design guidance are presented in the final chapters.

The first series of tests (Table 2.4) concentrated on low sloping timber groynes (Figure 2.4) in order to build up a base of data against which other groyne configurations could be compared. The low timber groynes were selected for these tests as they are both the most simple and the most common groyne form found along the UK coast. The only structural variable investigated during this series was groyne spacing.

During the course of this test series a number of wave and water level conditions were investigated. Wave parameters included heights (H_s) of 2m and 3m with steepness of 0.06 and 0.02. Water levels included depths of 2m, 3m and 4m as measured at the toe of the standard mobile beach. Wave direction was restricted to 30° (offshore) and all tests used the full calibration sediment input. The relationships investigated included the effect of these forcing conditions on beach development, groyne efficiency, cross-groyne transport and beach orientation.

The second series of tests (Table 2.5) investigated other structural variables including the use of rock, groyne elevation, length and the effect of eliminating the landward end of the structure. These tests concentrated on a limited sub-set of wave and water level conditions, including a wave height of 2m (H_s), steepnesses of 0.06 and 0.02 and a water level of 4m. Only one configuration was also tested at the 3m water level. All tests were run with a wave direction of 30° (offshore).

A third series of tests (Table 2.6) investigated the influence of wave direction on several groyne configurations, including low timber. The tests were run using an offshore wave direction of 15°. It had been intended that a further direction of 45° should be investigated, however time was not available.

3.2 Presentation of results

The results of this work are discussed in the following sections. The terms groyne efficiency and pinch point are used extensively; they are defined in Section 2.4 and in Figure 2.9, and are denoted by η and X_{pp} respectively. The important results are presented within the text along with the associated figures and tables. Additional supporting figures showing all of the measured beach profiles, the groyne efficiency curves and the cross-shore distributions of transport are presented in Appendices 2, 3 and 4. Photographs of the final beach responses are presented in Appendix 5.

The beach profile plots presented in Appendix 2 contain all the profiles for each test. The updrift and downdrift profiles from three groyne bays are plotted against the appropriate groyne profile. The tests are generally paired to allow comparison of the swell (S = 0.02) and storm (S = 0.06) profiles for each layout. In some cases either the swell or storm conditions were not run, in which case the pairing is based on contrasting different groyne spacing (ie Tests 17 and 20), different sediment input rates (ie Tests 25 and 26) or different groyne configurations (ie Tests 33 and 34.3 or Tests 42 and 43).

Most tests started with the beach remoulded to the standard initial profile with a slope of 1:7.5, however several storm tests were run on from swell tests without remoulding. In these cases the upper beach profile was partly formed by the relict profile of the previous test. This applies to Tests 32.2, 32.4, 34.2 and 34.3; the relict portions of these profiles have been edited for the analysis of profile areas. The plots for the 15° offshore wave direction tests (Tests 37 - 40) have only 2 sets of profiles. The inshore wave conditions were not consistent across the model face and an area of low wave energy developed at the downdrift end of the mobile beach, causing uncharacteristic accretion. The affected profiles are therefore not presented.

During the analysis and interpretation of the profile data only the profiles from the central groyne bay of the mobile beach are used. It is hoped that this selective analysis has ensured that any longshore model effects are minimized and consistent results are produced.

The efficiency curves for the groynes tests are presented in Appendix 3. These curves relate the drift output to the calibrated input over time. In most cases η started at 100% and then decreased over time, to eventually reach zero as the initial beach profile developed to its final form. Several tests did not start from the standard initial profile, but ran on from a previous test, and therefore the curves do not fit the general pattern. This applies to the storm condition Tests 32.2, 32.4 and 34.2 which ran on from the previous swell conditions. It also applies to the zero input tests (26, 33, 34.3, 42 and 43) which all commenced with a fully developed beach and ran on until η had risen to about 80%.

Appendix 4 contains the cross-shore transport distribution plots. This information was collected for 12 tests of the three timber layouts. Information on the rock layouts can be inferred from recorded observations and from the timber groyne data. The plots relate the cross-shore transport distribution to the updrift profile of the central groyne bay, and to the open beach transport distribution measured during calibration. The transport distribution curves are the mean of at least three sets of measurements. These were subject to wide variations, but still provide a good indication of the trends. The open beach distributions are fixed according to the position of the beach crest updrift of the groyne.

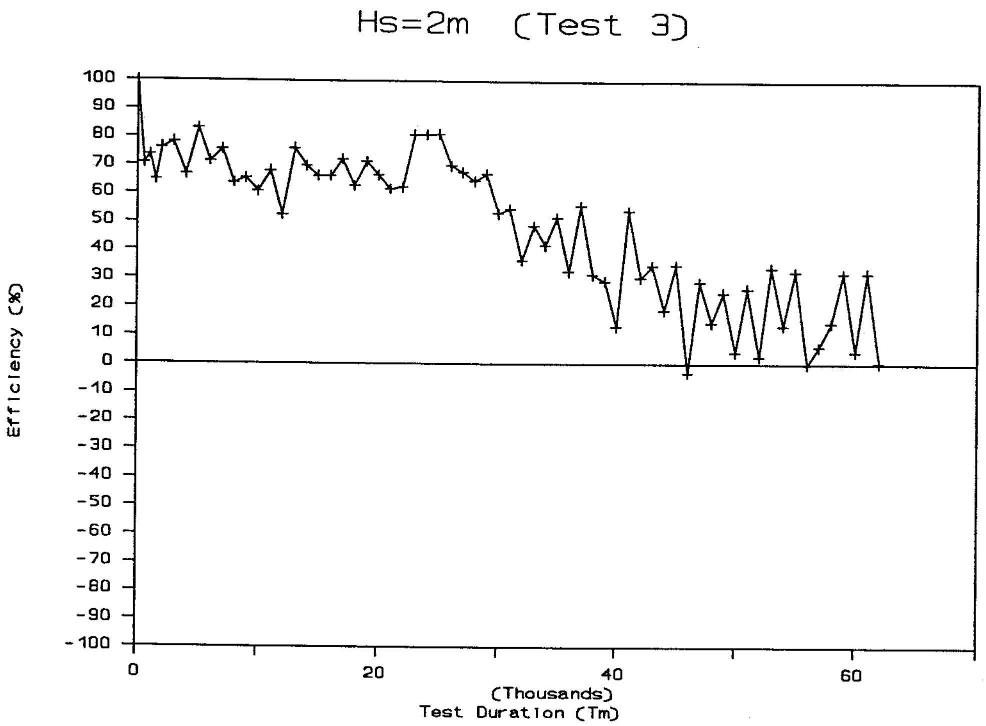
End of test overhead photographs are presented in Appendix 5. They are arranged in the same order as the profile plots to allow comparison of related tests. The photos show clearly the crest, SWL and toe of the beach.

3.3 Series 1 tests

3.3.1 Effect of wave conditions

Wave height
Wave conditions with H_s values of 1m, 2m and 3m were calibrated for the test programme. The 1m conditions were not used during testing as the drift rates were too low to be measured consistently. The majority of tests were run with a 2m H_s, with the exceptions being Tests 17 and 20 that were run with a 3m H_s. A 2m H_s was used for all Series 2 and 3 tests.

During the 3m H_s tests the groynes appeared to be ineffective in terms of their influence on beach response. η dropped rapidly to below 50% then decreased to zero, and there was no updrift development of beach crest. These observations contrast sharply with the comparable 2m H_s conditions for which η remained at 70% for some time before decreasing to zero, and the final



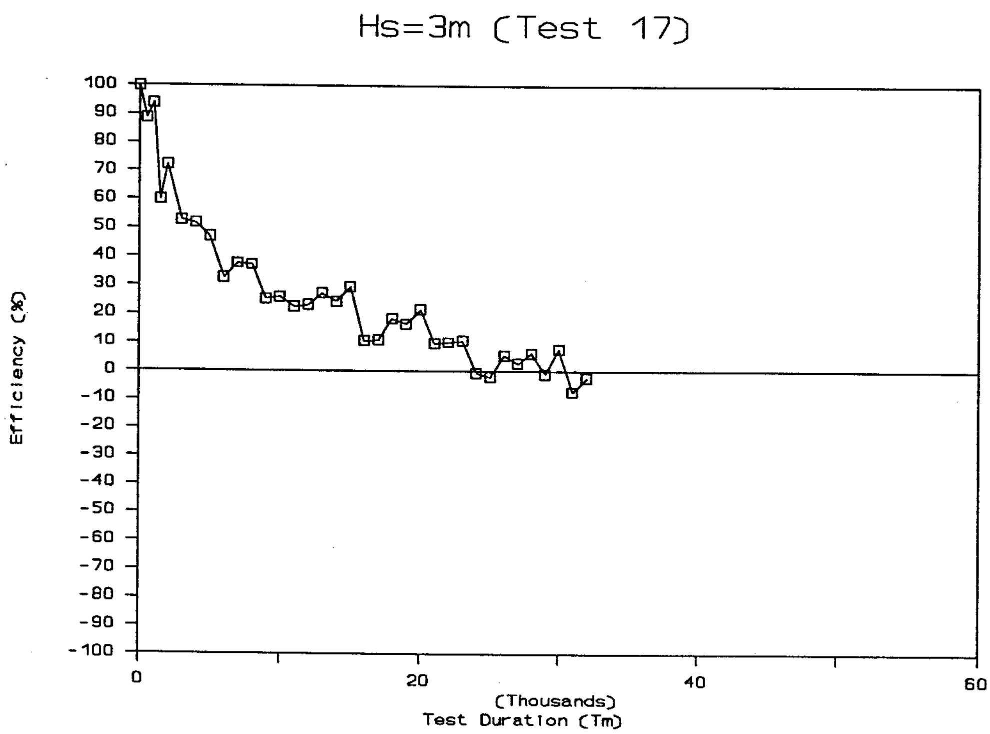


Figure 3.1 **Comparison of groyne efficiency curves for 2m and 3m H_s waves conditions**

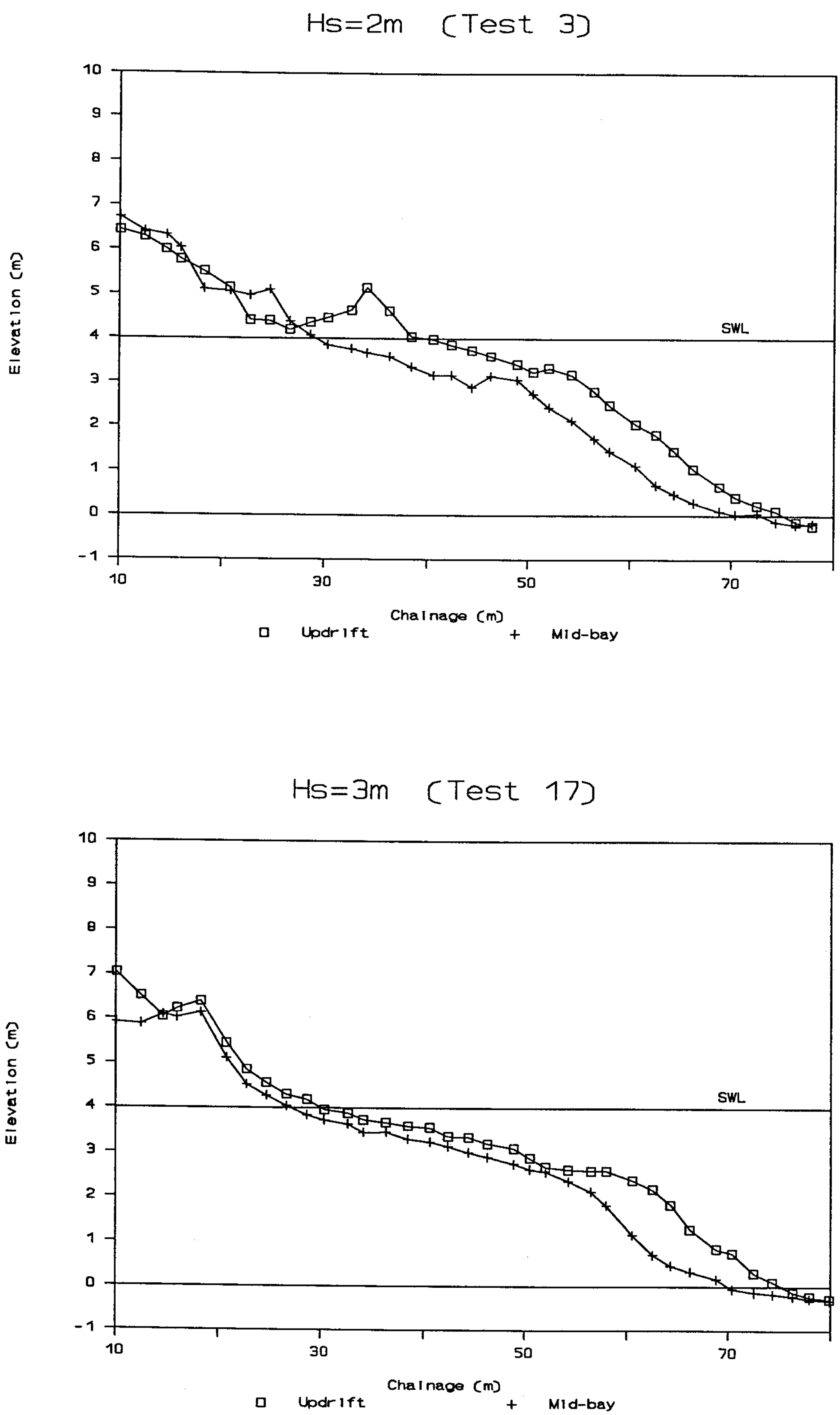


Figure 3.2 **Comparision of mid-bay and updrift profiles for 2m and 3m H_s wave conditions**

profiles showed a substantial updrift beach crest development relative to the mid-bay. Figures 3.1 and 3.2 contrast the efficiency curves and the updrift and mid bay profiles for Test 3 (2m H_s) and Test 17 (3m H_s).

The lack of updrift crest development for the 3m H_s tests was a result of localized scour along the groyne face. This response replicated results obtained during previous groyne research at HR Wallingford (Reference 2) during which the low timber groynes were compared with equivalent low rock groynes and found to be much less effective at retaining an updrift beach during severe conditions.

The cross-shore distribution of transport for the 3m H_s condition was found to have shifted offshore relative to the open beach distribution and relative to 2m H_s condition; the 2m H_s distribution also shifted offshore, but not significantly in relation to the confidence limits of the measurement method. The shift of the 3m H_s distribution was a result of the wave interactions with the upper groyne face which prevented updrift beach build up. The lower beach did build out (Figure 3.2) allowing transport to occur at the calibrated open beach rate. Figure 3.3 contrasts the cross-shore distribution of transport for the 2m and 3m H_s conditions using Test 22 (2m H_s) and Test 20 (3m H_s). Plates A11 and A12 in Appendix 5 show the planshape of the 3m H_s tests.

3.3.2 Wave steepness

The low timber groynes were tested with 2m H_s waves at two steepnesses - 0.06 (storm) and 0.02 (swell). The conditions caused differences in the profiles, planshapes and efficiencies. Previous research has established the expected profile response for the long and short period waves (Reference 1). Longer waves cause greater run up, and therefore higher crests, than shorter waves, as profile Figures A1-A15 in Appendix 1 illustrate. Observations of beach development at the commencement of each test indicate that under the storm conditions the beach immediately updrift of the groynes suffered localized scour that was subsequently obscured by accretion. This scour did not occur with the swell waves.

The beach planshapes differed due to the beach crest orientation between the updrift and mid-bay profile lines. Figure 3.4 presents these differences. The results are scattered but show a mean increase of about 3° between swell and storm conditions, due to differences in wave refraction. The efficiency curves for comparable swell and storm conditions (Figures A1-A5: Appendix 3) show similar patterns, but η remained higher for storm conditions. Under the storm conditions, η tended not to reach zero as some of the input material was drawn offshore rather than being deposited in the downdrift sediment traps. Measurement and analysis of the offshore material indicated that up to 10% of the input material could be lost offshore, and that it was always very coarse. It is not clear whether this loss is a model effect or a phenomenon that can be observed along shingle beaches; there is some field evidence that supports the model observations, but as yet this is inconclusive.

3.3.3 Effect of water levels

Reductions in the SWL from 4m through 3m to 2m caused a lowering of the active beach profile, and a reduction in the drift rate as the breaker zone moved seaward to the non-mobile model bed. However, the position of the toe of the mobile beach and pattern of cross-shore transport distribution remained essentially unchanged. Figure 3.5 presents the storm condition response for a 4m and 3m water level and illustrates the limited influence of

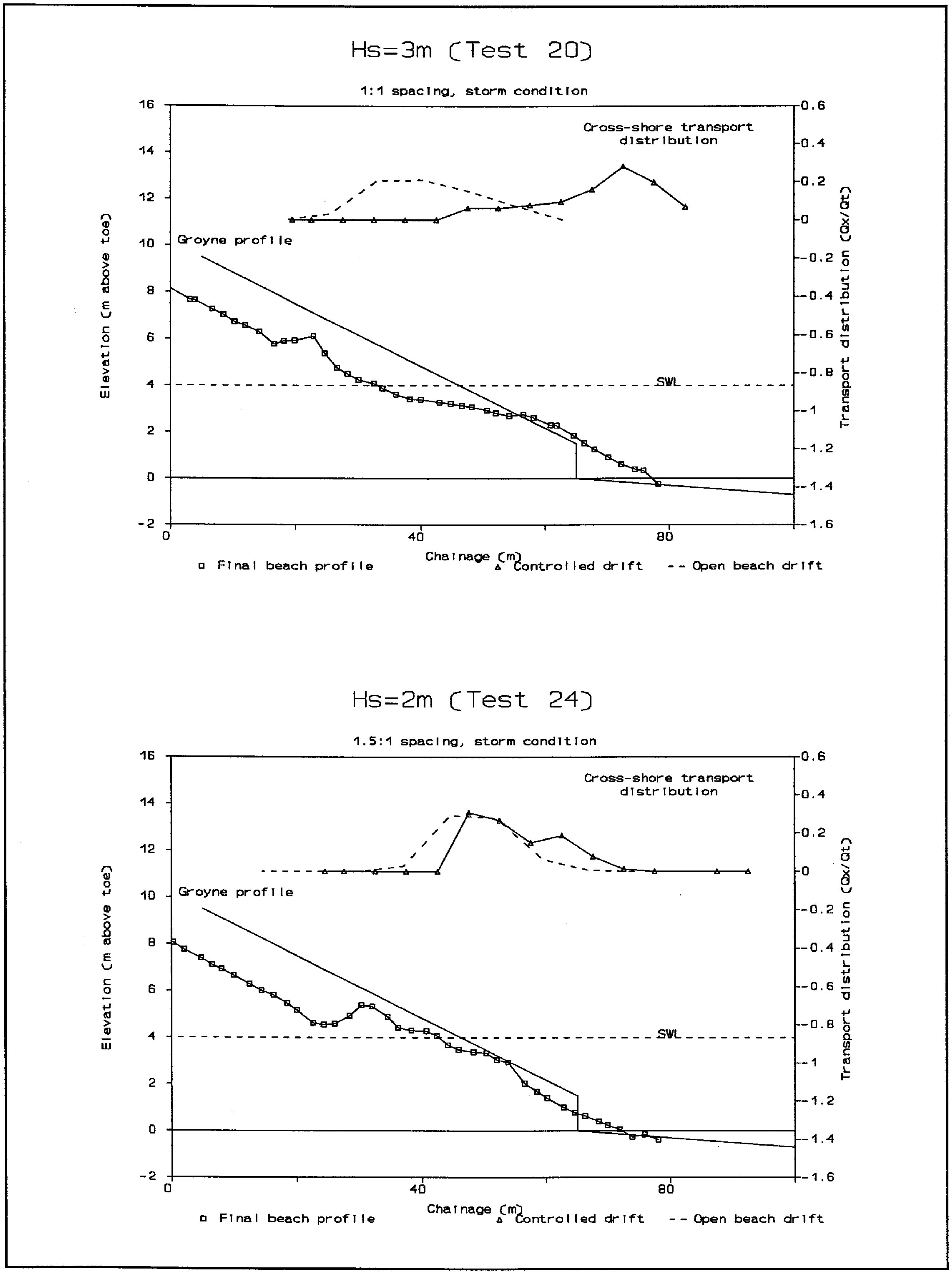


Figure 3.3 Comparison of cross-shore transport distributions for 2m and 3m H_s waves conditions

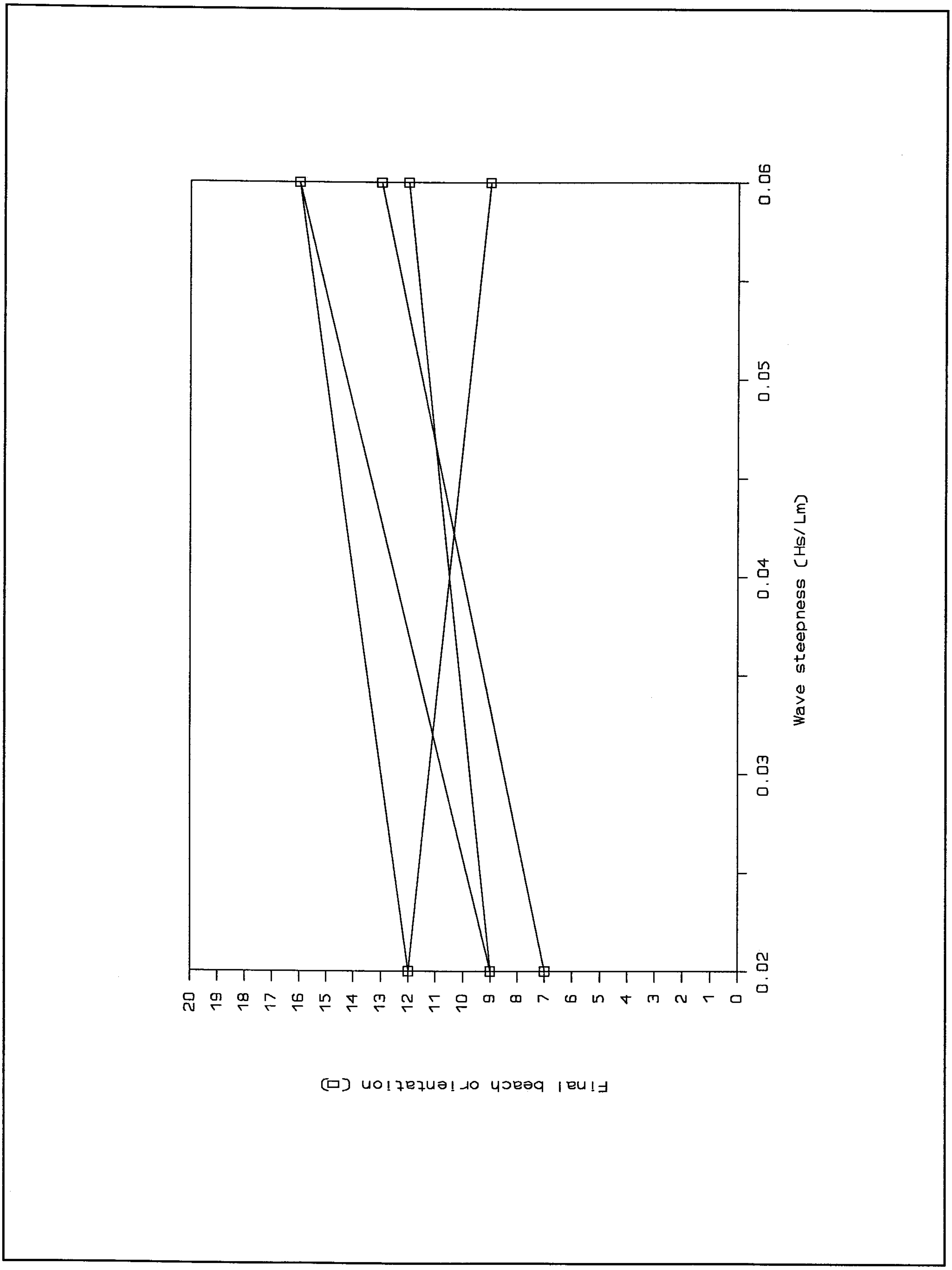


Figure 3.4 Beach orientations relative to wave steepness for Series 1 tests

water depth on the transportation distribution and the beach toe. This lack of impact is important in the design of the groyne systems at meso and macrotidal locations as it suggests that groynes designed to optimize beach response for high water levels will perform well at lower water levels.

3.3.4 *Effect of groyne spacing*

Groyne spacing was found to have a linear effect on pinch point chainage, and therefore beach volumes, for the fully accreted beaches. Figure 3.6 presents the measured data for the Series 1 tests. These data suggest the linear relationship:

$\Delta X_{pp} = (0.14 \Delta S_g)$

where ΔX_{pp} = change in pinch point chainage

and ΔS_g = change in groyne spacing

This relationship gives a beach orientation of about 8° from the updrift profile to the pinch point of each bay regardless of groyne spacing, subject to the calibrated input rate and a 30° offshore wave direction. Beach orientation is investigated further in a later section.

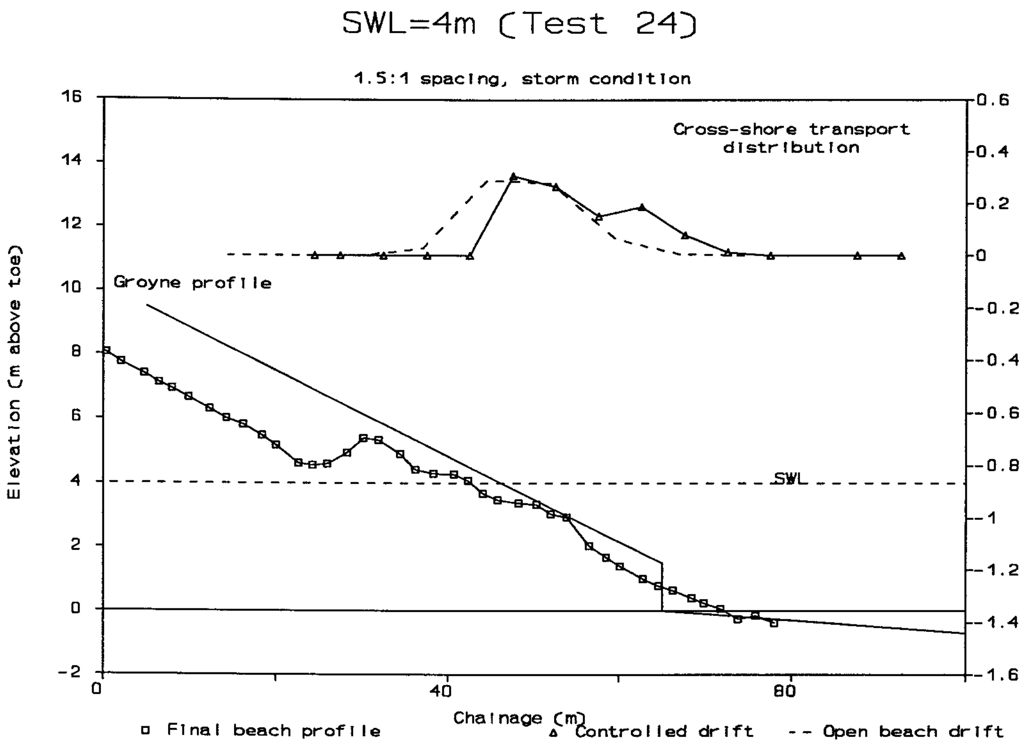
Groyne spacing had no noticeable effect on profile shape, but did affect the efficiency curves. Spacings of twice the effective groyne length had lower initial η values, which decreased gradually towards zero, compared to the 1:1 spacings that gave high initial values which then dropped very rapidly to zero when the groyne bays approached their final volumes (Figures A1 - A5 in Appendix 3). Later discussions suggest that groynes that produce these steep efficiency curves are more effective than those which produce shallow curves.

3.4 Results of series 2 tests

3.4.1 *Effect of construction materials*

Rubble mound groynes formed of 4T - 9T rock were compared to equivalent vertical timber groynes to determine any significant differences in beach response under 2m H_s conditions. The comparable configurations were low and high end groynes. The rock and timber structures had similar crest profiles except at the heads; the rock groynes sloped down to the hard moulding at 1:2, while the timber groynes had a vertical end. Figures 3.7 and 3.8 contrast the updrift beach profiles for the two groyne types under both swell and storm conditions. The low rock groynes caused the beach profiles to shift seaward by about 2m relative to the low timber groynes, while the high end rock groynes caused a shift of about 7m under the swell condition but no shift under the storm condition. The differences can be attributed to two factors, apart from the potential variability of the profiles. The first is the reduction in local turbulence which results from the rock groynes having a greater capacity to absorb energy, while the second is the greater length of the rock groynes due to the sloping seaward ends.

Earlier work on groyned beach response (Reference 2) investigated low timber and rock groynes under 3m H_s storm waves. This work suggested that the difference between rock and timber became much greater as wave energy increased, with rock groynes allowing the beach crest to remain much more stable.



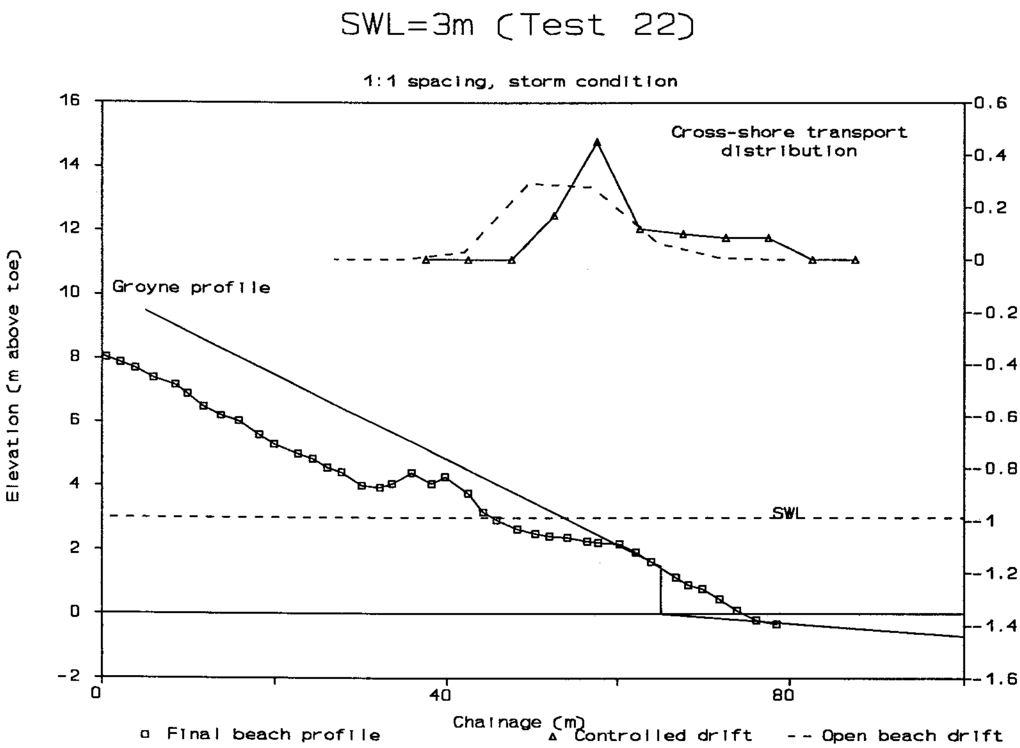


Figure 3.5 Comparison of cross-shore transport distribution for 4m and 3m water levels

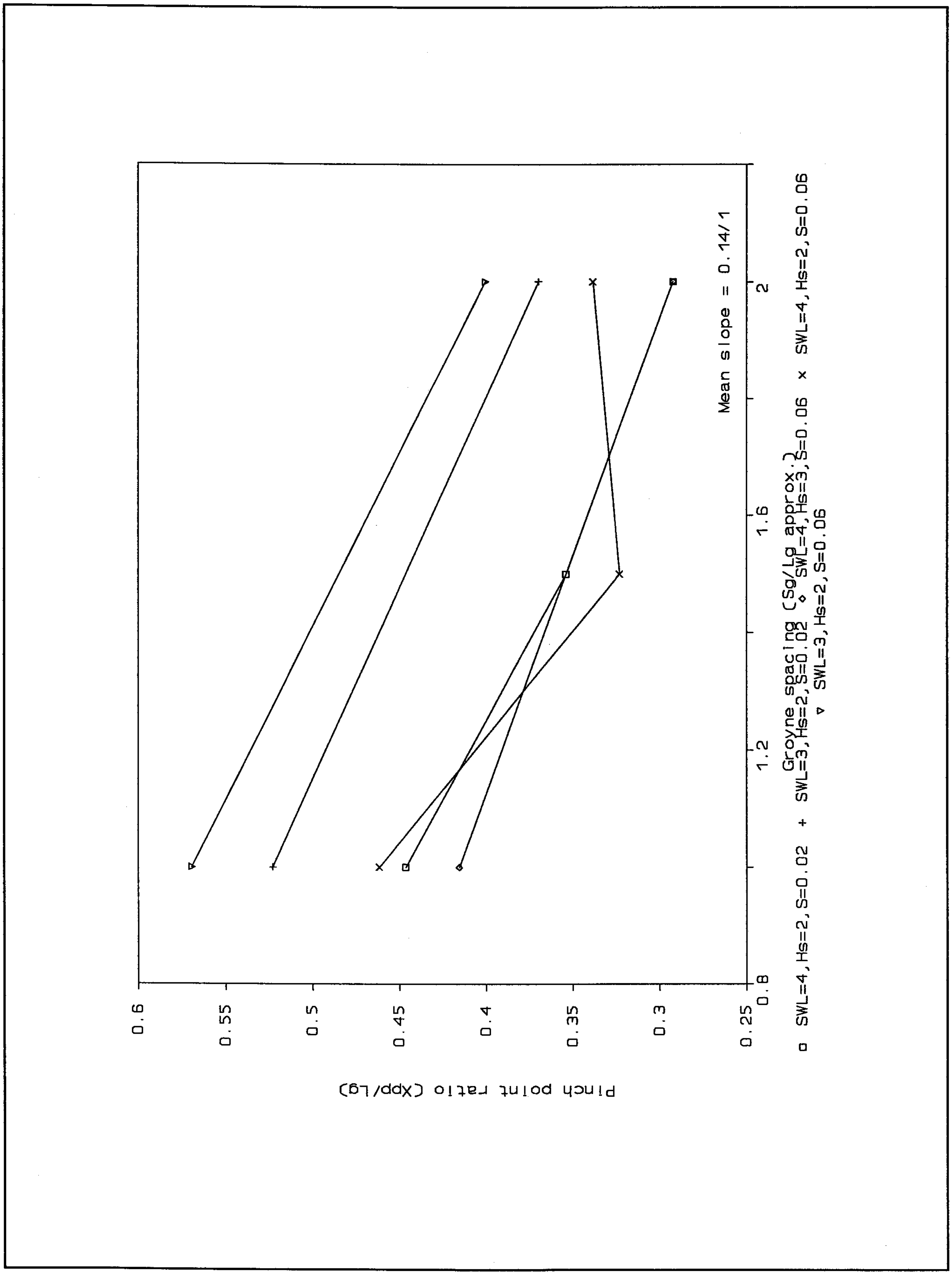


Figure 3.6 Influence of groyne spacing on pinch point chainage for Series 1 tests

Rock groynes have additional advantages that have not been investigated by this study. Groynes are subject to the destructive force of wave impacts, wave and current induced scour, abrasion by suspended shingle, and large lateral loading differentials due to the build up of beach material on their updrift side. Well designed rock groynes are better able to absorb these destructive forces than timber groynes, particularly over long time periods. Even if damage does occur rock groynes are less likely to suffer complete failure. Recent work at HR Wallingford addresses the probabilistic design of rock groynes on steep beaches (Reference 5).

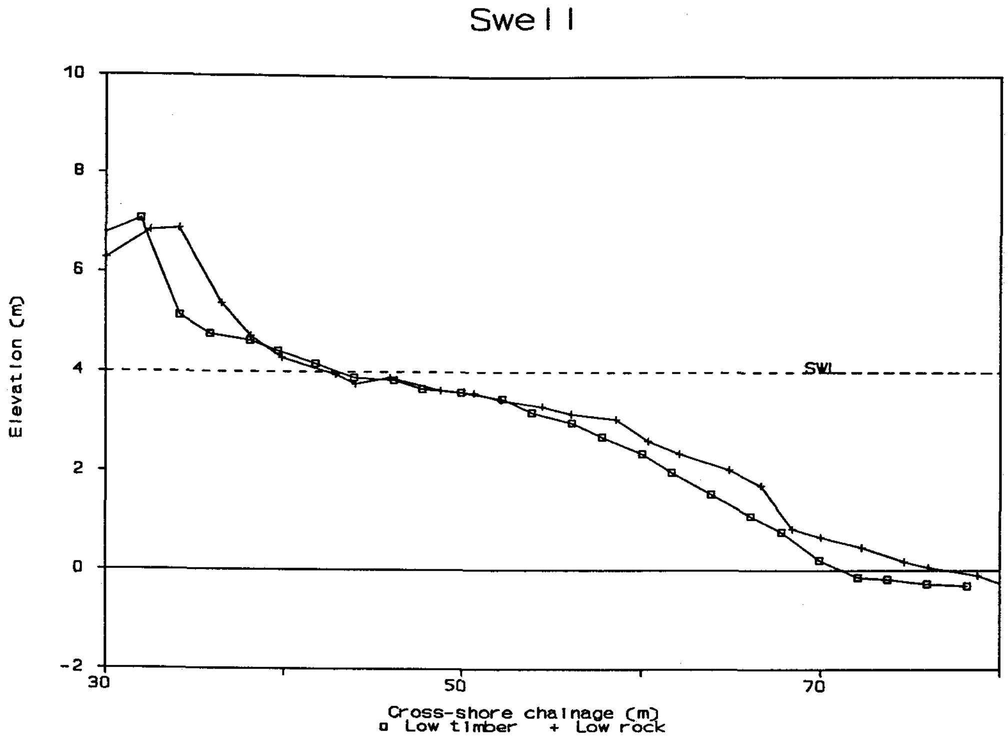
These results and observations clearly support the effectiveness of rock groynes in relation to beach response. In practice other considerations such as cost, construction methods, availability of materials, public acceptability and public safety may influence design decisions. Further useful information on rock structures can be found in the CIRIA/CUR "Manual on the use of rock in coastal and shoreline engineering" (Reference 8).

3.4.2 Effect of groyne dimensions

Traditionally groyne length has been taken as an important parameter in groyne design. However, the simple physical length is not always relevant as part of the groyne may be ineffective in relation to beach response. The following discussions suggest that it is more important to use the effective area of a groyne as a design parameter.

During the investigation of beach response to variations in groyne dimensions initial effort was concentrated on the beach pinch point chainage (X_{pp}) as the main dependant variable due to its importance in defining the success of a groyne system. This approach was useful to some extent but inconsistencies in the pinch point chainage made the results inconclusive. Efforts were then switched to the use of the updrift beach profile as the dependant variable, with more positive results.

Initially the updrift beach profiles for each of the Series 2 groyne types were compared with the Series 1 profiles for the appropriate groyne spacings, wave conditions and water depths. This process revealed that the profiles could be split into two parts, upper and lower, with the division being at the transition point as defined by previous beach profile research (Reference 1) and illustrated as point P_t in Figure 3.9. The lower part of the profiles had a consistent shape for every groyne type with the only difference being the cross-shore position. The upper part of the profiles had distinct differences in shape.



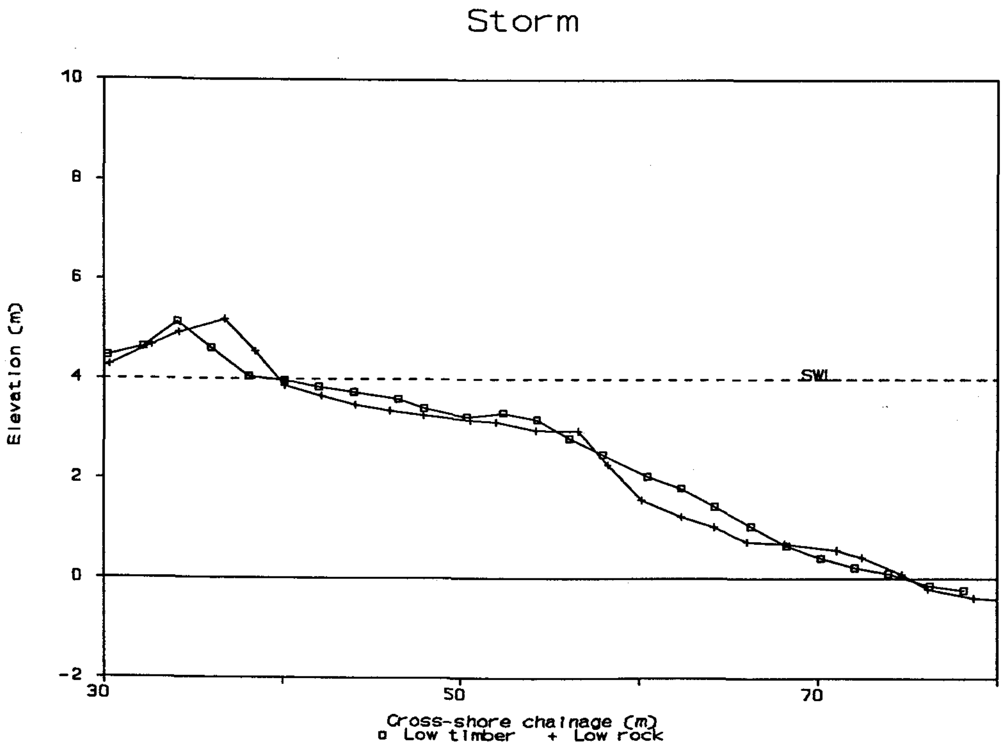


Figure 3.7 Comparison of updrift profiles for low timber and rock groynes

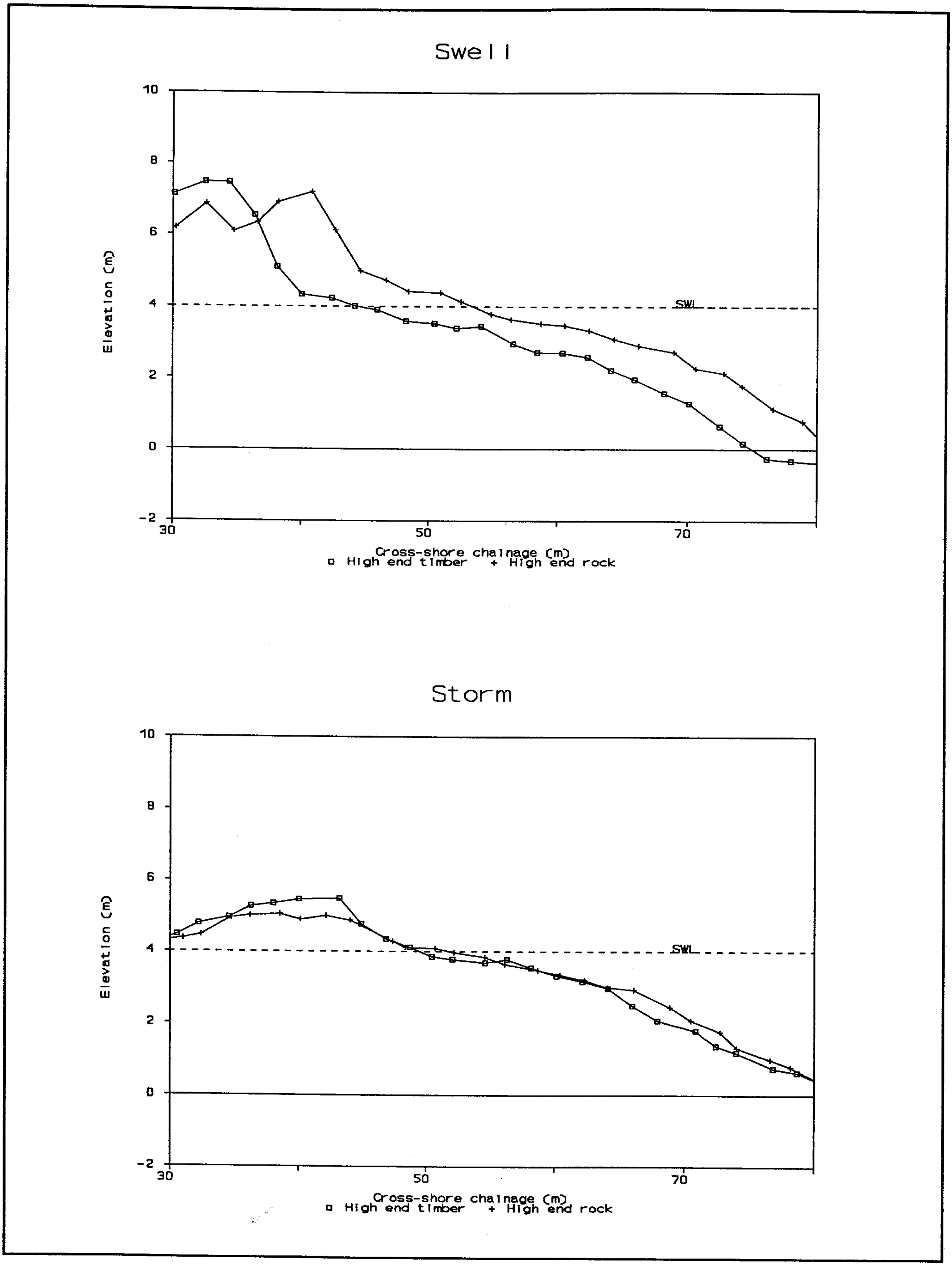


Figure 3.8 Comparison of updrift profiles for high end timber and rock groynes

Further investigation of the factors controlling the cross-shore position of the lower part of the profile revealed that the chainage to a specific elevation along the groyne crest profile had an important influence. Under a 4m SWL this point was found to be at 2.75m above the initial beach toe elevation, or 1.25m below SWL. The beach profile at this point was slightly above the groyne at an elevation of 3.0m. Figure 3.10 presents a plot of updrift profile area against the chainage of the 2.75m intersect along each groyne type. The plot produces a linear relationship for both the swell and storm wave conditions, with the storm data points showing little deviation. The swell condition data points are more scattered suggesting that other factors also influence the profile area. The position of this crest elevation is important as it defines the effective length of the groyne. Any further length of groyne below this elevation has no significant effect on effectiveness. Unfortunately the position is dependant on water level and wave height, which are varying continuously in nature.

Examination of the updrift profiles for the rock barriers indicates that the upper beach is influenced by the emergent seaward portion of the groyne under swell wave conditions. The crest position on these profiles developed significantly seaward of the position on all lower groynes, as shown by Figure 3.11 (swell), which contrasts the profiles for the low rock and long rock barrier groynes (the lower portion of the profiles have been brought into coincidence).

The storm wave conditions did not produce any significant differences in the profile shapes for the low and high groynes. The storm wave tests of the short and long rock barriers were run on from the swell wave tests without a remould, and therefore the upper beach profile area includes the relict swell profile. However, when this relict portion of the profile is edited, as in Figure 3.11 (storm), then the remaining profiles are shown to be similar for the low rock and long barrier groynes. The profile areas used in Figures 3.10 and 3.12 are also edited for these tests.

The influence of the emergent portion of the groynes is also shown by Figure 3.12. The effective groyne area, taken as the area below 6m and seaward of the 6m intercept on the low timber groyne crest (31.25m from the beach head), is plotted against the updrift profile area. The scatter about the swell regression line is considerably less than for the groyne length plot (Figure 3.10) indicating the importance of the effective area as a design parameter in situations where swell waves influence the design sea conditions.

The influence of the landward portion of rock groynes was also investigated using groynes truncated to leave only the seaward portion. These tests (Tests 27a, b and c) were qualitative only, with no measurements or photographs. They were undertaken to investigate an observation recorded during earlier research (Reference 2), that the seaward portion of a groyne is dominant over the landward portion in determining beach response. Testing commenced with only the submerged portion of the low rock groynes. The sea state throughout was the swell condition at the 4m SWL. The structure was successful in retaining the lower beach, but wave run up around and over the landward end of the truncated groynes prevented the beach crest from forming and reduced the groyne efficiency to near zero. The structure was gradually extended upwards and landwards while the beach response was continually observed. The beach crest updrift of the groynes did not begin to accrete seawards until the groyne had been extended landwards to the point of maximum wave run up. This response indicated that under swell conditions a full groyne profile is required to achieve a satisfactory response. Storm conditions were not tested.

Insufficient data are available to define the most effective groyne profile for use on specific sites. However the following points appear to be important :

- groyne length landward of a threshold elevation is important. Length seaward of this threshold has a negligible effect on beach response assuming a constant SWL. For the sea conditions used during the analysis the threshold elevation was 2.75m. This was 1.25m below the test SWL, and corresponds approximately with 0.75 of the breaking wave height (H_b) for the 2m H_s storm waves. According to previous research (Reference 2) the cross-shore distribution of transport for an open beach peaks at a chainage of $0.8X_b$ (where X_b is the cross-shore chainage of the breaker zone); insufficient data are available at present to confirm any definite relationship between these points but further research might provide a useful design equation.

- Increases in the effective groyne area (ie, above the threshold elevation and below the beach crest elevation under storm conditions) cause a linear increase in the updrift beach profile at a ratio of about 1:1 under swell conditions but only about 1:1½ under storm conditions.

- Groyne crest elevations above the maximum storm beach crest elevation were not tested. It is assumed that the influence of increasing the crest above the storm beach level would have little benefit in practice.

3.4.3 Effect of sediment input rates

The majority of tests throughout the three test series were run with updrift sediment input rates as calibrated at the outset of the programme. A number of tests were also run with a zero input to investigate beach response during erosion. The layouts tested under both conditions included the high end timber (Tests 25 and 26), high timber (Tests 35 and 43), short rock barriers (Tests 34.2 and 34.3 : S=0.06 and Tests 34.1 and 43 : S=0.02) and long rock barriers (Tests 32.3 and 33). Each zero input test was run until the groyne efficiency had risen to 80% and the rate of erosion had levelled out. Figure 3.13 contrasts the full and zero input profiles for the high end timber and short rock barrier tests and also presents the cross-shore distribution of transport for the full input tests. Cross-shore distributions for rock groynes were not measured, so the distributions from the nearest equivalent timber groyne tests are displayed. Table 3.4 presents measured data for differences in X_{pp} and beach volume.

Several interesting observations emerged from these test results that have important consequences for groyne design. Tests of the high end timber groynes (Tests 25 and 26) and short rock barriers (Test 34.1 and 42) are used to illustrated the observations.

Figure 3.14 compares the difference in X_{pp} values for the two groyne types under conditions of zero and full sediment input (swell waves). Under the full input conditions the groynes caused similar X_{pp} responses, but under the zero input conditions the responses were very different. With the short rock barriers in place the beach eroded by 9m, while it eroded by 18m for the high end timber groyne.

The updrift profiles and cross-shore transport distributions for the two pairs of tests are presented in Figure 3.13. Under the full input conditions the updrift profiles are very similar, except that the emergent portion of the short rock barrier allowed the crest to accrete further seaward. Under the zero input conditions the beach eroded to different profiles. Updrift of the high end timber groynes the beach eroded back over the full profile until it was at least 0.6m below the groyne crest line. In contrast, the beach updrift of the short rock barriers only eroded above the water line, leaving the lower profile virtually unchanged.

The cross-shore distribution of transport and the efficiency curves for the two groynes indicate the reasons for the different beach responses. Figure 3.13 presents the transport distributions under the full input conditions. The transport of beach material over the high end timber groynes occurred across the whole beach profile. As the beach eroded during the zero input test sediment transport would have continued in the same pattern but with gradually reducing rates as the groyne crest became more elevated relative to the beach profile. At the conclusion of the test, transport was still occurring around the SWL indicating that significant amounts of beach material were being carried in suspension at a level of about 0.6m above the beach profile. The efficiency curves for both the full input and zero input tests (Tests 25 and 26, Figure A9 Appendix 3) indicate that the high end timber groynes allowed a constant loss of material, resulting in slow beach development and a large volume difference between the fully developed and full eroded beach.

The beach response to the rock barriers was quite different. The cross-shore transport distribution was skewed to the lower beach, with no material passing over the groyne. During the erosion test, the profile crest cut back and transport around the groyne head dropped rapidly. The efficiency curve for the full input test suggests that the rock barriers retain over 90% of drift material until the beach volume reaches a threshold point, after which efficiency drops rapidly to zero. During erosion the reverse occurs, with the η value shifting rapidly from zero to around 50% and then increasing more gradually to over 80% (Tests 3.41 and 42, Figures A14 and A17: Appendix 3). This rapid shift in transport rate is beneficial to beach management as the beach volume remains relatively constant, ensuring that recharge material is retained rather than being constantly lost as observed for the lower profile groynes.

3.5 Results of the Series 3 tests

3.5.1 Effect of 15° wave direction

The 15° offshore wave direction was run with several groyne types and with both zero and full inputs. The wave direction had no significant effect on profile shape, but did affect the efficiency curves and planshapes. All of the groyne types acted as near total barriers to drift until the groyne bay volumes reached a threshold level, after which η dropped rapidly to zero. The final beach planshapes under the full input conditions had orientations of around 6°.

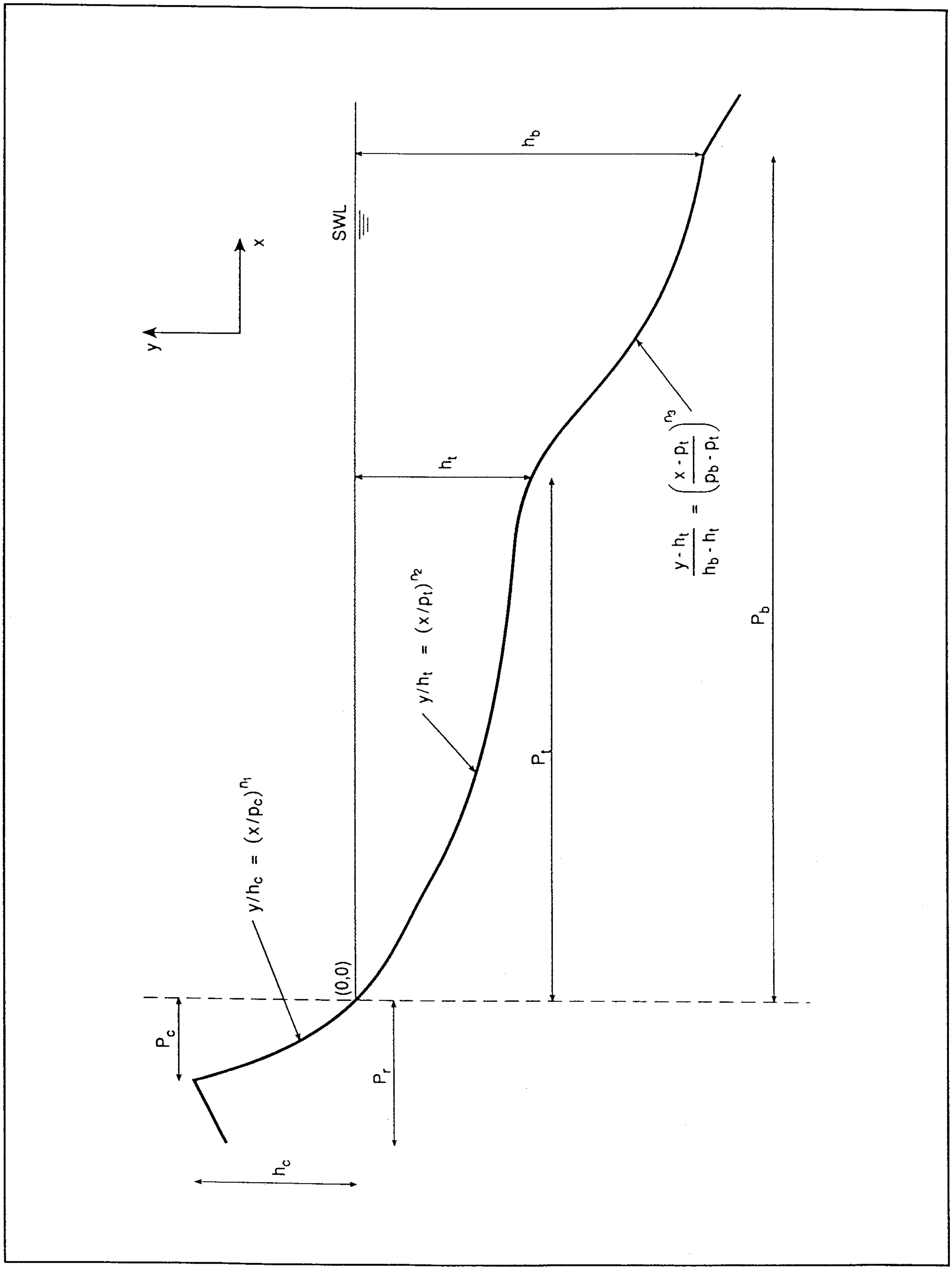


Figure 3.9 Schematized beach profile from SHINGLE model (Reference 1)

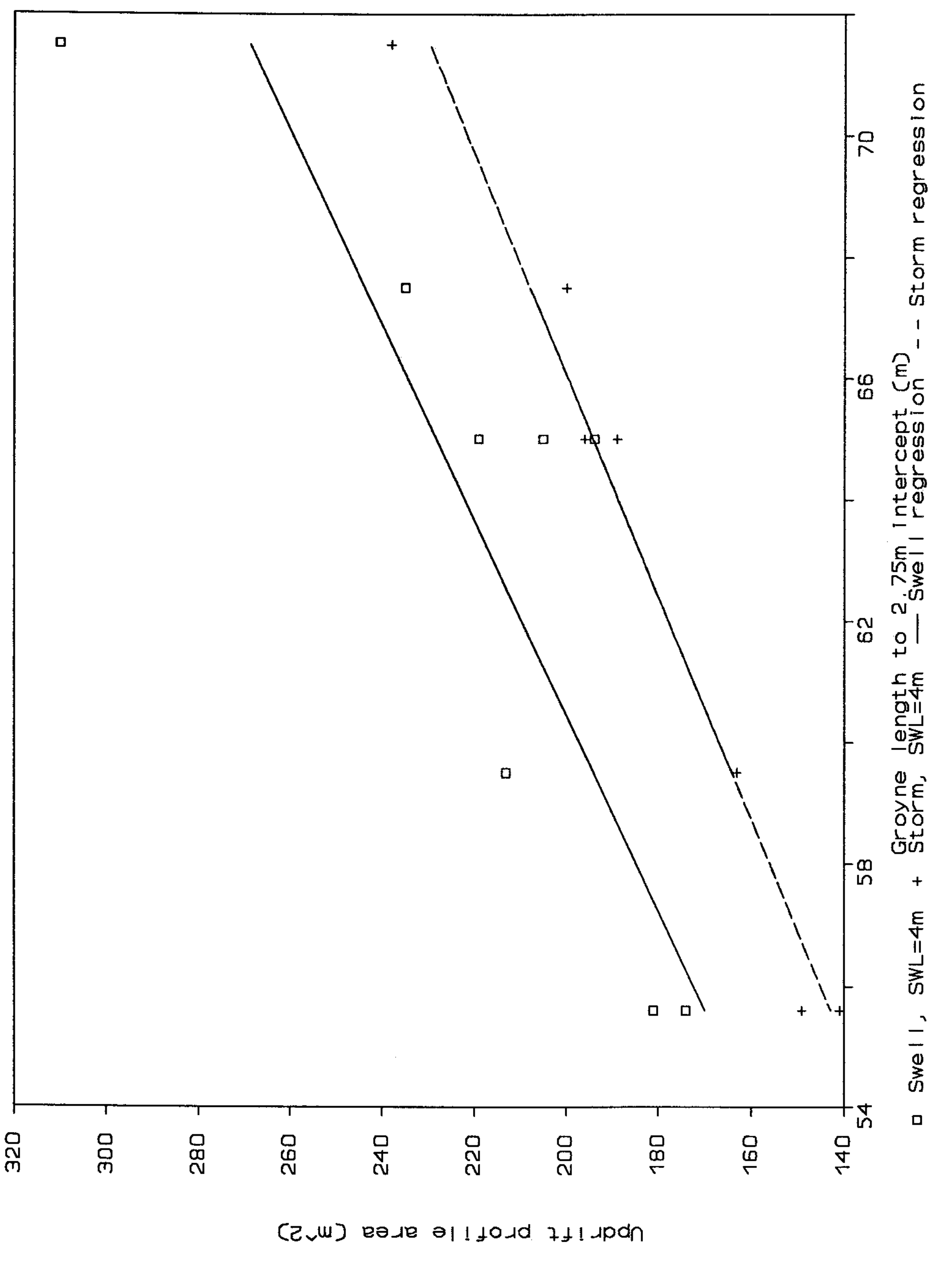


Figure 3.10 Effect of groyne length on updrift beach profile area

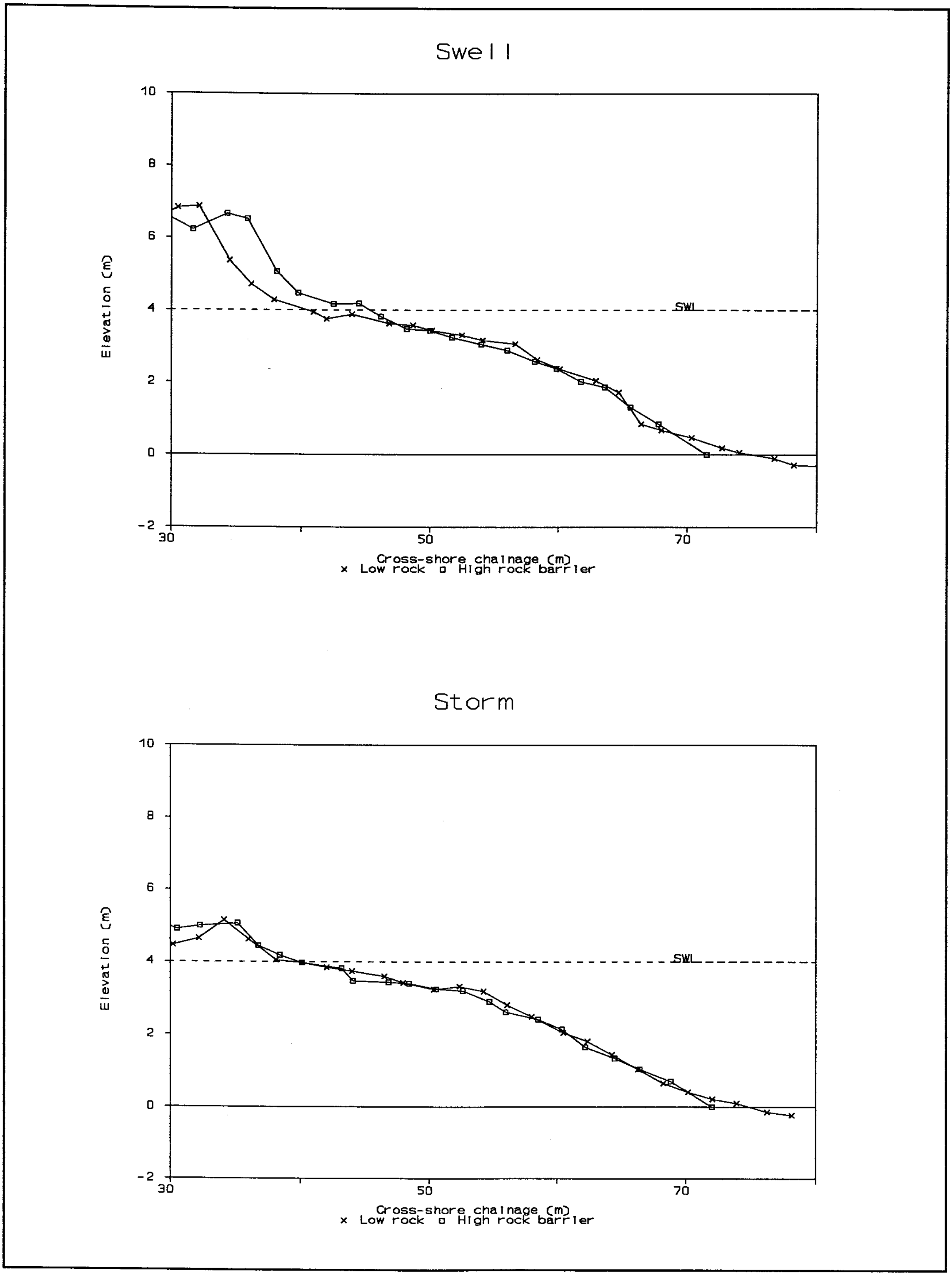


Figure 3.11 Effect of emergent portion of groyne structure on updrift beach profile

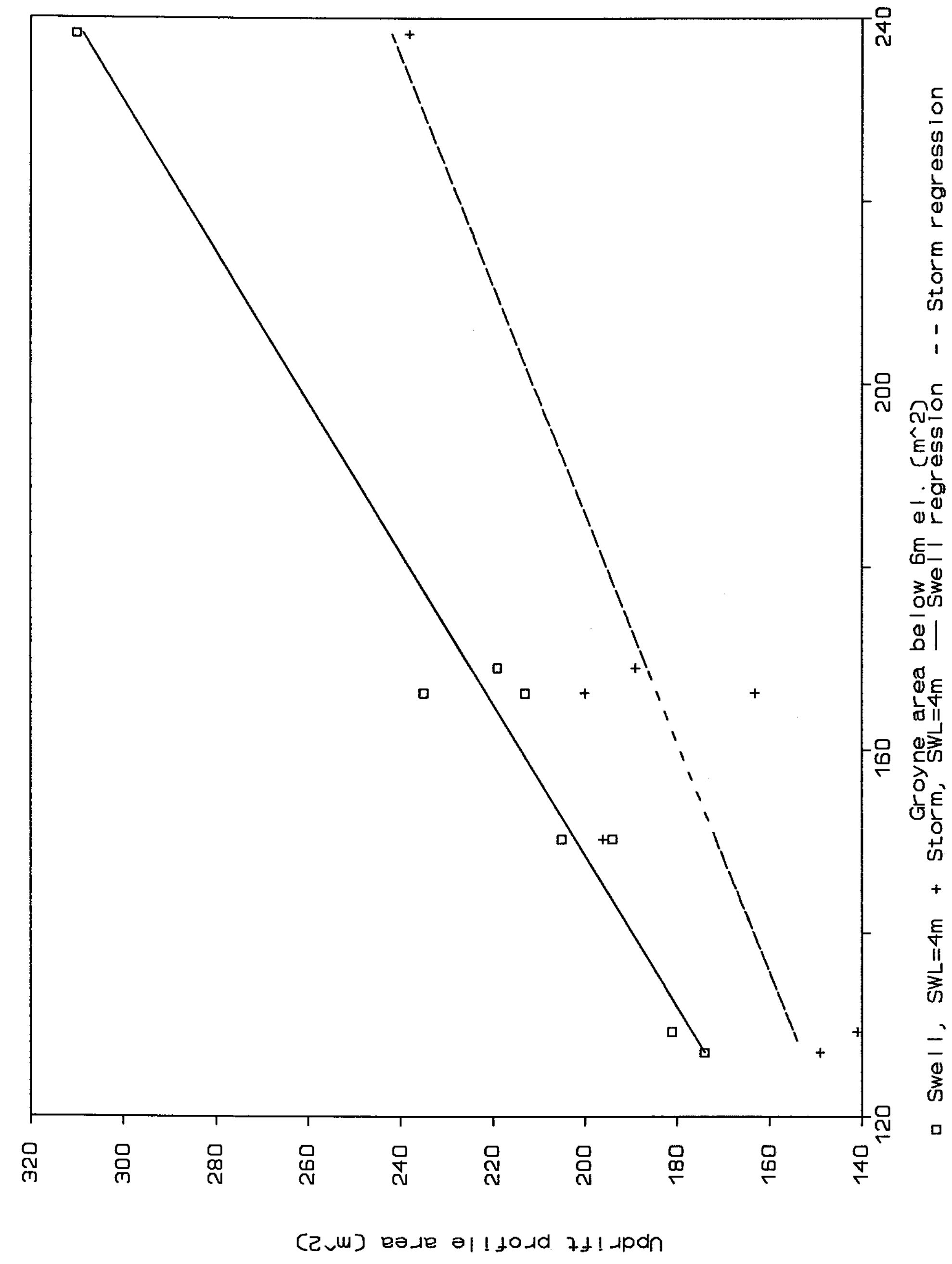


Figure 3.12 Effect of groyne area on updrift beach profile area

3.5.2 Wave direction analysis

The influence of wave direction on beach response was investigated in association with groyne efficiency for both the 15° and 30° offshore wave directions. As a basis for the analysis it was assumed that if η=0 then the beach orientation would equal the open beach situation (ie. parallel to the beach threshold line) and that if η=100 then the orientation updrift of the groynes would tend towards the breaking wave angle. The results of the test programme did not appear to support the first assumption as the final beach orientations between the updrift and mid-bay profile lines tended towards 8-10° for the 30° offshore direction, and 5-7° for the 15° direction. This apparent anomaly was further investigated by reviewing the calibrated input rates. The calibrated input drift rates were based on the mean rate measured over 3 periods of 500 T_m. During testing waves were run over periods of 2000 T_m. It was suspected that the calibrated rates were too low for the test period, so a second calibration was completed for several wave conditions using 3 periods of 2000 T_m. The rates obtained were approximately 20% higher than the first rates, implying that the efficiency curves and ratios derived for each test were not correct. By applying the new rates to the orientation analysis the assumption that the groyned beach would develop to the open beach orientation was viable, but not proven.

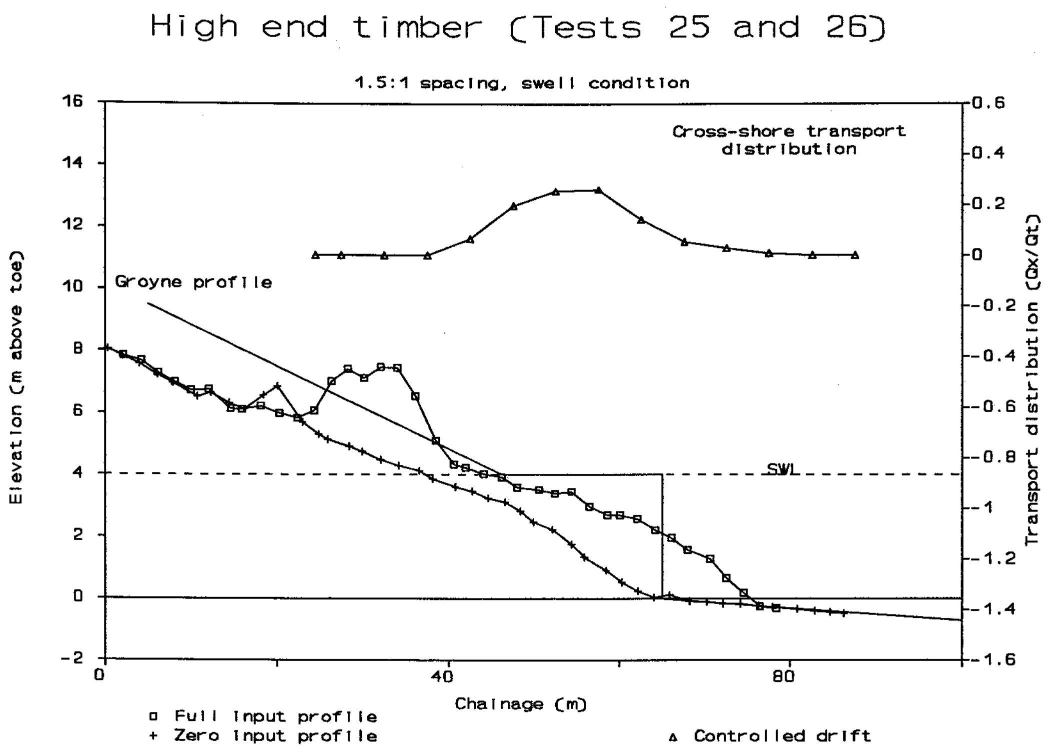
The efficiency curves and beach planshapes were reviewed to find beach orientations corresponding to a range of η values. The results were plotted and a non-linear regression curve fitted to determine the beach orientation for any η value. At 100% efficiency the curve predicted a beach orientation of 16.5° for a 30° offshore direction. Insufficient data was available to fit a curve to the 15° offshore direction.

A simple refraction analysis, using the methods of the "Shore Protection Manual" (Reference 9), was then applied to the 30° offshore waves. This suggested a wave direction at 5m depth of 19° and a direction at breaking of 12°. The predicted 5m depth direction disagreed with the measured direction of 23° for that point (Section 2.4.1). This disagreement was considered to be a function of the model bathymetry , which had a 1:10 approach slope to the foreshore, and so a refraction analysis was run based on the 23° value for the 5m contour. This approach gave a wave direction at breaking of 17°, correlating with the regression curve analysis. This process was repeated for the 15° waves, giving a 9° breaking wave direction. A general curve for determining beach orientation which considered both offshore direction and groyne efficiency was derived by combining the 15° data with the 30° data. This curve is shown in Figure 3.15 and is described by the equation:

$$\theta^\circ = \theta^\circ_b \,(0.253\, \eta^{0.3})$$

where
θ° = beach orientation
θ°_b = breaking wave orientation
η = efficiency

Unfortunately this study has shown that η is time dependant and that no single value of η can be applied to any groyne type. For all practical design purposes η must be taken to be 100%, giving the conservative result that between the groyne and the mid point of the groyne bay the beach will align with the breaking wave direction.



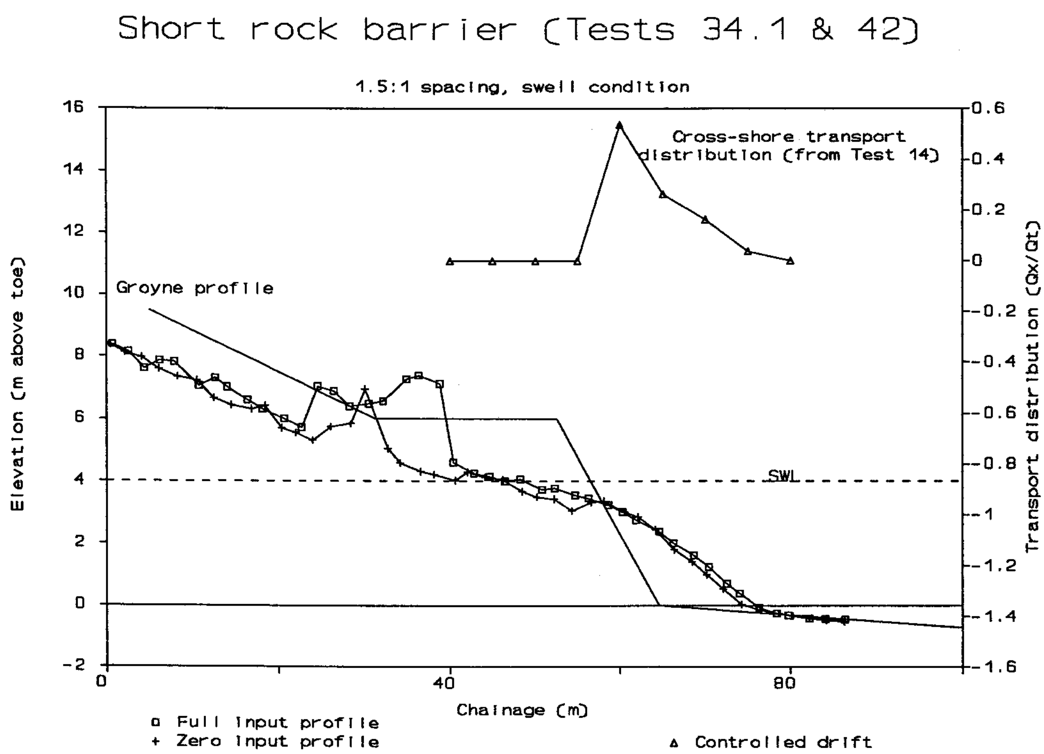


Figure 3.13 Updrift profile response under full and zero sediment input - High end and short rock barriers

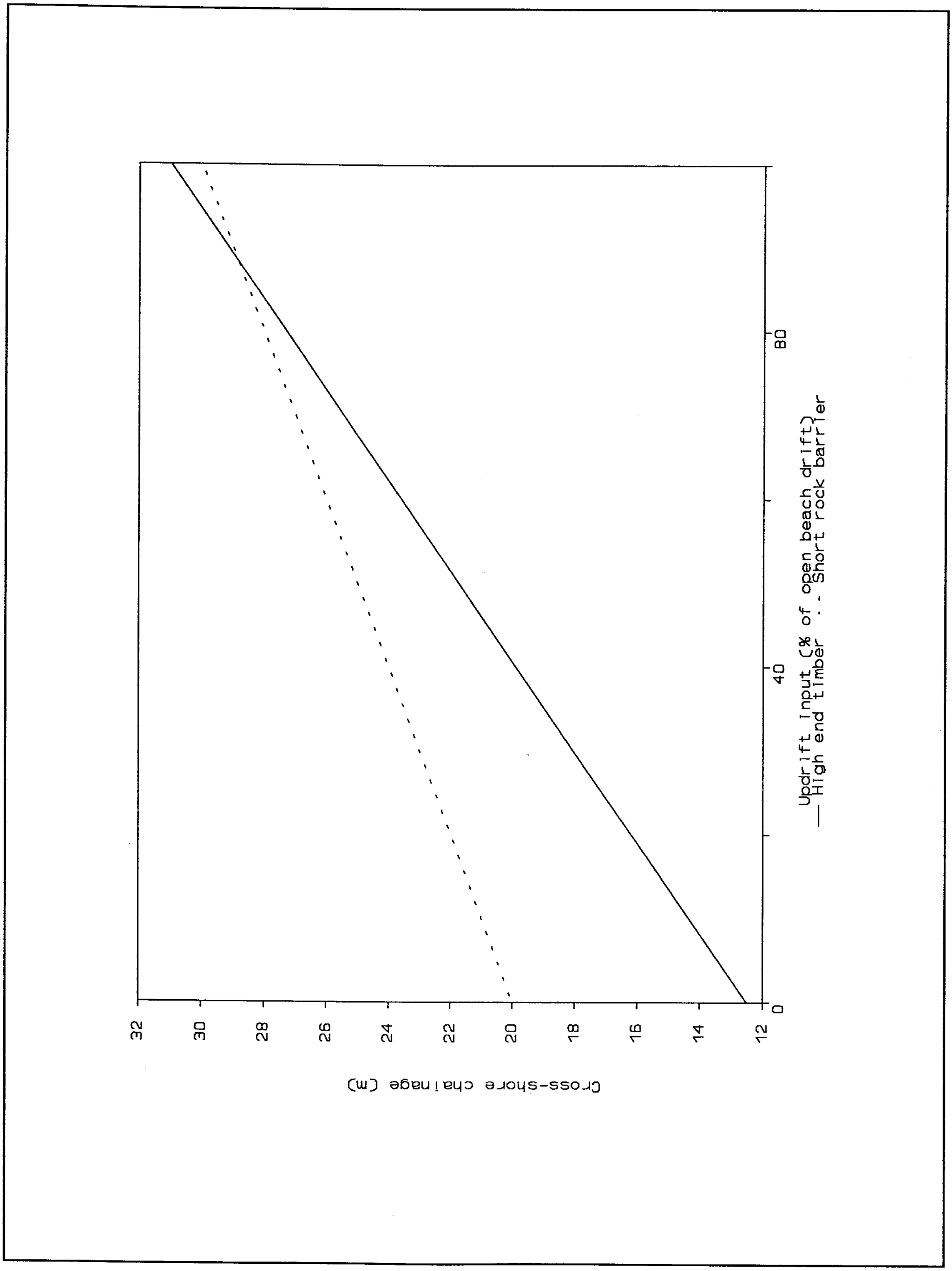


Figure 3.14 Effect of drift input rate on pinch point chainage

3.6 Comparison with existing numerical models

Numerical models have been developed by HR Wallingford to predict beach profile response (Reference 1 and 2) and long term beach plan shape development (References 3 and 4). This section considers the results of the present study in relation to the models.

The beach profile model, known as SHINGLE, was developed from physical model results in a wave flume at a scale of 1:17 (Reference 1) Figure 3.16 compares the updrift beach profiles from Series 1 tests with SHINGLE predictions. The measured and predicted profiles are in very good agreement for the storm conditions and for the upper portions of the swell conditions. The lower portion of the swell profiles does not show good agreement. The predicted profile assumes a concave curve, whereas the measured profiles shows a convex curve.

This difference is not necessarily important as groyne design will normally be based on design storm conditions with steepness values approaching 0.06, for which the SHINGLE profiles agree with the model profiles.

Of greater importance to groyne design are results from the present study which contradict three of the assumptions used in the longterm beach development model (Reference 3) and the recently proposed modifications for modelling transport at groynes (Reference 4). The assumptions are that:

- the cross-shore distribution of transport at a groyne must remain the same as that of an open beach;
- a groyne can be assigned a single efficiency value; and
- shingle is transported as bed load throughout the surf zone.

The first of these assumption implies that long-shore processes dominate transport at any point on a shingle beach and that the full potential drift rate can only resume at a groyne when the full beach profile is above or seaward of the groyne. The profile and cross-shore distribution plot presented in Figure 3.9 clearly contradicts this assumption and indicates that cross-shore processes can dominate at a groyne, allowing full transport to concentrate into very narrow cross-shore zones at the points of least resistance.

The second and third assumptions are also clearly contradicted. The efficiency plots presented in Appendix 3 all show η varying over time as the beach volume builds up within the groyne bays, indicating that a single efficiency value for a given groyne is an over-simplification. Similarly the transport distribution plots for the full input tests, presented in Appendix 4, show that transport occurred when the groyne crest was well above the beach profile indicating that the beach material was transported in suspension as well as bedload, particularly in the most turbulent area of the breaker zone. Although shingle is known to be put into short term suspension under turbulent conditions there must be some doubt as to whether the extent of suspension is as great as the physical model results suggest. Field measurements must be obtained to verify the model.

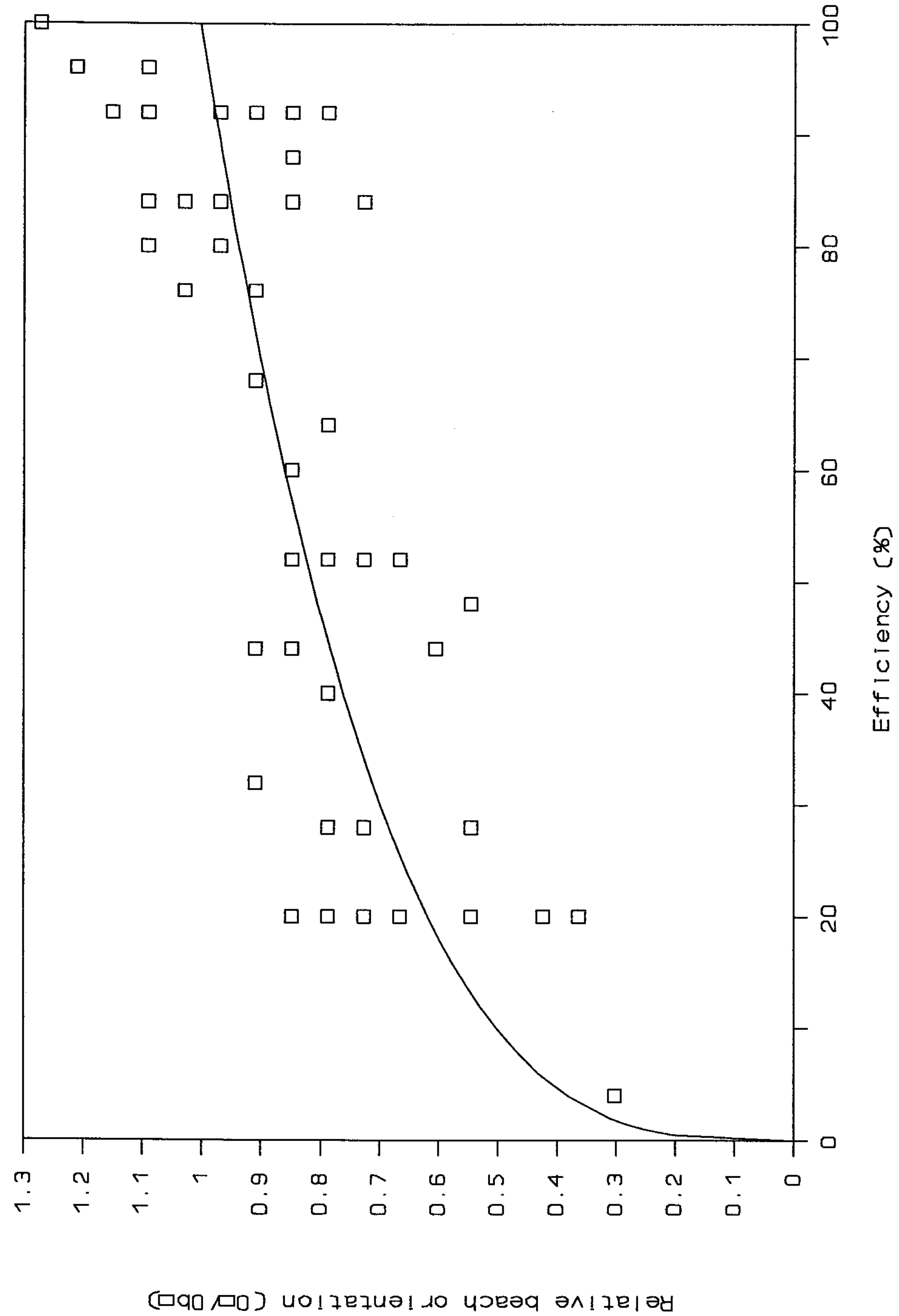


Figure 3.15 Beach orientation relative to breaking wave direction and groyne efficiency

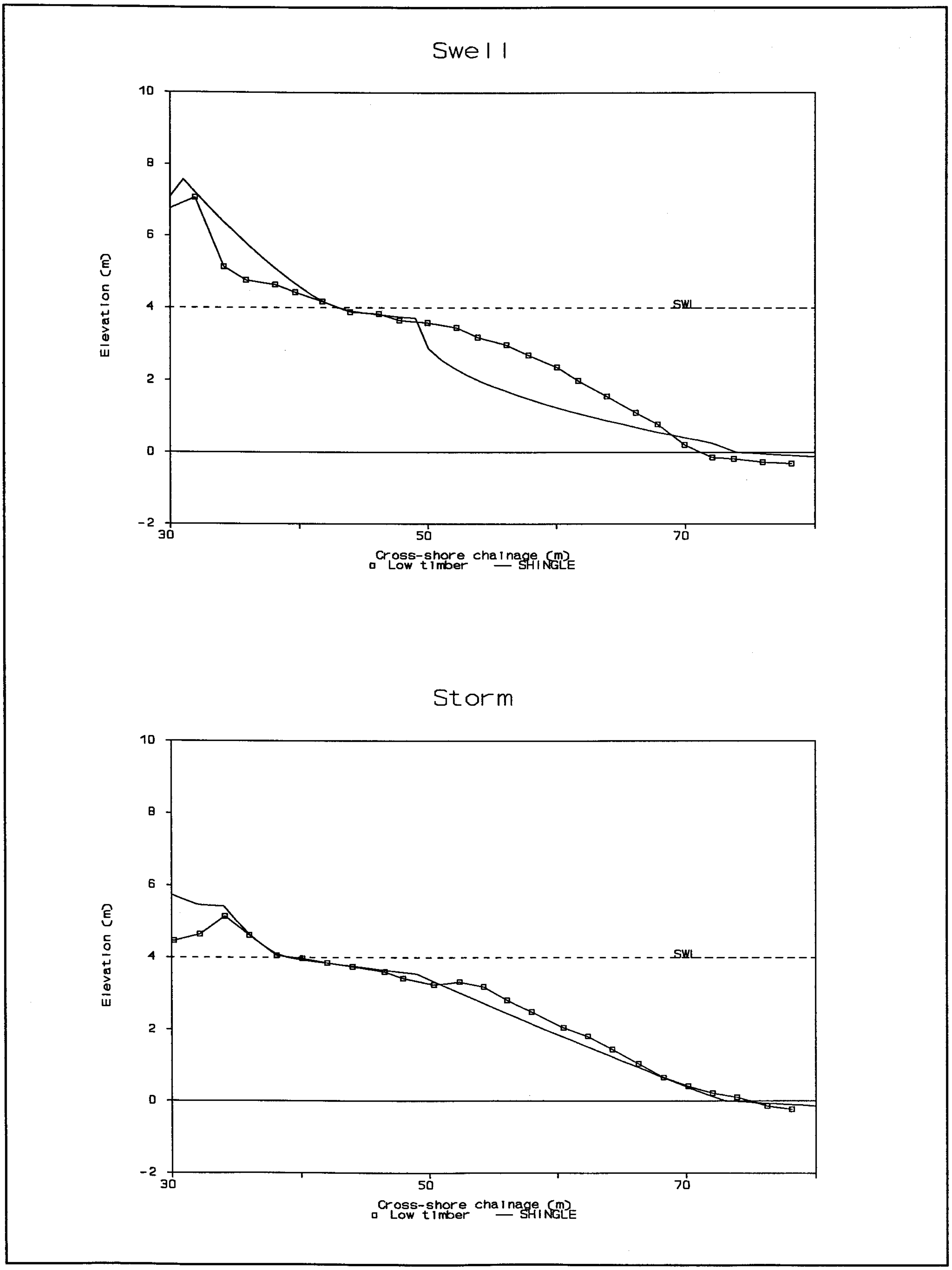


Figure 3.16 Comparison of measured updrift beach profiles with SHINGLE model predictions

3.7 Beach material distribution

The cross-shore distribution of sediment was analysed for both storm and swell wave conditions. Samples were taken from the model beach surface at the crest, transition point (step) toe and offshore. Figure 3.17 presents the grading curves for these points, along with the initial beach grading.

Under swell conditions coarser material moved onshore to the crest and, to a lesser extent, the transition point. Material collected offshore was from the finest 5% of the beach material and did not account for a significant volume.

Under storm conditions the material at the crest and transition point were not quite so coarse. The beach toe was also finer than under the swell condition, though there were distinct longshore variations with pockets of very coarse material in the lee of groynes. The offshore samples showed the most significant difference relative to the swell conditions. Samples were taken from the relatively large volumes of very coarse material that were carried offshore from the mobile beach and transported right across the hard moulding of the lower beach. This material accounted for up to 10% of the total drift volume and consisted of material from the coarsest 20% of the initial beach material.

These results are, in general, as expected and fit in well with field observations. However, the offshore loss of coarse material during storm conditions is not well documented in field studies, though some evidence from beach recharge schemes does lend support. Further work is necessary to confirm whether the model results reflect a significant potential problem for shingle recharge schemes using wide sediment grading.

3.8 Current monitoring

Titanium dioxide was used as a neutrally buoyant tracer to monitor suspended sediment transport paths and possible rip currents. Both swell and storm waves were investigated for several different groyne types. Observations were recorded but rigorous methods were not used, so the results are only subjective.

Tracer injections along the mobile beach migrated quickly to the breaker zone, where the dye was transported along-shore and gradually dispersed. Injections close to the updrift face of the more substantial groyne types showed cross-shore currents, but these were often no more defined than at other points within the groyne bays. Injections downdrift of the groynes during the early part of a test showed a zone of low currents with intermittent reversals of direction. As the beach developed over the course of each test these low energy areas disappeared and currents became uniformly downdrift.

No observations of significance to groyne design were recorded.

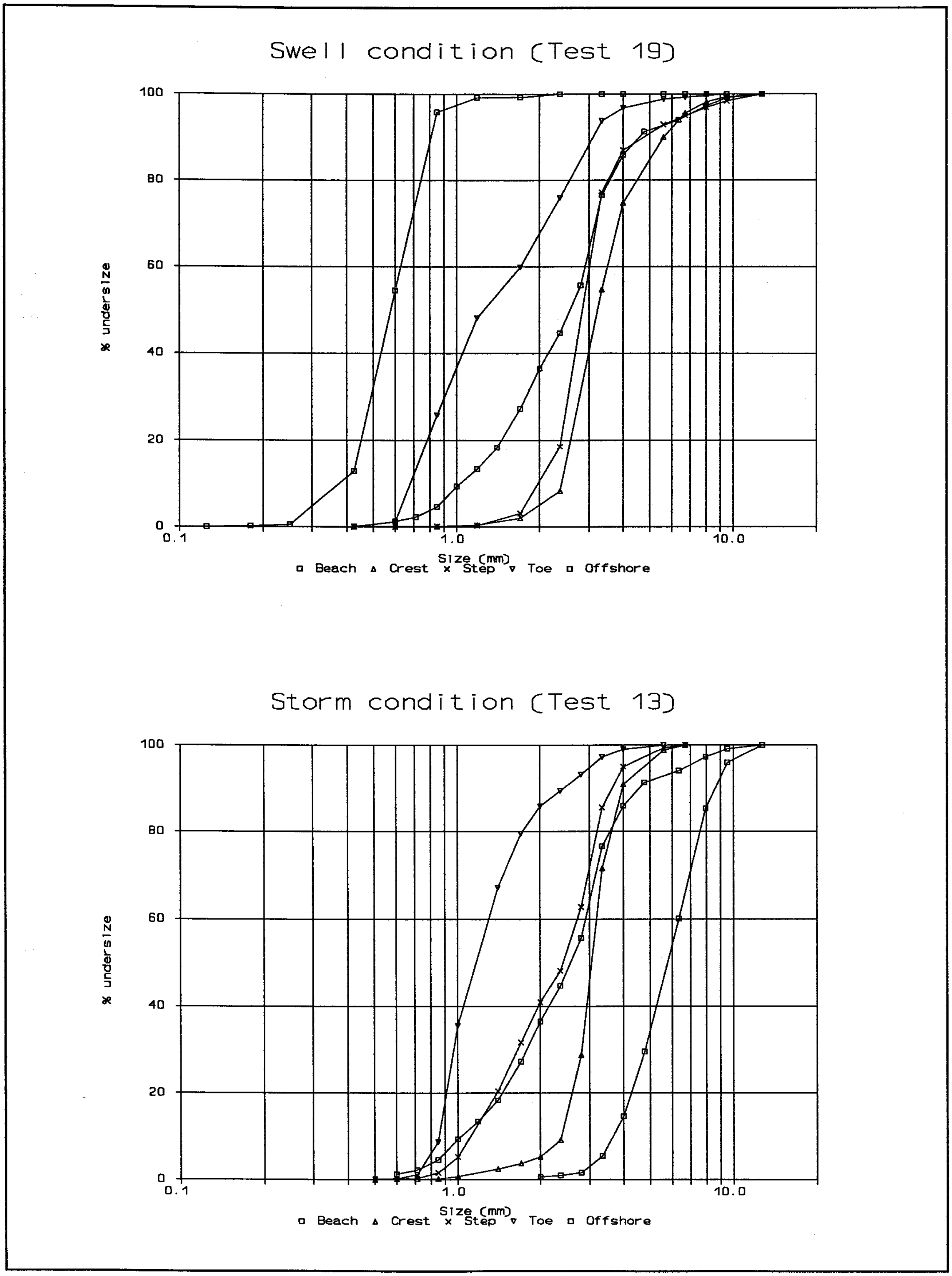


Figure 3.17 Cross-shore distribution of model beach material

4 Discussion of results - Breakwaters

4.1 Introduction

Detached breakwaters, unlike groynes, have not been used for beach control in the UK until quite recently. Existing design guidelines are based on experience in other areas of the world, with the most recent publication being the CERC Technical Report "Engineering Design Guidance for Detached Breakwaters as Shoreline Stabilization Structures" (Reference 10). The majority of research and experience available relates to sand beaches in micro or mesotidal environments with shore normal wave attack, and are therefore not directly applicable to many UK situations. In addition, much of the research concentrates on determining the shape and extent of the beach development in the lee of the structures; this approach only considers the symptoms and not the causes, as it does not directly consider the sediment transport regime at the design site.

Existing UK based research relates to numerical modelling of wave transformations around breakwaters and to ongoing field measurements of waves and beach response in the lee of a breakwater at the Elmer site on the south coast (Reference 11). Morphodynamic numerical models have been developed to consider sand beach response under limited conditions, but, as yet, little work has been directed at shingle.

The aim of the work presented in this report is to establish a basis for developing design guidelines for detached breakwaters on shingle or shingle/sand beaches, based on the principle of providing shoreline protection while maintaining the existing sediment transport regime. The approach adopted was to investigate beach response in the lee of a single breakwater, following by a preliminary investigation of response to pairs of breakwaters. A very limited investigation of wave induced currents was also included for the single breakwaters.

The terms salient, efficiency, potential drift rate and actual drift rate are used in the discussions, and are defined as follows:

- Salient refers to the seaward development of the beach in the lee of a breakwater. If this development reaches the breakwater at the water line then the salient is referred to as a tombolo. This distinction becomes somewhat blurred in areas with high tidal ranges, where a shingle salient at high water may become a sand tombolo at low water.

- Efficiency has been used during discussions of groynes and is defined in Section 2.4 and by Figure 2.9.

- Potential and actual drift rates are important in relation to breakwater design. The potential drift rates is the volume of beach material that the incident wave condition could move along an open beach if the supply of material was unlimited - in the model this is the calibrated input rate. The actual drift rate on a given beach is often much less than the potential rate due to a lack of available material or the influence of artificial structures or channels; erosion of the foreshore often occurs as a result of the difference between these two rates.

The model study showed that the dimensions of a breakwater could be tuned to reduce the potential drift rate in its lee to the level of the actual drift rate, thereby providing a stable beach over its area of influence. The results presented are, at present, only applicable to a limited range of sea conditions, but it is anticipated that the approach will be developed for general application.

An important limitation of this work is that the transport of sand was not considered. Field experience accumulated subsequent to the model study indicates that this maybe a fundamental issue. The results of this study are by no means invalid but must be considered as a first step in developing a design, rather than an end product.

The structural variables investigated included:

- crest length (L_s)
- distance offshore (X_s)
- freeboard (R_c)
- gap length (G_s)

The sea condition variables were restricted to wave steepness (S) and water level (SWL). All tests were run with a 2m H_s wave height and a 30° offshore wave direction. Wave induced currents were investigated in a single test using S=0.02 and a range of water levels from 1m to 4m. Tables 2.6 and 2.7 summarise the test conditions, with further details of sea conditions and structural variables in Tables 2.2 and 2.3. Figure 2.8 illustrates the basic design of the structures.

The data obtained are presented and discussed in the following sections. The discussions include the limitations of the results with respect to design. Conclusions, recommendations for further research and practical design guidelines are presented in the final chapters.

4.2 Presentation of results

The results of the breakwaters study are presented within the following sections, together with illustrative figures and tables. Figures and overhead photographs showing the final beach response for all tests are presented in Appendices 6 and 7.

The beach planshapes presented in Appendix 6 show the final contours of the beach crest, SWL and toe, with the exception of Figure A1 which shows the Test 1 beach after 24,000 T_m and at completion. The cross-shore chainage datum is taken as the 3m water line on the initial standard beach. The planshapes are included to complete the measurement data set but are not discussed in detail.

Most of figures in Appendix 6 present two test planshapes, paired to compare the influence of different water levels. In some cases two water levels were not run for the same breakwater configuration and wave steepness, in which case the pairing is based on contrasting different stages of development (Test 1) or different offshore distances (Tests 30 and 31). Three figures are not paired (Tests 4, 13 and 34).

The photographs in Appendix 7 are arranged in the same order as the beach planshapes. The photographs illustrate the final beach development, and also indicate the progressive development of the beach through the relict crest lines which are visible across the salient.

It should be noted that the efficiency graphs in the text present comparisons of equivalent test conditions. As the graphs are on only two axis and there are three structural and two sea state variables, then simplifications have been required to the labelling of plotted lines. However, each graph is intended to indicate the impact of a single variable, while their combined effects are presented in the final contour plot.

4.3 Results of single breakwater tests

4.3.1 Effect of test duration on efficiency and beach development

The breakwater tests commenced with a standard beach which was subjected to a 2m H_s wave condition with a steepness of either 0.02 (swell) or 0.06 (storm) at water levels varying from 3.0m to 4.5m relative to the initial beach toe. Sediment was added to the updrift end of the model at rates approximately 20% higher than those used for the groynes test (the new rates were based on a second set of drift calibration tests and represent the potential drift rates for the beach and sea conditions). The beach was allowed to develop until the measured downdrift sediment output rates stabilized, usually after 12 - 14,000 T_m. Thereafter the input rates were reduced to equal the stabilized output, and the model beach length was shortened on the updrift side of the breakwater to the point at which the beach salient started to form. The tests continued for a further 12000 T_m to ensure stability with minor modifications being made to the input if necessary. The confirmed transport rates along the beach were then used to determine the efficiency (η) of each breakwater configuration under the relevant sea conditions.

This procedure was followed during all tests except Test 1, which was run with a constant input at the calibrated rate. The test was continued until a tombola had built out to the breakwater and transport had resumed along the seaward face. Plate A1 in Appendix 8 illustrates the final beach response . Although this final result was not of use to the establishment of a η value, it did serve to illustrate the potential impact of a breakwater that is too effective relative to the actual beach drift rate. The structure efficiency reached a stable level of 86% after about 20,000 T_m, but as the beach continued to develop η increased gradually to almost 100%. This increase in efficiency over time is an important consideration in design, as a structure that is too effective will block longshore transport, causing downdrift erosion, until the beach has built out sufficiently to allow bypassing to occur.

4.3.2 Effect of wave steepness on efficiency

Tests were run with wave steepnesses of 0.02 and 0.06. Figure 4.1 illustrates that the structures were 30% more efficient under the shorter period waves. The relationships between steepness, structure length and offshore distance are discussed later. Longer waves lose less energy through diffraction and transmission, and they are more likely to overtop structures than equivalent shorter period waves.

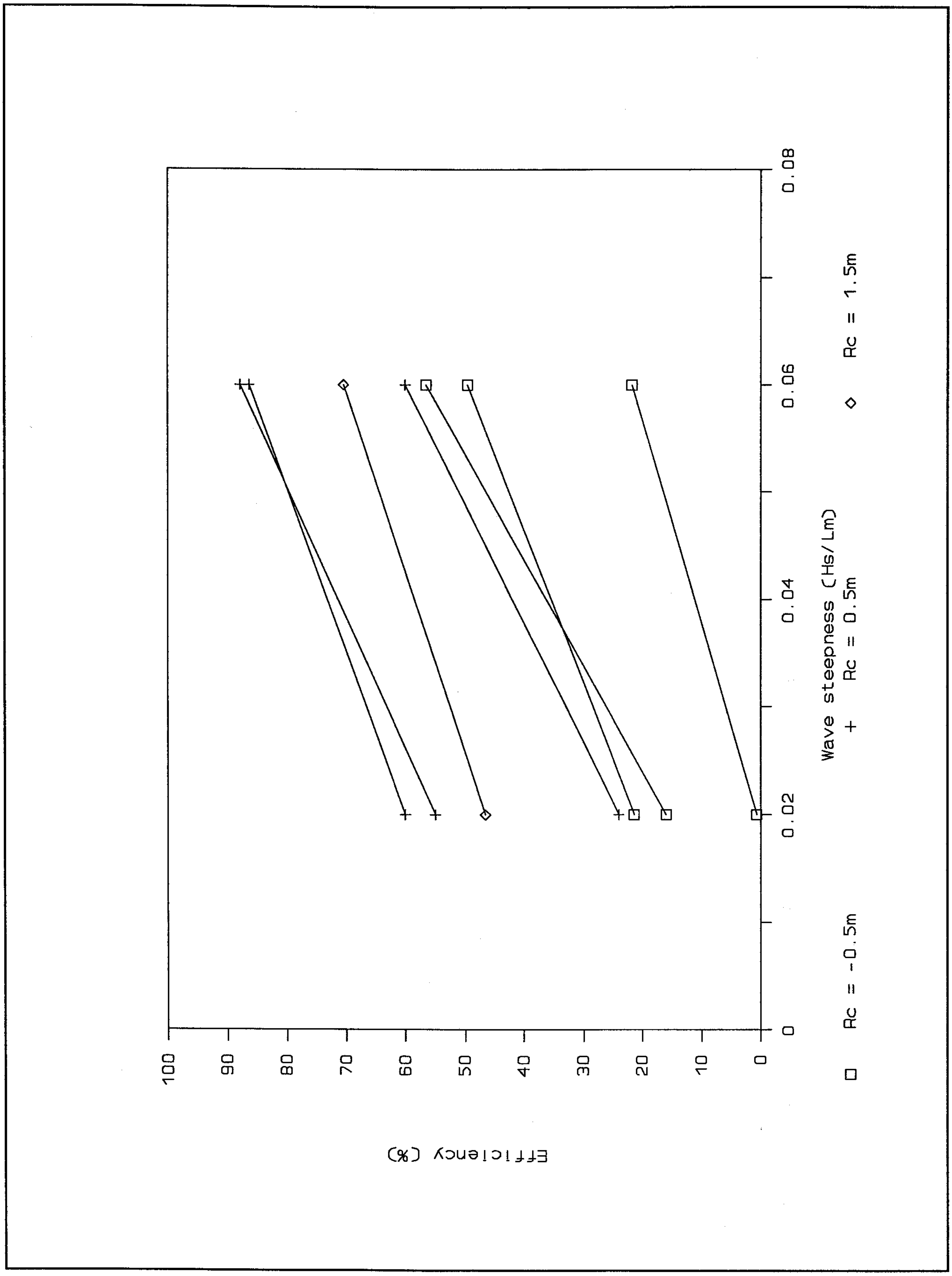


Figure 4.1 Influence of wave steepness on breakwater efficiency

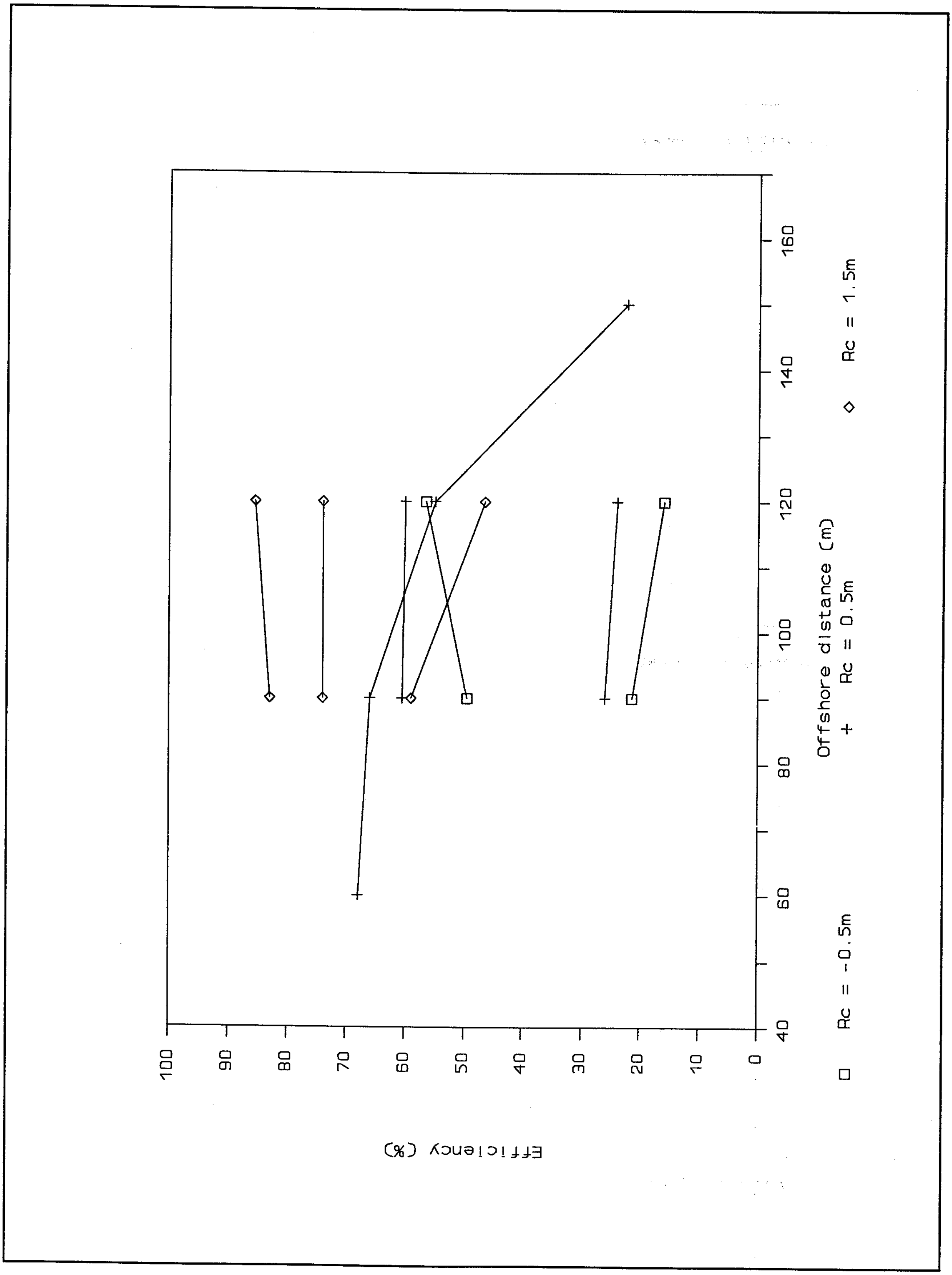


Figure 4.2 Influence of offshore distance on breakwater efficiency

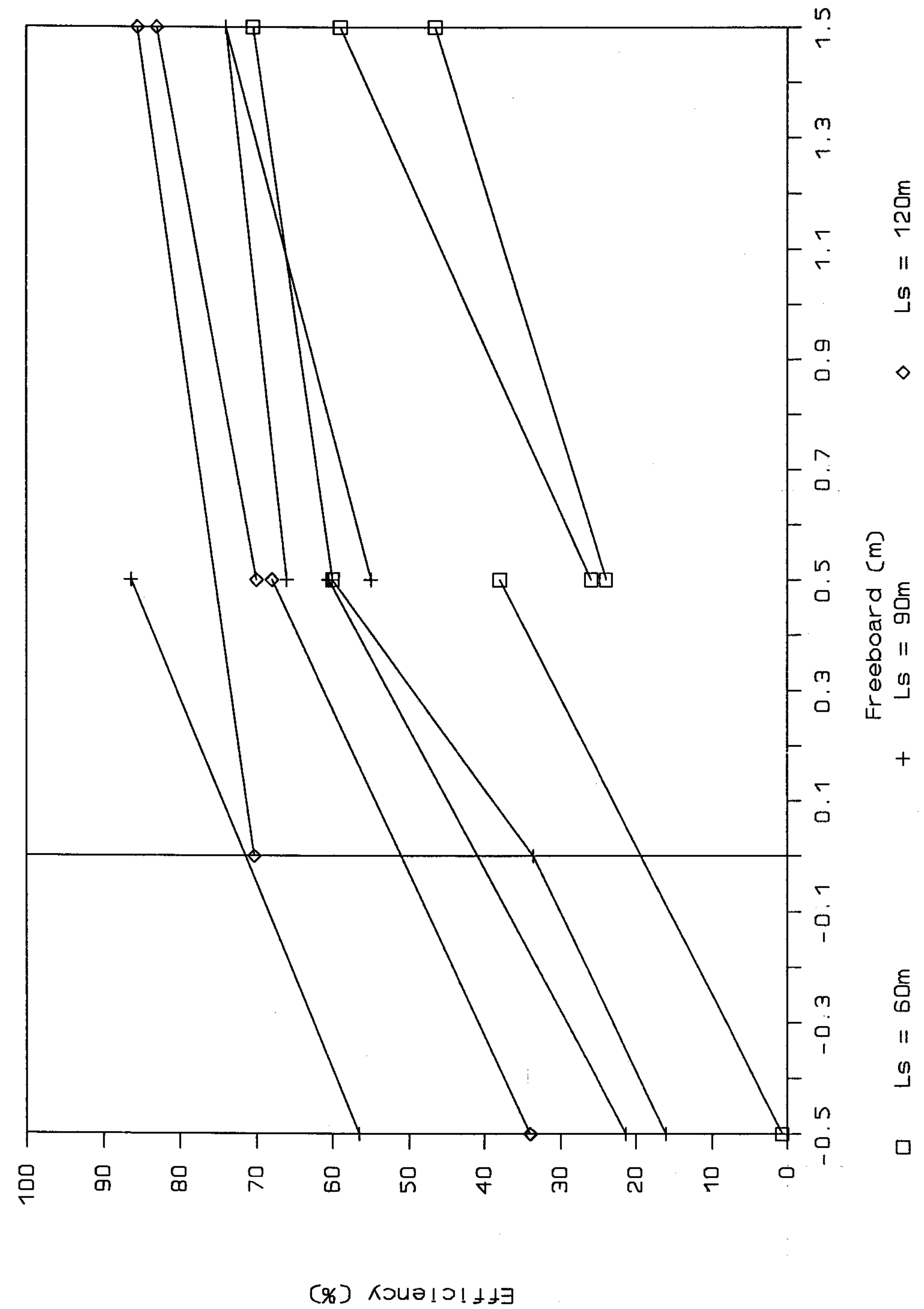


Figure 4.3 Influence of freeboard on breakwater efficiency

4.3.3 Effect of offshore distance (X_s) on efficiency

X_s was measured from the structure centre line to the 3m contour line on the initial standard beach. Most of the tests were run with X_s values of either 90m or 120m, with only one test run with each of 60m and 150m.

Figure 4.2 illustrates that structures set at 60m, 90m and 120m had very similar impacts on efficiency. The only distance which showed a significant effect was 150m, which caused a substantial decrease in efficiency.

Insufficient data was collected for low and high values of X_s to establish any definite trends, however it is apparent that, over the limited range of 90m and 120m, X_s is not a dominant factor.

4.3.4 Effect of freeboard (R_c) on efficiency

R_c is the difference in elevation from the structure crest to SWL. Changes in R_c influence the amount of wave energy that can pass over or through the breakwater. By varying both the structure crest elevation, from 3m to 4m, and the SWL from 3.0m to 4.5m, R_c values from -0.5m to 1.5m were tested. Figure 4.3 illustrates the effect of R_c on η; the lines join data points for tests with equal structure lengths (L_s). These results suggest that R_c is an important factor in beach response to breakwaters.

It is apparent from Figure 4.3 that R_c has a non-linear effect on efficiency. η increased by 30 - 40% as R_c increased from -0.5m to 0.5m. As R_c increased further to 1.5m then η only increased by 10 - 30% with the higher values being associated with the shorter structure lengths.

4.3.5 Effect of crest length (L_s) on efficiency

L_s values of 60m, 90m and 120m were tested. Figure 4.4 illustrates the relationship between L_s and efficiency, with the lines joining data points of equal R_c.

Beach response to L_s was again non-linear. Changes in L_s from 60m to 90m resulted in efficiency increases of between 20 - 40% while changes from 90m to 120m resulted in increases of only 10 - 20%.

4.3.6 Relationship between wave length and structural dimensions

The model data for wave steepness (or wave length), X_s and L_s were further investigated in combinations to determine any relationships. Figure 4.5 illustrates the influence of L_m/X_s on η. There are insufficient results to form any definite conclusions but the figure suggests that η may reach a minimum when $0.4<L_m/X_s<0.9$.

Figure 4.6 illustrates the influence of L_m/L_s on η. Though the results show some scatter, there is an obvious trend of decreasing η with increases in L_m/L_s.

4.3.7 Wave induced currents

Titanium dioxide was used as a neutrally buoyant tracer to monitor currents along the model beach and in the lee of the breakwaters. This work was subjective only.

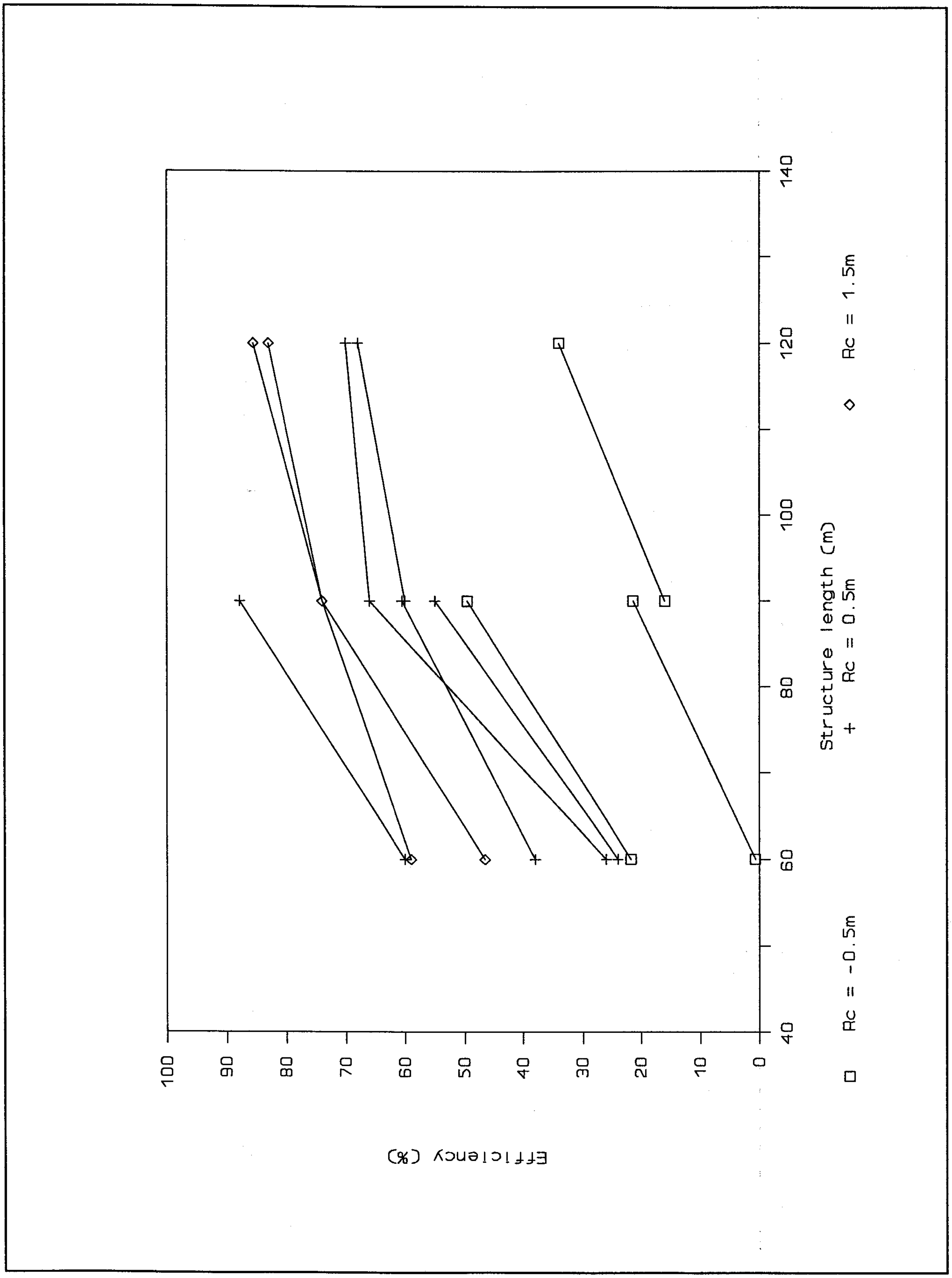


Figure 4.4 Influence of structure length on breakwater efficiency

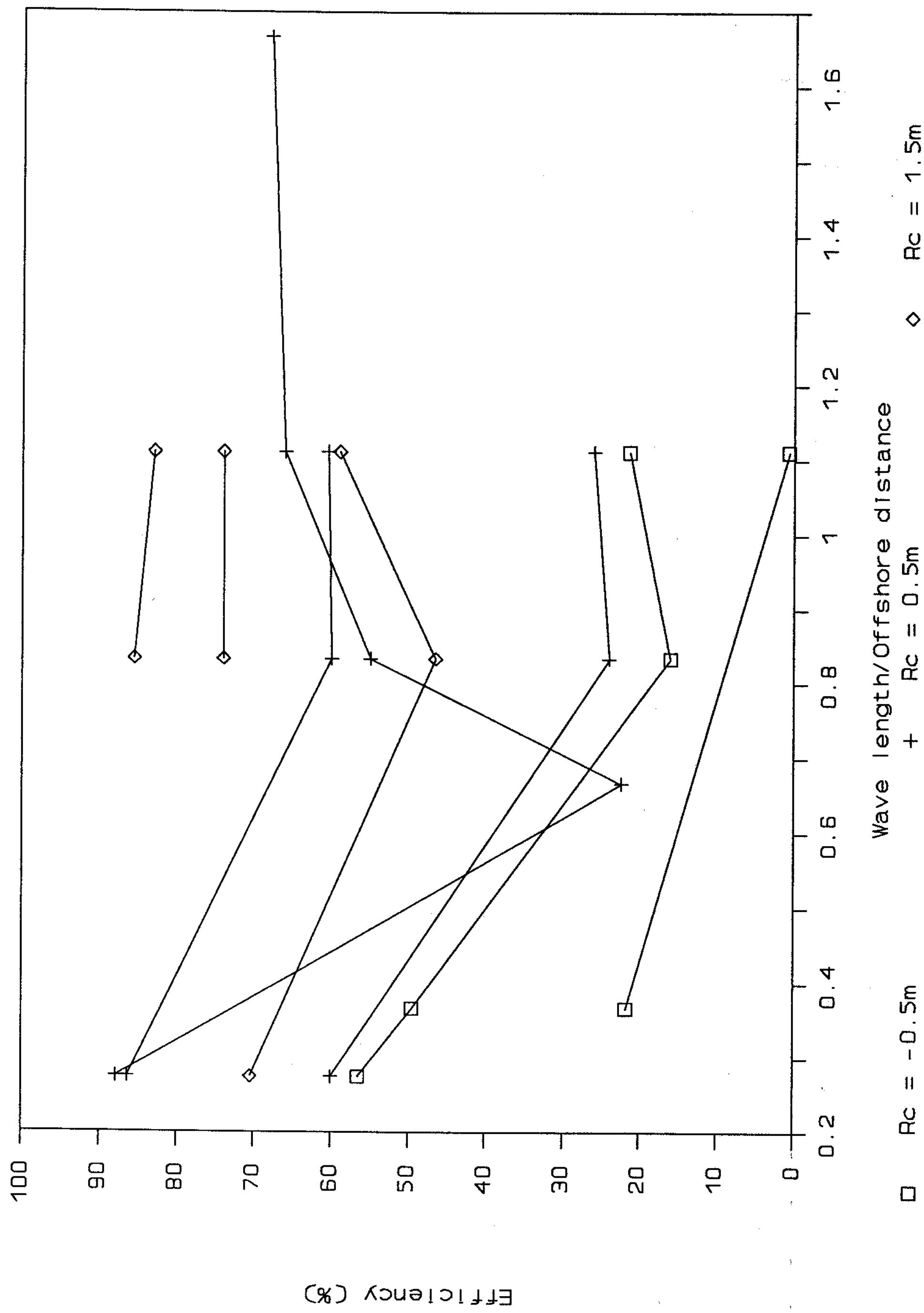


Figure 4.5 Influence of wave length relative to offshore distance of structure on breakwater efficiency

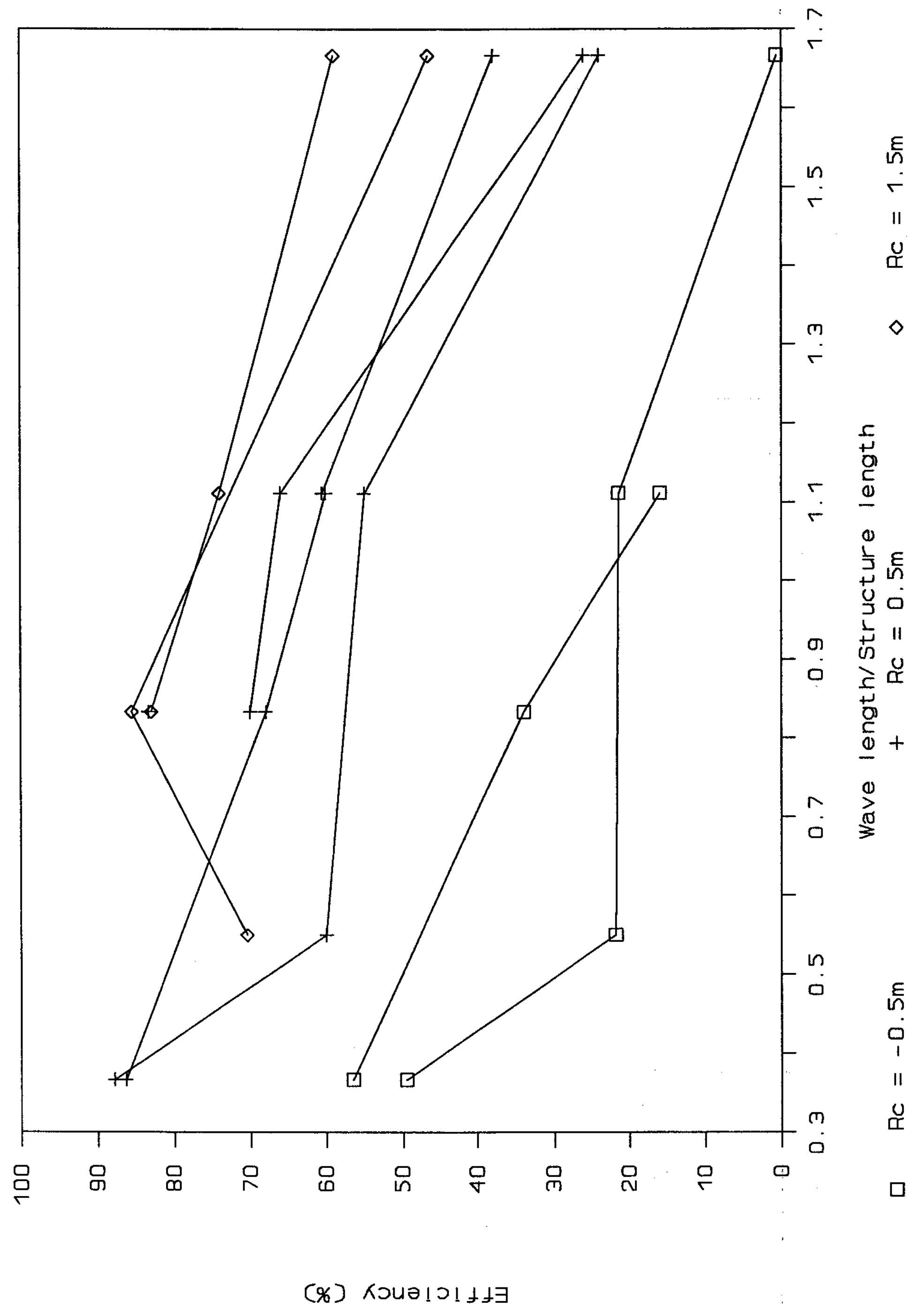


Figure 4.6 **Influence of wave length relative to structure length on breakwater efficiency**

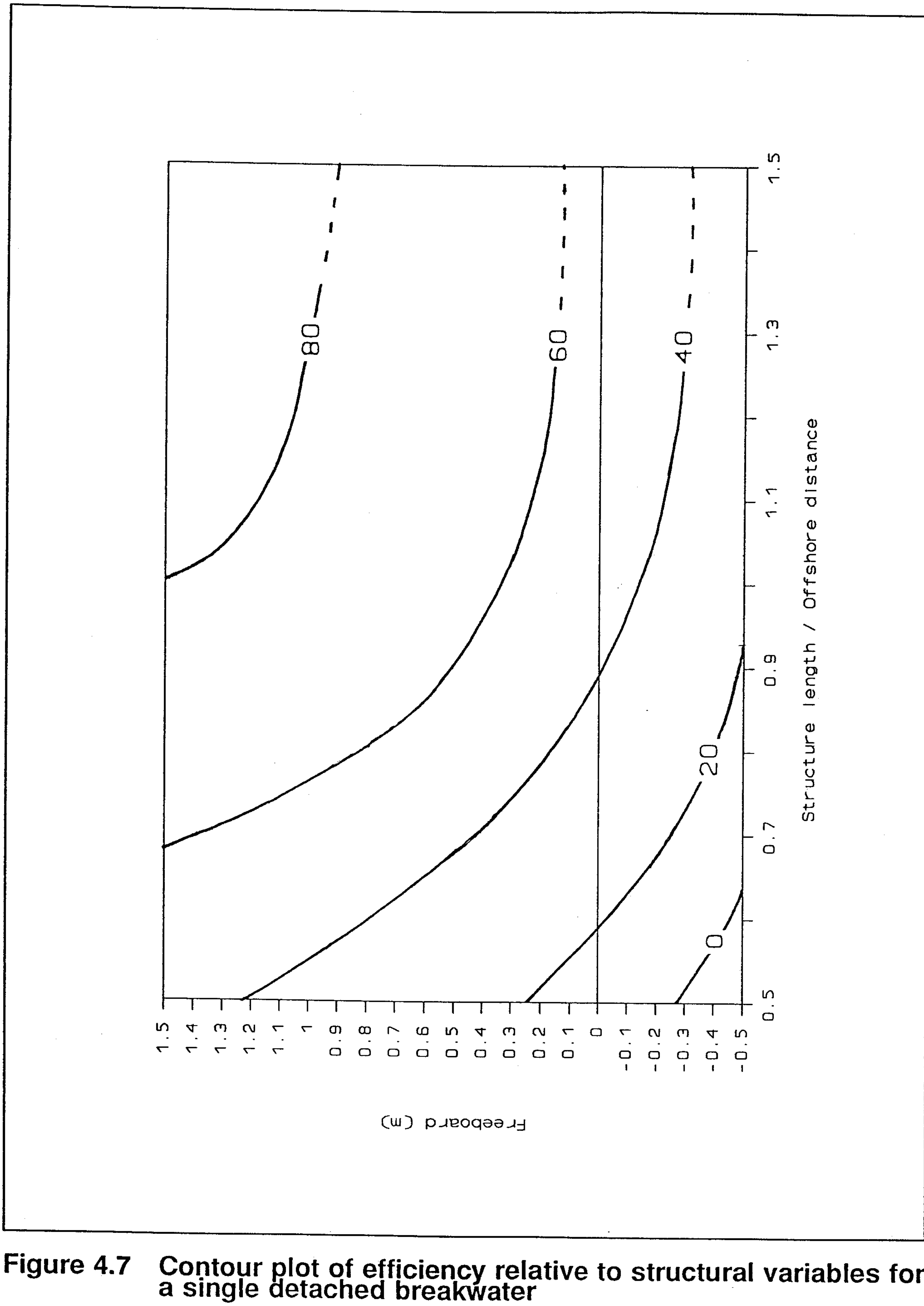


Figure 4.7 Contour plot of efficiency relative to structural variables for a single detached breakwater

Tracer was injected in the lee of a 90m breakwater during swell wave conditions with a range of water levels. Wave induced currents transported dye from the updrift beach and the immediate downdrift beach to the head of the beach salient. From this point the dye was transported intermittently seaward around the down drift end of the breakwater. The intermittent currents had a period of about 10 - 15 seconds (model), equivalent to 8 - 12 incident wave periods. The seaward current reached strong peak velocities, particularly at water levels of between 1m and 3m. Currents at higher water levels were less well defined.

4.3.8 Combined influence of structural variables

The individual effects of length, offshore distance and freeboard are combined into a single efficiency contour plot in Figure 4.7. The data set for the plot was extended by combining both the swell and storm values using the relationship η (swell) = η (storm) - 30; the plot is therefore set out for wave steepnesses of 0.02, and is only applicable to 2m H_s waves. Tidal currents are not considered in this plot.

The contour plot can be used to determine the optimum combination of structural dimensions for a given situation. Apart from the requirement to acheive a particular level of longshore transport, other design considerations might include cost of materials, cost of stabilizing the substrate, visual impact, navigational safety, desirability of providing safe mooring for small craft and possible use as an amenity platform. Although the model variables were limited, the method used to obtain the contour plot could be extended to form a full set of design plots for detached breakwaters. Alternatively, the information derived from the study could be used to calibrate existing numerical models which predict wave energy in the lee of structures.

As noted in the introduction to the backwaters discussion, this work does not consider the transport of sand around structures which are designed to control a shingle beach. Field experience accumulated subsequent to completion of the model programme shows clearly that sand from both alongshore and cross-shore can be deposited in the lee of the structures. These deposits can be extensive and can substantially increase the efficiency of the structures, causing downdrift erosion and unwanted beach development. Although this study does not address this important issue, the results are not invalid. They continue to form the first step in the development of a design procedure. Future work must address the problem from the outset.

These results qualitatively support available field observations. Further field work should be undertaken as areas of potential rip currents or deposition will be important to breakwater design.

4.4 Results of paired breakwater tests

Only three tests of breakwater pairs were carried out as a post-script to the single structure study. The results are presented in Table 4.1 (Tests 32, 33 and 34). They are insufficient for rigorous analysis, but the following observations can be made.

As might be expected, a larger gap width (G_s) resulted in a slightly lower efficiency, while a change of freeboard from +0.5 to -0.5 resulted in a substantial drop. Comparison against single structures indicated that two emergent 60m long breakwaters with a 60m gap have a η value approximately equal to a single 60m structure, indicating that the efficiency data collected for the single breakwaters may be applicable to multiple structures if approprate gap widths can be determined. However, when submerged the pair of structures became significantly more effective than a single submerged structure suggesting that the relationships may not be straight forward. Further work is required before any conclusions can be drawn on the influence of multiple structures.

5 Conclusions

Shingle beach response in the presence of groynes and detached breakwaters was investigated using a 1:50 mobile bed physical model. The study is part of an ongoing coastal research programme at HR Wallingford.

The groynes study looked at both timber and rock structures. Structural variables included groyne length, head elevation, profile shape and spacing. Sea condition variables were wave height, wave steepness, offshore wave direction and water level. Groyne effectiveness was determined from measurements of longshore transport, cross-shore distribution of transport and beach profiles, plan shapes and volumes.

The breakwaters study concentrated on single detached rubble mound structures, but concluded with a brief series of tests on pairs of structures Structural variables included length, crest elevation, distance offshore and gap width. The sea conditions only varied in wave steepness and water level.

Groynes
The groynes study acheived a number of useful conclusions. However it was not successful in acheiving general design guidelines due to the limited number of variables tested and, to some extent, the complexity and inconclusive nature of some of the results. The following conclusions can be drawn.

1 Groyne efficiency (the percentage of the potential drift retained) changes over time as the updrift beach builds out. Therefore numerical modelling using a single defined efficiency for a given groyne type is not appropriate in many situations.

2 Groynes reach a point of zero efficiency while their crests are still above the beach over at least part of the active beach profile. Drift over and around groynes is controlled by onshore-offshore transport processes which may dominate longshore processes. Drift concentrates at the points of least resistance, resulting in a local cross-shore distribution of drift that may be quite different from that of an open beach. Numerical modelling that does not consider the influence of onshore-offshore transport will over predict the development of the updrift beach and therefore the beach volume.

3 Shingle can be transported as short term suspended load, as well as bedload, within the breaker zone. The physical model suggests that under 2m H_s waves a percentage of the drift can be transported over a groyne that is up to 1m above the beach profile and that a groyne crest that is less than 0.25m above the beach does not present a significant barrier to longshore transport. These elevations are probably greater than would be found in prototype due to the distortion in the geometric

scale of the model material. However, the potential for significant suspension of shingle within the breakers zone has been identified and should be considered.

4 The study indicates that the effective length of a groyne can be defined by the intercept of the groyne profile with an elevation approximately equal to 0.75 of the breaking wave height for storm waves (S=0.06). This relationship assumes a groyne slope of between 1:2 and 1:7.5, and a constant water level. During the model tests any further seaward extension of a groyne at a lower elevation was redundant. The updrift beach profile was about 0.25m higher than the groyne at this threshold point. This conclusion has limited application in a tidal environment in which both the SWL and the wave conditions are constantly varying.

5 Comparison between the model results and the numerical beach profile response model SHINGLE showed very good correlations for storm wave conditions and for the upper beach under swell waves. The lower beach profile shape under swell waves was not well represented by the numerical model.

Assuming that groynes are designed to acheive a satisfactory response under storm conditions and that the beach profile can be located relative to the groyne profile based on the potential for cross-groyne transport (Conclusion 3), then the SHINGLE model can be used to determine the cross-shore position of the updrift beach.

6 A relationship between offshore wave direction, groyne efficiency and beach orientation was derived. However, for design purposes the beach orientation to the mid-bay point should be taken as normal to the breaking wave direction. The tests did not produce any consistent information regarding beach planshapes immediately downdrift of the groynes. Until more conclusive evidence is available then a conservative approach to the potential for downdrift erosion will have to be applied.

7 Timber groynes tend to cause updrift scour of the beach crest when available drift volumes are limited. Scour is much less evident with rock groynes. Surface emergent rock groynes were found to allow the beach crest to develop further forward, relative to the transition point, than lower rock or timber groynes.

8 Barrier type rock groynes were found to have the least updrift beach volume difference between a fully developed beach and a fully eroded beach. Minimizing volume change ensures that recharge material is retained, while natural longshore drift suffers minimal interruption. As these criteria are critical to the success of a controlled beach scheme then the barrier type rock groynes showed the best results of the configurations tested. Under tidal conditions it is likely that the barrier profile could be modified to optimize performance and construction costs.

9 Low timber groynes were ineffective during tests with 3m H_s waves and an unrestricted input of drift material. The beach profiles and planshape were essentially the same as for an open beach, except that the groynes caused some localized scour at the beach crest. No other groyne configurations were tested with the 3m waves.

Detached breakwaters.
The breakwaters study achieved conclusive results within the limits of the variables investigated. The following conclusions can be drawn.

1 The modelling work that has been completed provides a substantial data base for use in further physical modelling of single or multiple structures and for the development and calibration of numerical models based on wave transformation in the lee of breakwaters. The models reproduced the response of shingle beaches, and assumed that the sand portion of the beach had no impact on the shingle. Field experience has shown this to be an oversimplification. Future work must consider the effect of the structure on sand and the relationship between sand and shingle deposition.

2 Breakwaters can be used to stabilize an existing or recharged beach where the natural drift has a strong dominant direction and both the potential and actual drift rates can be determined. Successful design depends on matching the breakwater geometry to the actual natural drift under the dominant wave and water level condition. Therefore, breakwaters on beaches with high gross transport, but low nett transport are likely to cause unwanted areas of scour and accretion, as the structures can only be designed correctly for one drift direction.

3 An efficiency contour plot was derived from the study which relates the structure length/offshore distance ratio to freeboard. Application of this plot is limited as the range of sea conditions tested was restricted and no field verification is available. However it provides a useful first step in developing design guide lines and can be used to optimize structural geometry with respect to construction costs and other design factors.

4 The relationships of efficiency with structure length (L_s) and freeboard (R_c) were clearly established. R_c is dominant when the structure crest is submerged due to the high level of wave energy transmission through and over the structure. L_s becomes the dominant factor as R_c increases above zero.

5 The influence of the offshore distance of the structures (X_s) was less conclusive. Most tests were run with an X_s of 90m or 120m. These distances were too similar to show any distinct trend. Comparison of efficiency with the ratio of wave length to offshore distance (L_m/X_s) suggests that there may be an efficiency minimum when the ratio is between 0.4 and 0.9. However the available data set is insufficient to confirm this possibility.

6 Tests on wave induced currents showed the potential for strong rip currents around the downdrift end of the breakwaters. The currents observed were intermittent at periods associated with between 8 and 12 incident waves. They were also dependant on the depth of water in the lee of the structure, with peak currents being observed at depths of between 1m and 3m. The study did not investigate the influence of tidal currents on beach response.

7 Breakwater efficiency also appears to be dependant on the size of the beach salient. As the salient extends out towards the structure the efficiency increases due to more of the wave energy being dissipated on

the beach and less being available for transport. This applies particularly to structures with a large freeboard where the transmitted energy level is low. It is therefore important not to over design structures initially, but to allow for minor modifications after construction based on monitoring results. Monitoring should include adjacent beaches as well as the immediate leeward beach.

8 The selected design wave and water level conditions should represent the range of conditions which cause the greatest proportion of the annual drift. Designing for storm protection is liable to cause over design and tombolo development which will give rise to downdrift erosion.

6 Recommendations

The present study has gone some way towards developing design guidelines for groynes and breakwaters. Further work is necessary before the results are of general use. The following recommendations suggest ways to achieve this goal.

Groynes

1. Field studies are needed to verify the model results. In particular the transport of shingle along open and controlled beaches must be studied in greater depth to give confidence to the prediction of potential and actual drift rates. The development of the Electronic Pebble System will make this possible (Reference 12). Studies of the suspension of shingle in the surf zone are necessary to verify the conclusions of the groynes study regarding cross-groyne transport, and studies of offshore losses of coarse material are necessary to varify this observed model process.

Offshore breakwaters

2. Further model studies on breakwaters are needed to investigate additional wave directions, wave heights, wave lengths and offshore distance. The multiple structure study initiated here needs to be extended into a full programme and should include an investigation of submerged structures. Prior to this work, serious consideration must be given to the problems of combining sand and shingle transport, as field experience has shown the interdependance of the two processes.

3. The influence of tidal currents on beach response in the lee of breakwaters needs investigation. This study should also look at the potential for the deposition of fines and the occurrence of rip currents around structures.

4. Existing numerical models of wave transformation in the lee of breakwaters should be extended to include longshore transport ratio, using the model results and field measurements for calibration.

7 Engineering applications

The conclusions and recommendations of this study acknowledge the limitations of the physical model study and the lack of field verification data, and therefore the lack of any directly applicable design tools for coastal managers and engineers. This final chapter is intended to address this deficiency by providing design guidance based on the study and on experience gained elsewhere. The reader should be aware that this work is continually developing and that each coastal situation is unique and must be fully understood before any design procedures are applied. As yet there are no numerical design models which are able to simulate longshore shingle beach development in the presence of control structures. Site specific physical models remain the most effective design tool available to coastal engineers.

Most shingle beaches in the UK are of a type known as shingle upper/sand lower (Reference 7), and it is this type of beach that is referred to by this study. Under open beach situations, the shingle fraction of these beaches tends to remain relatively independent of the sand and the majority of shingle transport activity takes place over a relatively narrow zone. Under extreme storm conditions the upper shingle beach will be cut back and may be overtopped but, given a reasonable supply of material, will recover under subsequent swell conditions. Very little material will be lost offshore, though there is field evidence that larger clasts may be removed. The purpose of the beach control structures is to ensure that the volume of shingle retained along the shoreline is sufficient to prevent storm cut back from extending landward to a point where there is a risk of significant damage to property or the environment. This retention of material must not be at the expense of downdrift beaches under either storm or more frequent conditions, so beach recharge will normally be required in association with the structures. The need to manage beaches under all wave conditions necessitates compromises between the need for short term storm defence and long term beach stability, though achieving the latter should also achieve the former except in very extreme situations.

Design of groynes and breakwaters are discussed independently , followed by a brief discussion of their use in combination. The discussions assume a meso to macro tidal range (>2m) and the potential for extreme events combining high water levels with storm waves.

Groynes

Groynes are shore connected structures set approximately normal to the coastline. They range from short sloping timber structures to the massive hybrid structures referred to as shore connected breakwaters. Their purpose is to retain beach material as a means of providing local shoreline protection. Their effectiveness and area of influence depend on their dimensions, interactions with local wave and tidal processes, and the availability of drift material. In general they should only be considered in combination with a beach rechange.

The design approach discussed here assumes that a successful groyne field achieves the following:

- retention of a beach which provides erosion and flood protection to a specified threshold line along the full scheme frontage under the design storm conditions;

- minimal impact on downdrift frontages due to disruption of longshore transport;

- maximum stability of the groyne bays by minimizing the profile and planshape variations between fully eroded and fully developed.

In general, groynes are most appropriate to frontages with high gross drift, but low nett drift and where an equal level of protection is required along the full scheme length.

This study and others, including field observations, conclude that highly reflective, vertical sided groynes are effective under conditions of high shingle availability and moderate wave conditions. If the shingle supply is limited, which is often the case for natural beaches, or if inshore wave conditions are extreme, say 2.5m H_s waves breaking on the shingle, then vertically faced groynes tend to cause updrift scour which may lead to exposure of the groyne root and foundations with consequent potential for backshore and structural damage. Under these potential risk conditions the use of less reflective structures formed of rock or concrete armour units will be required. Non-vertical structures are also appropriate to frontages with a steep angle of wave attack. Large differences in cross-groyne beach profile elevations, typical of steep wave angle beaches, will result in structural instability of vertical structures.

The cross-shore distribution of longshore transport in the presence of groynes can be very different from the open beach distribution. On an open beach longshore processes dominate and measurements produce a distribution curve with a broad peak between the water line and the seaward edge of the breaker zone. Groynes will cause the curve to be substantially redistributed as local cross-shore transport processes will move material to the point of least resistance along the line of the groyne. This redistribution may be shorewards if the groyne has a low crest, or seawards if the crest is above run-up limit of the waves. This process is not accounted for by existing numerical beach development models.

The physical model results provide an approximation of the minimum effective elevation of a groyne, below which the structure ceases to have any significant effect on longshore processes. This level is approximately SWL - 0.75 (H_b). Unfortunately this elevation is dependant on water level and wave height, which are not constant in the real world. The designer must consider these variations when deciding on the length of a low level groyne, although in practice groynes will normally extend across the full width of the shingle portion of a sand/shingle beach or down to low water, whichever comes first.

The often quoted concept of designing groynes based on the total length and spacing is of limited use. A better approach is to define the shape, dimensions and spacing of the groynes based on the potential variations in beach planshape. The process is as follows:

- define the required beach threshold line, that is the line beyond which erosion must not pass, for the full scheme frontage;

- determine the range of breaking wave orientations for the dominant storm directions, thereby establishing the envelope of beach orientations.

- determine the potential envelope of beach profiles under the design storm conditions using a numerical profile model that considers the influence of sediment grading.

- set out groynes so that the groyne berm length and groyne spacing prevent the beach crest from cutting back to the threshold line under the worst case design storm condition (Figure 7.1). Obviously this can be achieved using long groynes with wide spacings or short groynes with narrow spacings. However, the wider the spacing the greater the volume of shingle needed to maintain a satisfactory beach. Conversely the narrower the spacing the greater the number and cost of groynes. Optimization may also consider non-hydraulic factors such as environmental and visual impact.

- define the groyne face profile based on the beach profile envelope. The groyne should be about 1.0m above the intended design profile to reduce cross-groyne transport. The groyne crest need not be any higher than the design beach crest, and could be slightly lower as very little transport will occur at the limit of wave run-up. The groyne profile can follow the design beach profile, though many successful groynes are near horizontal structures which only allow transport around the seaward end. These massive structures are obviously more costly to construct and there is no existing method for predicting the maximum extent of beach development.

- set out the volume of beach recharge material required, using the dominant storm wave breaking direction to determine orientation and a shingle beach profile model to determine profile shape.

- overfill the pinch point area to allow extra volume which may be lost as the beach settles.

Detached breakwaters

Detached breakwaters are shore parallel structures, that can be either emergent or submerged and that control the beach by modifying the wave climate in their lee. They are generally constructed from armour stone, with or without a core. In general, the beach in their lee will develop into a salient, and may form a tombolo. If a tombolo forms then the structure becomes a total barrier to longshore drift in its lee and effectively becomes a shore connected breakwater. Effectiveness depends on structure length, freeboard, distance offshore, permeability and, in the case of multiple-structure systems, their spacing.

A successful breakwater scheme will reduce transport along the leeward beach to a rate which is in equilibrium with the available natural drift, thereby stabilizing the local shoreline and ensuring that the downdrift shoreline is not adversely affected. If the structures are combined with a recharge, then the recharge should remain in dynamic stability and should provide the required level of protection to the scheme frontage. Breakwaters are most appropriate to shorelines with a strongly dominant drift direction and where variable levels of protection are acceptable as the structures will result in greater protection

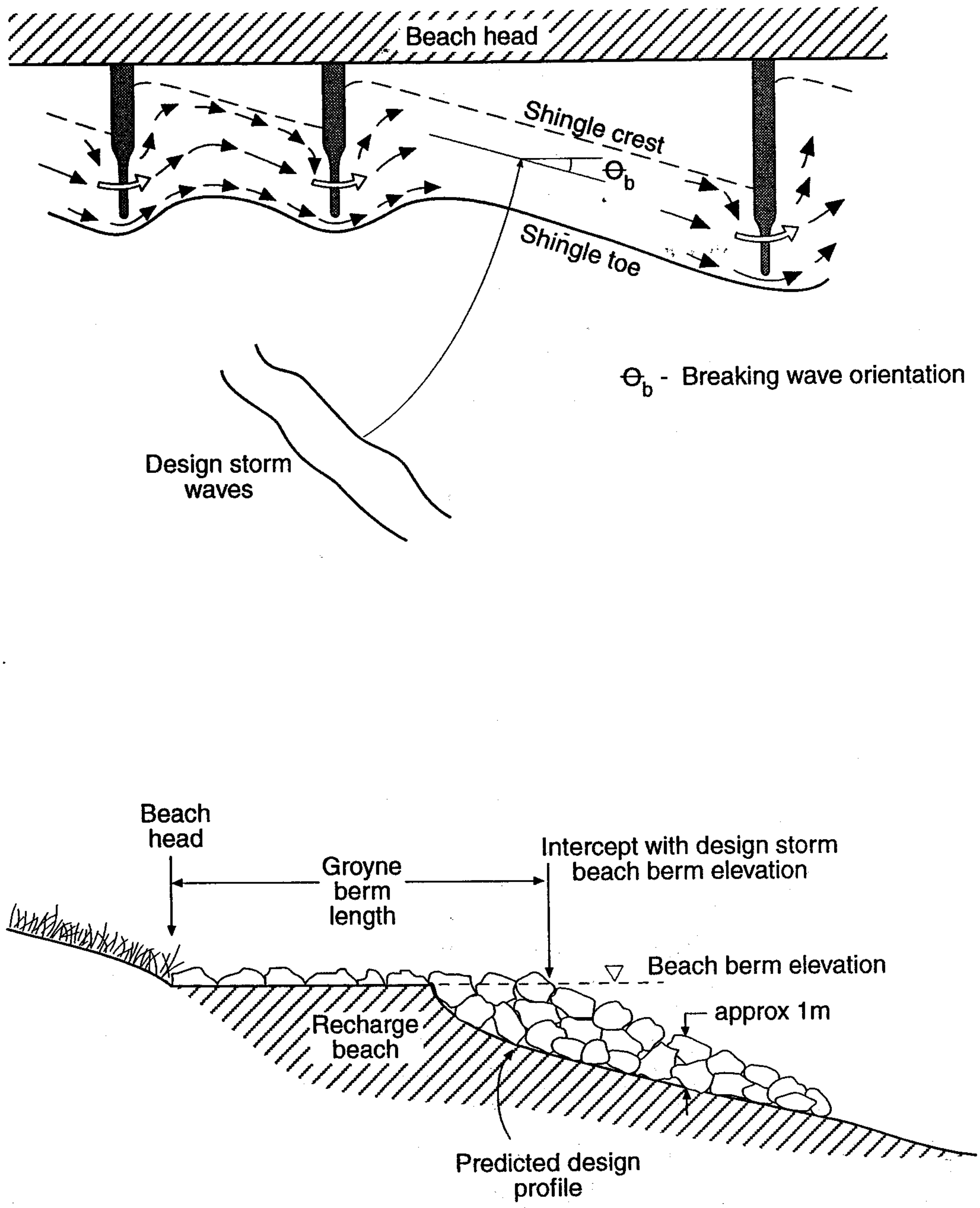


Figure 7.1 Design guidelines for groynes on shingle beaches

in their lee and lower protection in the lee of the gaps between structures. It should be noted that design procedures which concentrate on predicting the formation of salients or tombolos are of limited use - if these features form without being placed as beach recharge, then the structures are acting as barriers to natural drift and are therefore not successful according to the terms of this study.

The design procedure that has emerged from this study is still embryonic as there are insufficient data on variable wave heights and directions, and the influence of cross-shore structure location. However the following will be a useful starting point:

- determine the potential drift (Q_p) for the study frontage, based on the long term nearshore wave climate and the CERC formulae;

- determine the actual input drift rate Q_i, based on sediment budget studies;

- determine the required long term efficiency from $\eta = \left(1 - \frac{Q_i}{Q_p}\right) x\ 100$;

- use Figure 4.7 to determine the range of structural dimension combinations that will achieve the efficiency.

- select a combination that fulfils the secondary considerations of the scheme, such as:

  - need for storm protection at all points along frontage
  - visual impact
  - construction costs
  - construction methods
  - substrate stability
  - navigation safety
  - deposition of fines.

The present research has not investigated multiple structures in sufficient depth to suggest design procedures, nor has it covered the impact of tidal currents. Future work will be directed at these developments.

The proposed method only considers the shingle element of sand/shingle beaches. Site experience at Elmer indicates that this approach can lead to problems which may be difficult to resolve. Structures which are correctly designed for shingle transport may be too effective for sand, resulting in the formation of a sand tombolo at mid to low water levels. This result is due to the differences in cross-shore distribution of sand and shingle transport, and the greater availability of sand from both alongshore and nearshore sources. Deposition of sand in the lee of the structures will reduce the water depth and therefore the wave climate, resulting in a reduction of shingle drift. This cycle of drift reduction may ultimately lead to tombolo formation and erosion of downdrift beaches. Further work is required to address this problem.

8 Acknowledgements

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Tables

Table 2.1 Groyne layout summary

Layout	Tests	Type	Spacing (m)	Elevation relative to initial beach (m)	Head elevation [1] (m)	Crest length [2] (m)	Toe length [2] (m)	Length to 2.75m elevation [2] (m)	Effective area [3] (m^2)
1	1-11,17	Low timber	125	1.5	1.5	65	65	55.6	127
2	23,24,39	Low timber	95	1.5	1.5	65	65	55.6	127
3	18-22	Low timber	65	1.5	1.5	65	65	55.6	127
4	35,36,37,38,43	High timber	95	3.0	3	65	65	65.0	169
5	12-16	High end timber	125	1.5	4	65	65	65.0	150
6	25,26	High end timber	95	1.5	4	65	65	65.0	150
7	28,29	Low rock	125	1.5	1.5	65	65	55.6	129
8	30,31	High end rock	125	1.5	4	65	73	67.5	166
9	40,41	High end rock	95	1.5	4	65	73	67.5	166
10	32.1,32.2	Short rock barrier	125	1.5	6	53	65	59.5	166
11	34.1,34.2 34.3,42	Short rock barrier	95	1.5	6	53	68	59.5	166
12	32.3,32.433	Long rock barrier	125	1.5	6	65	77	71.5	238
13	27	Toe barrier	125	*	*	*	*	*	*

* Varied through test
1 Elevation measured relative to toe of initial beach
2 Length measured relative to beach head
3 Effective area has been taken as the groyne area below 6m and seaward of 26m relative to the beach head

Table 2.2 Breakwater layout summary

Layout	Tests	Length (m)	Offshore distance [1] (m)	Crest elevation [2] (m)	Gap (m)
1	1,9,10	120	120	4.5	--
2	2,3,4	90	120	4.5	--
3	5,6,7,8	60	120	4.5	--
4	18,19	120	90	4.5	--
5	20,21	90	90	4.5	--
6	22,23	60	90	4.5	--
7	30	90	60	4.5	--
8	31	90	150	4.5	--
9	16,17	120	120	3.5	--
10	11,12,13,14,15	90	120	3.5	--
11	27,28,29	90	90	3.5	--
12	24,25,26	60	90	3.5	--
13	32-33	60	90	3.5	60
14	34	60	90	3.5	90

1 Offshore distance measured relative to the 3m water line on the initial beach
2 Elevation measured relative to the toe of the initial beach

Table 2.3 Calibrated sea conditions

Condition	H_s (m)	T_m (s)	Steepness (H_s/L_m)	SWL [1] (m)	Open beach drift - 1st calibration (model kg/1000 T_m)			Open beach drift - 2nd calibration (model kg/1000 T_m)
					15°	30°	45°	30°
1	1	5.7	0.02	2	-	-	-	-
2	1	4.0	0.04	2	-	-	-	-
3	1	3.3	0.06	2	-	-	-	-
4	1	5.7	0.02	4	-	-	-	-
5	1	4.0	0.04	4	-	-	-	-
6	1	3.3	0.06	4	-	-	-	-
7	2	8.0	0.02	2	6.0	7.5		-
8	2	5.7	0.04	2	2.6	3.0	6.0	-
9	2	4.6	0.06	2	1.5	2.0	2.8	-
10	2	8.0	0.02	3	-	16.0		20
11	2	5.7	0.04	3	-	4.8		-
12	2	4.6	0.06	3	-	3.5		4.4
13	2	8.0	0.02	4	13.5	25.0	28.0	30
14	2	5.7	0.04	4	3.0	5.5	10.6	-
15	2	4.6	0.06	4	2.5	4.8	7.1	5.7
16	3	6.9	0.04	2	5.0	6.0	9.0	-
17	3	5.7	0.06	2	4.5	5.0	7.4	-
18	3	6.9	0.04	4	22.0	36.0	-	-
19	3	5.7	0.06	4	15.0	25.0	-	-

1 SWL measured relative to the toe of the initial beach

Table 2.4 Test programme and conditions: Series 1 - Low timber groynes

Test no	Groyne type	Groyne spacing	Wave steepness (H_s/L_m)	H_s required (m)	H_s actual (m)	Wave direction (°)	SWL (m)	Model drift input (kg/1000T_m)	Duration (Tm)
3	low timber	2:1	0.06	2.00	**1.86	30°	4.00	4.75	62000
4	low timber	2:1	0.02	2.00	1.92	30°	4.00	25	20000
6	low timber	2:1	0.06	2.00	1.99	30°	2.00	2	40000
7	low timber	2:1	0.02	2.00	2.03	30°	2.00	7.5	68000
9	low timber	2:1	0.02	2.00	2.05	30°	3.00	16	34000
10	low timber	2:1	0.06	2.00	2.02	30°	3.00	3.5	40000
17	low timber	2:1	0.06	3.00	**2.83	30°	4.00	25	32000
18	low timber	1:1	0.06	2.00	2.05	30°	4.00	4.75	48000
19	low timber	1:1	0.02	2.00	1.97	30°	4.00	25	16000
20	low timber	1:1	0.06	3.00	2.99	30°	4.00	25	32000
21	low timber	1:1	0.02	2.00	2.04	30°	3.00	16	20000
22	low timber	1:1	0.06	2.00	2.08	30°	3.00	3.5	48000
23	low timber	1½:1	0.02	2.00	2.03	30°	4.00	25	18000
24	low timber	1½:1	0.06	2.00	**2.11	30°	4.00	4.75	44000

** values high or low by ≥ 5% reflective to required

Table 2.5 Test programme and conditions : Series 2 - Other groynes

Test no	Groyne type	Groyne spacing	Wave steepness (H_s/L_m)	H_s required (m)	H_s actual (m)	Wave direction (°)	SWL (m)	Model drift input (kg/1000T_m)	Duration (T_m)
12	high end timber	2:1	0.02	2.00	**1.88	30°	4.00	25	v36000
13	high end timber	2:1	0.06	2.00	**1.93	30°	4.00	4.75	116000
14	high end timber	2:1	0.02	2.00	1.98	30°	3.00	16	28000
15	high end timber	2:1	0.06	2.00	1.93	30°	3.00	3.5	56000
16	high end timber	2:1	0.02	2.00	1.98	30°	2.00	7.5	36000
25	high end timber	1½:1	0.02	2.00	2.02	30°	4.00	25	24000
26	high end timber	1½:1	0.02	2.00	2.02	30°	4.00	0	44000
27A	low rock ‡	1:1	0.02	2.00	1.93	30°	4.00	25	8000
27B	low rock ‡	1:1	0.02	2.00	2.08	30°	4.00	25	8000
27C	high rock ‡	1:1	0.02	2.00	**1.90	30°	4.00	25	8000
28	low rock	2:1	0.06	2.00	2.09	30°	4.00	4.75	40000
29	low rock	2:1	0.02	2.00	1.99	30°	4.00	25	24000
30	high end rock	2:1	0.02	2.00	2.00	30°	4.00	25	32000
31	high end rock	2:1	0.06	2.00	2.06	30°	4.00	4.75	60000
32A	short rock barrier	2:1	0.02	2.00	1.97	30°	4.00	4.75	20000
32B	short rock barrier	2:1	0.06	2.00	1.98	30°	4.00	4.75	24000
32C	long rock barrier	2:1	0.02	2.00	1.94	30°	4.00	25	20000

** values high or low by ≥ 5% relative to required ‡ toe only, detached groyne

Table 2.5 Test programme and conditions : Series 2 - Other groynes (continued)

Test no	Groyne type	Groyne spacing	Wave steepness (H_s/L_m)	H_s required (m)	H_s actual (m)	Wave direction (°)	SWL (m)	Model drift input (kg/1000T_m)	Duration (T_m)
32D	long rock barrier	2:1	0.06	2.00	1.99	30°	4.00	4.75	48000
33	long rock barrier	2:1	0.02	2.00	2.04	30°	4.00	0	24000
34A	short rock barrier	1½:1	0.02	2.00	1.91	30°	4.00	25	20000
34B	short rock barrier	1½:1	0.06	2.00	2.02	30°	4.00	4.75	20000
34C	short rock barrier	1½:1	0.06	2.00	1.98	30°	4.00	0	7000
35	high timber	1½:1	0.02	2.00	2.04	30°	4.00	25	20000
36	high timber	1½:1	0.06	2.00	2.01	30°	4.00	4.75	76000
42	short rock barrier	1½:1	0.02	2.00	1.99	30°	4.00	0	27000
43	high timber	1½:1	0.02	2.00	1.93	30°	4.00	0	38000

** values high or low by ≥ 5% relative to required ‡ toe only, detached groyne

Table 2.6 Test programme and conditions : Series 3 - 15° wave direction

Test no	Groyne type	Groyne spacing	Wave steepness (H_s/L_m)	H_s required (m)	H_s actual (m)	Wave direction (°)	SWL (m)	Model drift input (kg/1000T_m)	Duration (T_m)
37	high timber	1½:1	0.02	2.00	2.07	15°	4.00	13.5	40000
38	high timber	1½:1	0.06	2.00	1.98	15°	4.00	2.5	152000
39	low timber	1½:1	0.02	2.00	2.03	15°	4.00	13.5	25000
40	high end rock	1½:1	0.02	2.00	**2.10	15°	4.00	13.5	36000
41	high end rock	1½:1	0.06	2.00	**2.15	15°	4.00	0	28000

** values high or low by ≥ 5% relative to required

Table 2.7 Test programme and conditions: Single breakwaters

Breakwater				Sea Conditions			
Test	Length (m)	Distance offshore (m)	Crest elevation (m)	H_s (m)	H_s/L_m	SWL (m)	Duration (T_m)
1	120	120	4.5	2	0.02	3	98,000
2	90	120	4.5	2	0.02	4	100,000
2b	90	120	4.5	2	0.02	4	8,000
3	90	120	4.5	2	0.02	3	28,000
4	90	120	4.5	2	0.06	4	32,000
5	60	120	4.5	2	0.02	3	28,000
6	60	120	4.5	2	0.02	4	24,000
7	60	120	4.5	2	0.06	4	36,000
8	60	120	4.5	2	0.06	3	24,000
9	120	120	4.5	2	0.02	4	24,000
10	120	120	4.5	2	0.02	4.5	28,000
11	90	120	3.5	2	0.02	3	26,000
12	90	120	3.5	2	0.06	3	40,000
13	90	120	3.5	2	0.02	3.5	28,000
14	90	120	3.5	2	0.06	4	30,000
15	90	120	3.5	2	0.02	4	24,000
16	120	120	3.5	2	0.02	3	24,000
17	120	120	3.5	2	0.02	4	24,000
18	120	90	4.5	2	0.02	3	20,000
19	120	90	4.5	2	0.02	4	24,000
20	90	90	4.5	2	0.02	3	24,000
21	90	90	4.5	2	0.02	4	24,000
22	60	90	4.5	2	0.02	3	24,000
23	60	90	4.5	2	0.02	4	24,000
24	60	90	3.5	2	0.02	3	24,000
25	60	90	3.5	2	0.02	4	24,000
26	60	90	3.5	2	0.06	3	24,000
27	90	90	3.5	2	0.06	4	24,000
28	90	90	3.5	2	0.02	3	24,000
29	90	90	3.5	2	0.02	4	20,000
30	90	60	4.5	2	0.02	4	16,000
31	90	150	4.5	2	0.02	4	20,000

Table 2.8 Test programme and conditions: Double breakwaters

Test	Length : L_s (m)	Spacing : G_s (m)	Distance offshore: X_s (m)	Crest elev : R_c (m)	H_s (m)	H_s/L_o	SWL (m)	Duration (T_m)
32	60	60	90	3.5	2	0.02	3	22,000
33	60	60	90	3.5	2	0.02	4	24,000
34	60	90	90	3.5	2	0.02	3	24,000

Table 3.1 Results - Series 1 Groynes

Test No	Pinch point (m)	Mean volume (m^3/m)	Updrift orientation ($\theta°$)	Updrift profile area (m^2)
3	22	250	9	149
4	19	243	12	174
9	24	228	9	-
10	26	233	12	-
17	19	241	3	-
18	30	224	13	-
19	29	233	7	-
20	27	231	-4	-
21	34	217	12	-
22	37	207	16	-
23	23	249	9	-
24	21	236	16	-

Table 3.2 Results - Series 2 Groynes

Test No	Pinch point (m)	Mean volume (m^3/m)	Updrift orientation ($\theta°$)	Updrift profile area (m^2)
12	22	250	13	205
13	33	294	11	196
14	26	252	12	-
15	27	250	14	-
25	30	269	9	-
26	12	204	15	194
28	16	236	18	-
29	18	260	12	141
30	29	300	12	181
31	21	275	17	235
32A	21	267	13	200
32B	22	251	14	213
32C	37	327	13	192
32D	35	315	14	310
33	24	285	17	272
34A	30	277	11	
34B	28	259	13	
34C	17	236	12	
35	30	286	12	219
36	40	302	5	189
42	20	247	12	
43	16	224	14	

Table 3.3 Results - Series 3 Groynes

Test No	Pinch point (m)	Mean volume (m^3/m)	Updrift orientation ($\theta°$)
37	37	310	5
38	41	302	3
39	24	263	9
40	29	294	8

Table 3.4 Comparison of full and zero input tests

Test No	Groyne Type	Pinch Point difference (m)	Volume difference (m^3/m)	Comment
25 and 26	High end timber	18	65	1½:1 spacing
35 and 43	High timber	14	62	1½:1 spacing
32c and 33	Long rock	13	42	2:1 spacing
34a and 42	Short rock	9	30	1½:1 spacing
34b and 34c	Short rock	10	23	1½:1 spacing, storm condition

Table 4.1 Breakwater efficiency

	Drift		
Test	Input (model kg/1000 T_m)	Output (model kg/1000 T_m)	Efficiency (%)
1	20.00	2.9	86
2	30.00	13.5	55
3	20.00	5.2	74
4	5.75	0.7	88
5	20.00	10.7	47
6	30.00	22.8	24
7	5.75	2.3	60
8	4.4	1.3	70
9	30.00	5.0	83
10	34.00	10.1	70
11	20.00	8.0	60
12	4.4	0.6	86
13	25.00	16.6	34
14	5.75	2.5	57
15	30.00	25.2	16
16	20.00	6.4	68
17	30.00	19.8	34
18	20.00	3.4	83
19	30.00	9	70
20	20.00	5.2	74
21	30.00	10.2	66
22	20.00	8.2	59
23	30.00	22.2	26
24	20.00	12.4	38
25	30.00	29.8	1
26	5.75	4.5	22
27	5.75	2.9	50
28	20.00	7.9	61
29	30.00	23.6	21
30	30.00	9.6	68
31	30.00	23.3	22
32	20.00	12.6	37
33	30.00	25.9	14
34	20.00	13.4	33

Appendices

Appendix 1

Scaling of model sediments

Scaling of model sediment

Selection of model sediment
Ideally the model sediment should satisfy three criteria:

- Permeability of the shingle beach should be correctly reproduced
- The relative magnitudes of the onshore and offshore motion should be correctly reproduced
- The threshold of motion should be correctly scaled

The first of these basically governs the beach slope, the second determines whether the beach will erode or accrete under given wave conditions, and the third determines the wave velocity at which sediment motion will begin.

Reproduction of Permeability
Yalin published a paper in 1963 describing a method for modelling shingle beaches with the correct permeability and drag forces. For the permeability he said that in an undistorted model the percolation slope must be identical to the prototype, where

$$J = k(Re_v)v^2/gD_{10}$$

with J = percolation slope
k = permeability, a function of ......
Re_v = voids Reynolds Number vD_{10}/υ
v = velocity through the voids
D_{10} = 10% undersize of the sediment
υ = kinematic viscosity

For identical percolation slopes in model and prototype this gives

$$\lambda_v^2\lambda_k/\lambda_D = 1$$

where λ is the model scale (prototype value/model value). Assuming that the model is operated according to Froude's Law then $\lambda_v^2 = \lambda$, the geometric scale, so that

$$\lambda\lambda_k/\lambda_D = 1 \qquad \text{(A.1)}$$

Unfortunately permeability is a non-linear function of Reynolds Number. For example, Yalin proposed a steady-flow law, and produced a recommended curve of k against Re_v. This curve can be approximated by the expression.

$$\log k = 3.17 - 1.134 \log Re_v + 0.155 \log^2 Re_v,$$

within the range $1 \leq Re_v \leq 200$

With such a non-linear expression the scaling law will depend on the representative value of the prototype permeability If this is designated k_p, and the Reynolds Number is Re_p, then

$$\lambda_k = k_p/k_m = \lambda_D/\lambda$$

or $\lambda_D = \lambda k_p/k_m$

Now $k_m = k(Re_m)$ where Re_m is the model Reynolds Number, so

$k_m = k(Re_p/\lambda_v\lambda_D) = k(Re_p/\lambda^{1/2}\lambda_D)$

By substituting this expression the implied equation for λ_D is obtained as

$$\lambda_D = \lambda k_p/k(Re_p/\lambda^{1/2}\lambda_D) \qquad \text{(A.1a)}$$

Assuming that k_p and Re_p are known, and the form of the function $k(Re_v)$ is known then this equation can be solved by successive approximation to define the particle size for the model sediment.

Reproduction of Onshore/Offshore Movement

Several authors have postulated that the relative tendency for sediments to move onshore or offshore depends on the dimensionless parameters H_b/wT, where H_b is the wave height at breaking, T is the wave period and w is the settling velocity of the sediment particles. Roughly speaking if $H_b/wT \leq 1$ then the sediment moves onshore, and if $H_b/wT \geq 1$ then offshore movement occurs (see for example Shore Protection Manual, section 4.525). In physical terms the parameter represents the ratio between the wave height and the distance which the sediment particle can settle during one wave period. For correct reproduction of the relative magnitudes of onshore and offshore movement the model scales must therefore be such that

$$\lambda_{H_b}/\lambda_w\lambda_T = 1$$

With a Froudian model $\lambda_T = \lambda^{1/2}$, and assuming that the beach slope is correctly modelled then $\lambda_{H_b} = \lambda$, so that we have $\lambda_w = \lambda^{1/2}$

In general, the settling velocity is given by

$$w = \left(\frac{1.33gD(\rho_s - \rho_f)}{C_D \qquad \rho_f} \right)^{1/2}$$

where ρ_s and ρ_f are specific gravities of the sediment and fluid respectively, and C_D is the drag coefficient for the settling particles.

For modelling purposes we therefore have

$$\lambda_w = \lambda_D^{1/2}\lambda_\Delta^{1/2}/\lambda_{C_D}^{1/2} = \lambda^{1/2}$$

$$\text{or} \quad \lambda_\Delta = \lambda\lambda_{C_D}/\lambda_D \qquad \text{(A.2)}$$

where Δ is $(\rho_s - \rho_f)/\rho_f$

Unfortunately C_D is also a non-linear function, in this case a function of the sediment particle Reynolds Number $Re_s = wD/\upsilon$. The actual scaling will again therefore depend on the typical value of the prototype drag coefficient. Denoting this prototype value as C_{D_p}, and the appropriate Reynolds Number Re_p we therefore have

$$\lambda_{C_D} = C_{D_p}/C_{D_m} = C_{D_p}/C_D(Re_m)$$

$$\lambda_{C_D} = C_{D_p}/C_D(Re_p/\lambda_w\lambda_D)$$

$$\lambda_{C_D} = C_{D_p}/C_D(Re_p/\lambda^{1/2}\lambda_D) \qquad \text{(A.2a)}$$

If C_{D_p} and Re_p are known, and λ_D has also been determined (for example from the permeability scaling) then equation A.2a can be solved for λ_{C_D}, and the value then inserted in equation A.2 to derive ρ_s, the specific gravity of the model sediment. If both model and prototype sediments are coarse grained(roughly greater than 4mm) then $\lambda_{C_D} \sim 1$, thus giving $\lambda_\Delta \sim \lambda/\lambda_D$

Threshold of Motion

For oscillating flow Komar and Miller (1973) proposed that for sediment sizes greater than 0.50 mm, which is expected to be the case for both model and prototype sediments, the threshold of movement was defined by the expression

$$\frac{U_m^2}{\Delta\, gD} = 0.46\,\pi\,\left(\frac{d_o}{D}\right)^{1/4}$$

where U_m is the peak value of the near-bed orbital velocity at the threshold of motion and d_o is the near-bed orbital diameter.

Since $U_m = \pi\, d_o/T$, this expression can be re-written as:

$$U_m^{7/4}/(\Delta\ D^{3/4}T^{1.4}) = 0.46\ \pi^{3/4}g$$

To the first order, the maximum orbital velocity near the bed is given by

$$U_m = \frac{\pi\ H}{T\ \mathrm{Sinh}(2\pi\ d/L)}$$

where L is the wavelength.

Substituting this expression, and rearranging, gives the threshold in terms of wave height and period as

$$H^{7/4}\ A^{7/4}/(\Delta\ D^{3/4}T^2) = 0.46\ g/\pi$$

where A is the depth attenuation factor 1/sinh $(2\pi\ d/L)$

For correct modelling we therefore have

$$\lambda_H^{7/4}\lambda_A^{7/4}/(\lambda_\Delta\lambda_D^{3/4}\lambda_T^2) = 1$$

In a Froudian model $\lambda_H = \lambda_L = \lambda_d = \lambda$ and $\lambda_T = ½$.

Therefore $\lambda_A = 1$. This then gives

$$\lambda_\Delta\lambda_D^{3/4} = \lambda^{3/4} \qquad \text{(A.3)}$$

Summary of Scaling Law

The preceding paragraphs have given the following equations for scaling the model material

For correct permeability: $\lambda_D = \lambda k_p/k(Re_p/\lambda^{½}\lambda_D)$ (A.1a)

For correct onshore/offshore movement:

$$\lambda_\Delta = \lambda\lambda_{C_D}/\lambda_D \qquad \text{(A.2)}$$

where $\lambda_{C_D} = C_{D_p}/C_D(Re_p/\lambda^{½}\lambda_D)$ (A.2a)

For correct threshold of motion

$$\lambda_\Delta\ \lambda_D^{3/4} = \lambda^{3/4} \qquad \text{(A.3)}$$

Assuming that the prototype values k_p, Re_p, etc are known, we then have four equations to solve for the four scale factors λ, λ_D, λ_Δ and λ_{C_D}. However the only solution to these four equations is the prototype situation, ie $\lambda = \lambda_D = \lambda_\Delta = \lambda_{C_D} = 1$. In practice it is necessary to select one of the scales (usually λ), and then decide which of the various scaling requirements are most important. Clearly, having selected one scale we have four equations to solve for three variables, and one of the equations therefore has to be relaxed.

Application

Application of these equations to the required beach grading selected for the present study yields the following sediment requirements at a model scale of 1:50.

Scale	Proto D_{50} (mm)	Model D_{50} (mm)	ρ_s (Threshold of motion)	ρ_s (Onshore/Offshore movement)
1:50	16mm	2.66	1.43	1.38

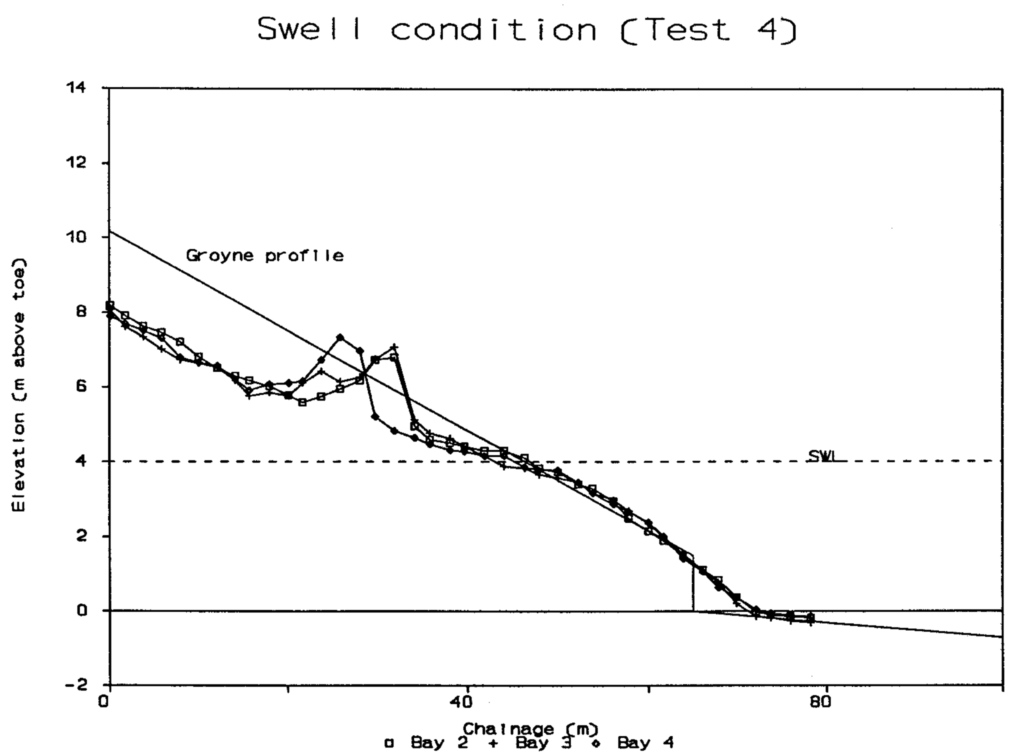
Anthracite has a specific gravity of 1.39 and thus satisfies the requirements for reproducing the correct onshore/offshore movement and threshold of motion. Moreover it is commercially available in a number of size gradings from which the required model mixes can be blended. It is therefore ideally suited for modelling shingle beaches at this scale.

Appendix 2

Beach profiles - Groynes

Appendix 2 Beach profiles - Groynes

Contents



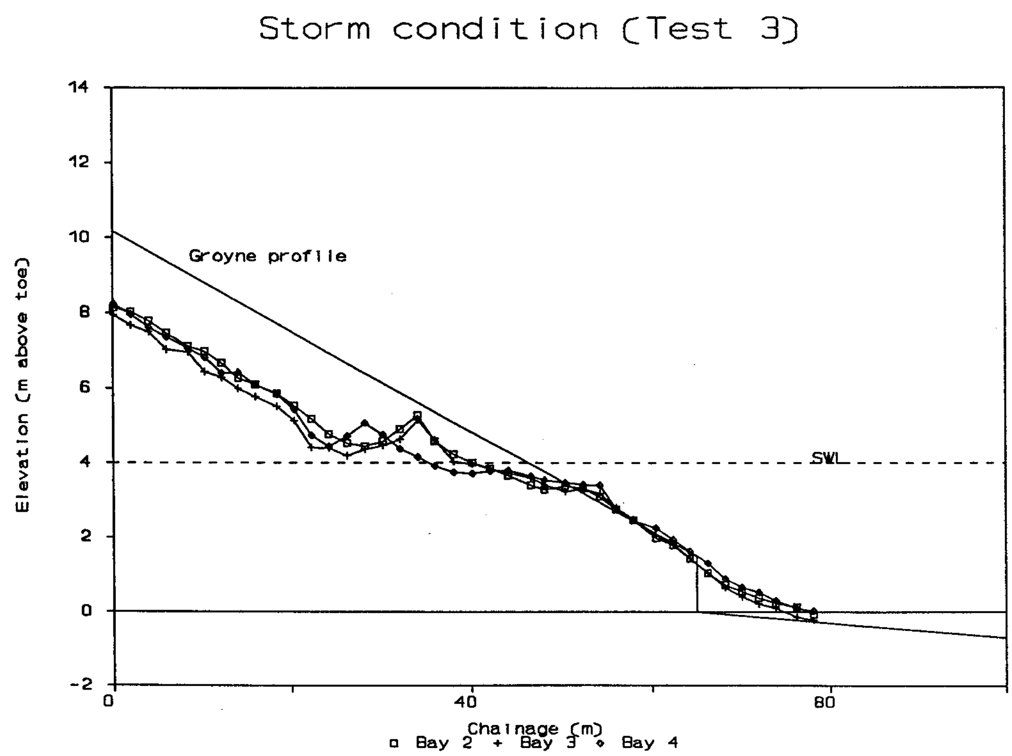
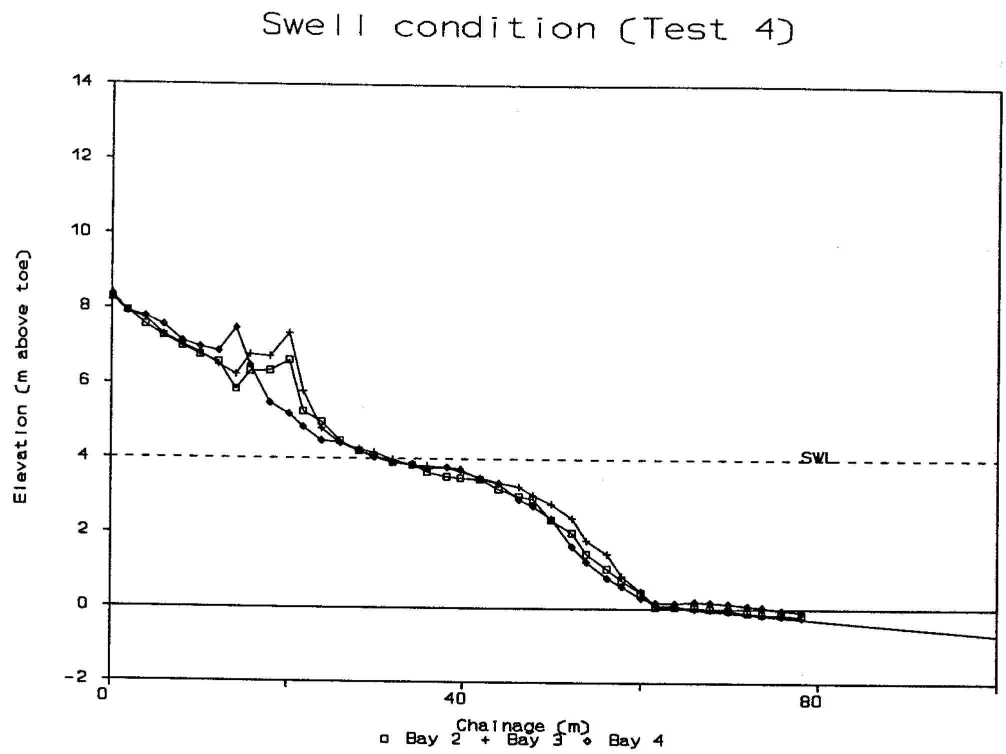


Figure A1 Updrift beach profiles: Low timber groynes, SWL = 4m, spacing = 2:1, 30° wave direction



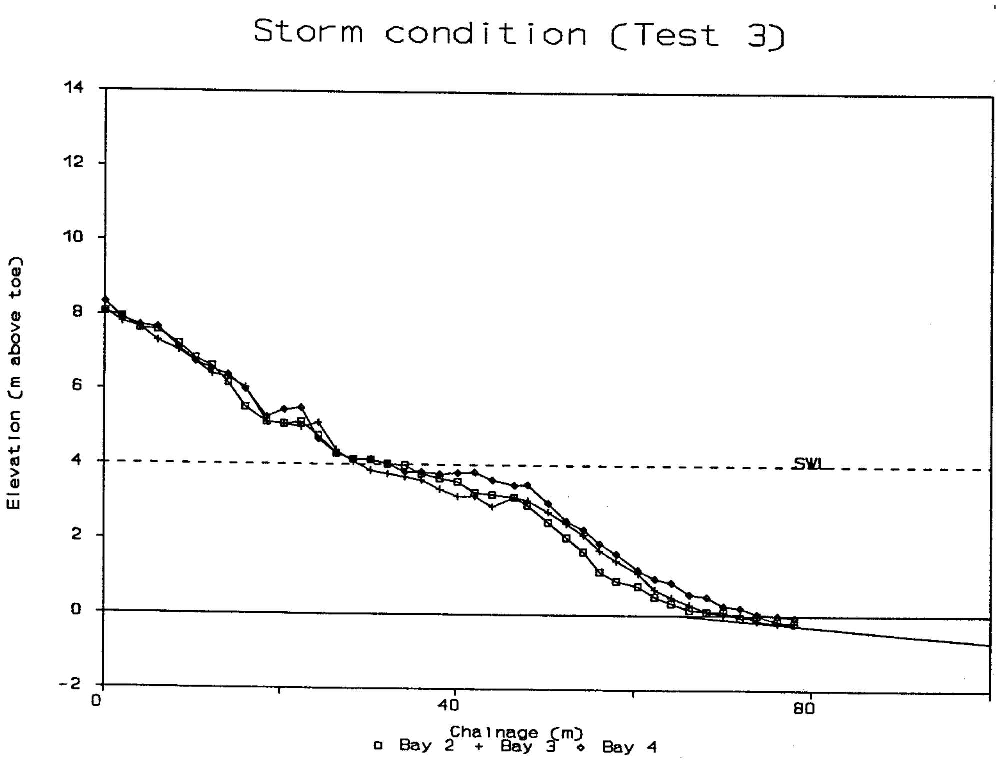
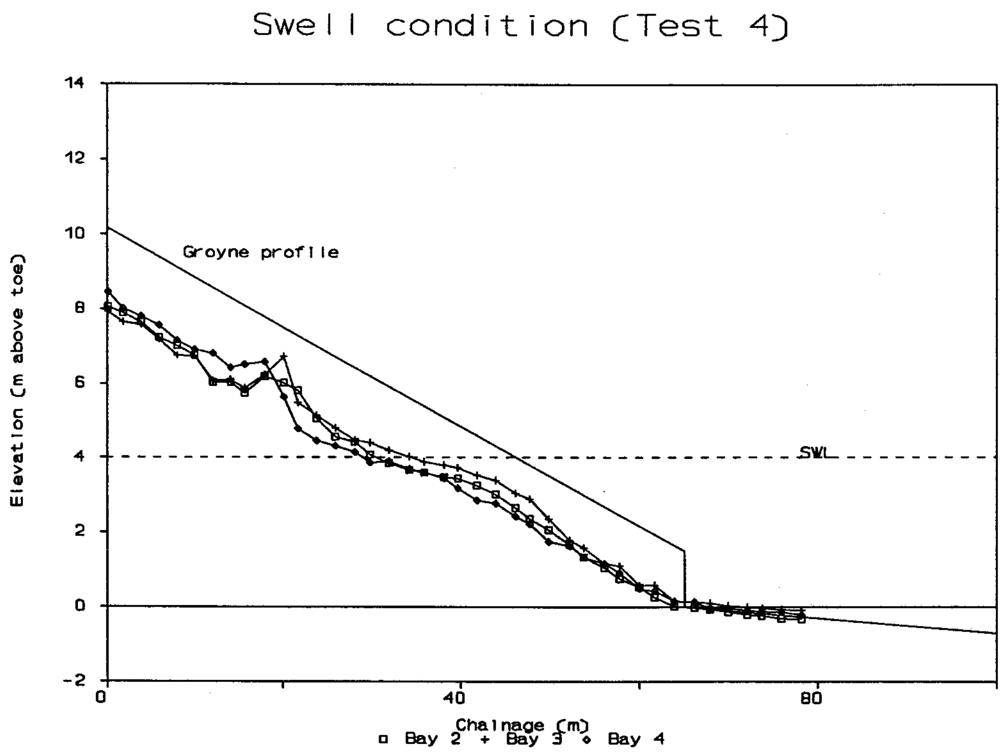


Figure A2 Mid-bay beach profiles: Low timber groynes, SWL = 4m, spacing = 2:1, 30° wave direction



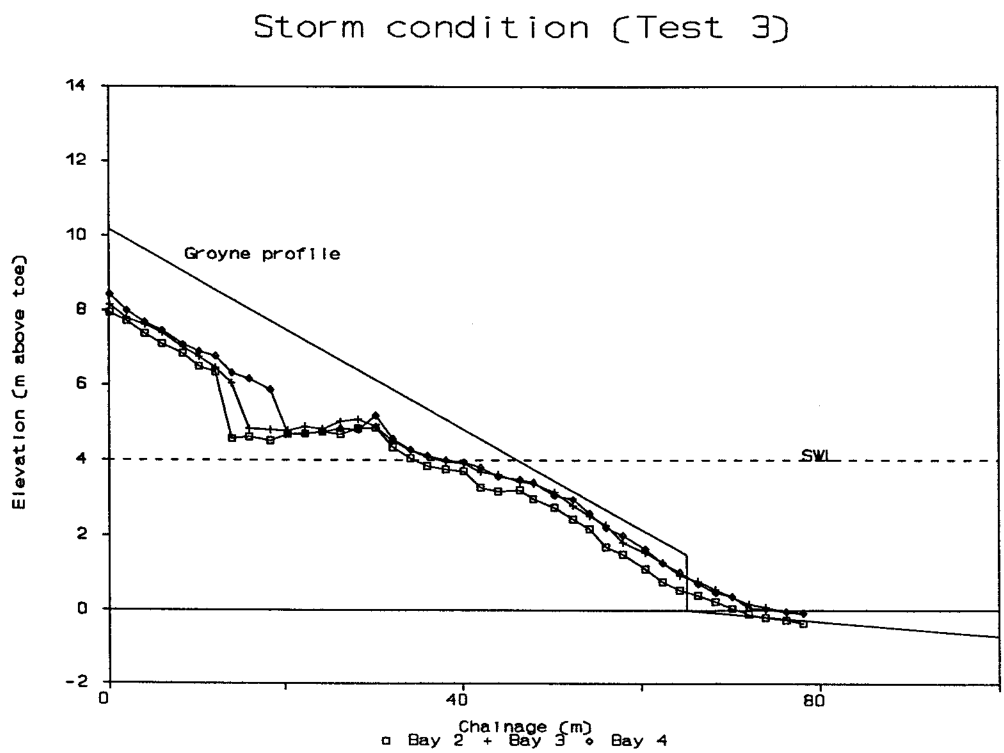
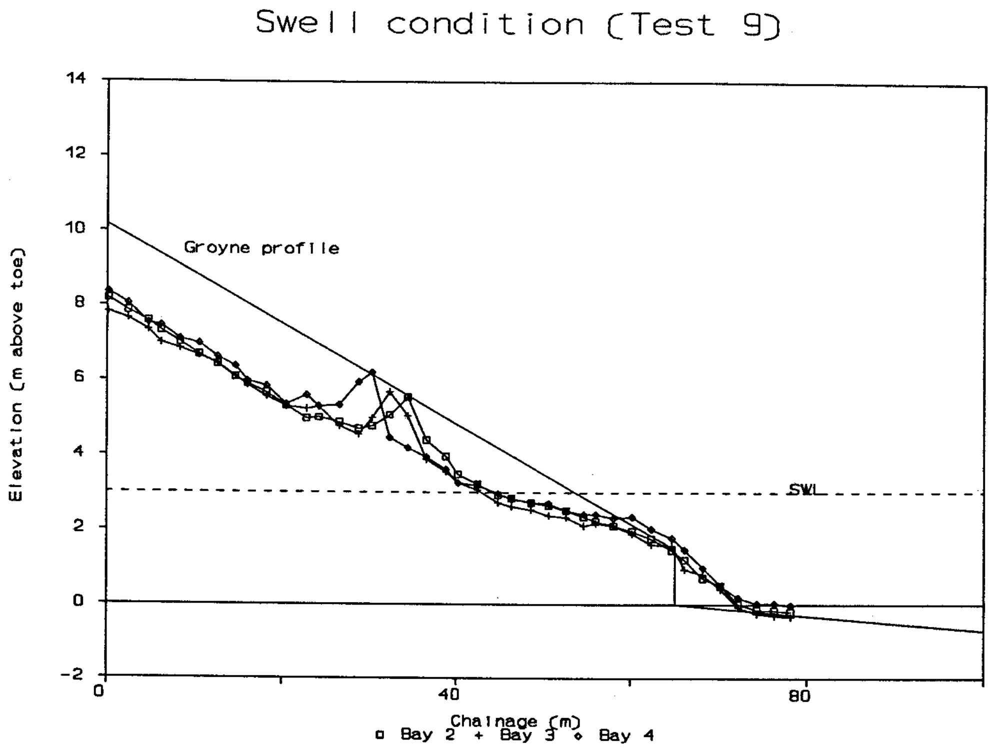


Figure A3 Downdrift beach profiles: Low timber groynes, SWL = 4m, spacing = 2:1, 30° wave direction



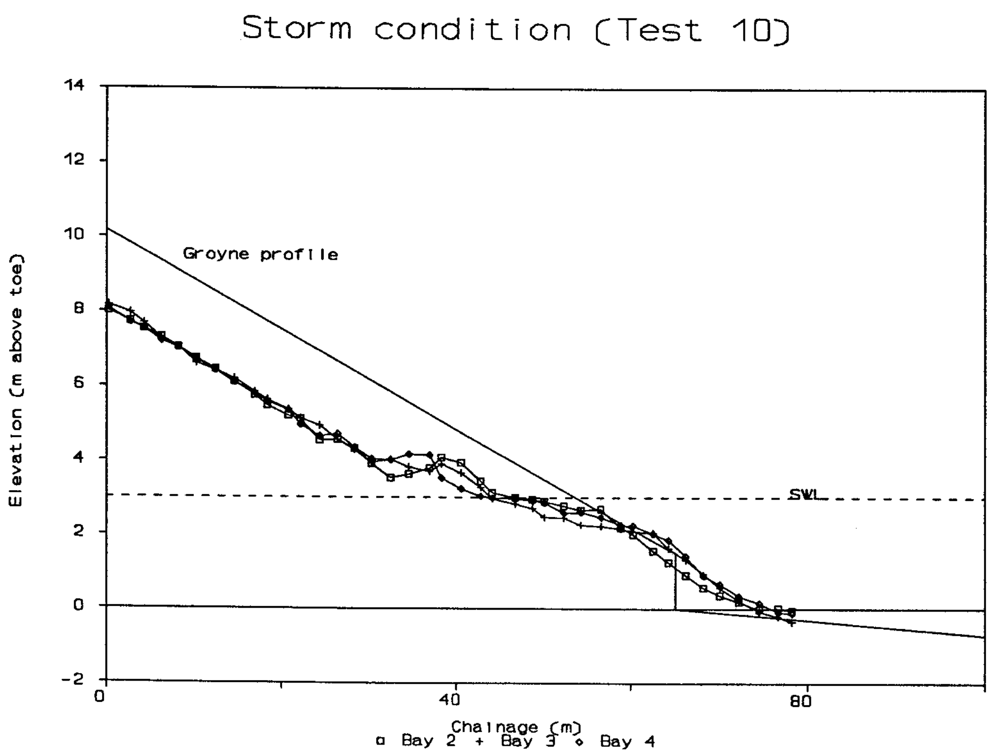
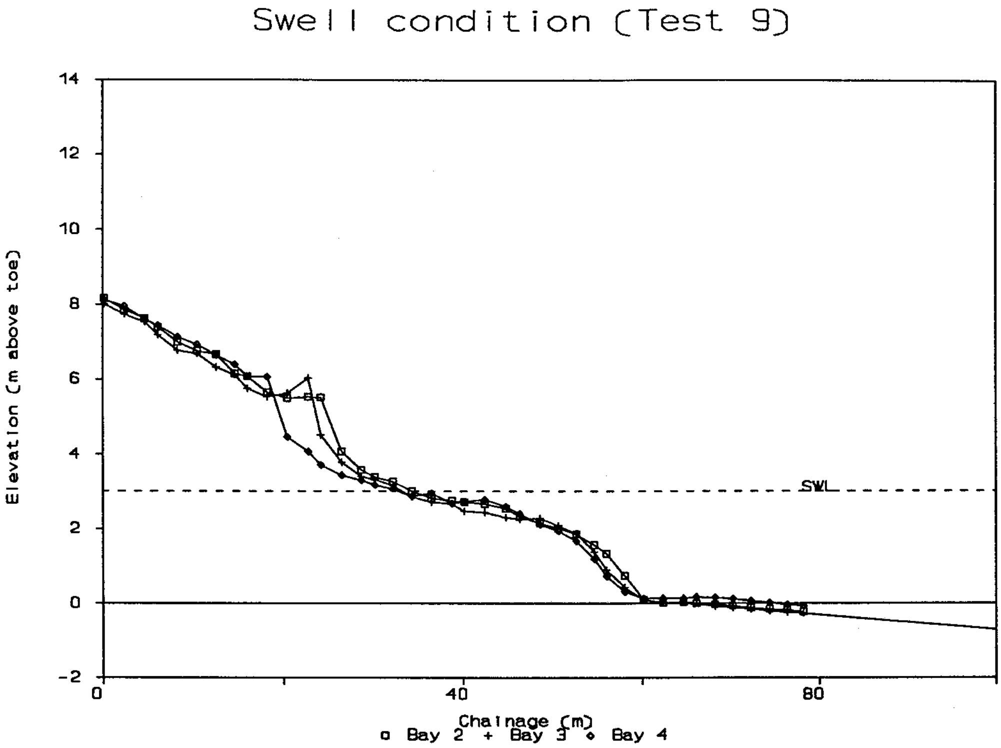


Figure A4 Updrift beach profiles: Low timber groynes, SWL = 3m, spacing = 2:1, 30° wave direction



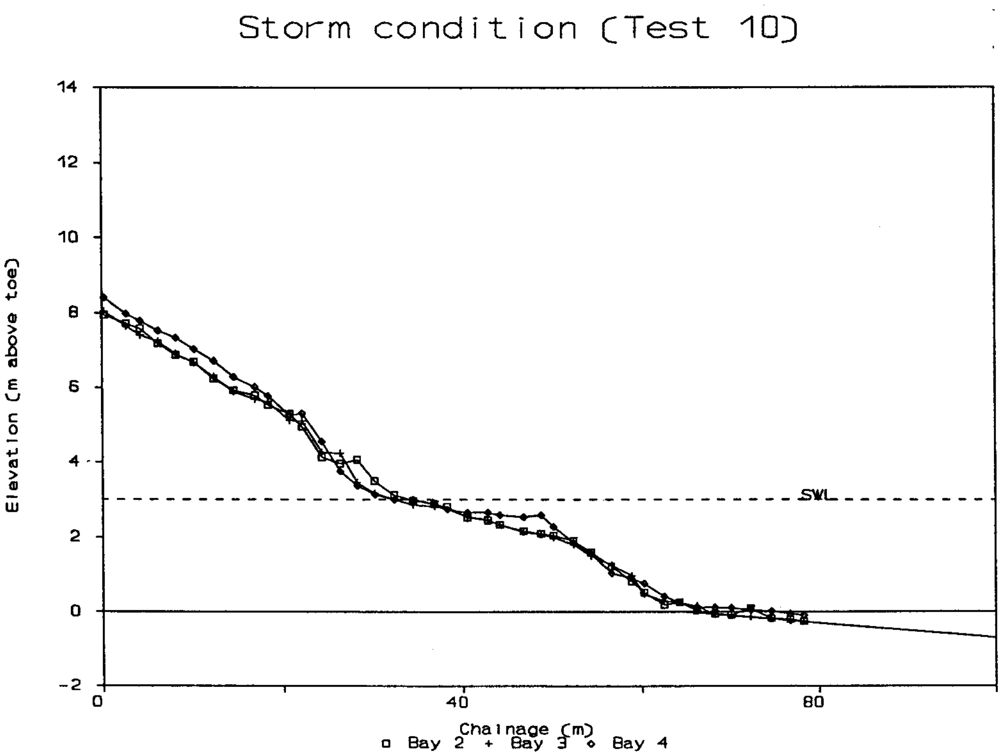
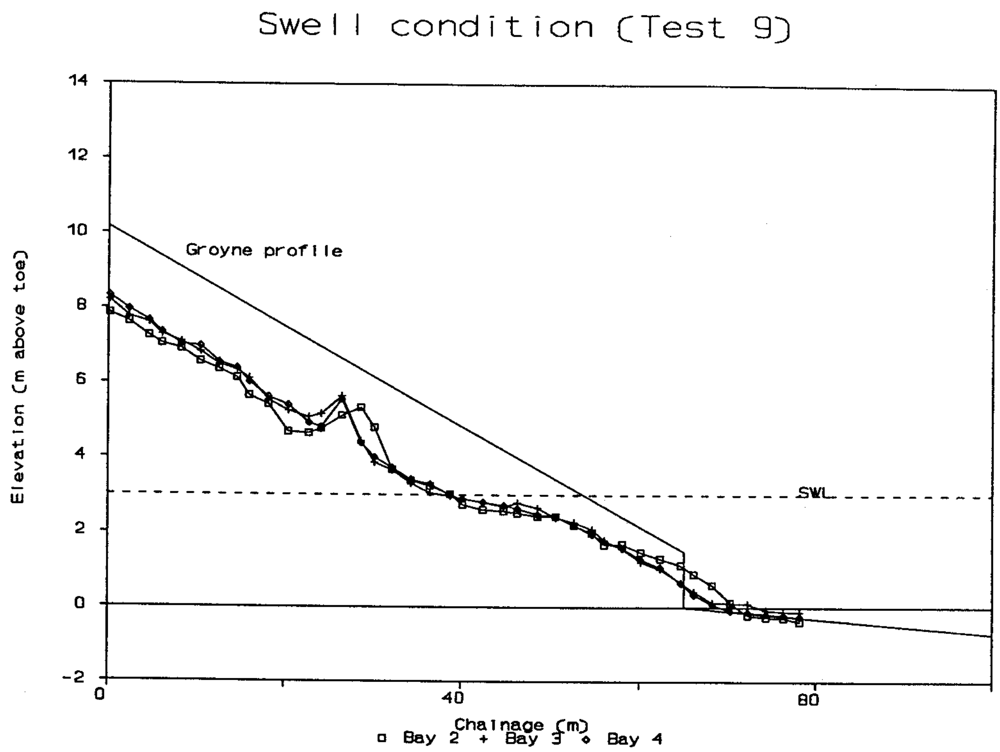


Figure A5 Mid-bay beach profiles: Low timber groynes, SWL = 3m, spacing = 2:1, 30° wave direction



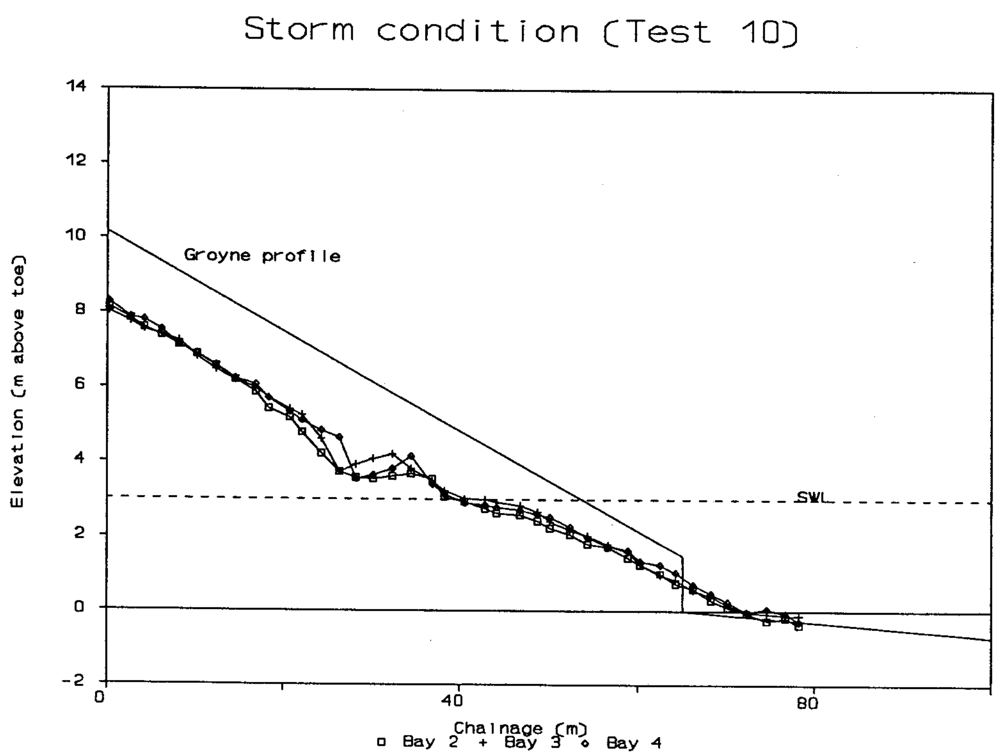
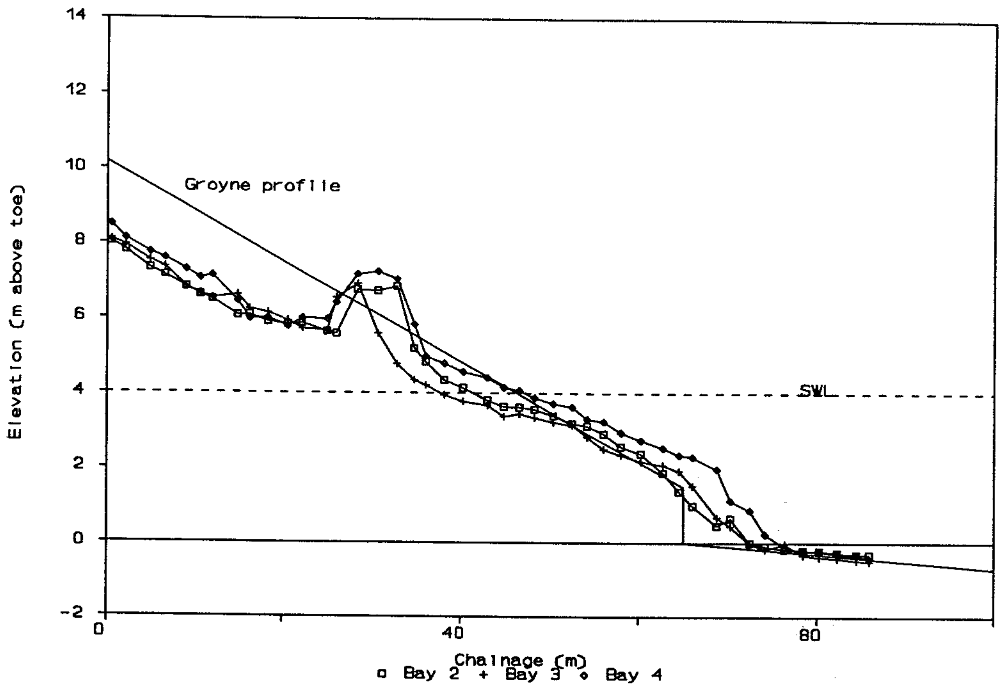


Figure A6 Downdrift beach profiles: Low timber groynes, SWL = 3m, spacing = 2:1, 30° wave direction

Swell condition (Test 23)



Storm condition (Test 24)

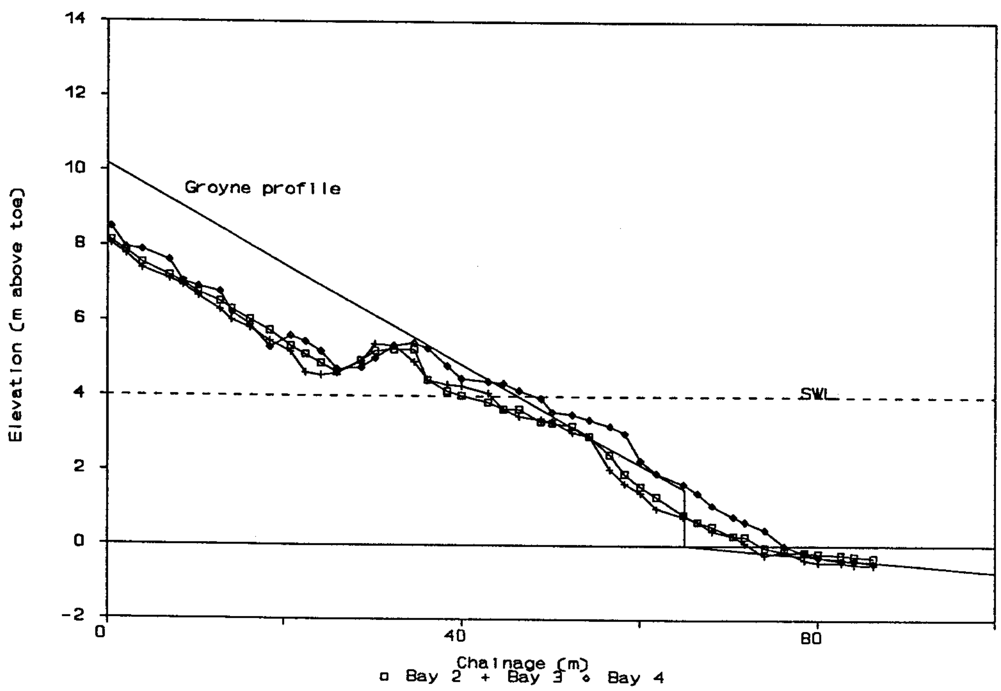
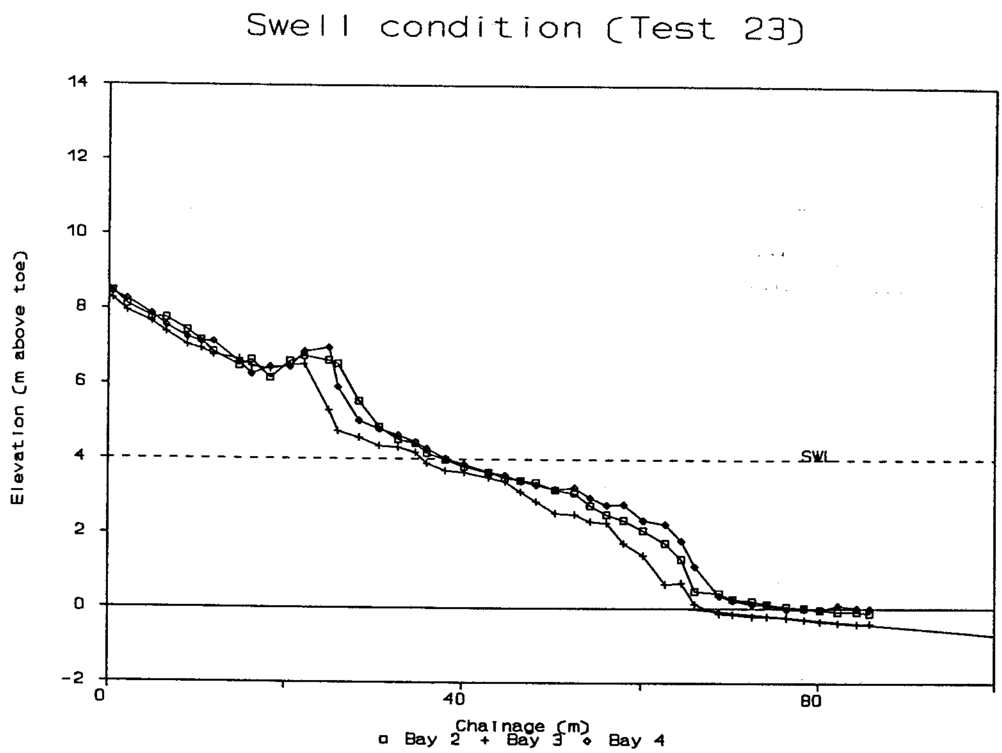


Figure A7 Updrift beach profiles: Low timber groynes, SWL = 4m, spacing = 1.5:1, 30° wave direction



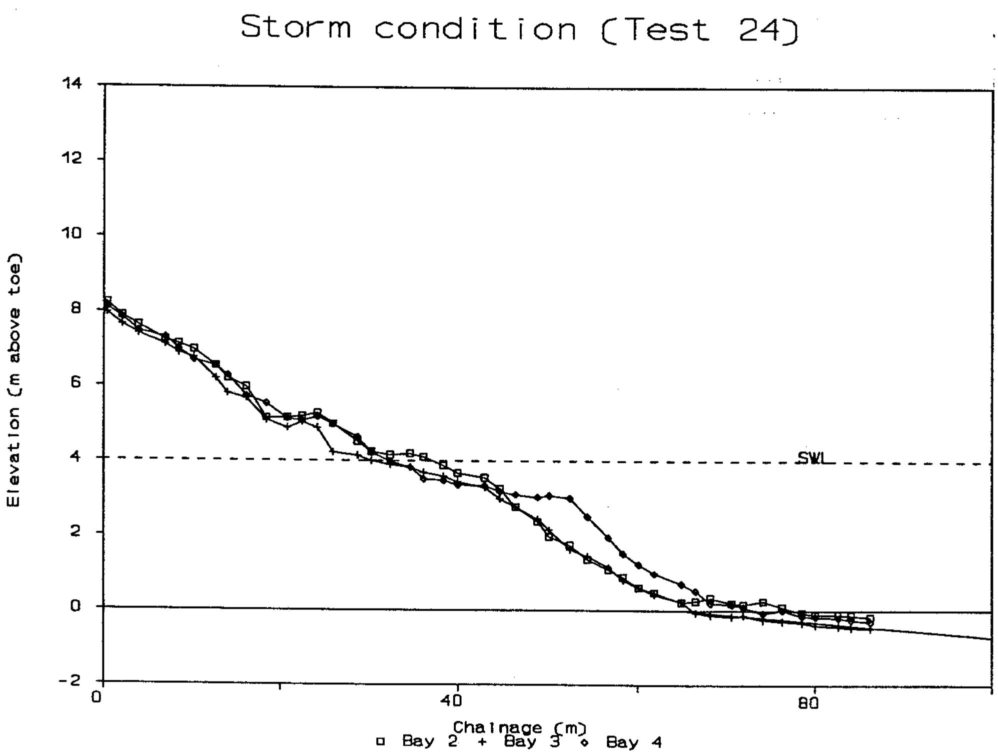
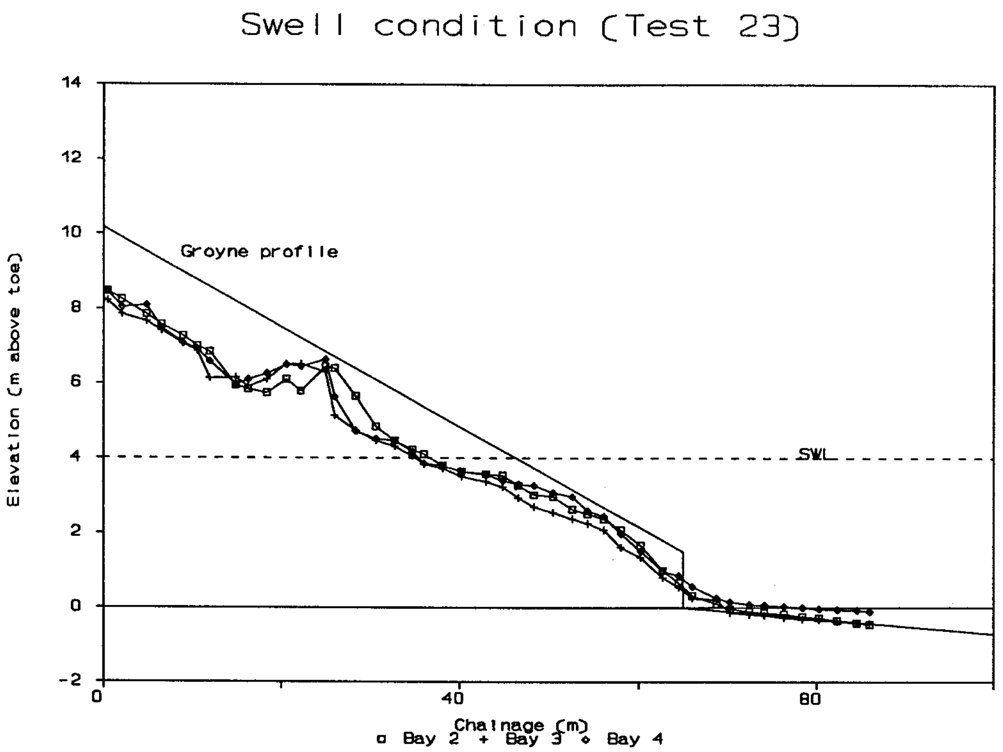


Figure A8 Mid-bay beach profiles: Low timber groynes, SWL = 4m, spacing = 1.5:1, 30° wave direction



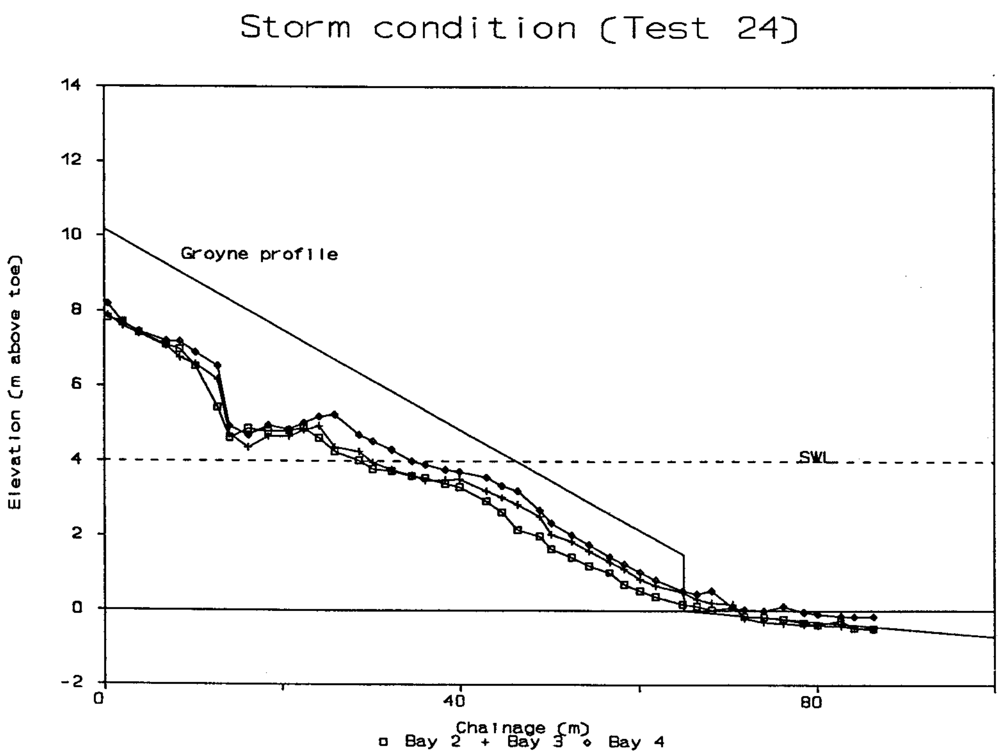
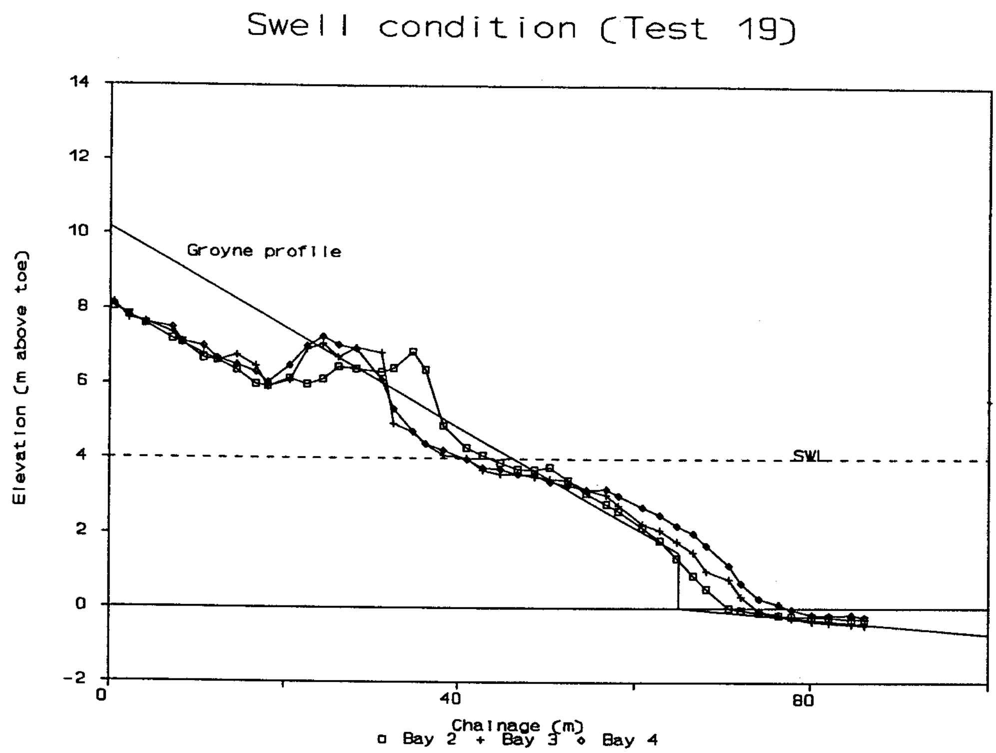


Figure A9 **Downdrift beach profiles: Low timber groynes, SWL = 4m, spacing = 1.5:1, 30° wave direction**



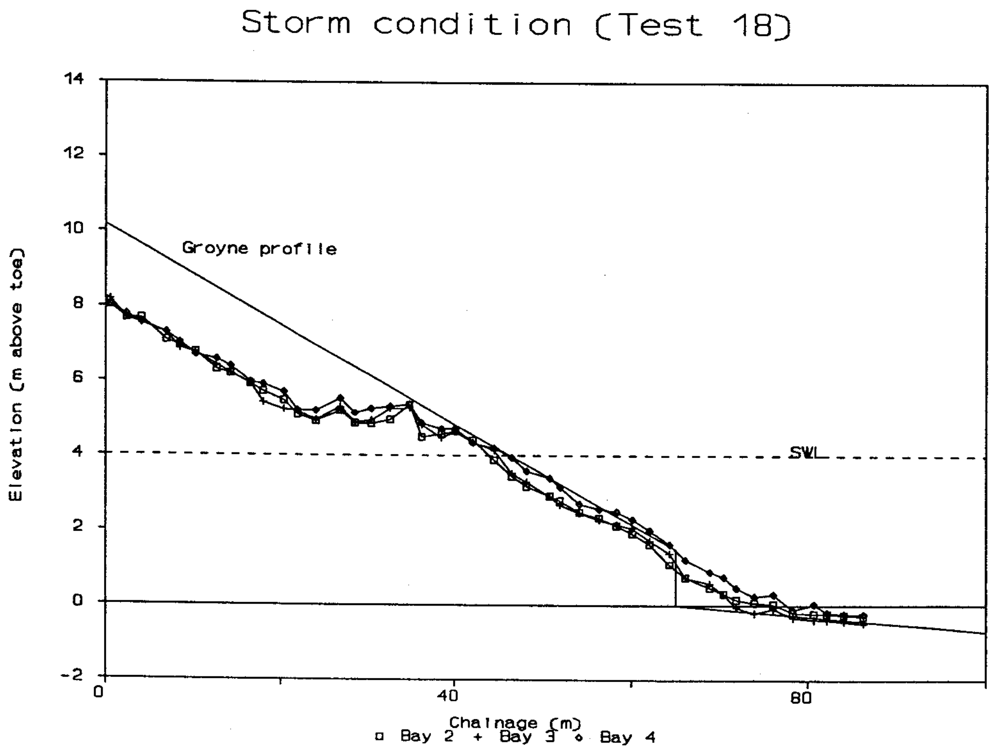
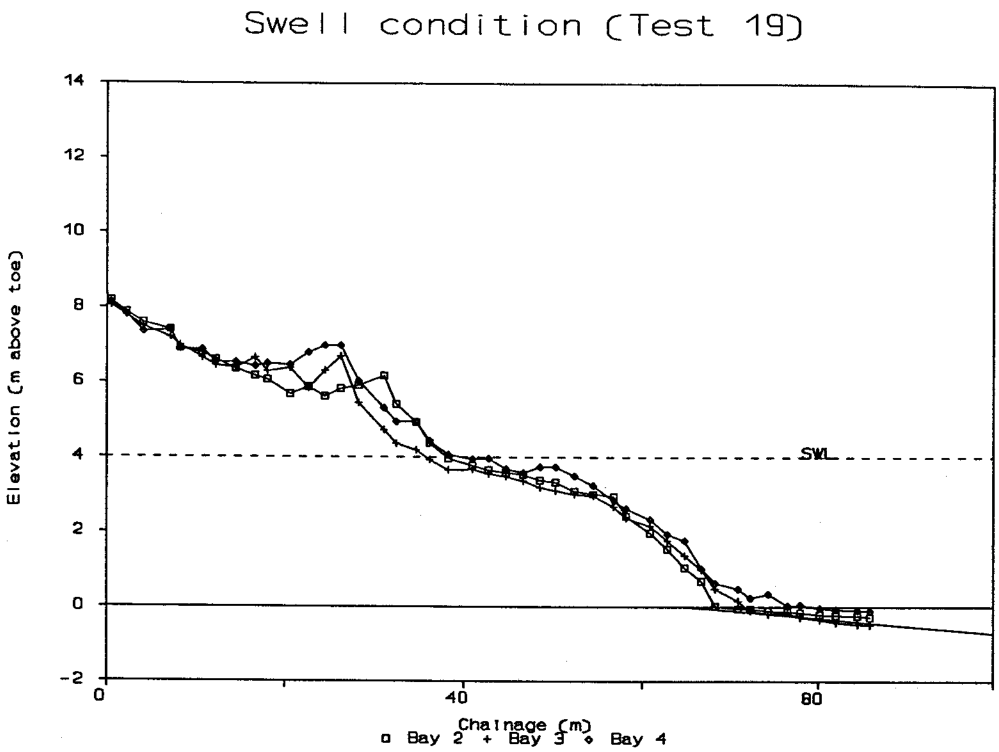


Figure A10 Updrift beach profiles: Low timber groynes, SWL = 4m, spacing = 1:1, 30° wave direction



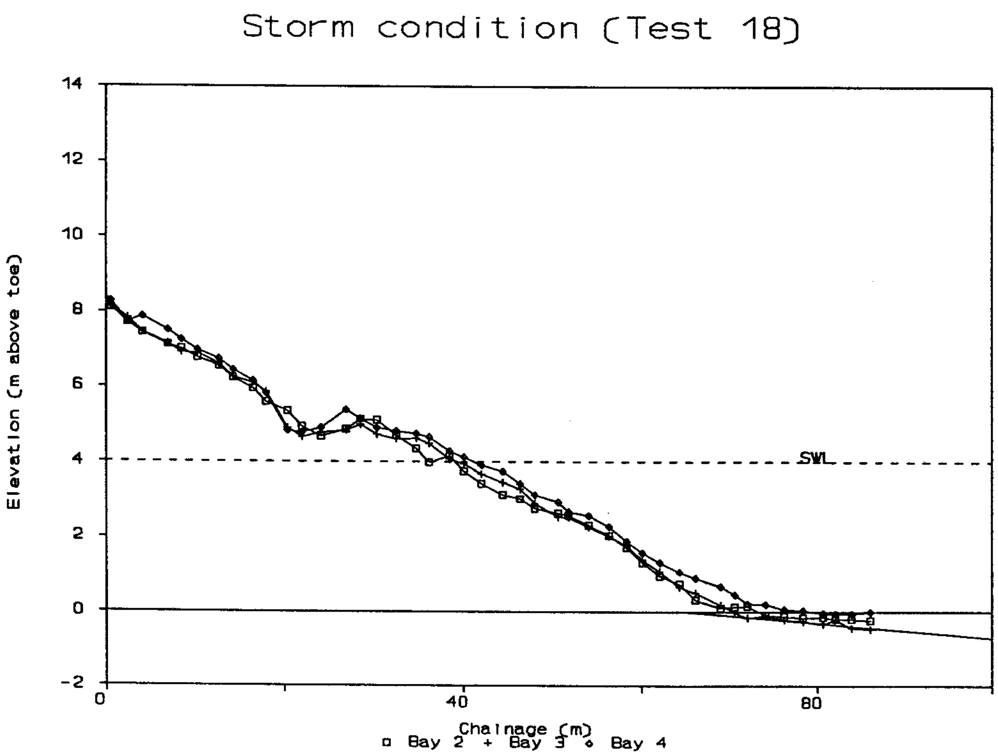


Figure A11 Mid-bay beach profiles: Low timber groynes, SWL = 4m, spacing = 1:1, 30° wave direction

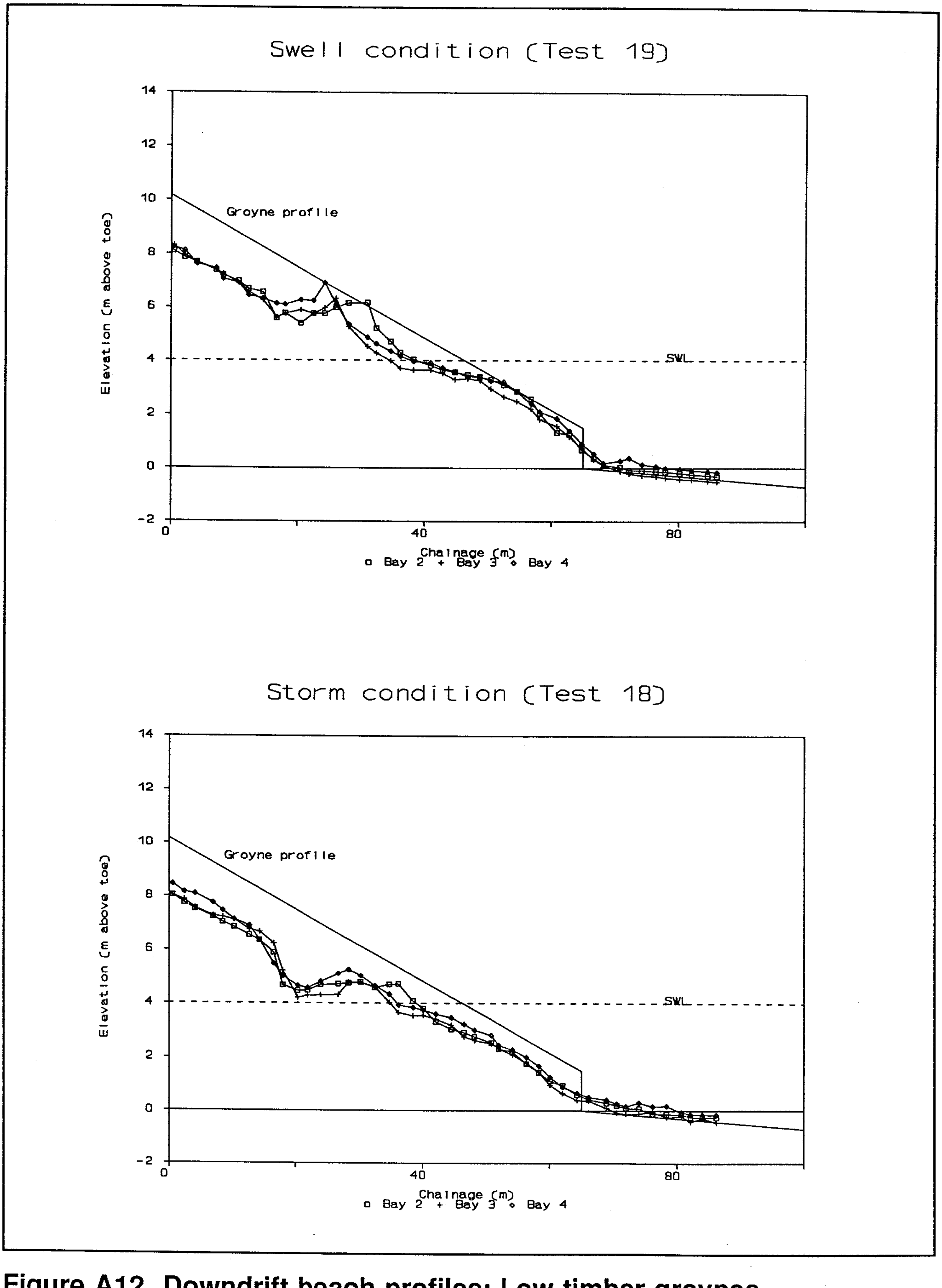
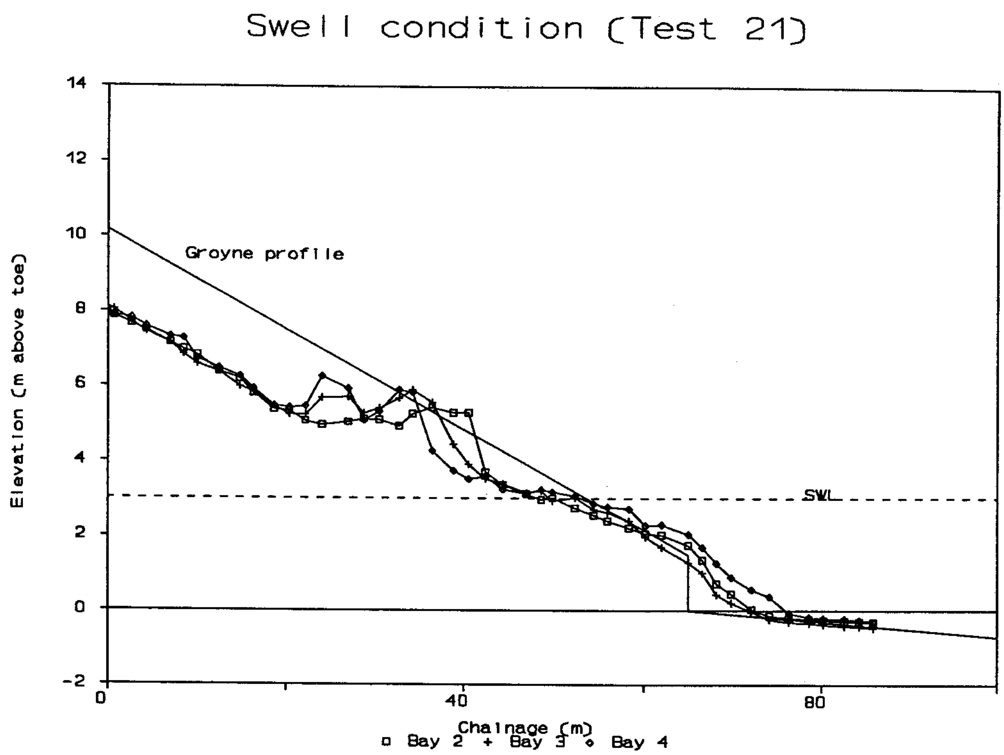


Figure A12 Downdrift beach profiles: Low timber groynes, SWL = 4m, spacing = 1:1, 30° wave direction



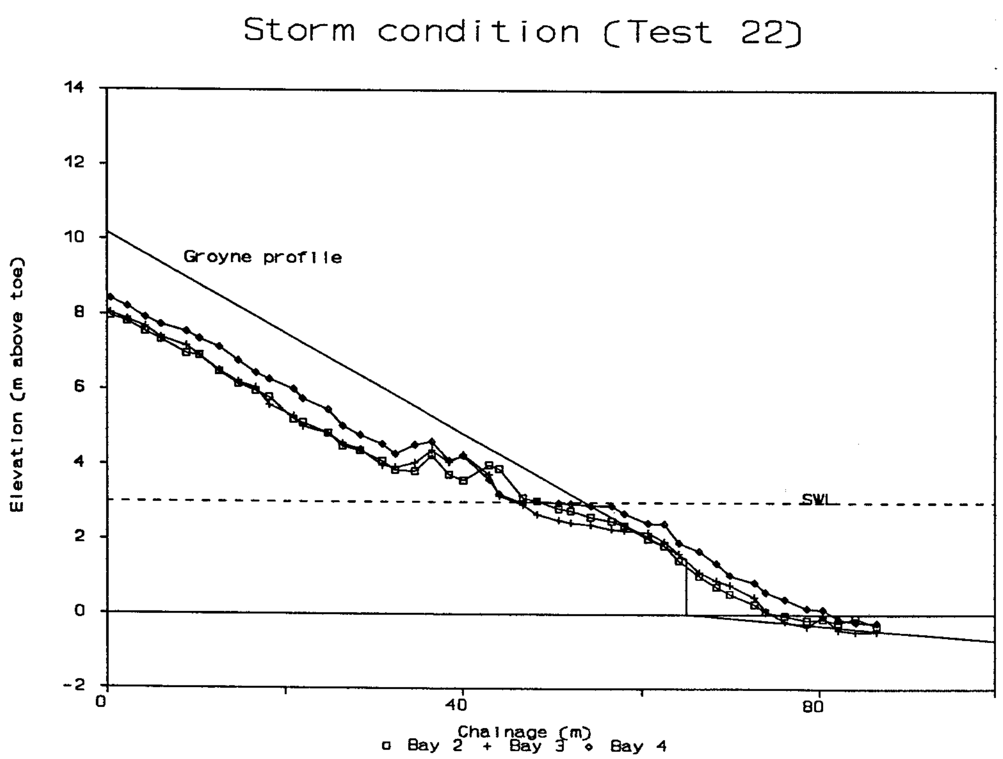


Figure A13 Updrift beach profiles: Low timber groynes, SWL = 3m, spacing = 1:1, 30° wave direction

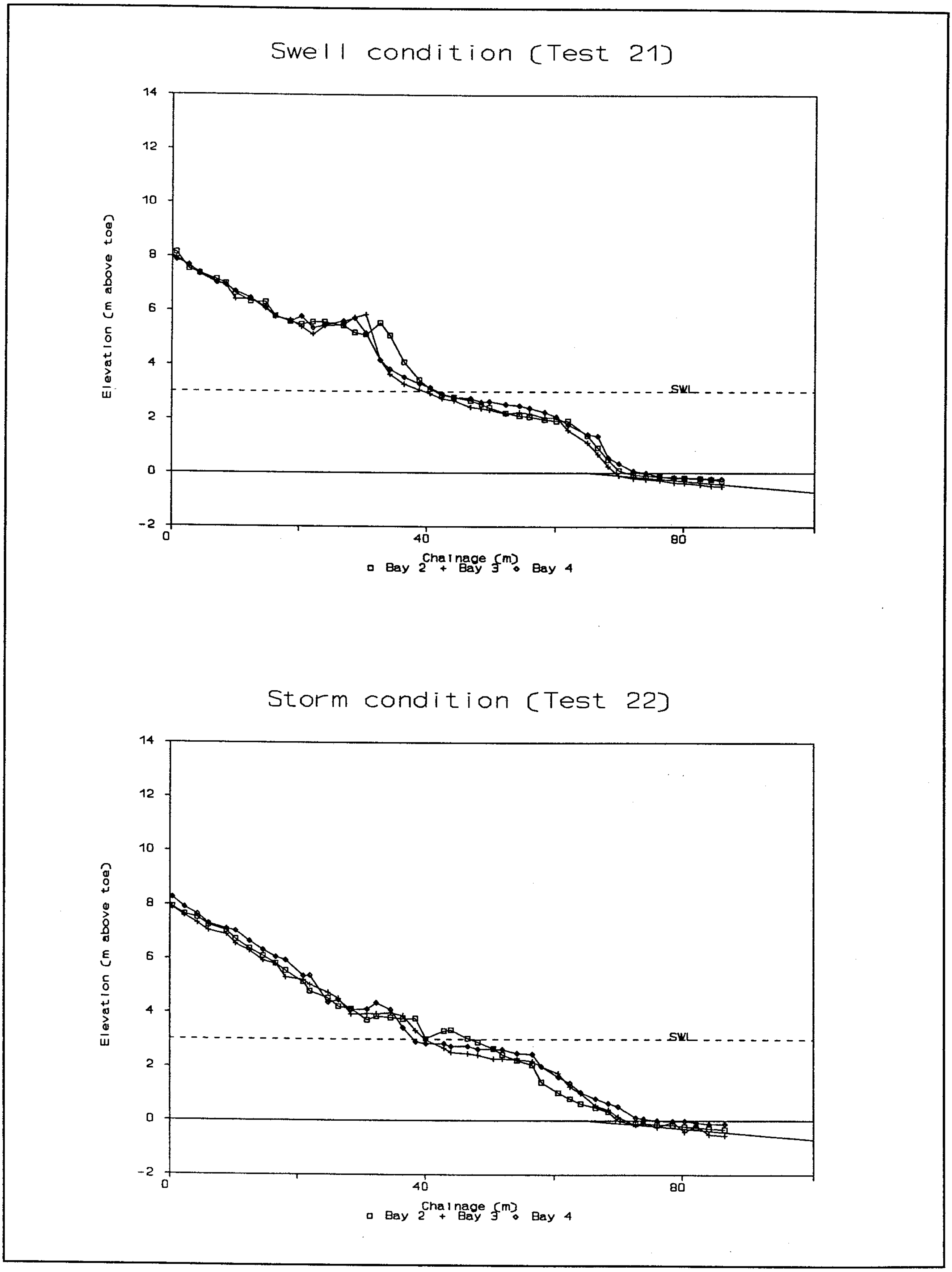
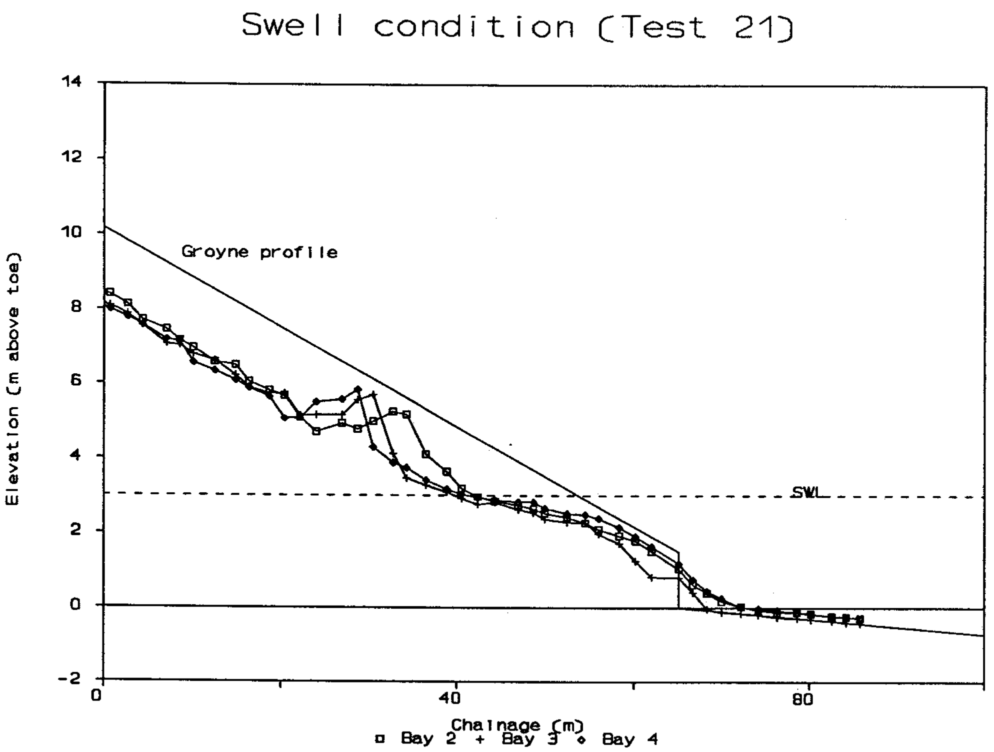


Figure A14 Mid-bay beach profiles: Low timber groynes, SWL = 3m, spacing = 1:1, 30° wave direction



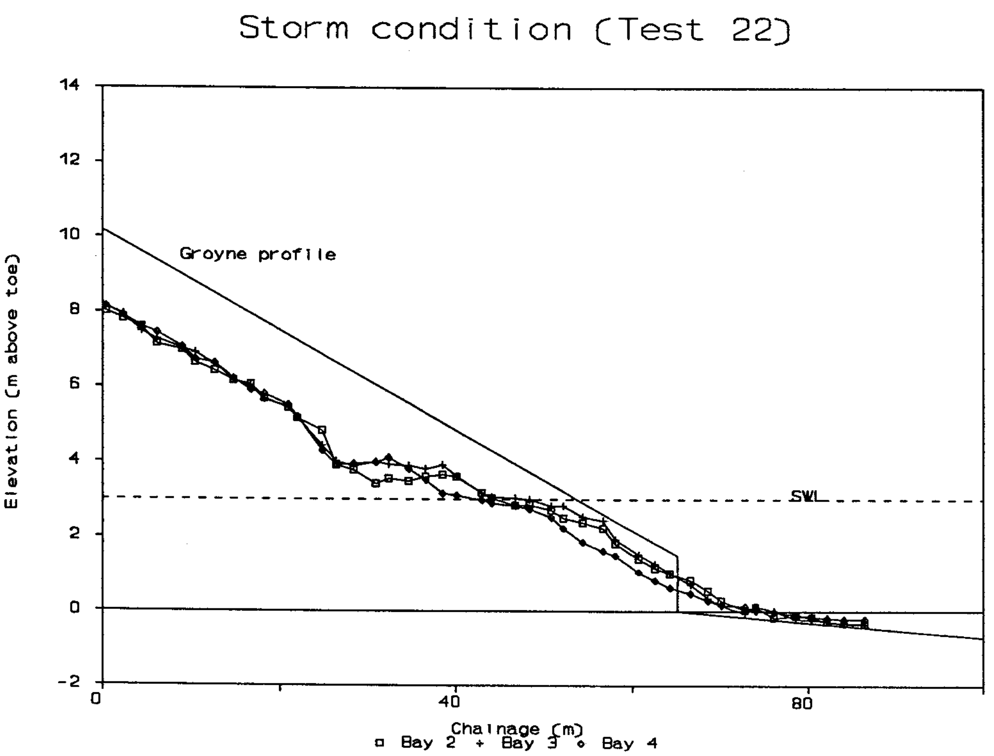
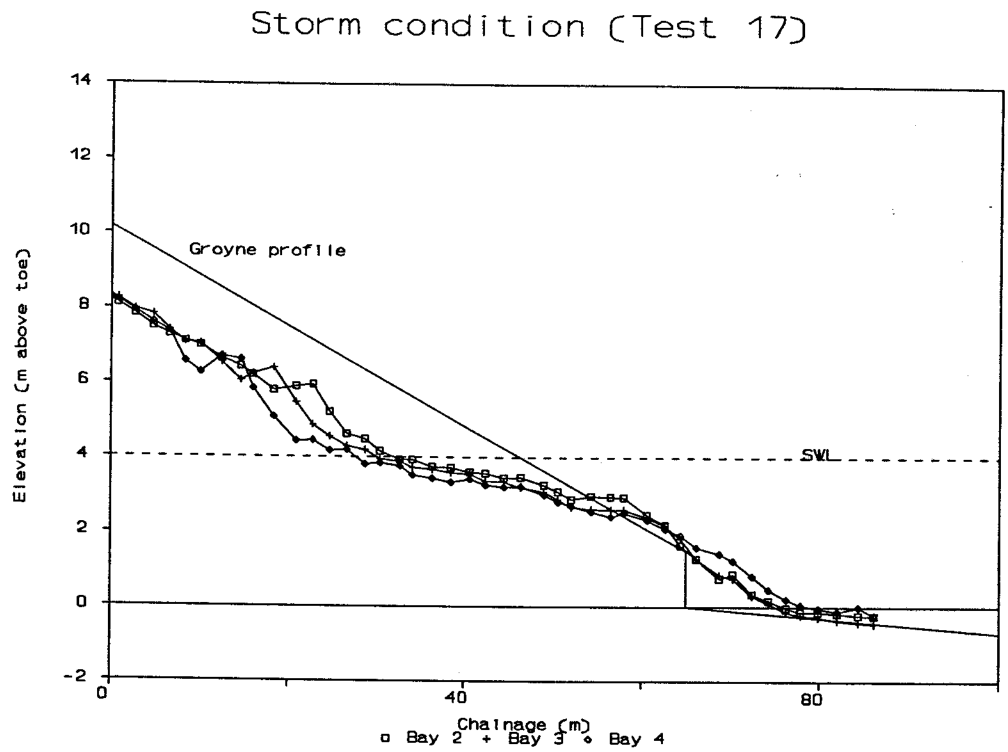


Figure A15 Downdrift beach profiles: Low timber groynes, SWL = 3m, spacing = 1:1, 30° wave direction



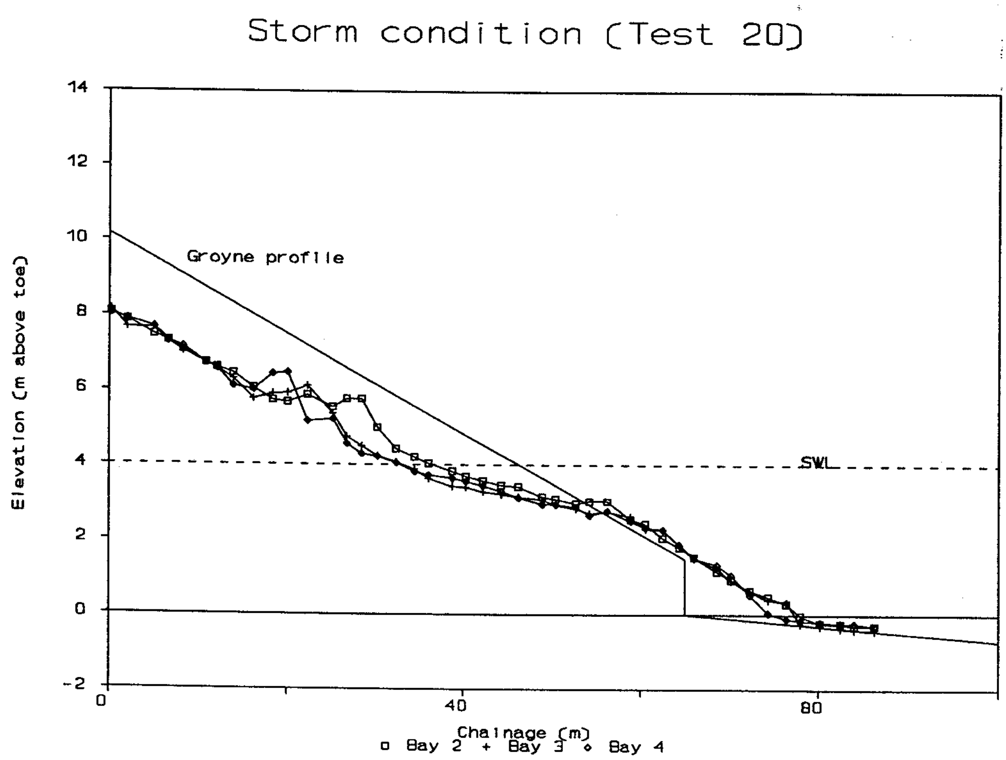
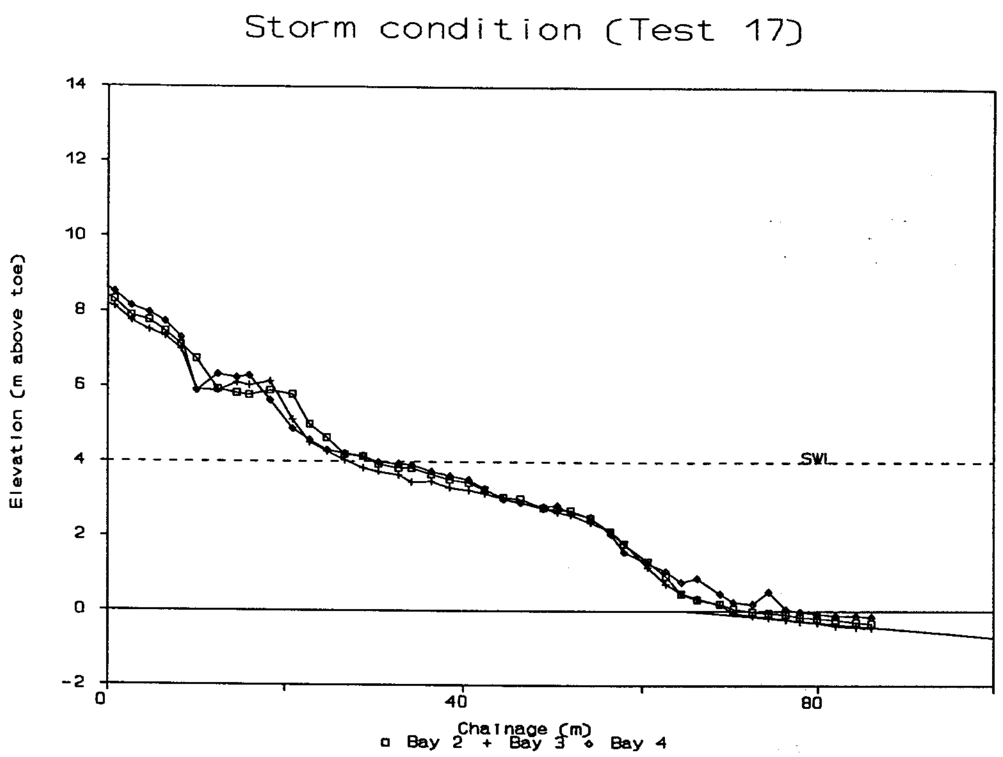


Figure A16 Updrift beach profiles: Low timber groynes, SWL = 4m, spacing= 2:1 and 1:1, 30° wave direction



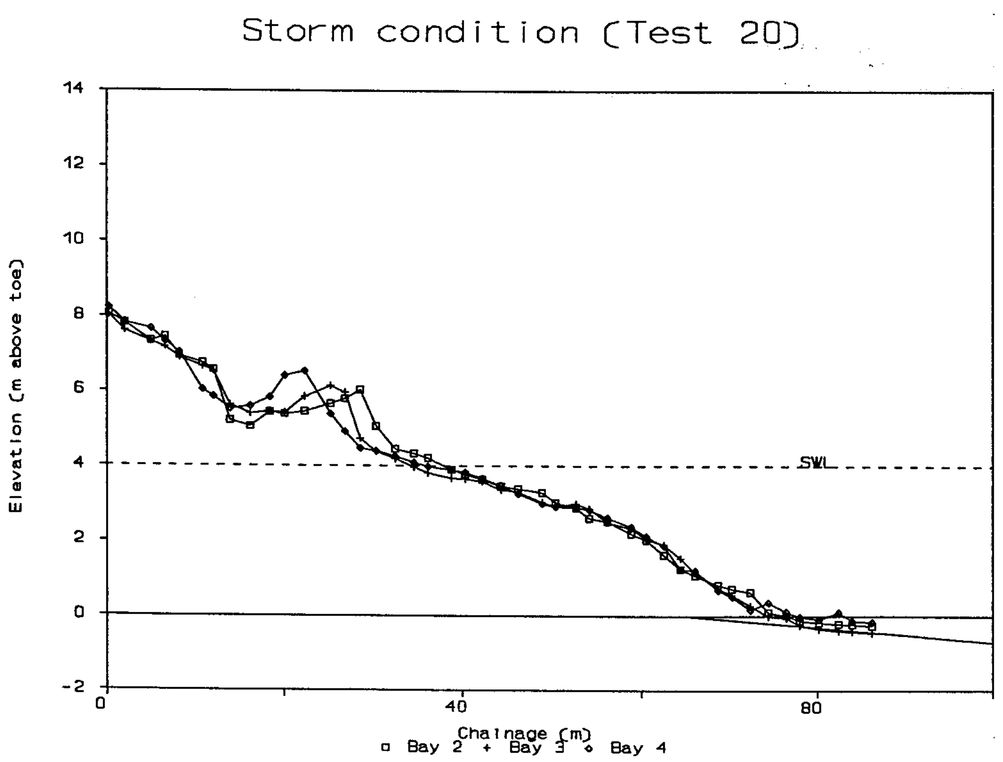
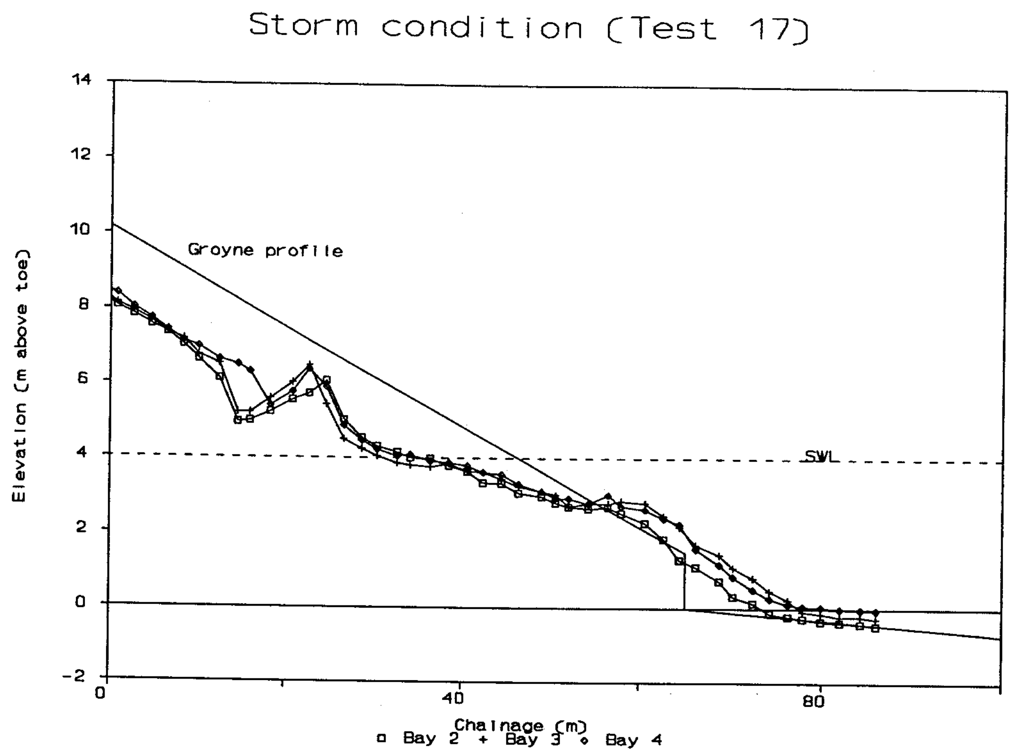


Figure A17 Mid-bay beach profiles: Low timber groynes, SWL = 4m, spacing= 2:1 and 1:1, 30° wave direction



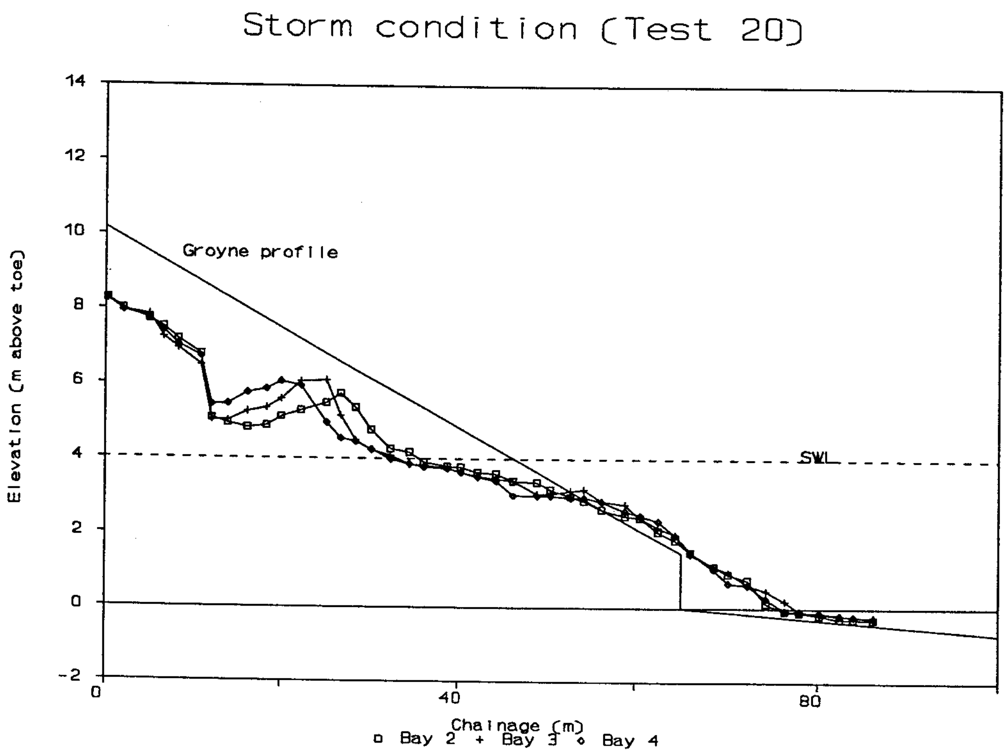
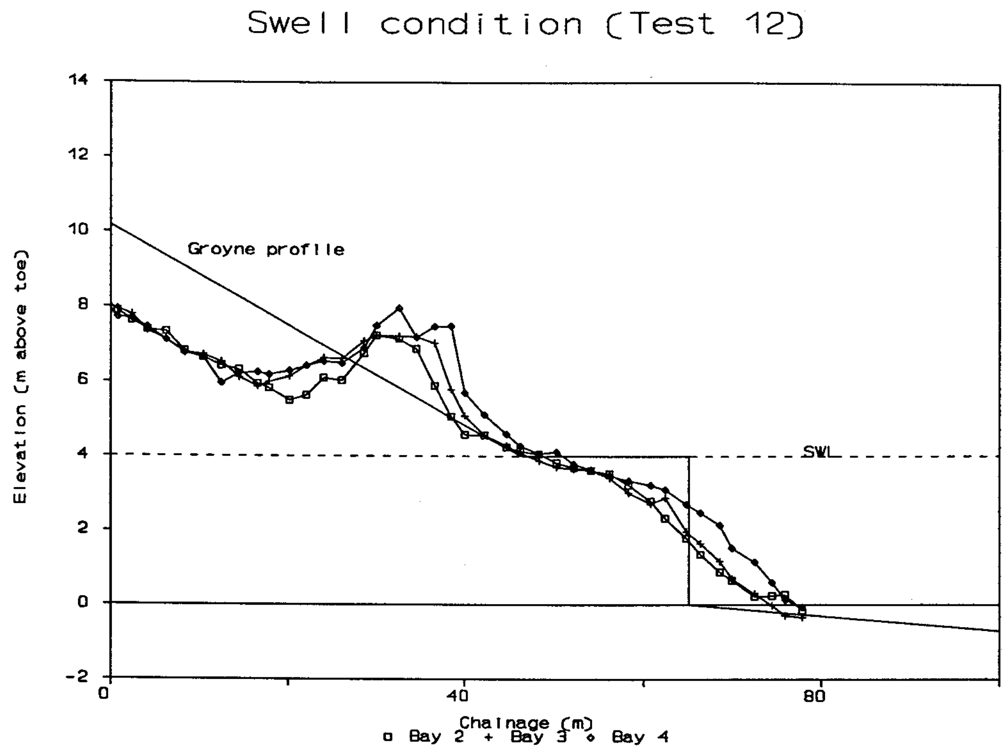


Figure A18 Downdrift beach profiles: Low timber groynes, SWL = 4m, spacing= 2:1 and 1:1, 30° wave direction



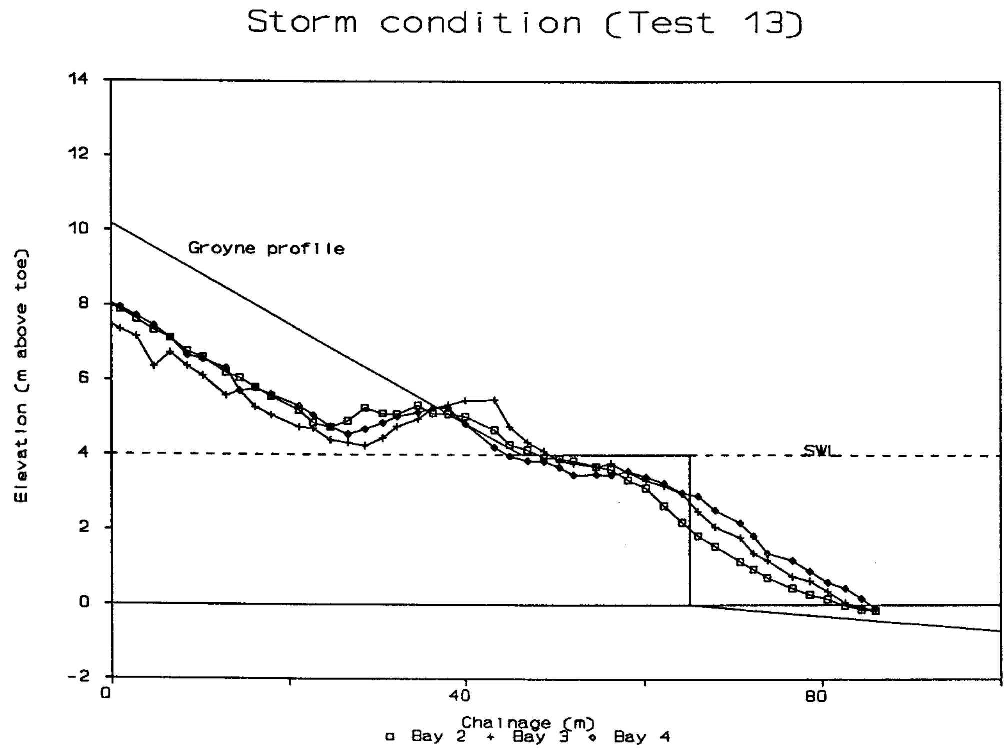


Figure A19 Updrift beach profiles: High end timber groynes, SWL = 4m, spacing = 2:1, 30° wave direction

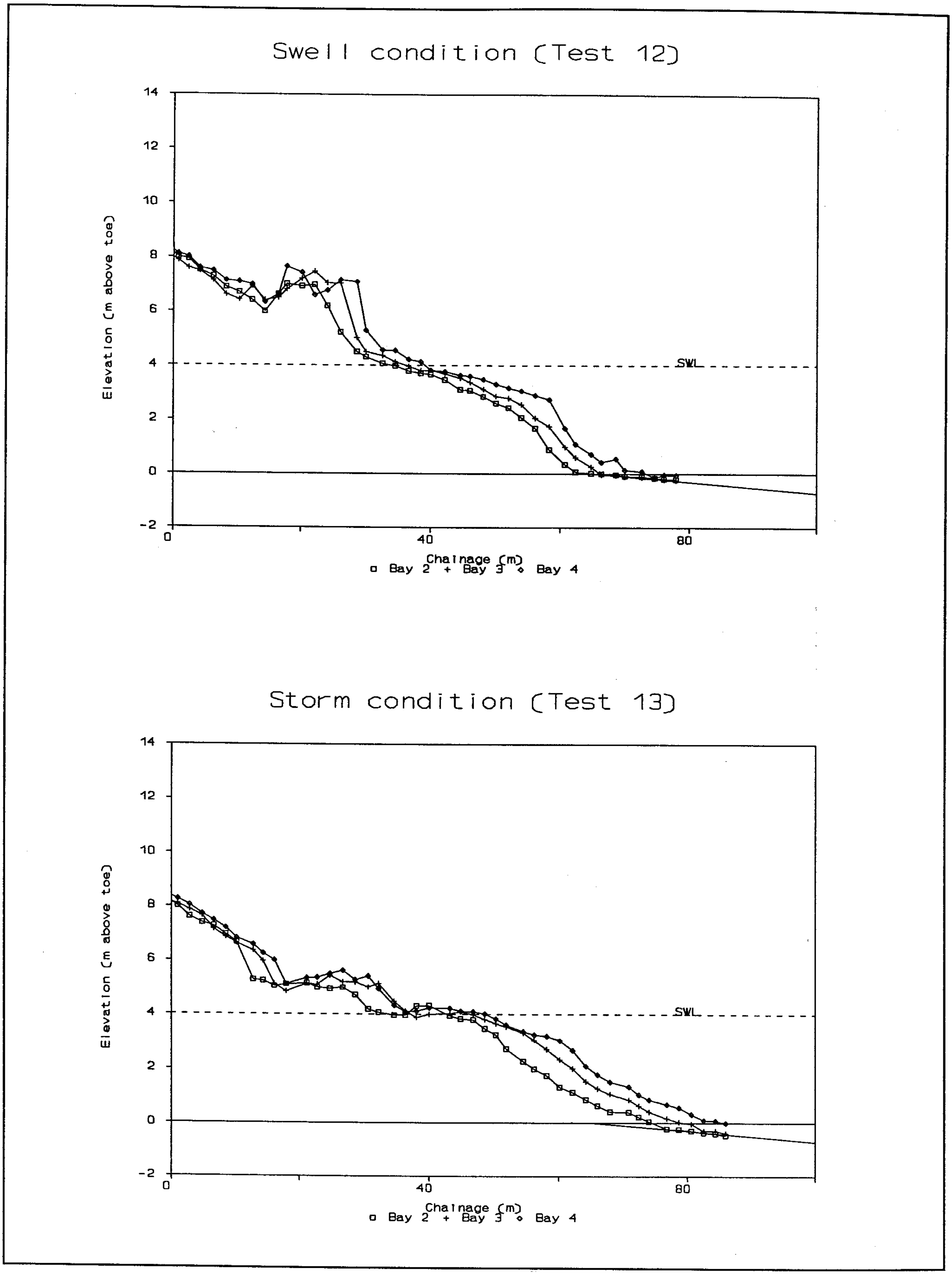
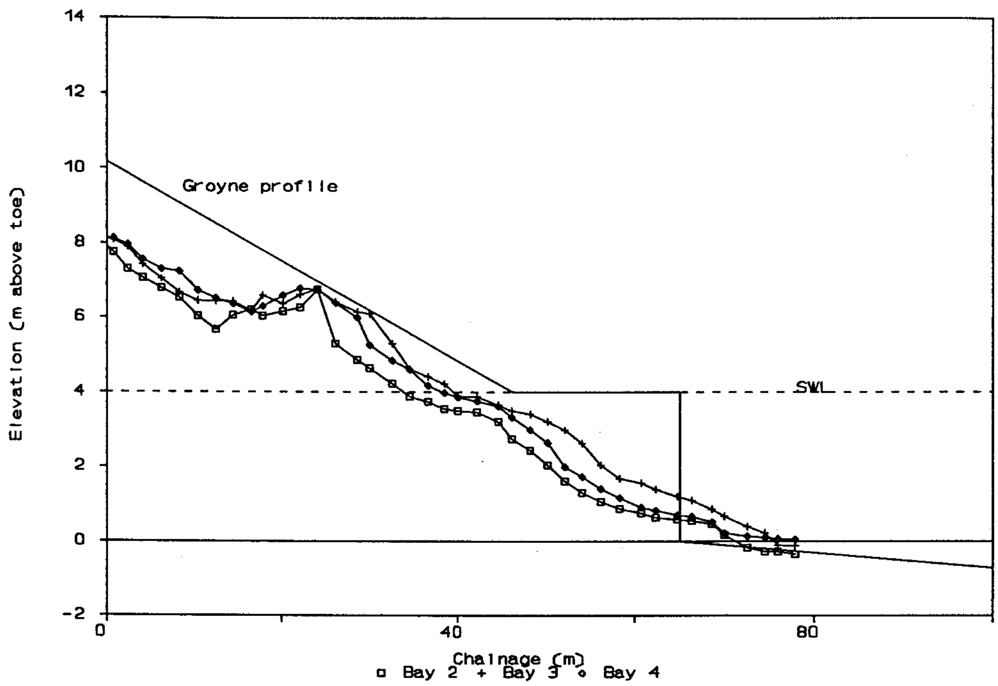


Figure A20 Mid-bay beach profiles: High end timber groynes, SWL = 4m, spacing = 2:1, 30° wave direction

Swell condition (Test 12)



Storm condition (Test 13)

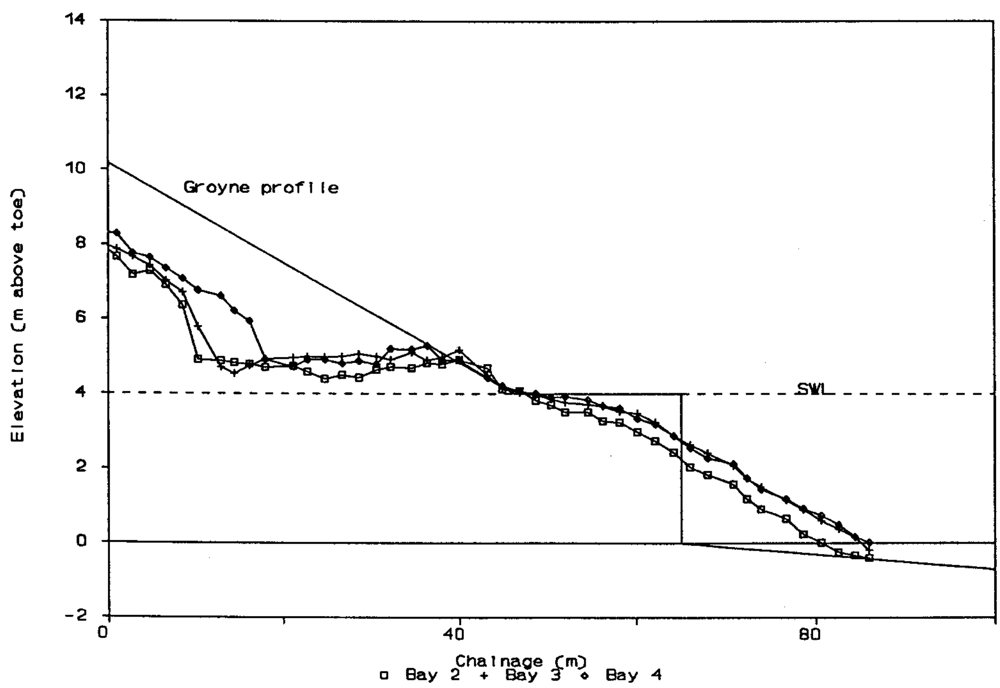
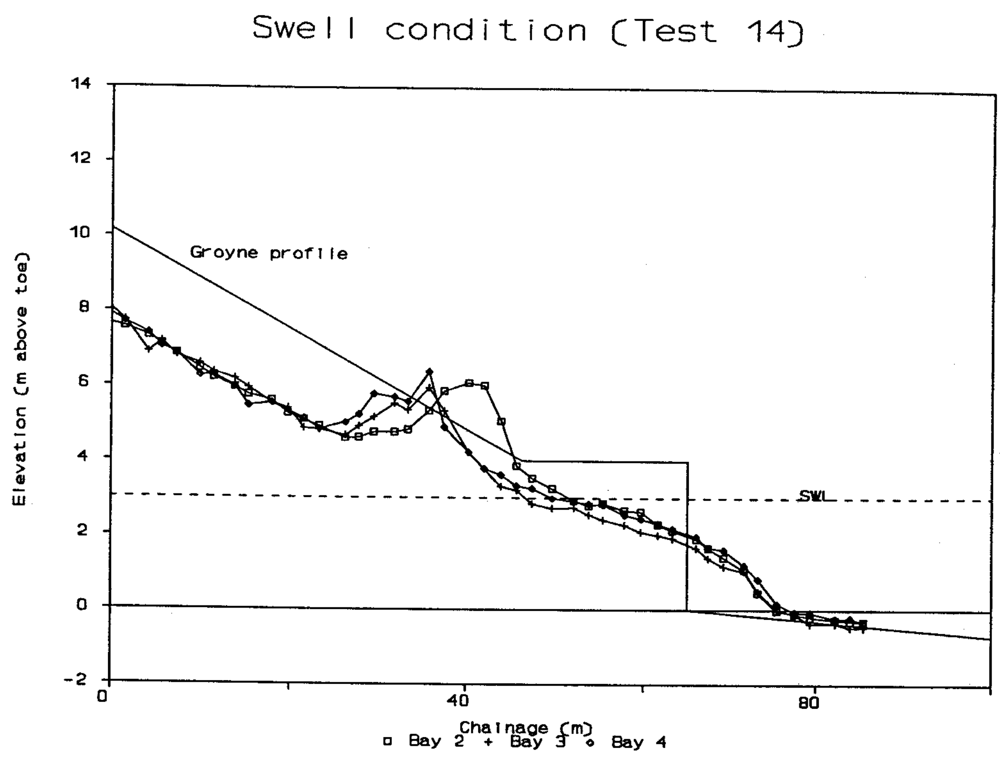


Figure A21 Downdrift beach profiles: High end timber groynes, SWL = 4m, spacing = 2:1, 30° wave direction



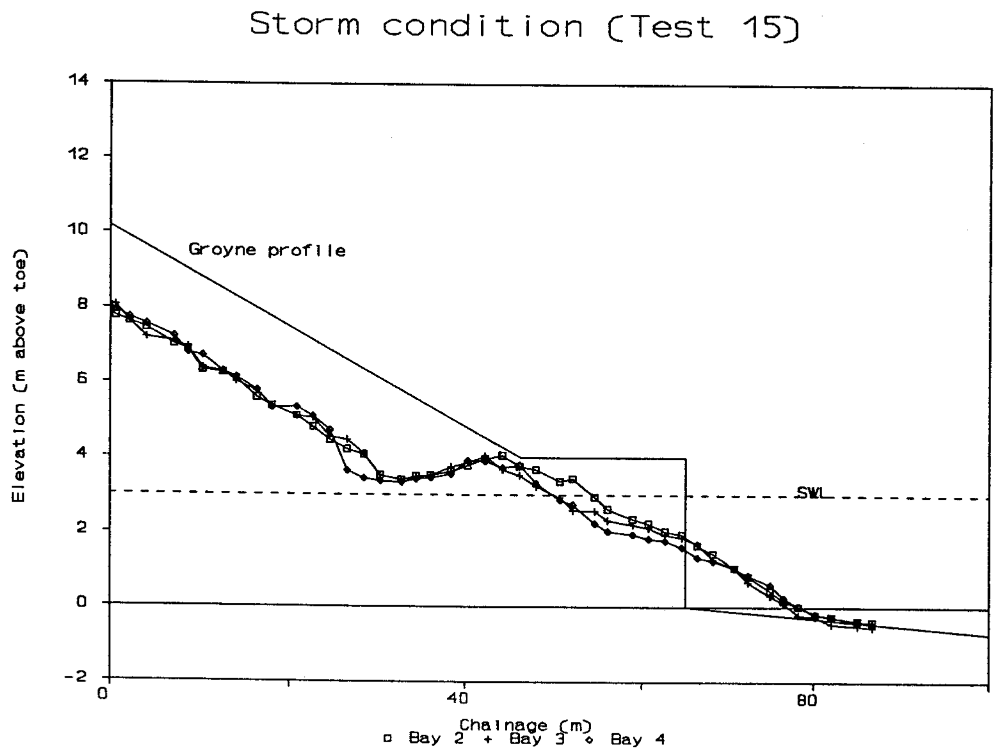
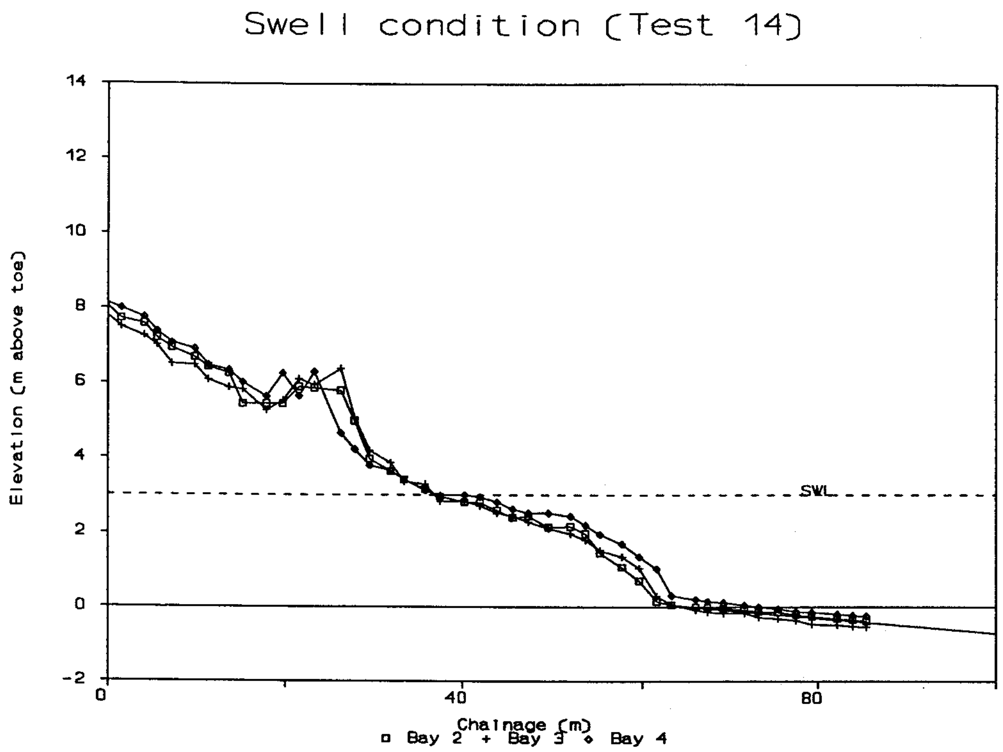


Figure A22 Updrift beach profiles: High end timber groynes, SWL = 3m, spacing = 2:1, 30° wave direction



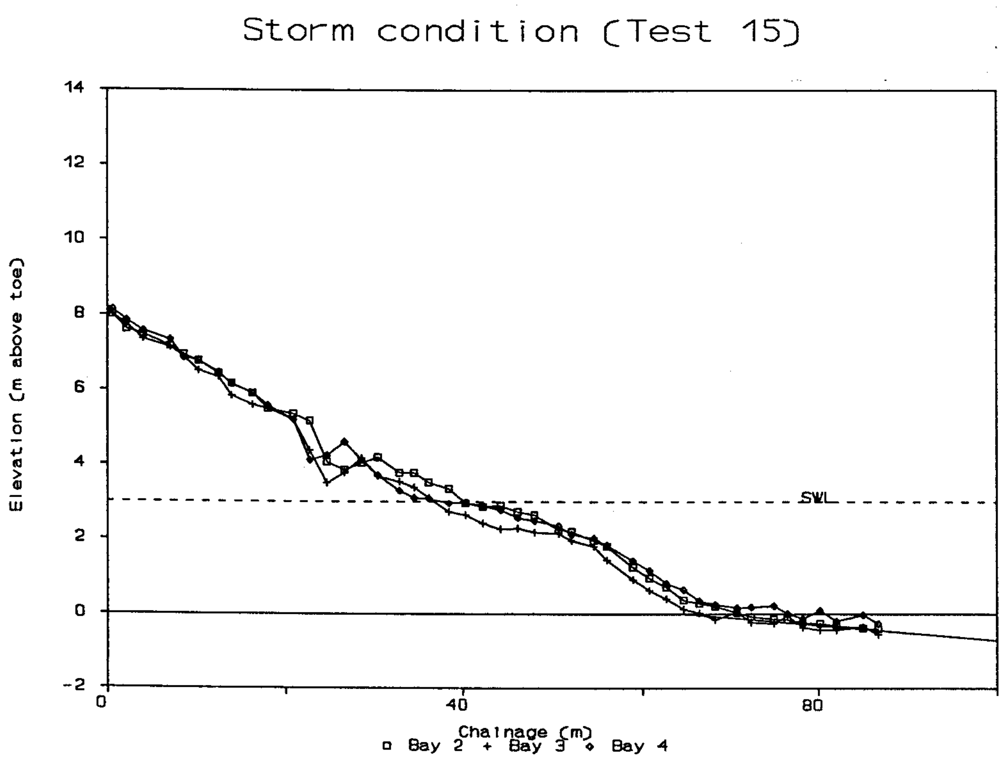
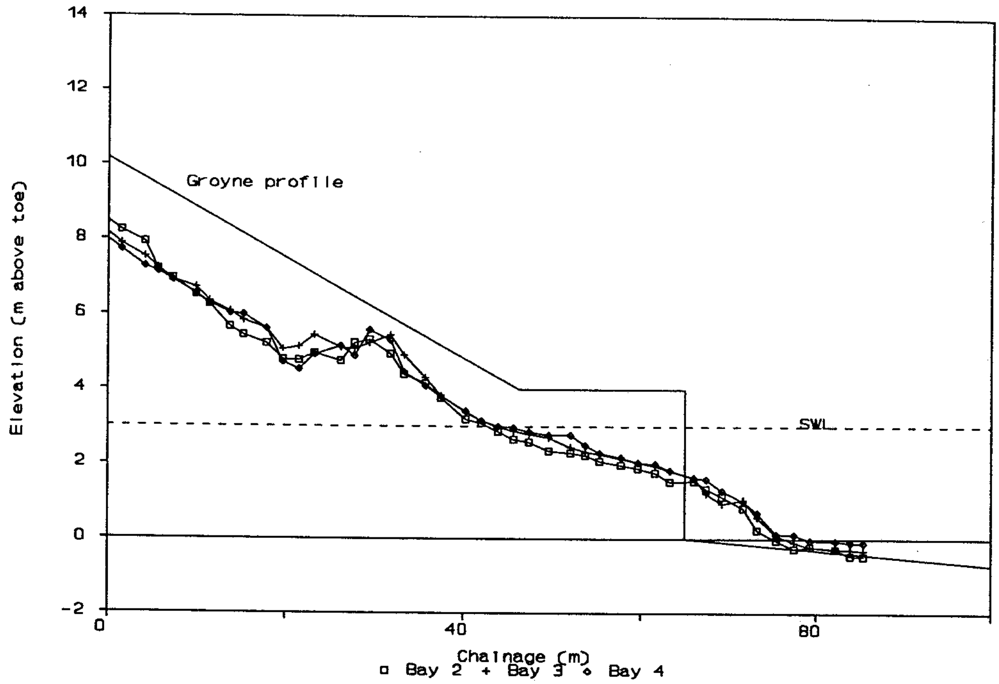


Figure A23 Mid-bay beach profiles: High end timber groynes, SWL = 3m, spacing = 2:1, 30° wave direction

Swell condition (Test 14)



Storm condition (Test 15)

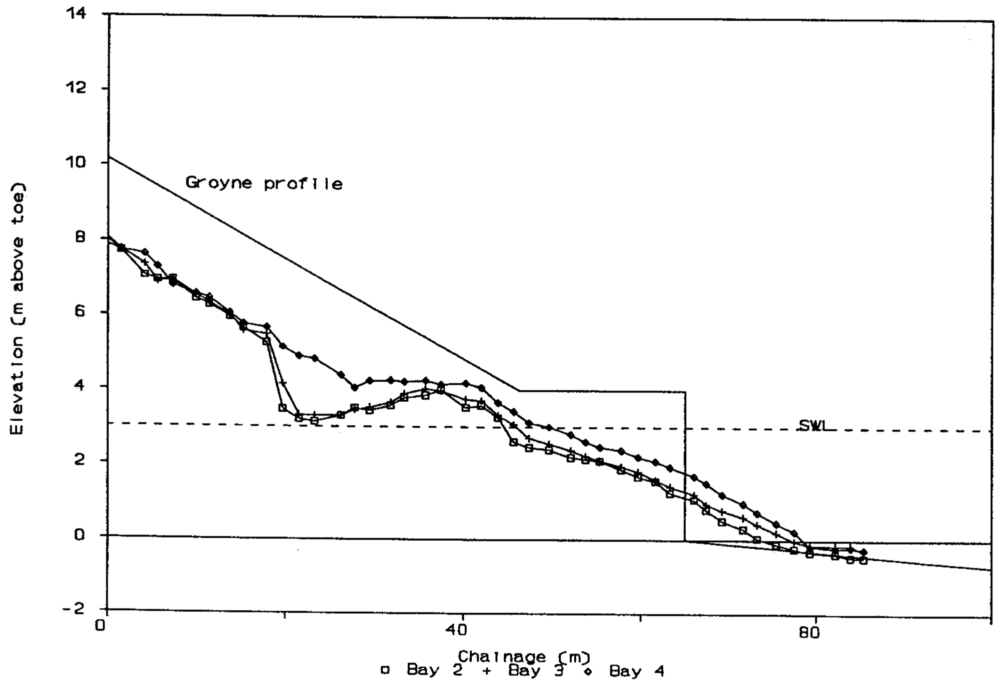
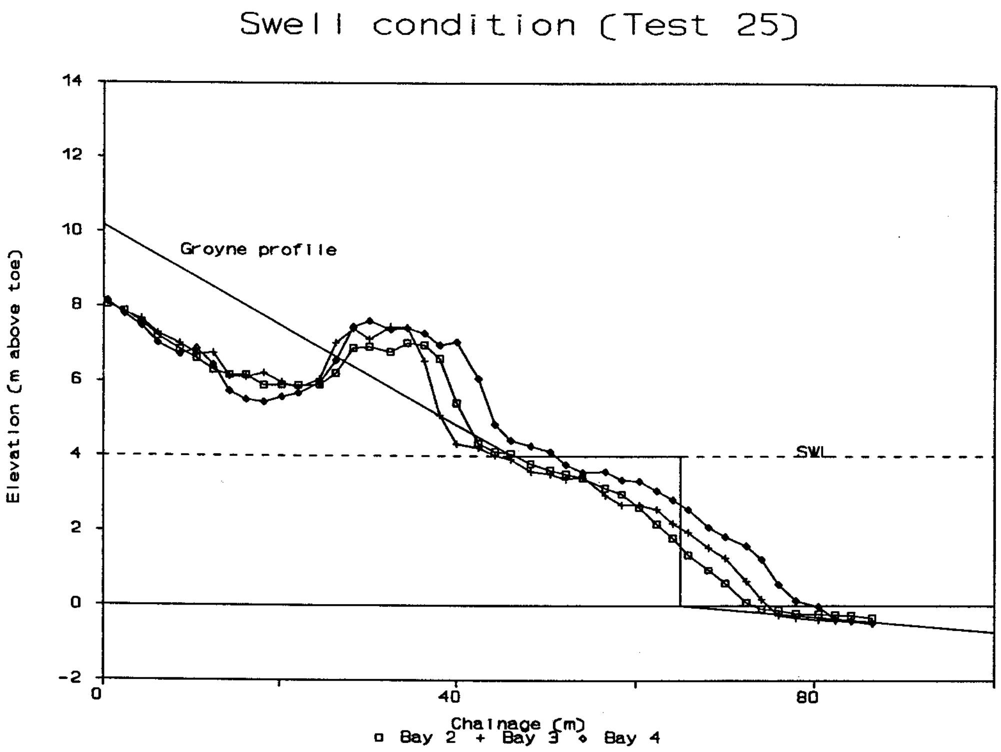


Figure A24 Downdrift beach profiles: High end timber groynes, SWL = 3m, spacing = 2:1, 30° wave direction



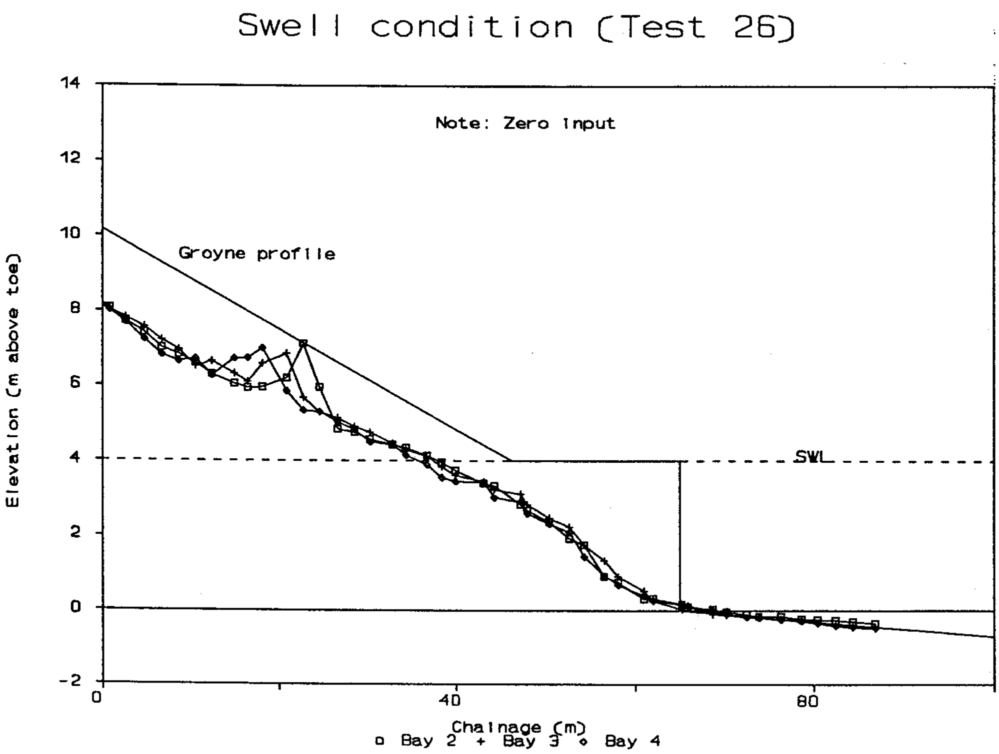
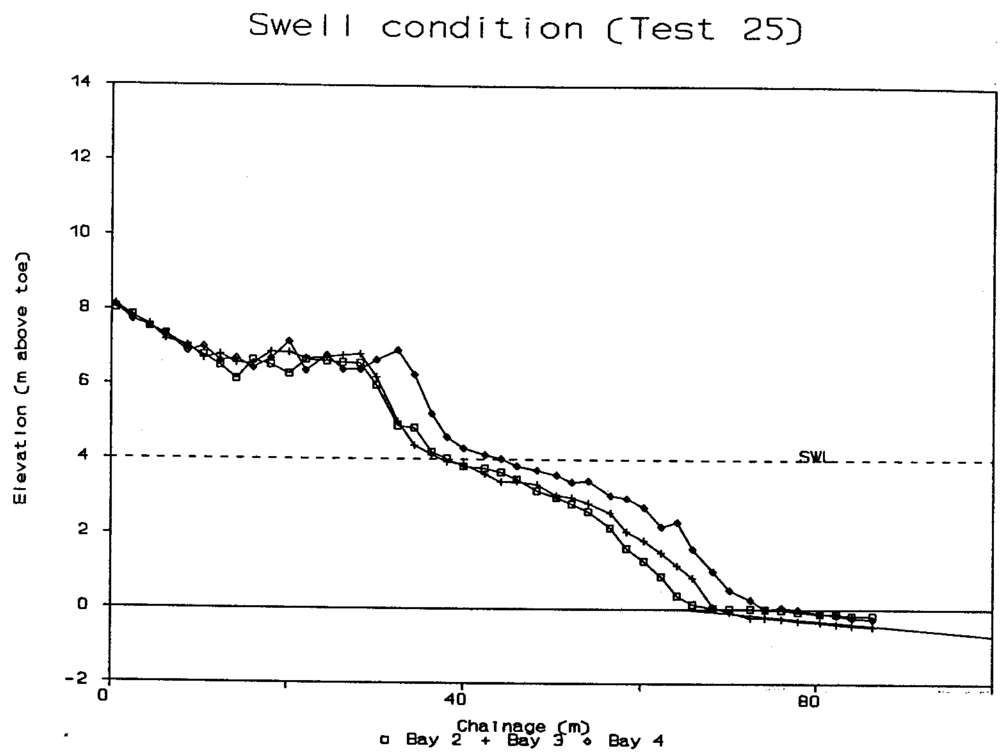


Figure A25 Updrift beach profiles: High end timber groynes, SWL = 4m, spacing = 1.5:1, 30° wave direction



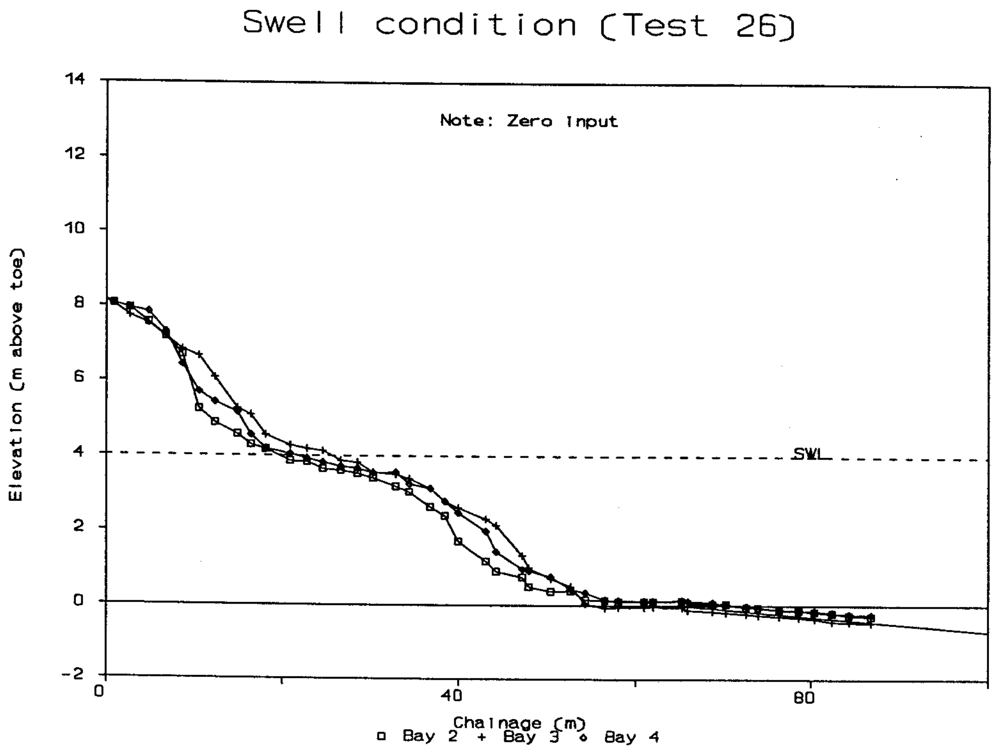
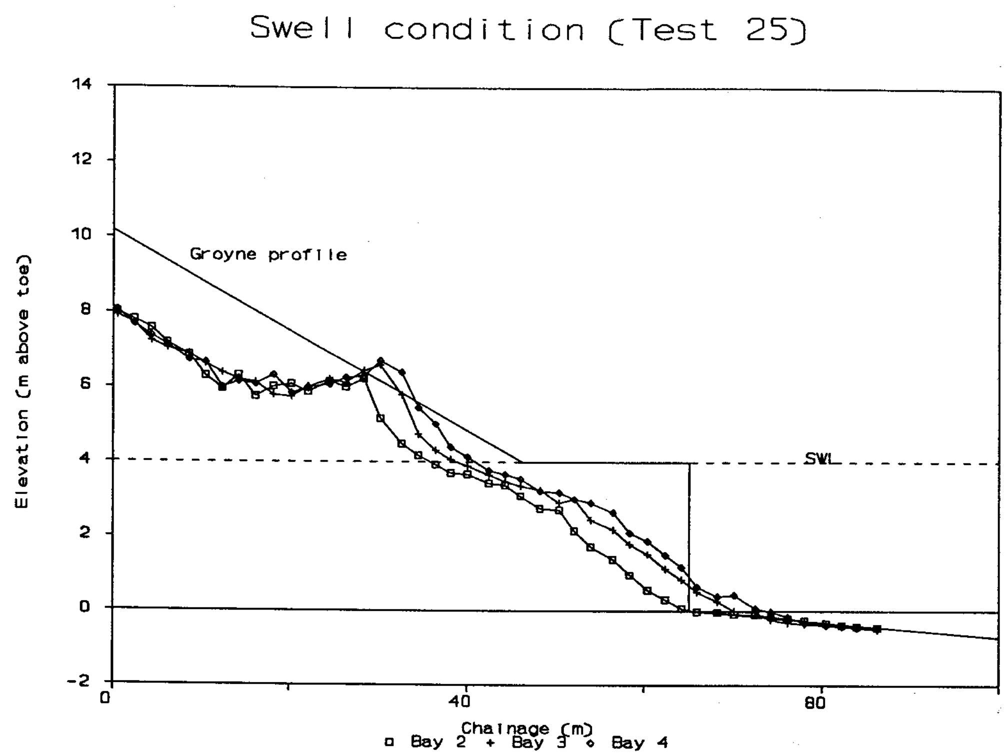


Figure A26 Mid-bay beach profiles: High end timber groynes, SWL = 4m, spacing = 1.5:1, 30° wave direction



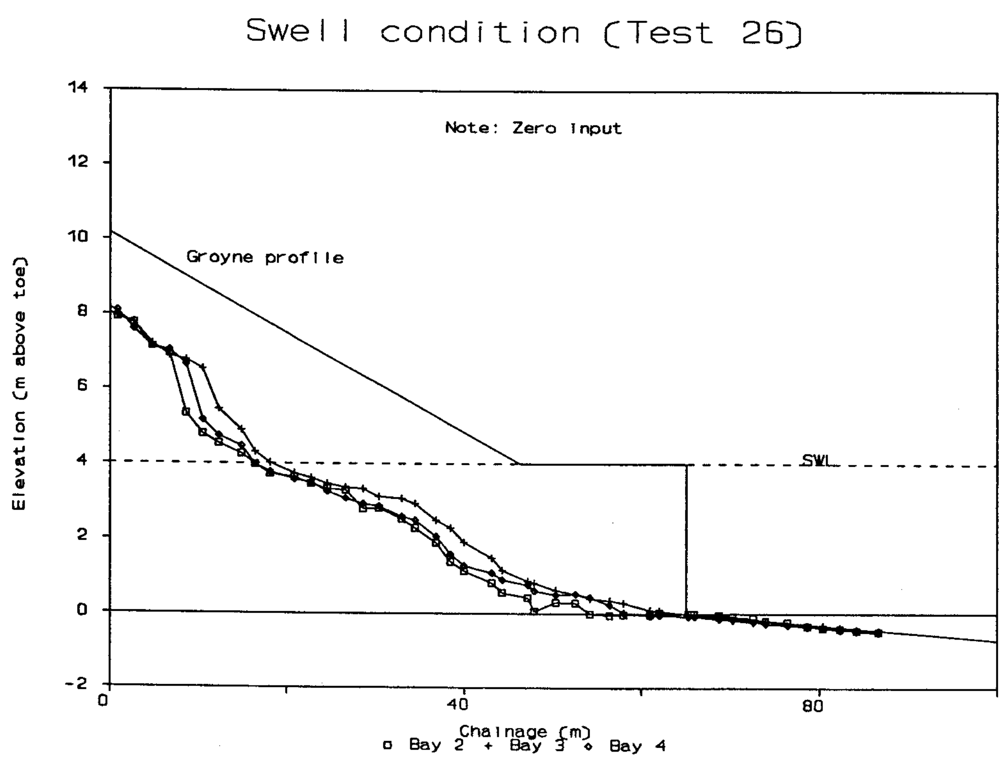
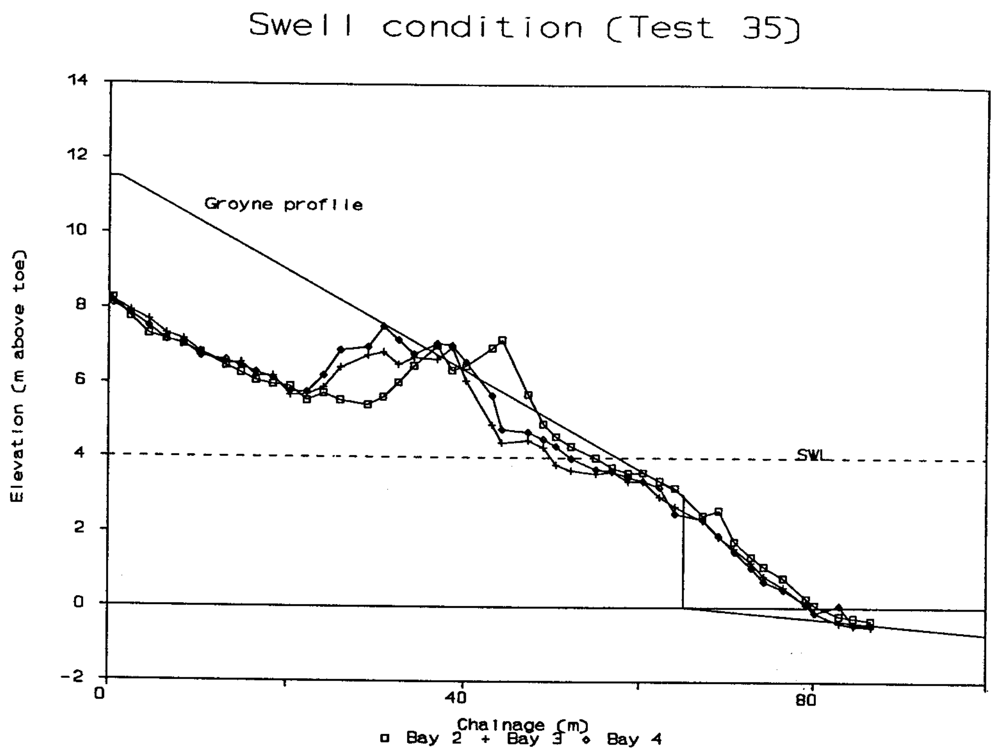


Figure A27 Downdrift beach profiles: High end timber groynes, SWL = 4m, spacing = 1.5:1, 30° wave direction



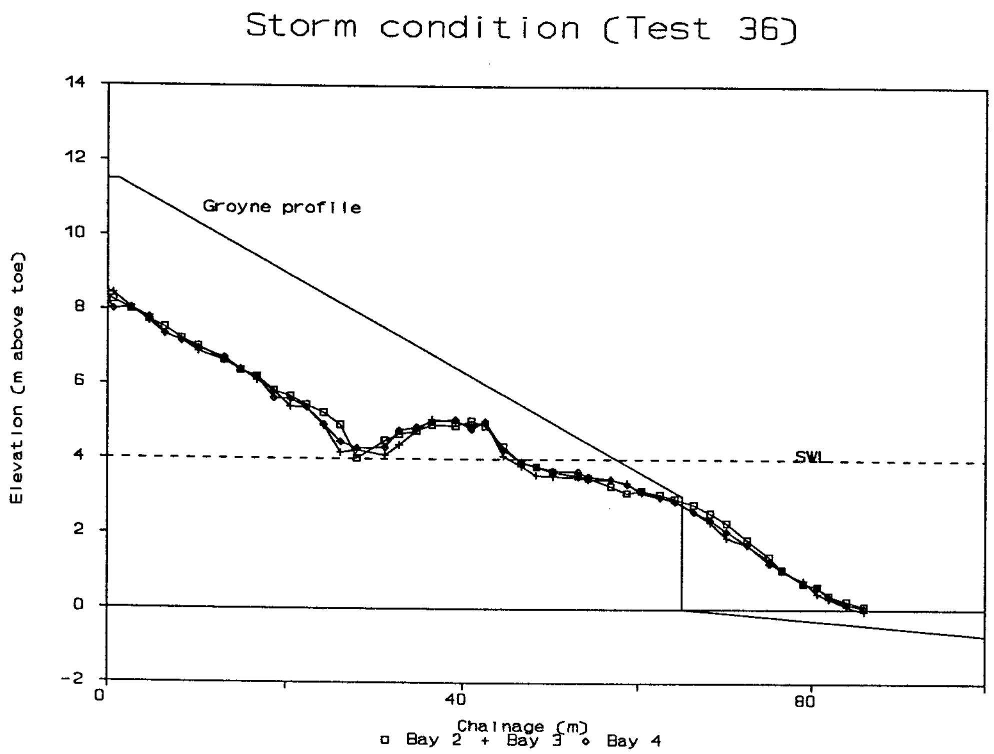
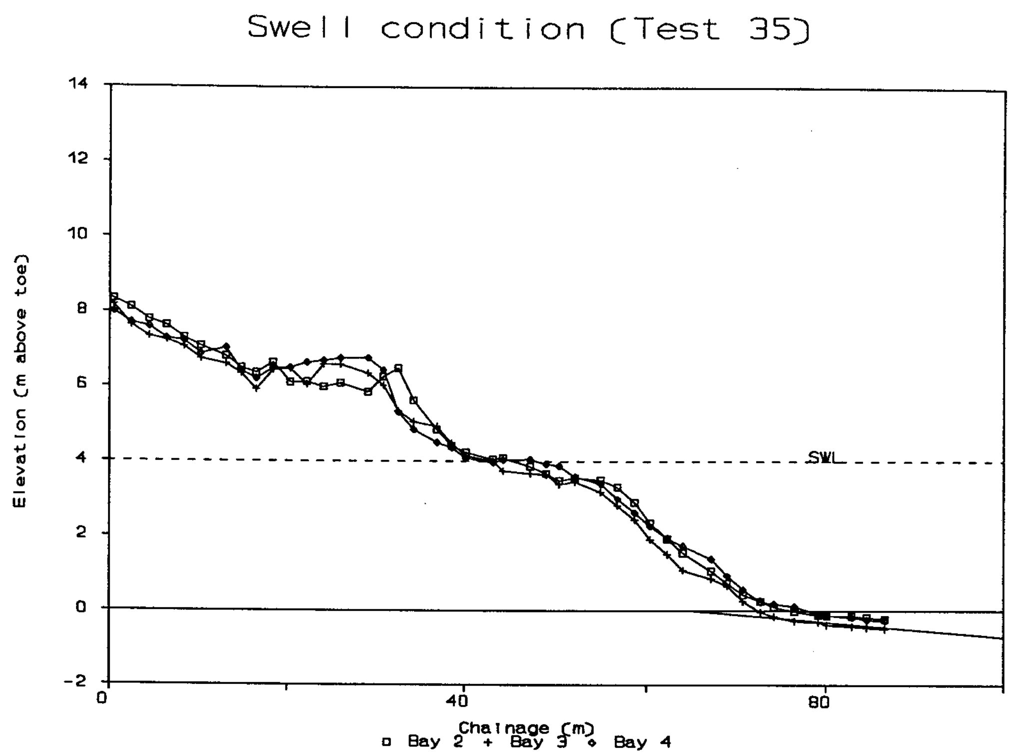


Figure A28 Updrift beach profiles: High timber groynes, SWL = 4m, spacing = 1.5:1, 30° wave direction



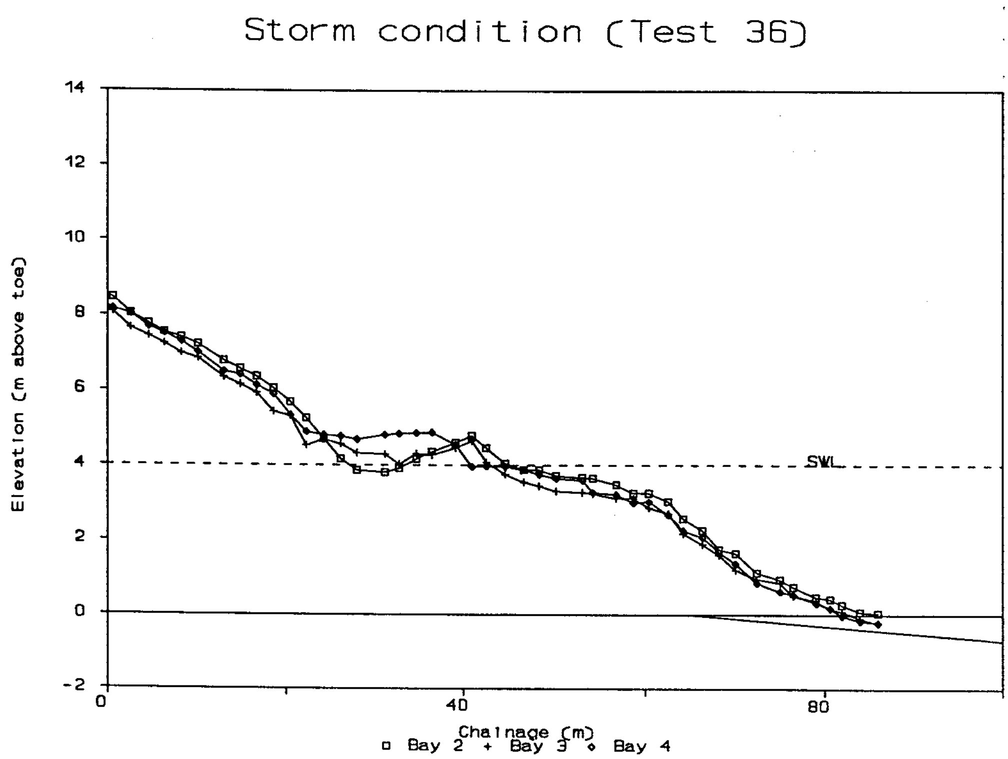
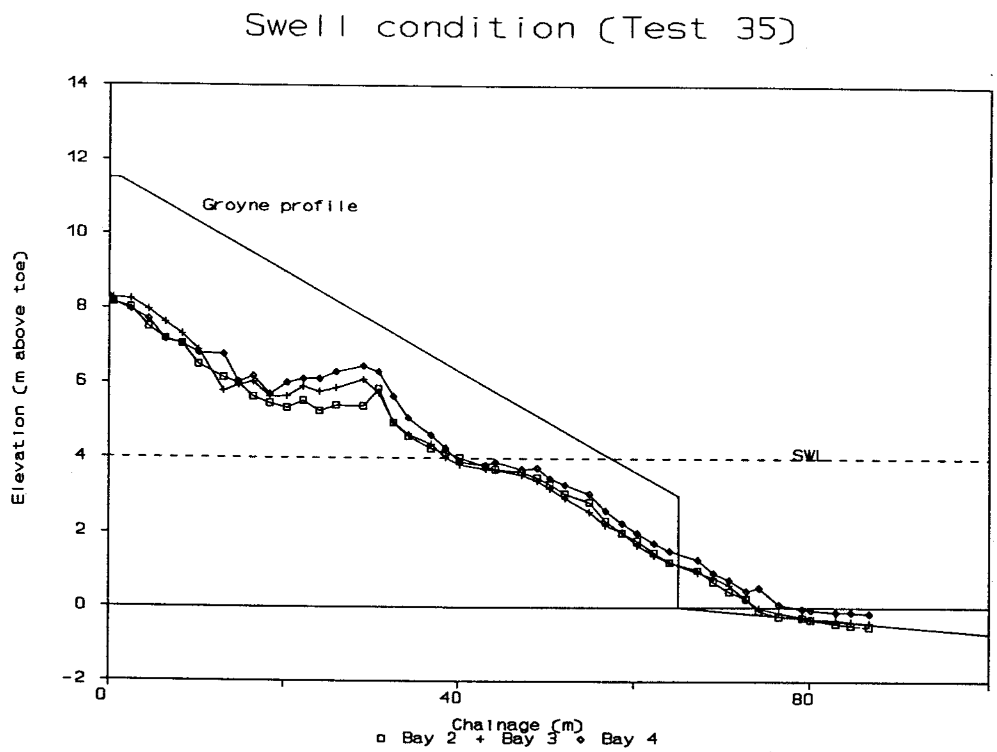


Figure A29 Mid-bay beach profiles: High timber groynes, SWL = 4m, spacing = 1.5:1, 30° wave direction



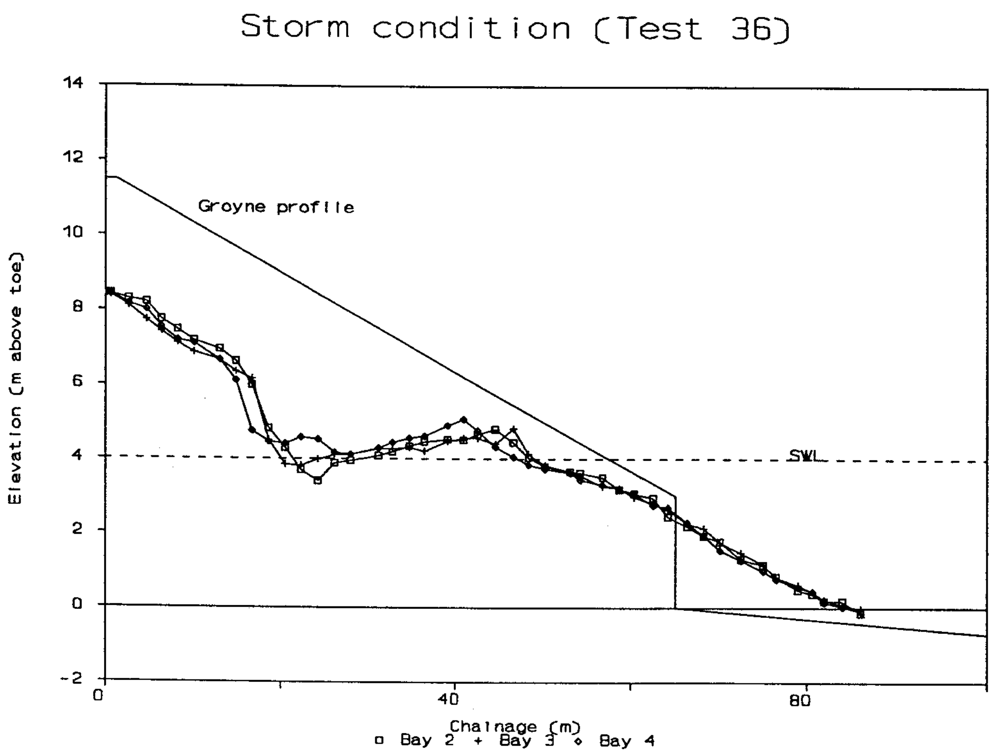
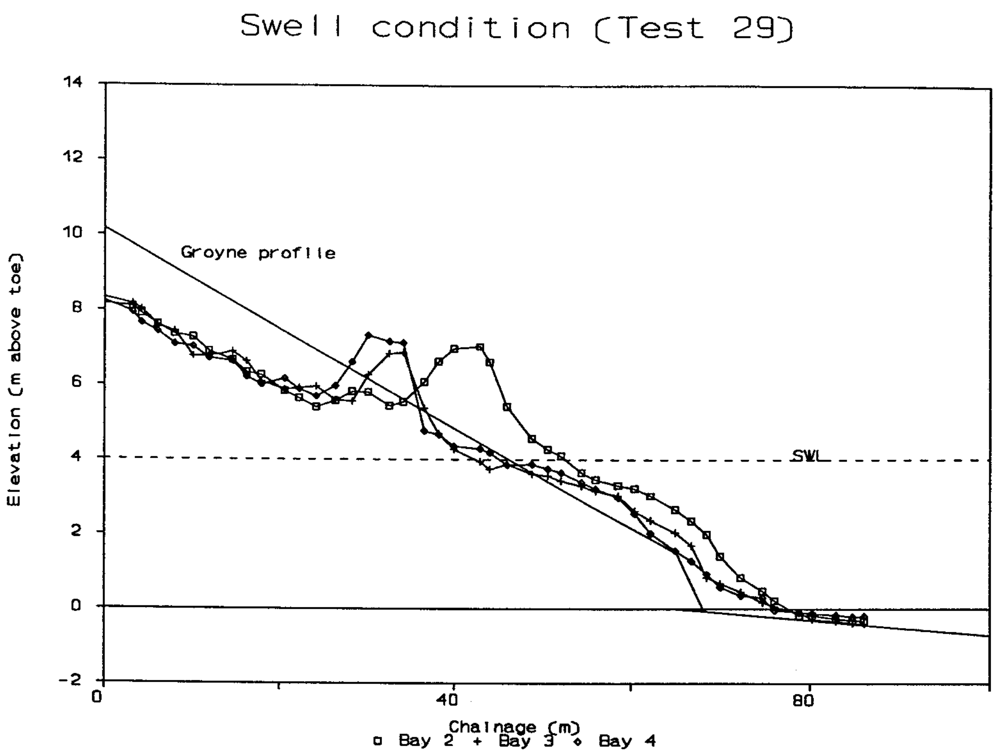


Figure A30 Downdrift beach profiles: High timber groynes, SWL = 4m, spacing = 1.5:1, 30° wave direction



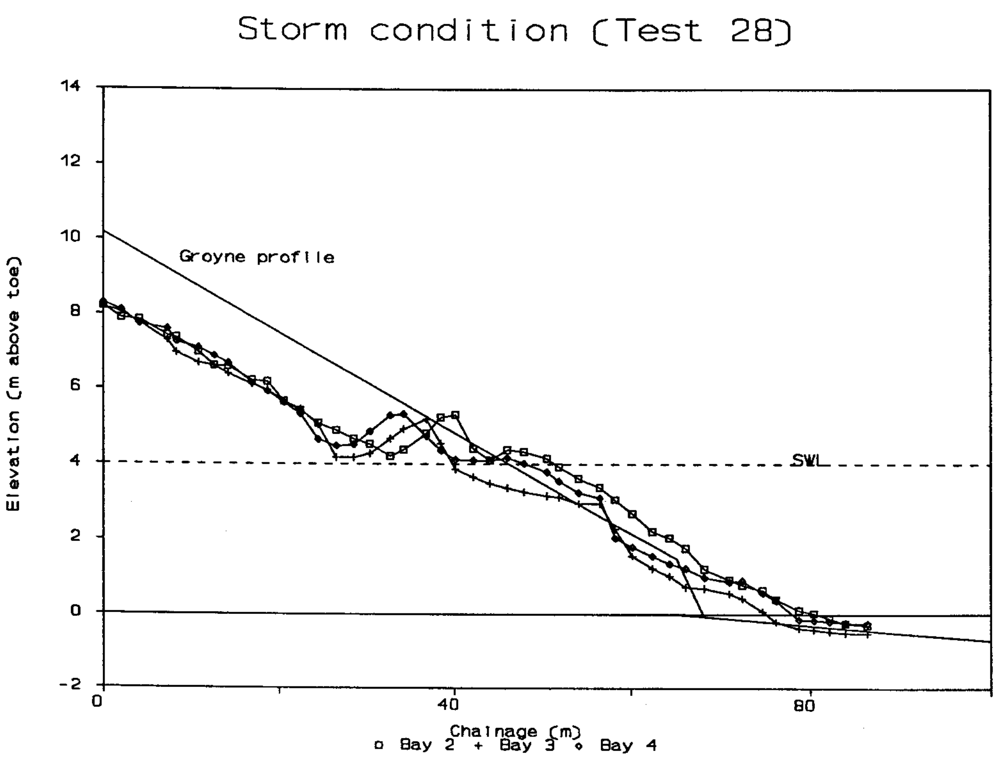
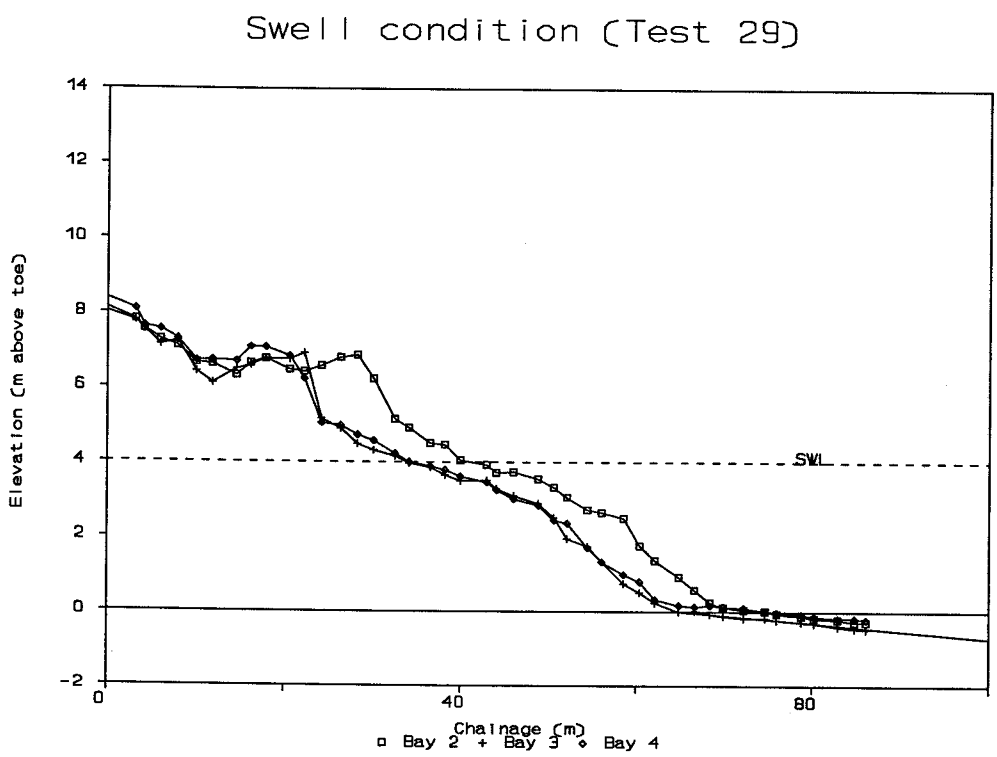


Figure A31 Updrift beach profiles: Low rock groynes, SWL = 4m, spacing= 2:1, 30° wave direction



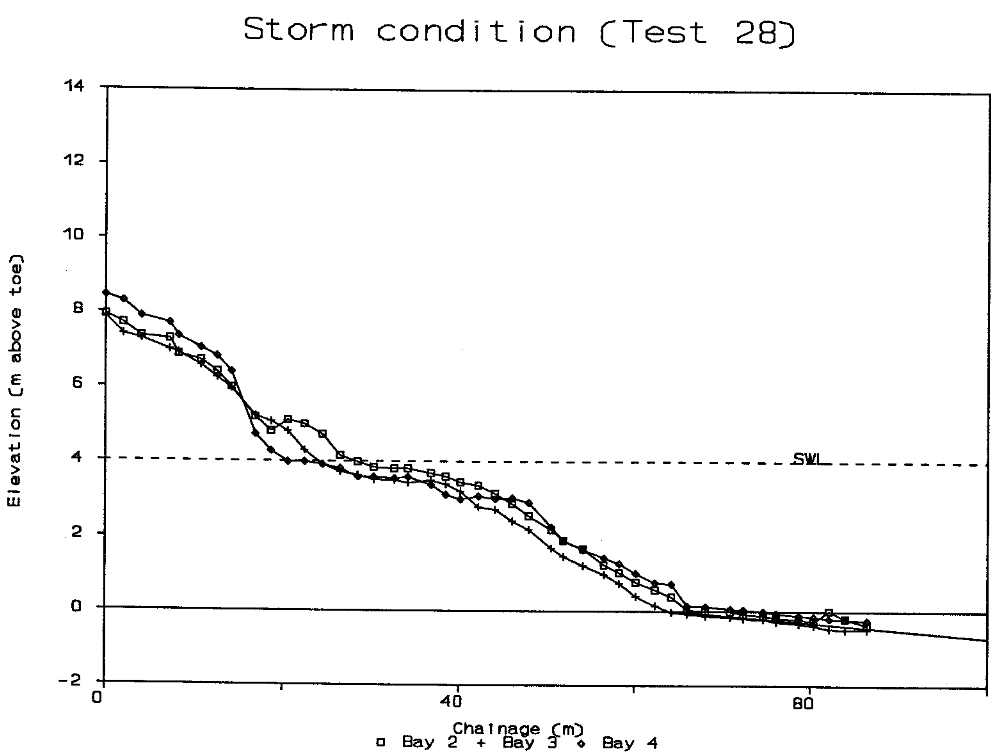
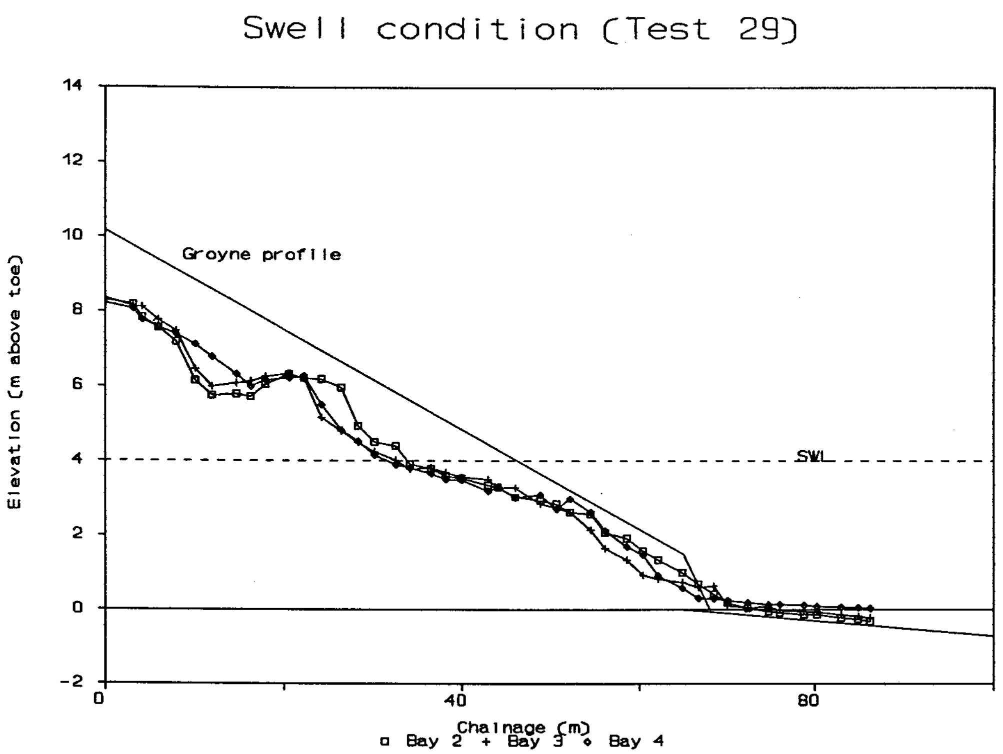


Figure A32 Mid-bay beach profiles: Low rock groynes, SWL = 4m, spacing= 2:1, 30° wave direction



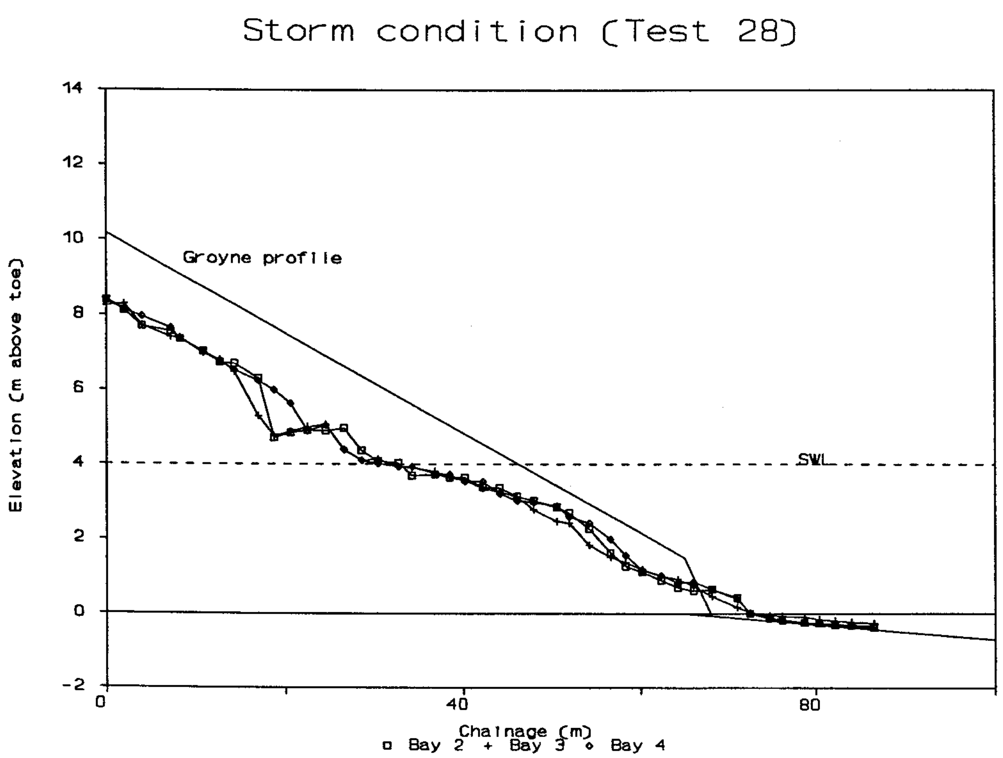


Figure A33 Downdrift beach profiles: Low rock groynes, SWL = 4m, spacing= 2:1, 30° wave direction

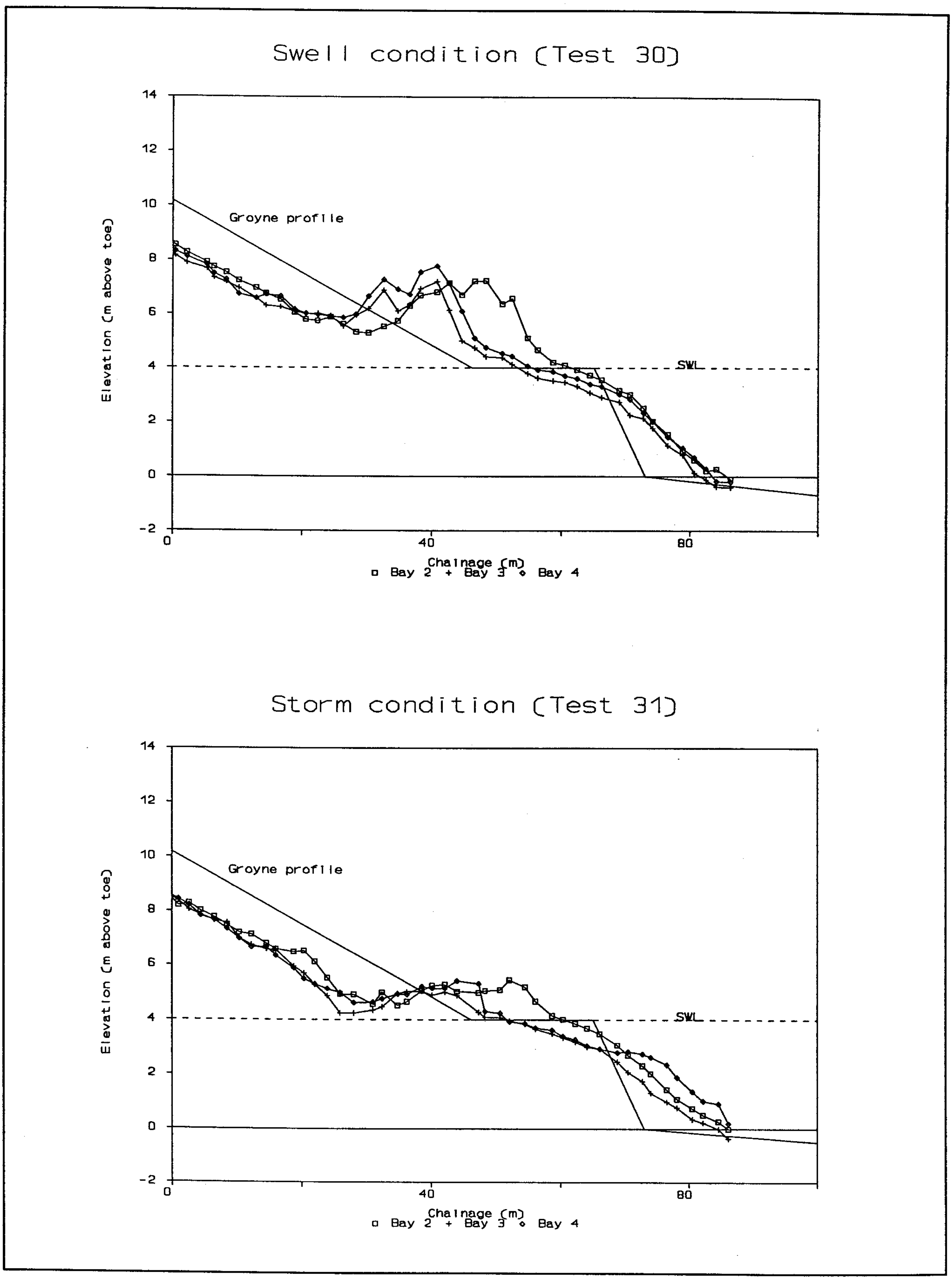


Figure A34 Updrift beach profiles: High end rock groynes, SWL = 4m, spacing = 2:1, 30° wave direction

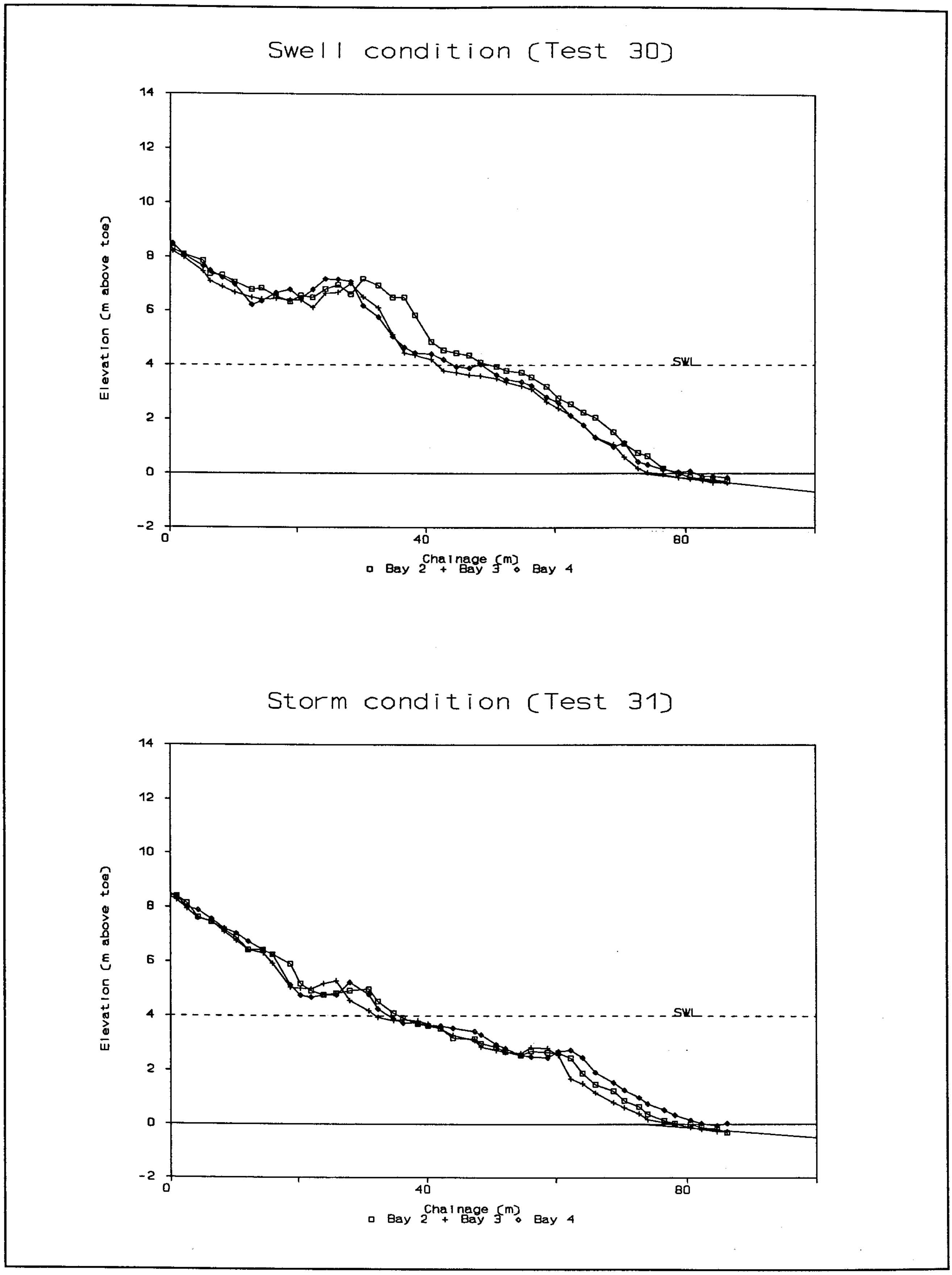
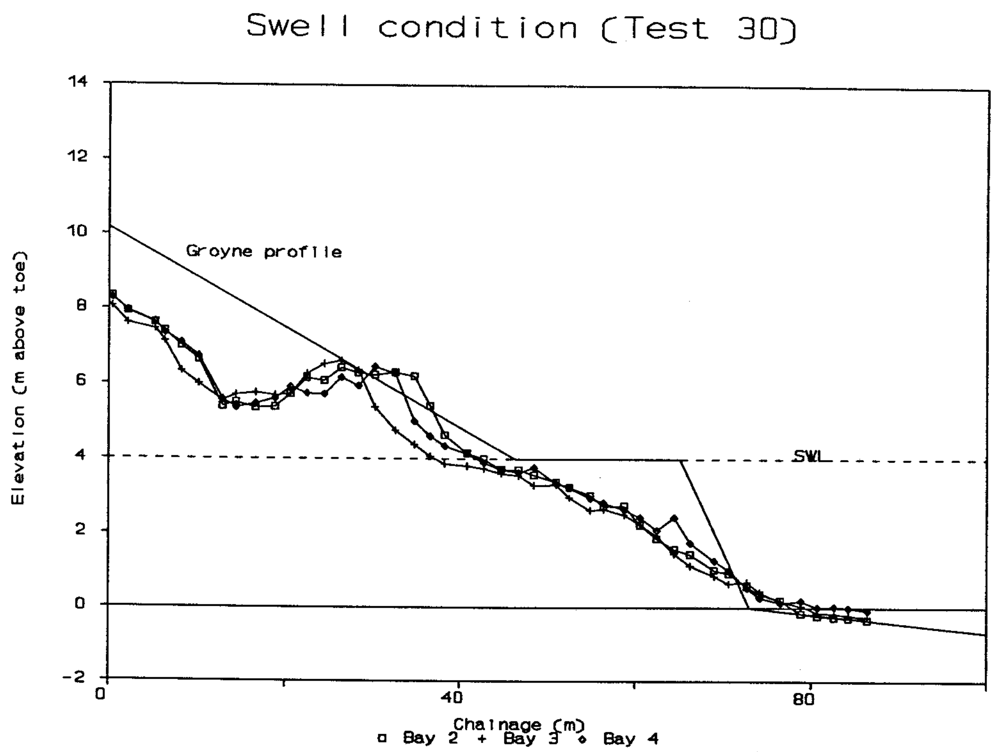


Figure A35 Mid-bay beach profiles: High end rock groynes, SWL = 4m, spacing = 2:1, 30° wave direction



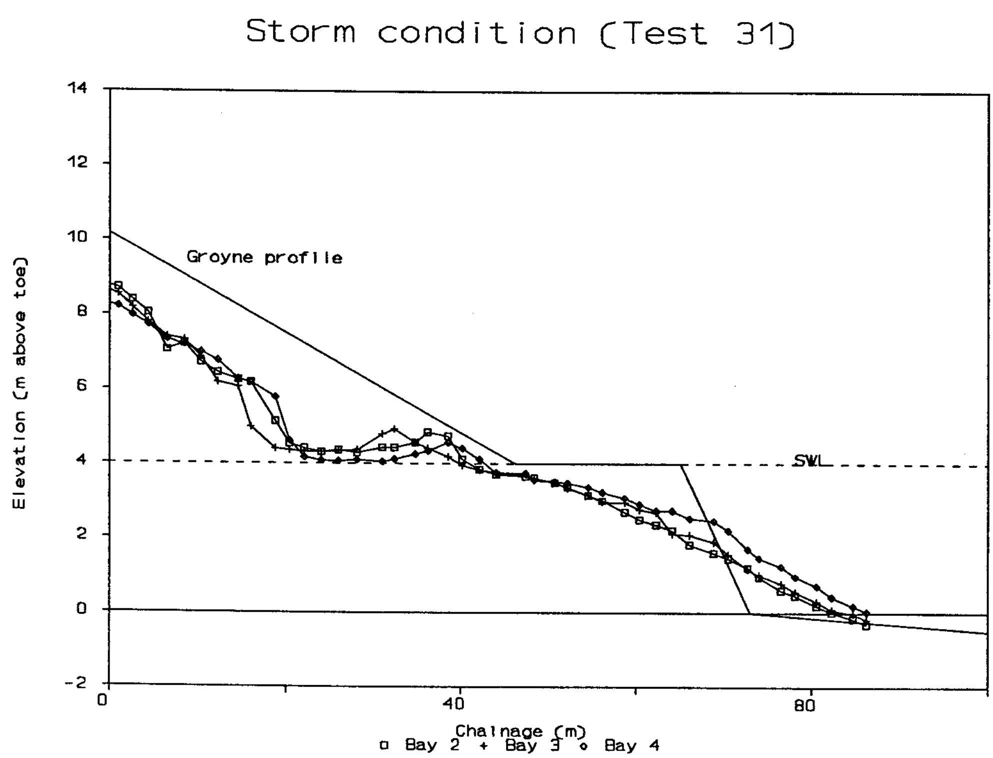
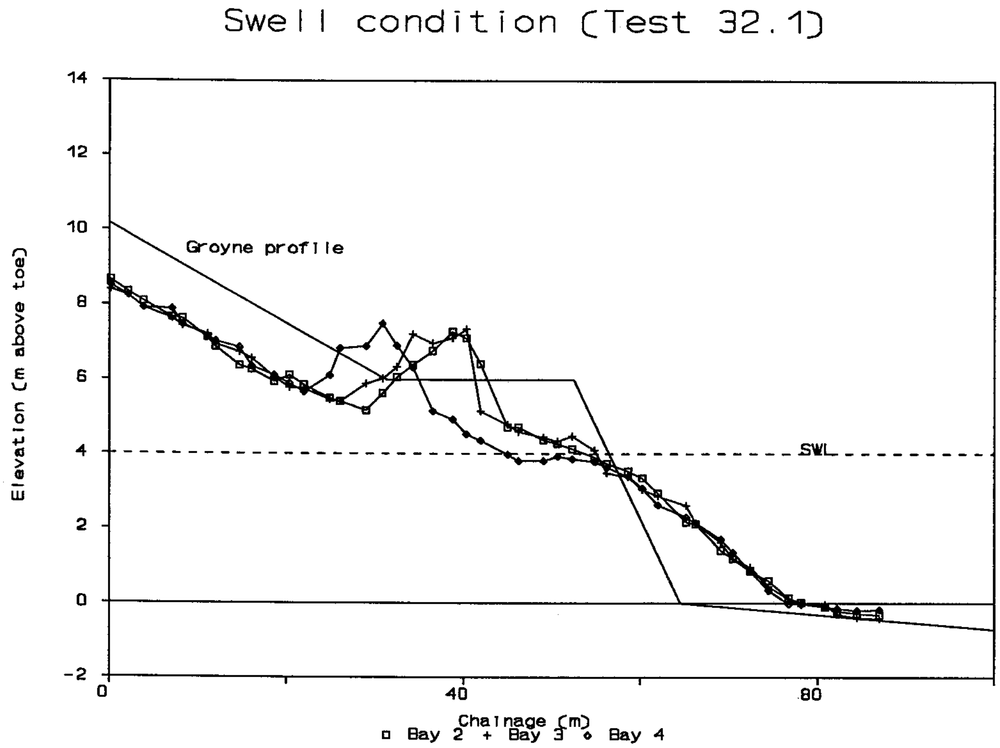


Figure A36 Downdrift beach profiles: High end rock groynes, SWL = 4m, spacing = 2:1, 30° wave direction



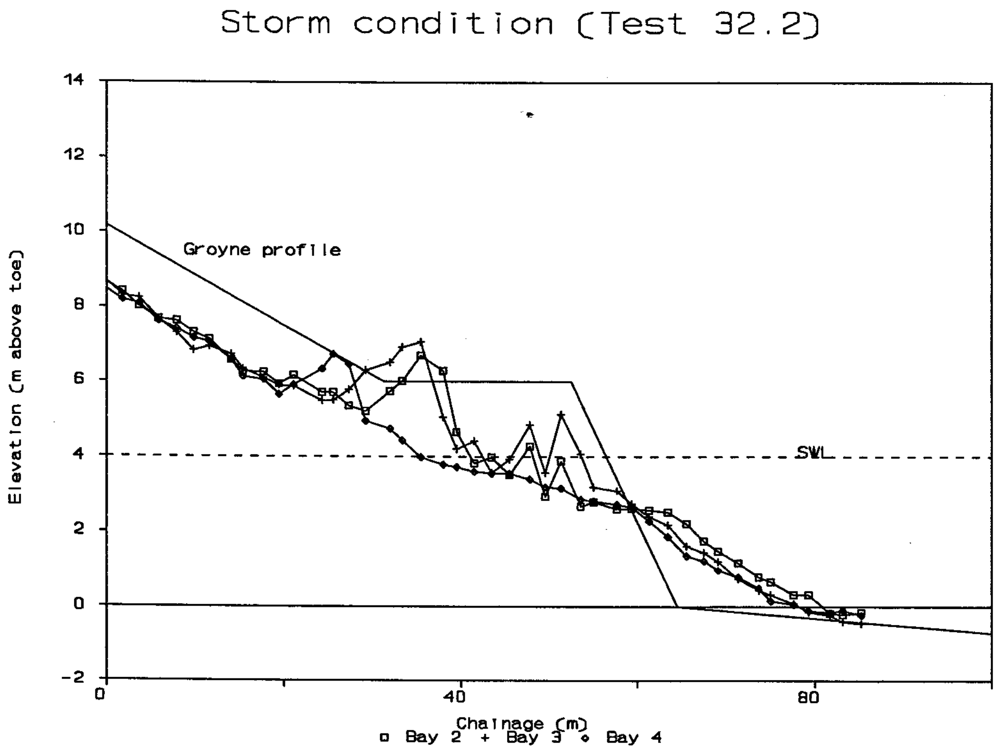


Figure A37 Updrift beach profiles: Short rock barriers, SWL = 4m, spacing = 2:1, 30° wave direction

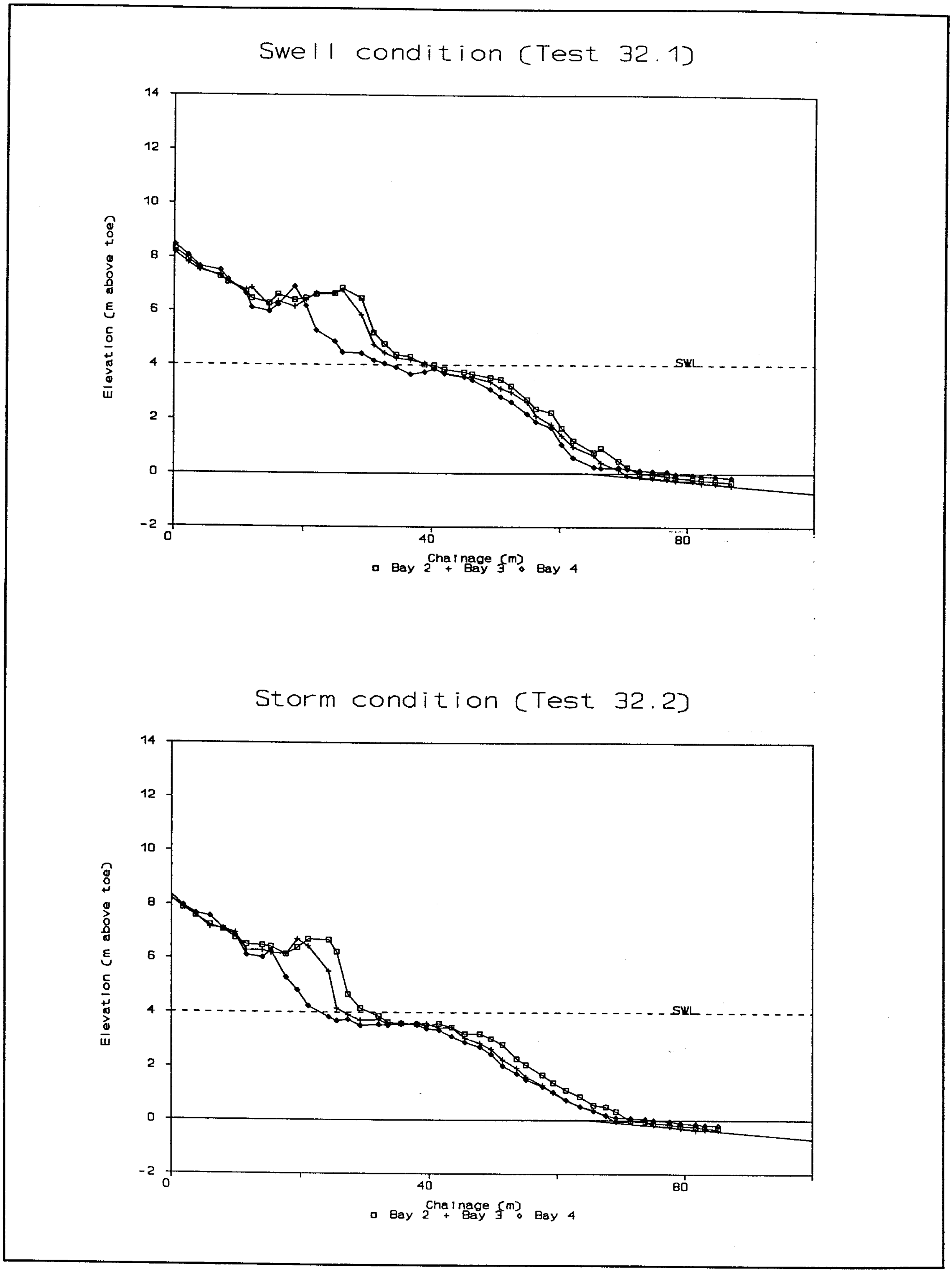
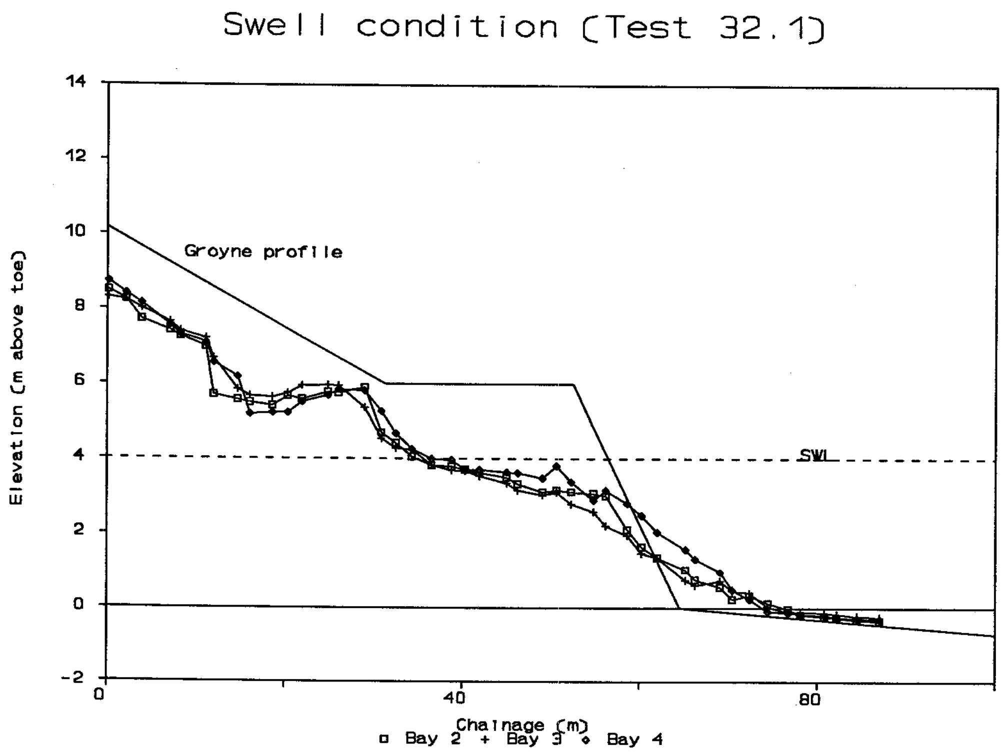


Figure A38 Mid-bay beach profiles: Short rock barriers, SWL = 4m, spacing = 2:1, 30° wave direction



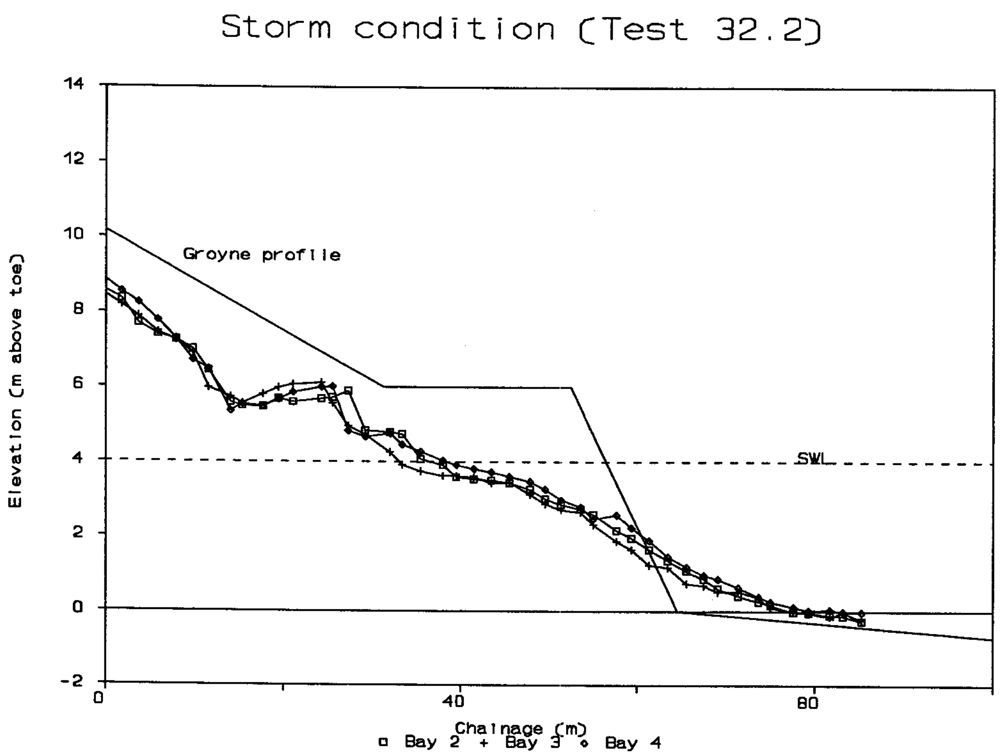
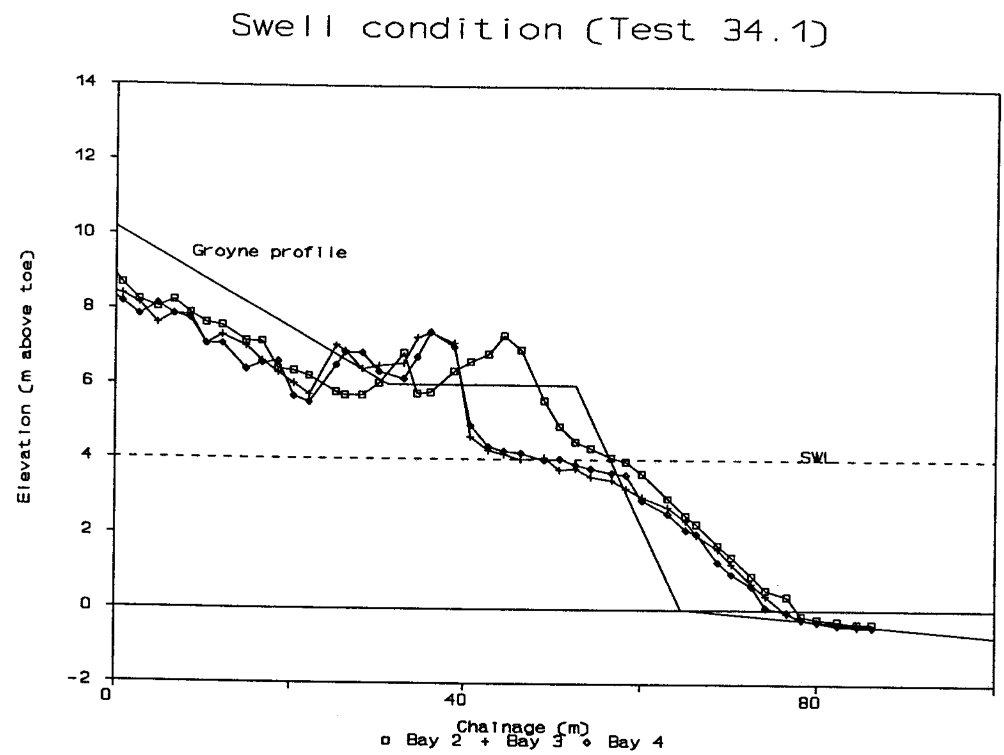


Figure A39 Downdrift beach profiles: Short rock barriers, SWL = 4m, spacing = 2:1, 30° wave direction



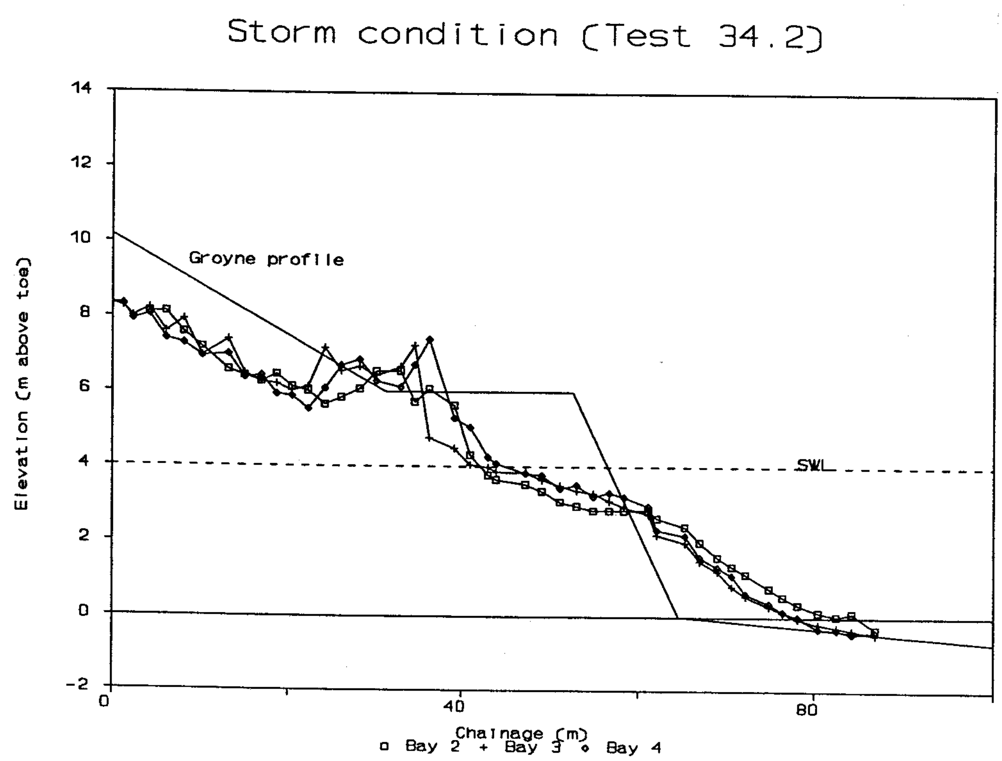


Figure A40 Updrift beach profiles: Short rock barriers, SWL = 4m, spacing = 1.5:1, 30° wave direction

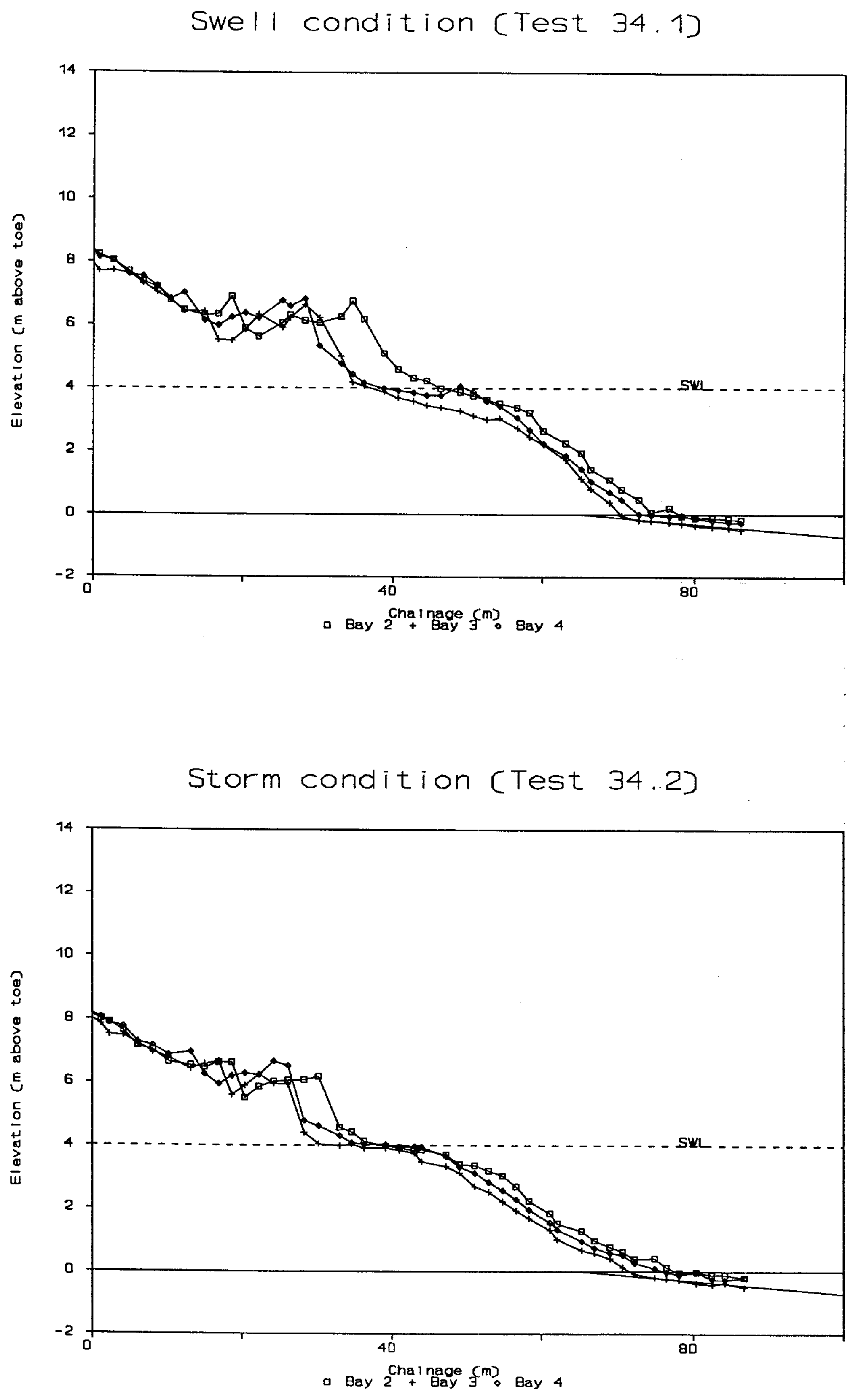
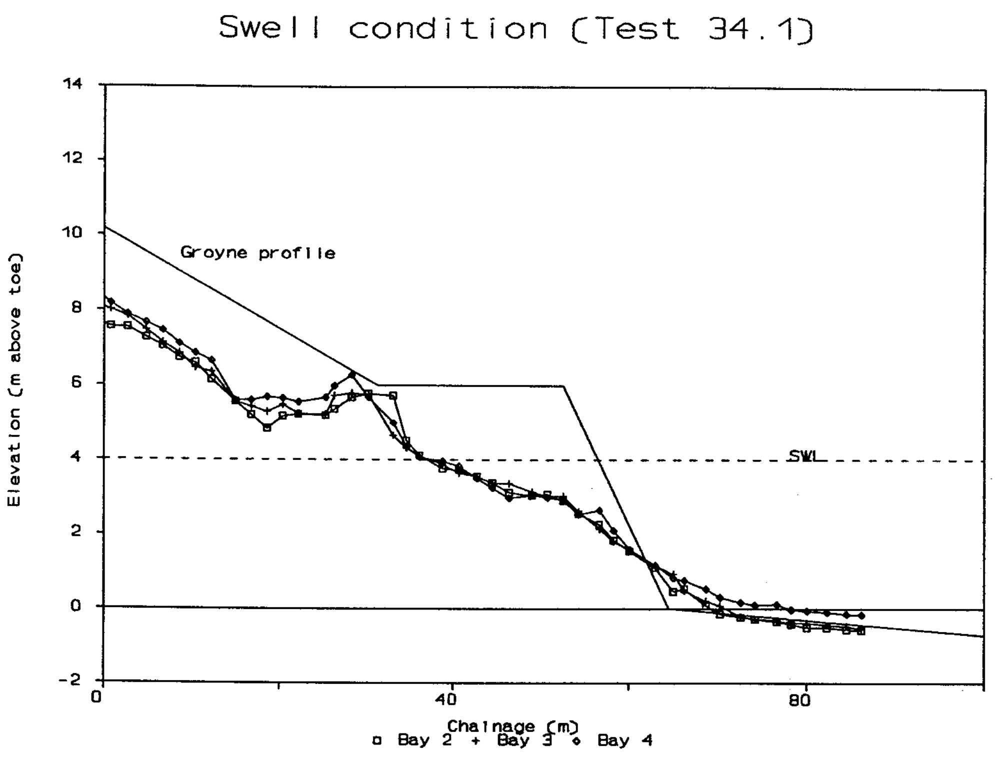


Figure A41 Mid-bay beach profiles: Short rock barriers, SWL = 4m, spacing = 1.5:1, 30° wave direction



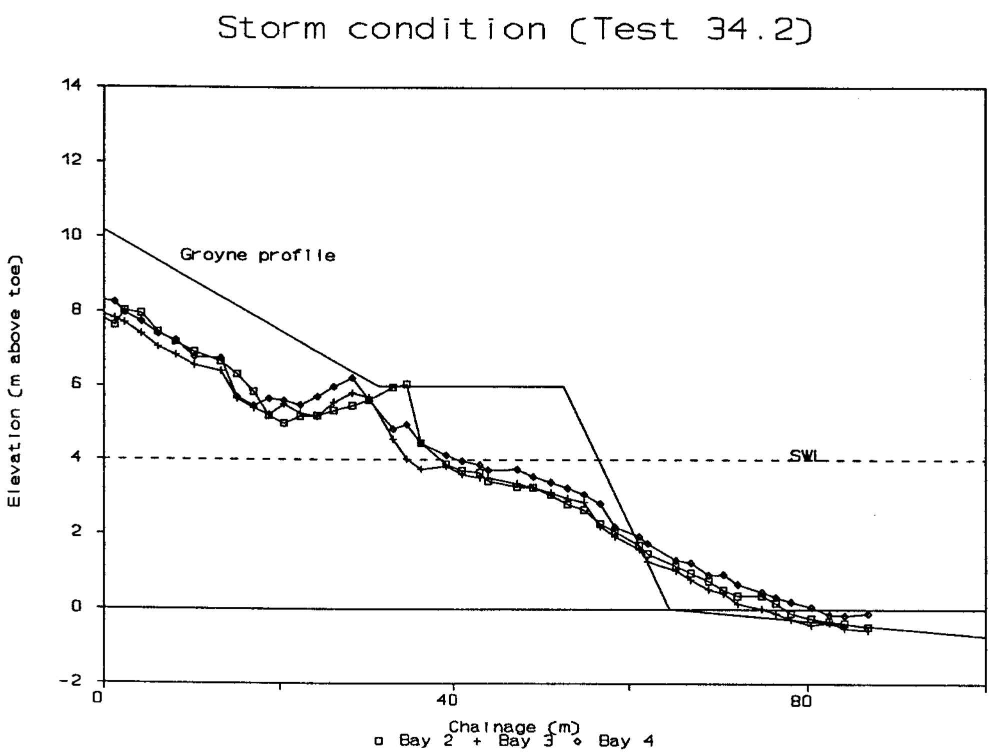
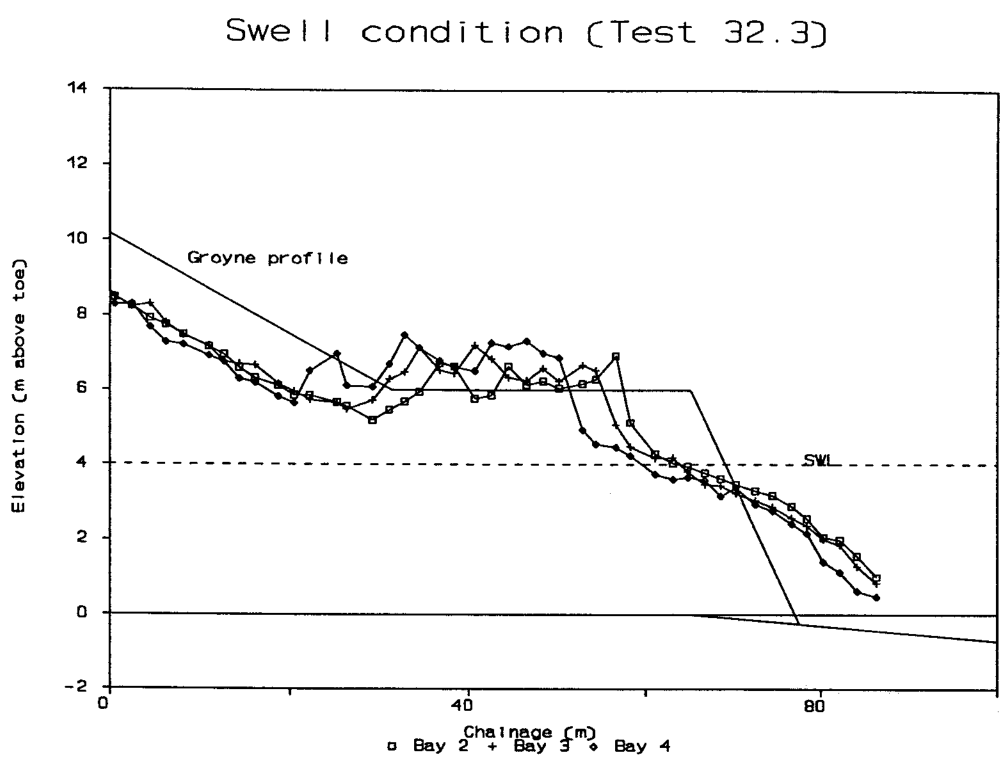


Figure A42 Downdrift beach profiles: Short rock barriers, SWL = 4m, spacing = 1.5:1, 30° wave direction



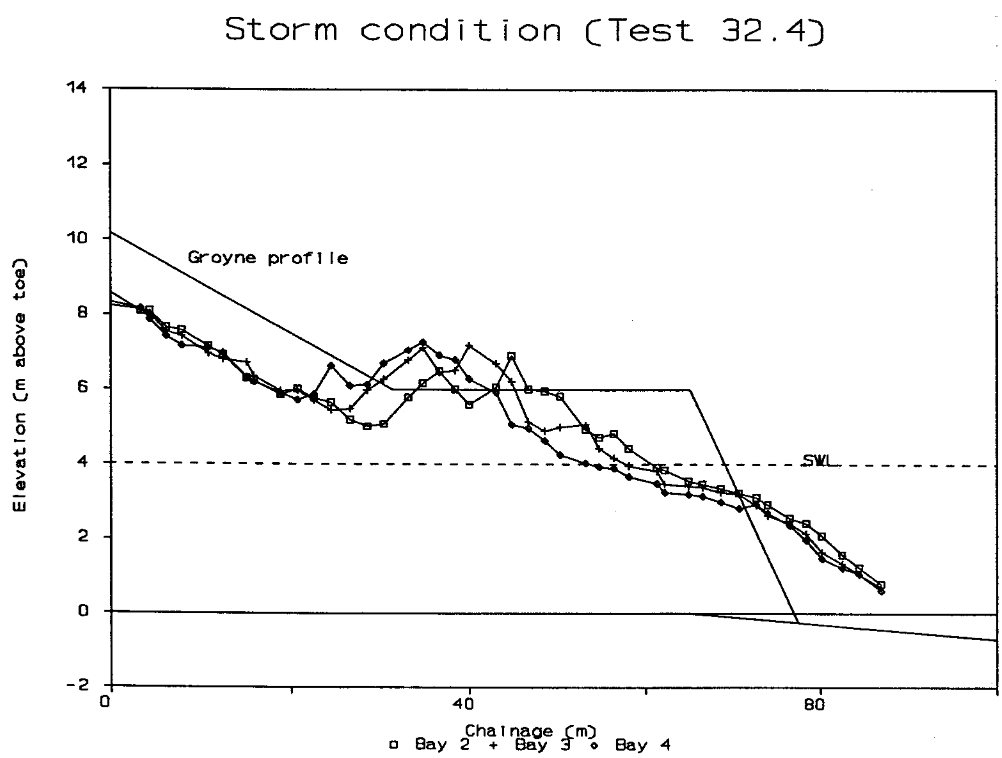
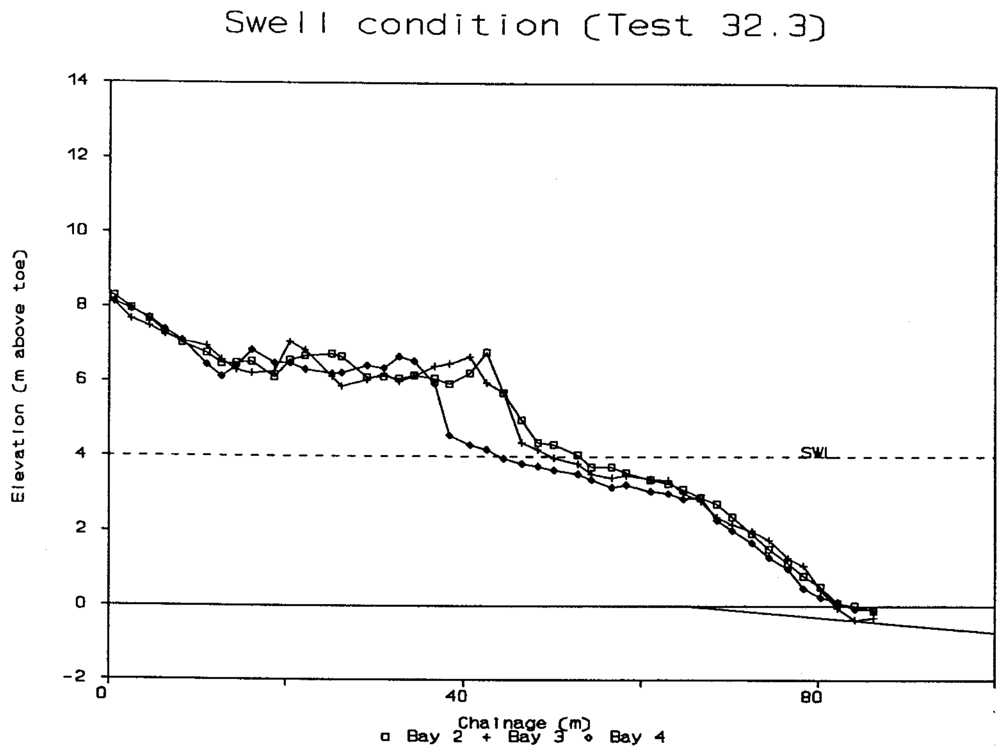


Figure A43 Updrift beach profiles: Long rock barriers, SWL = 4m, spacing = 2:1, 30° wave direction



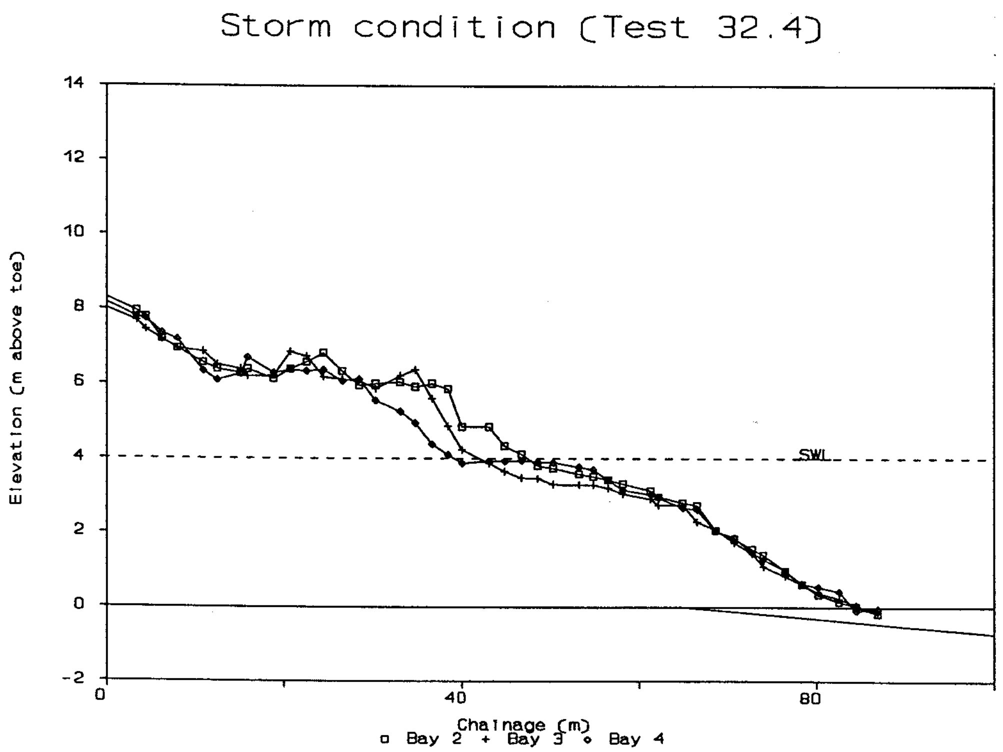
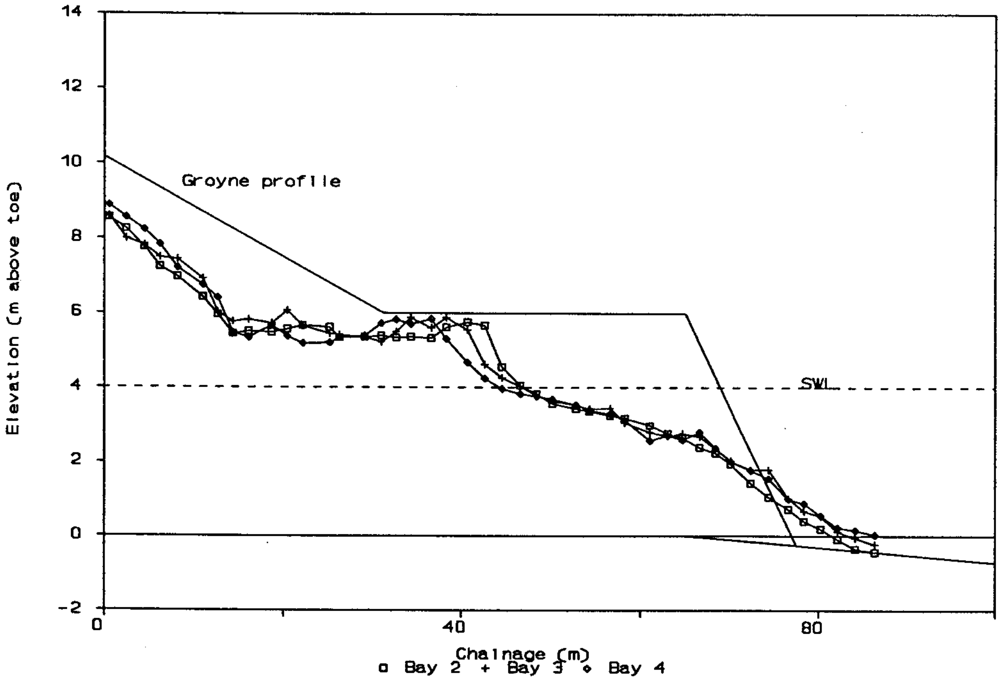


Figure A44 Mid-bay beach profiles: Long rock barriers, SWL = 4m, spacing = 2:1, 30° wave direction

Swell condition (Test 32.3)



Storm condition (Test 32.4)

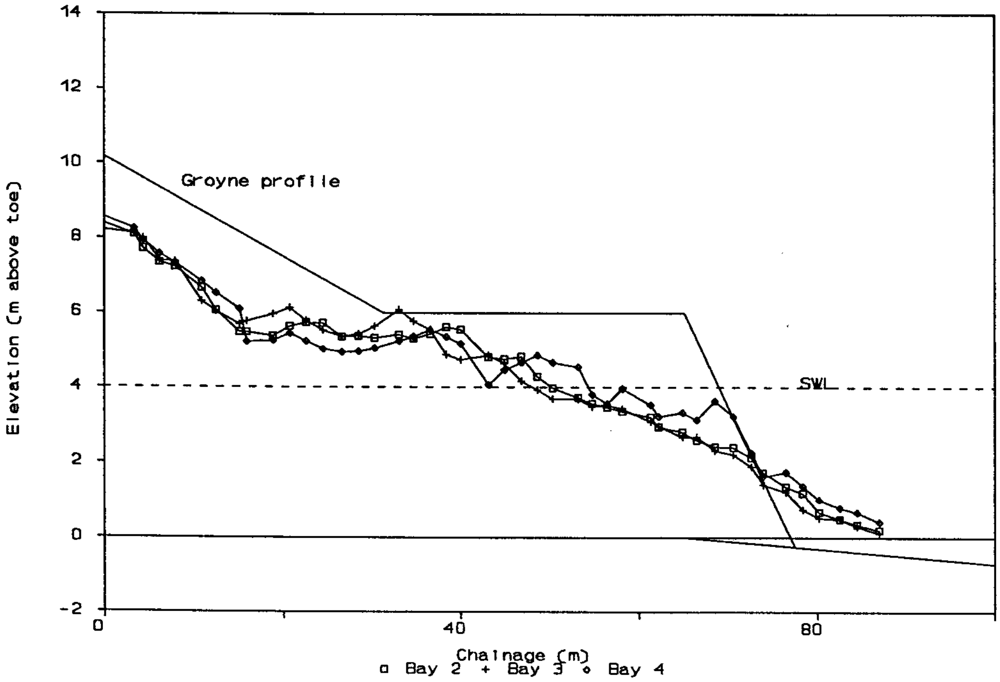
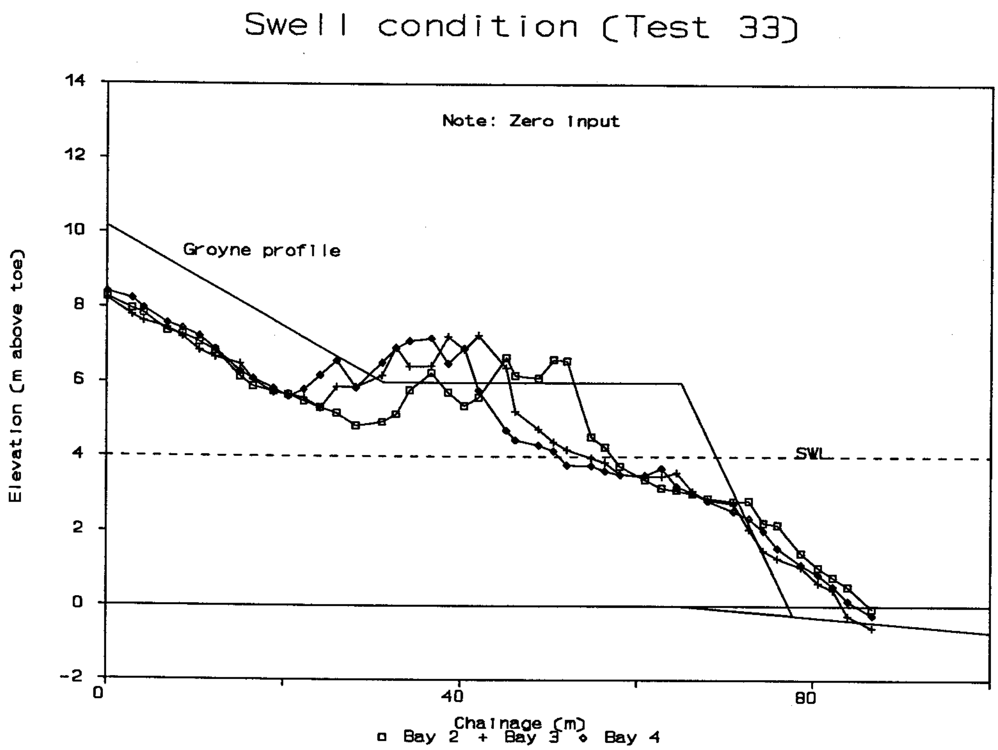


Figure A45 Downdrift beach profiles: Long rock barriers, SWL = 4m, spacing = 2:1, 30° wave direction



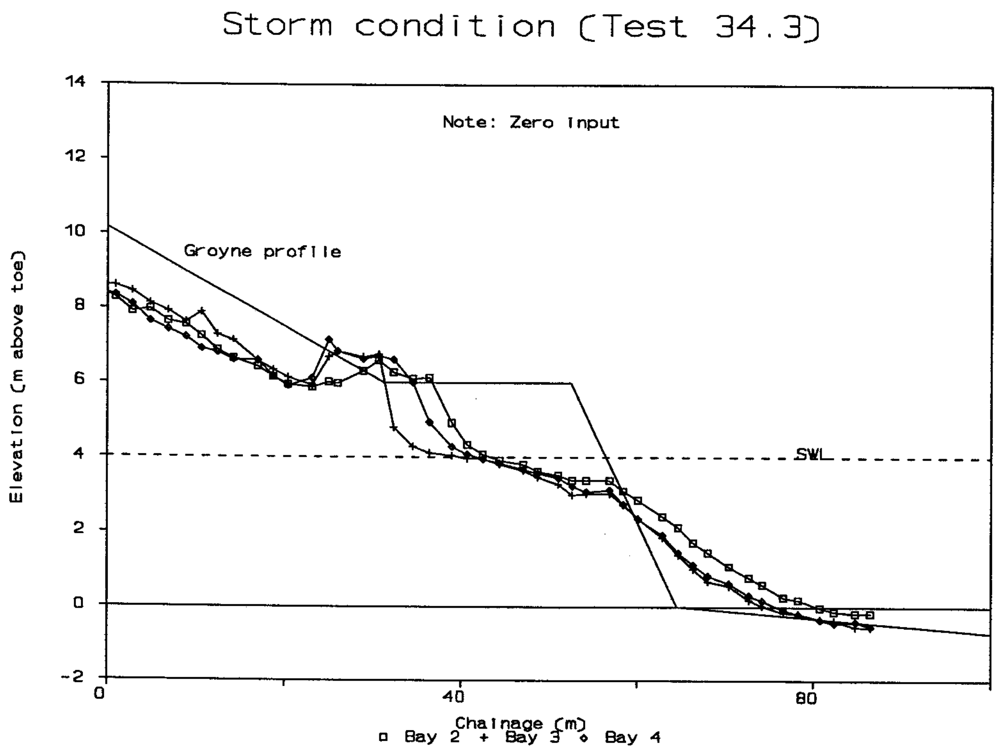


Figure A46 Updrift beach profiles: Long and short rock barriers, SWL = 4m, spacing = 1.5:1, 30° wave direction

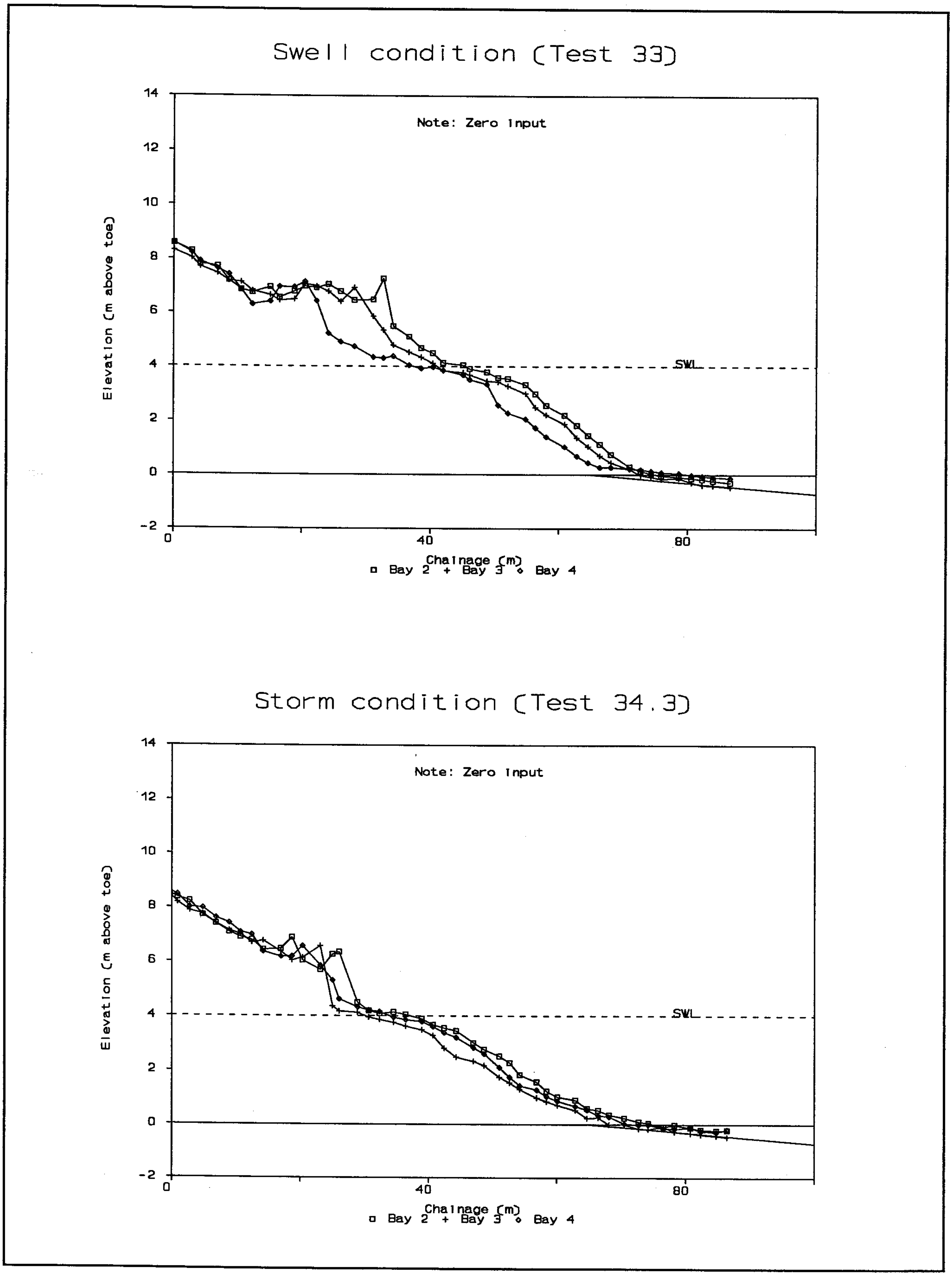
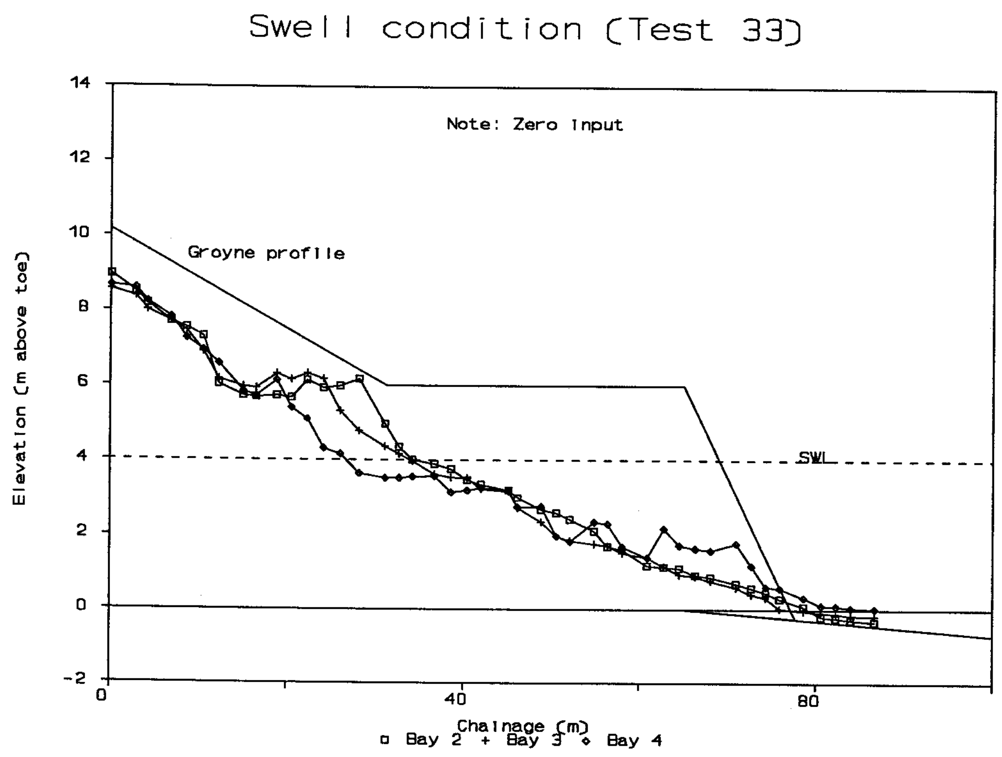


Figure A47 Mid-bay beach profiles: Long and short rock barriers, SWL = 4m, spacing = 1.5:1, 30° wave direction



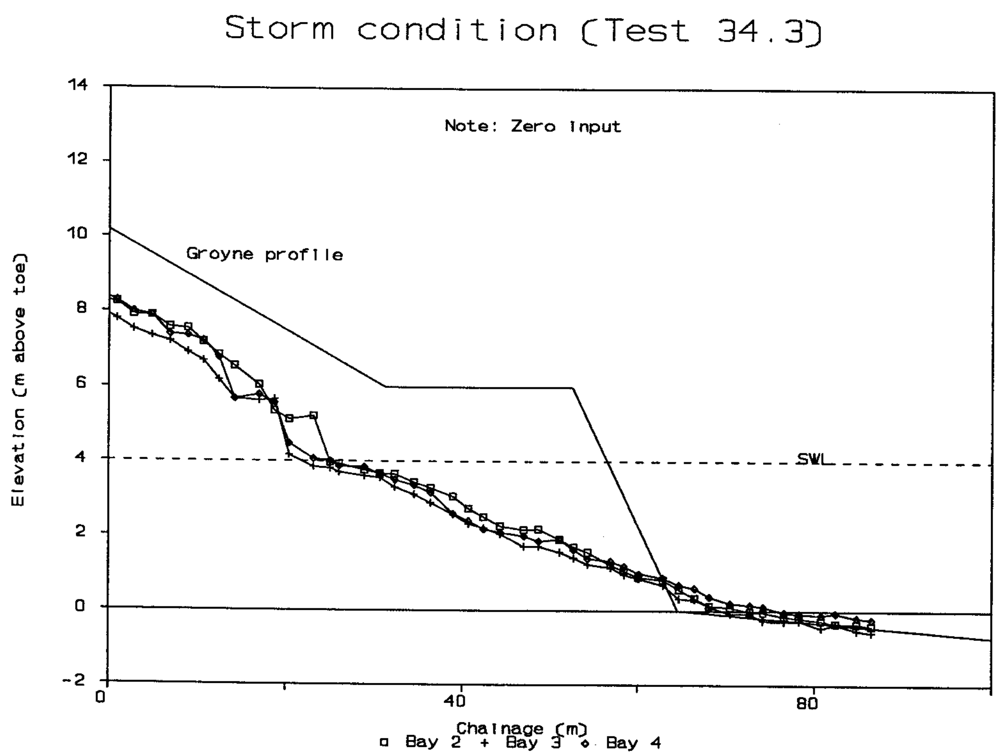
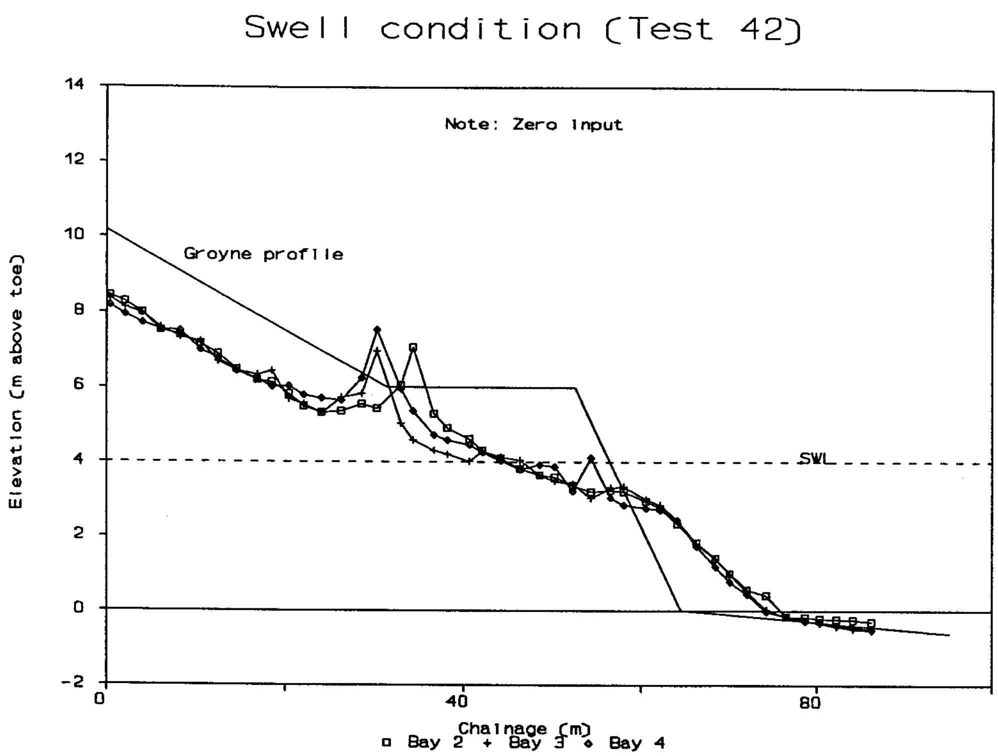


Figure A48 Downdrift beach profiles: Long and short rock barriers, SWL = 4m, spacing = 1.5:1, 30° wave direction



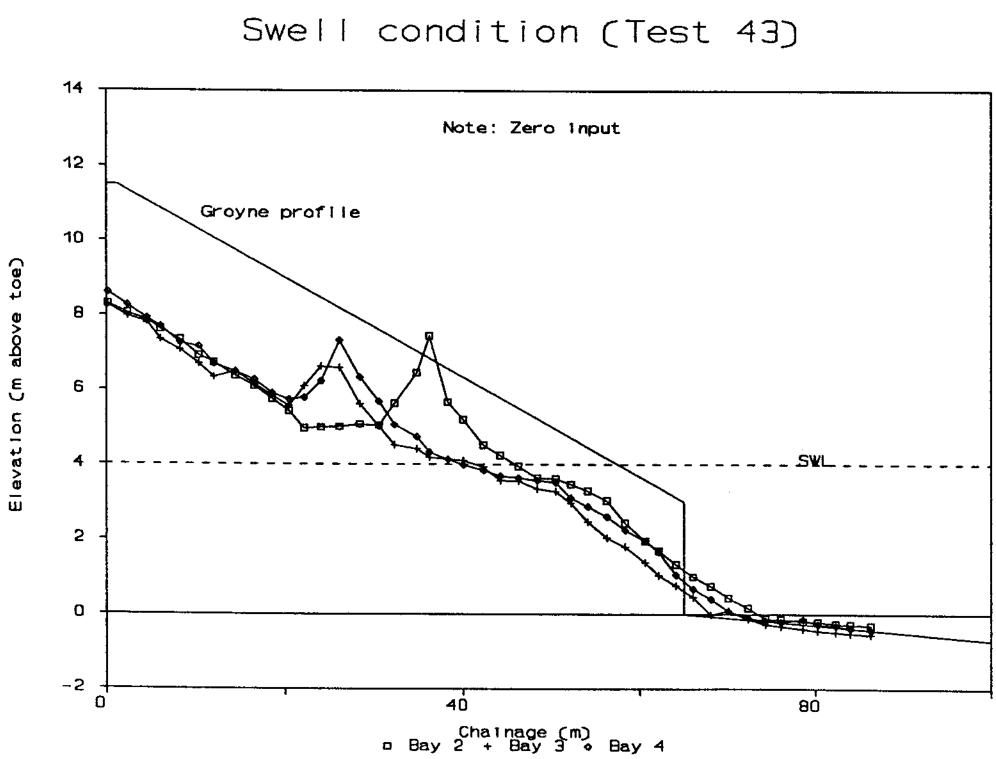
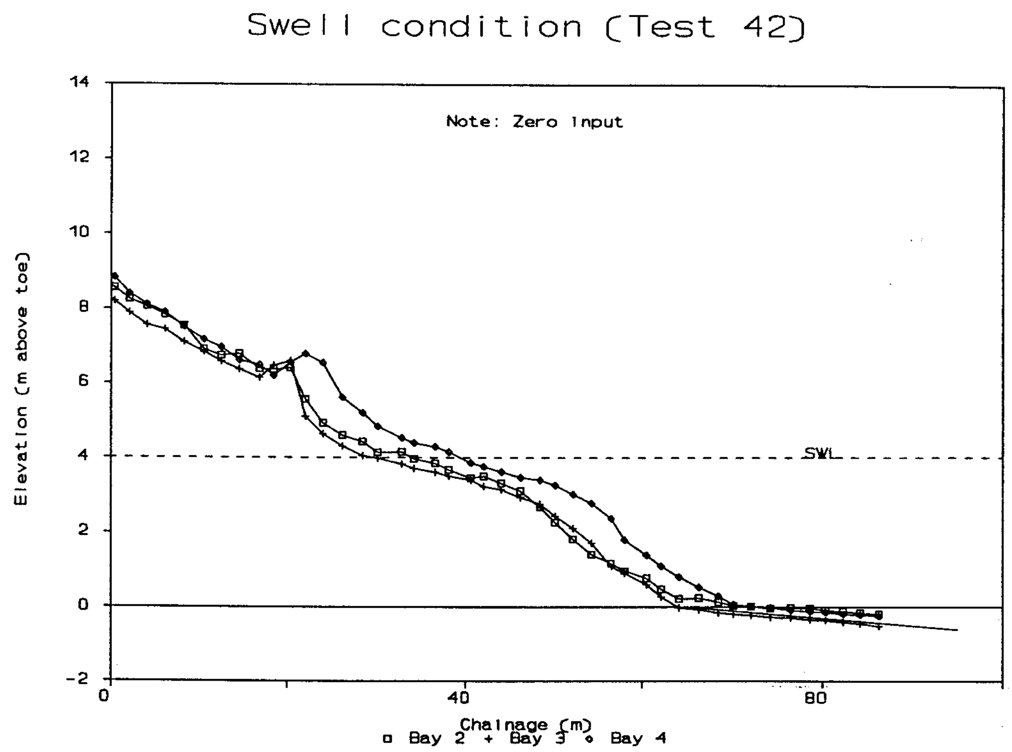


Figure A49 Updrift beach profiles: Short rock and high timber, SWL = 4m, spacing = 1.5:1, 30° wave direction



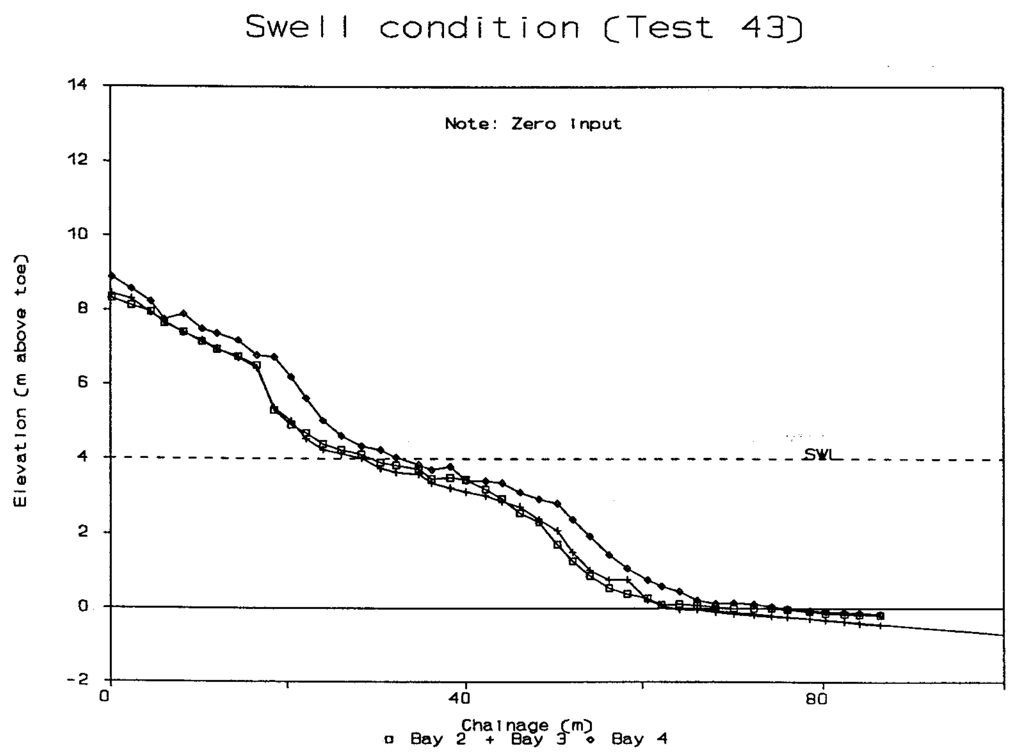
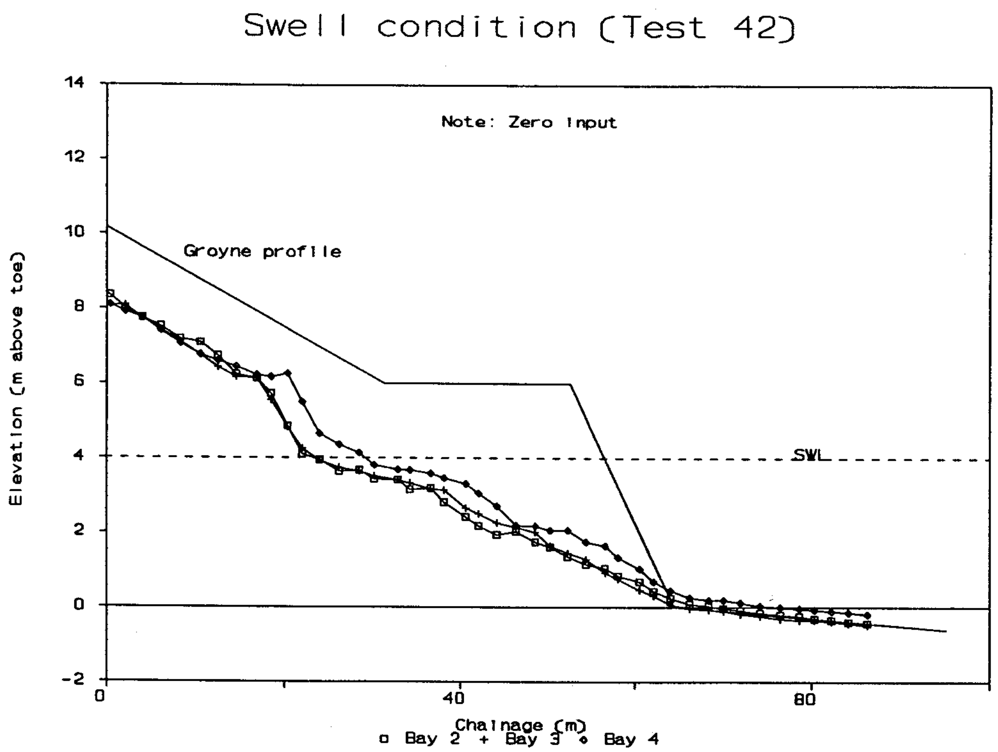


Figure A50 Mid-bay beach profiles: Short rock and high timber, SWL = 4m, spacing = 1.5:1, 30° wave direction



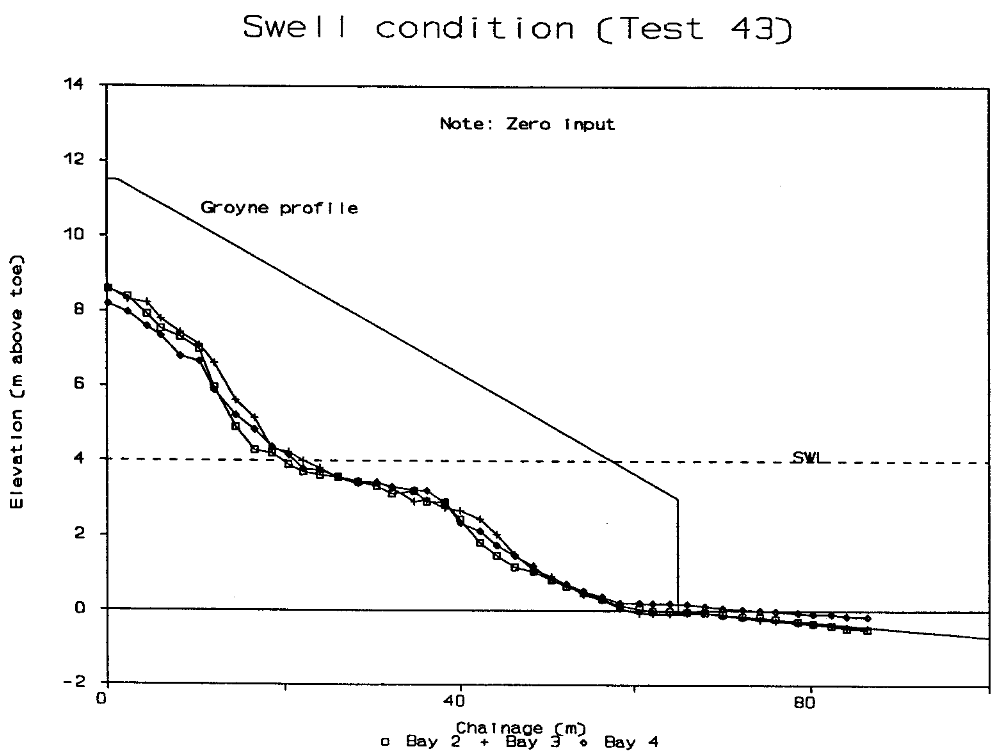


Figure A51 Downdrift beach profiles: Short rock and high timber, SWL = 4m, spacing = 1.5:1, 30° wave direction

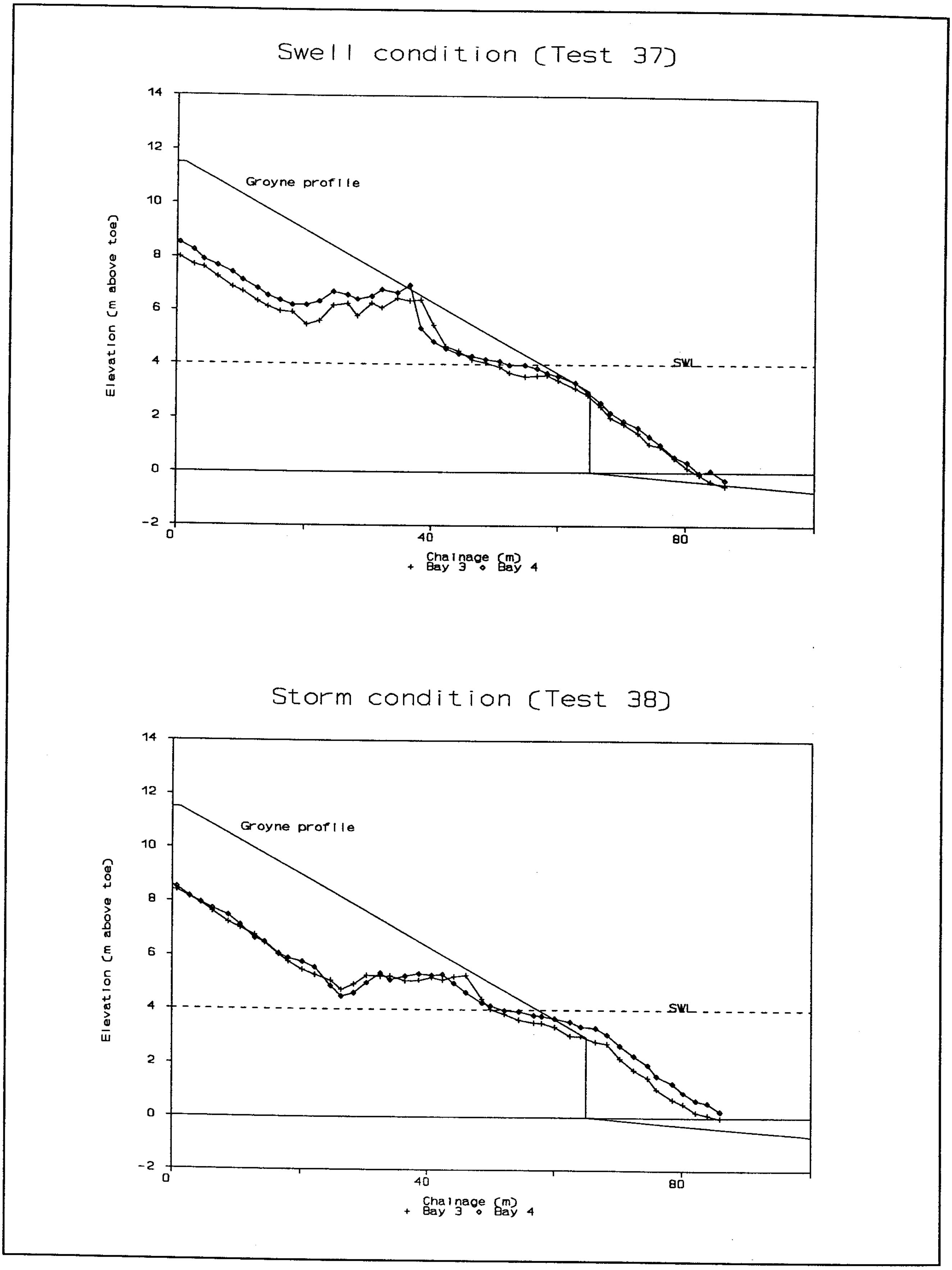
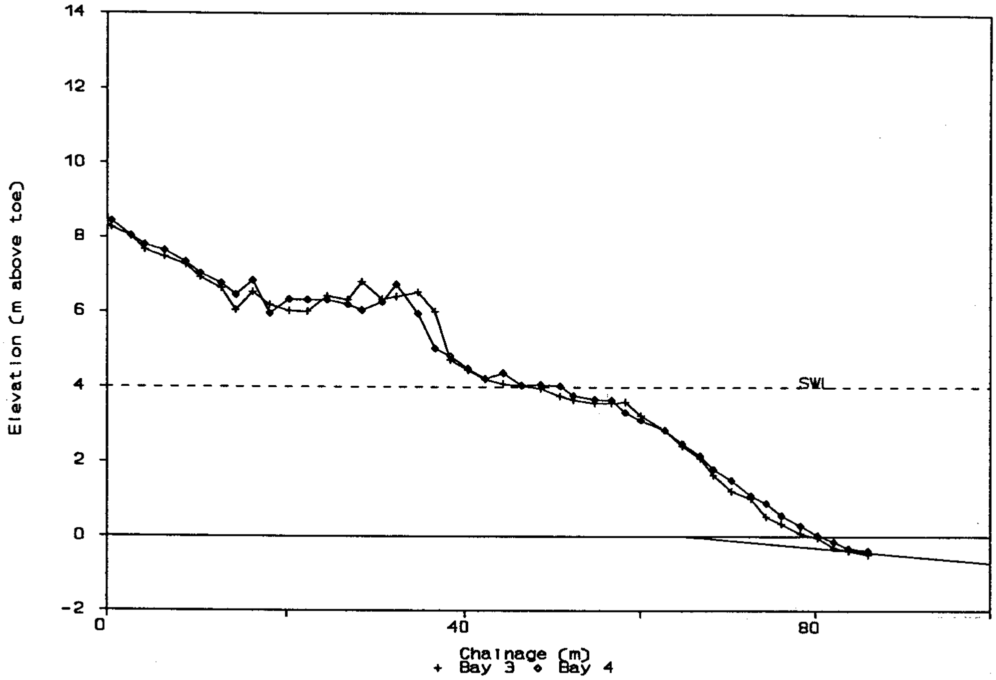


Figure A52 Updrift beach profiles: High timber groynes, SWL = 4m, spacing = 1.5:1, 15° wave direction

Swell condition (Test 37)



Storm condition (Test 38)

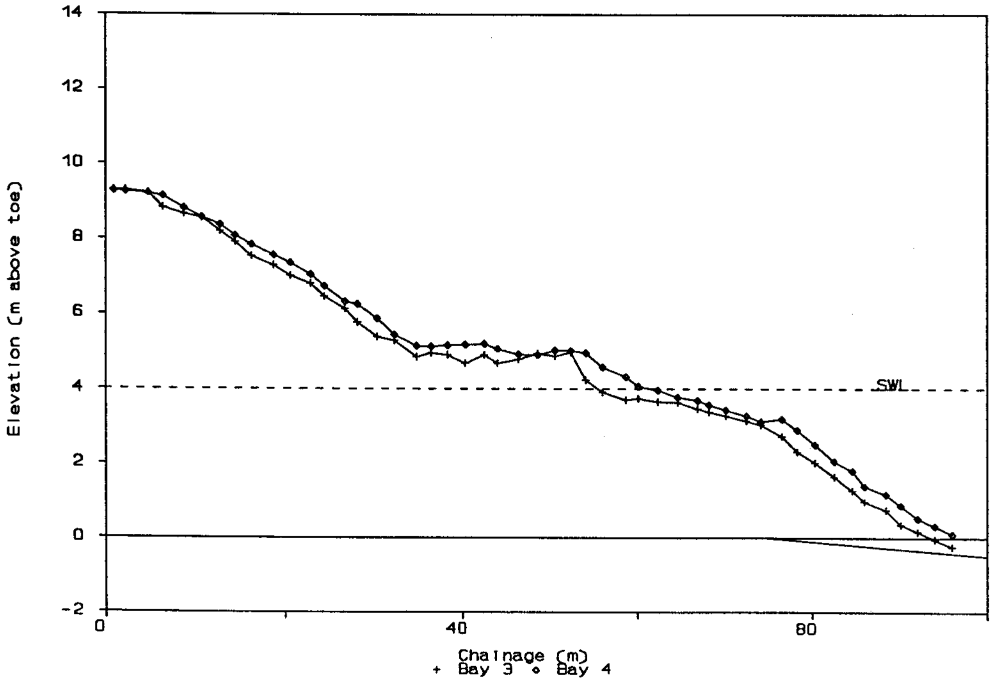
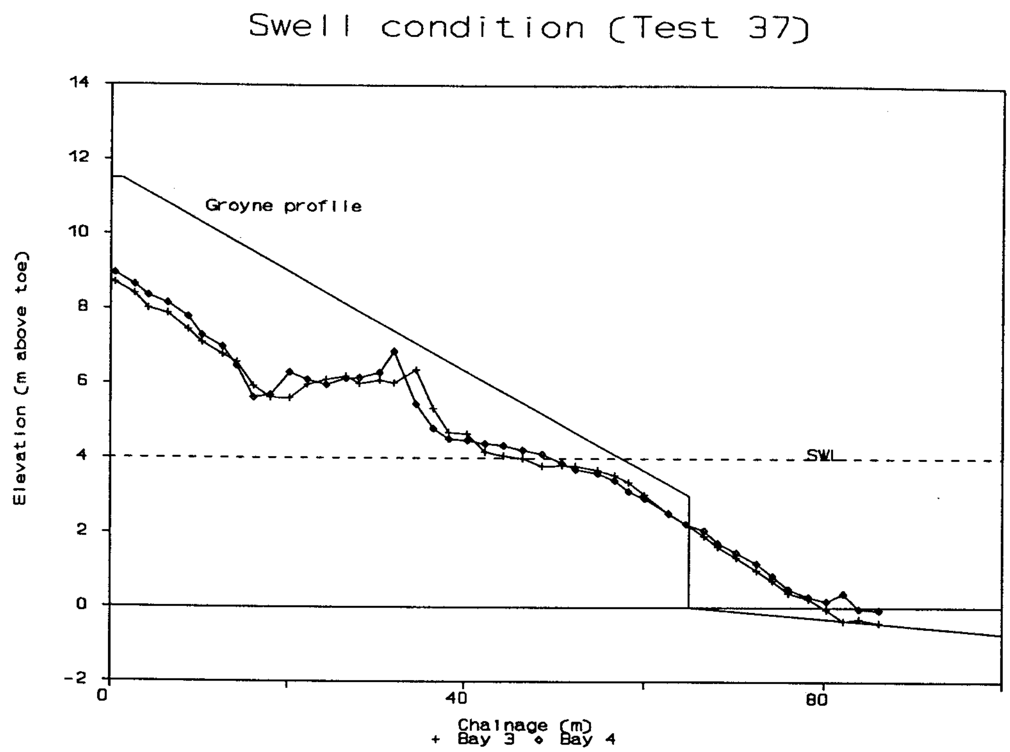


Figure A53 **Mid-bay beach profiles: High timber groynes, SWL = 4m, spacing = 1.5:1, 15° wave direction**



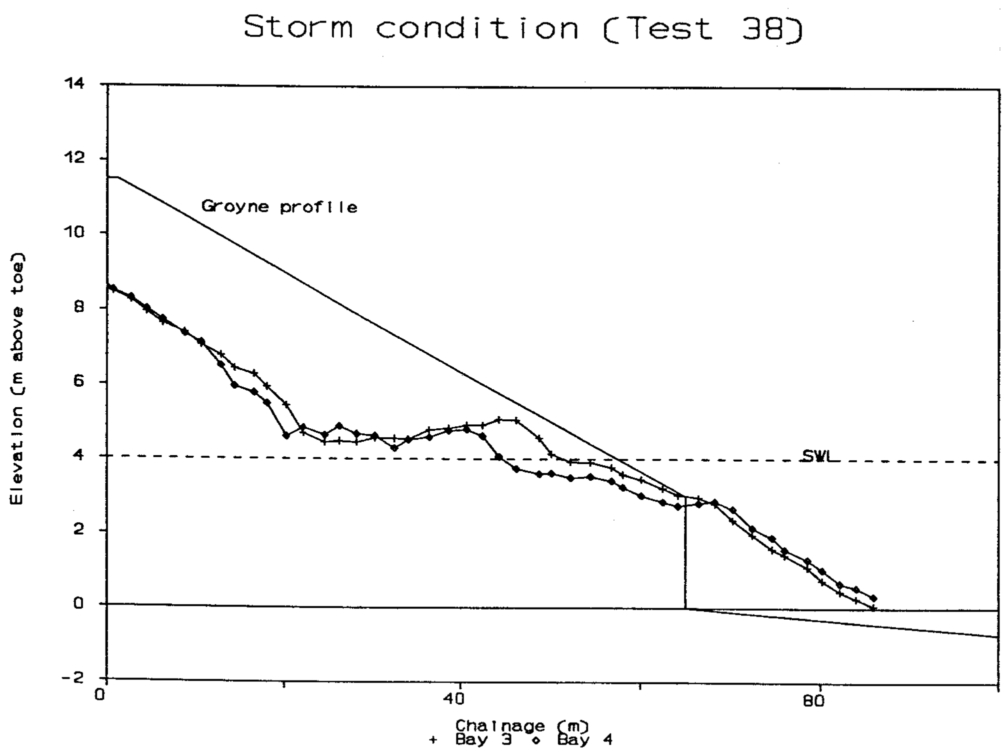
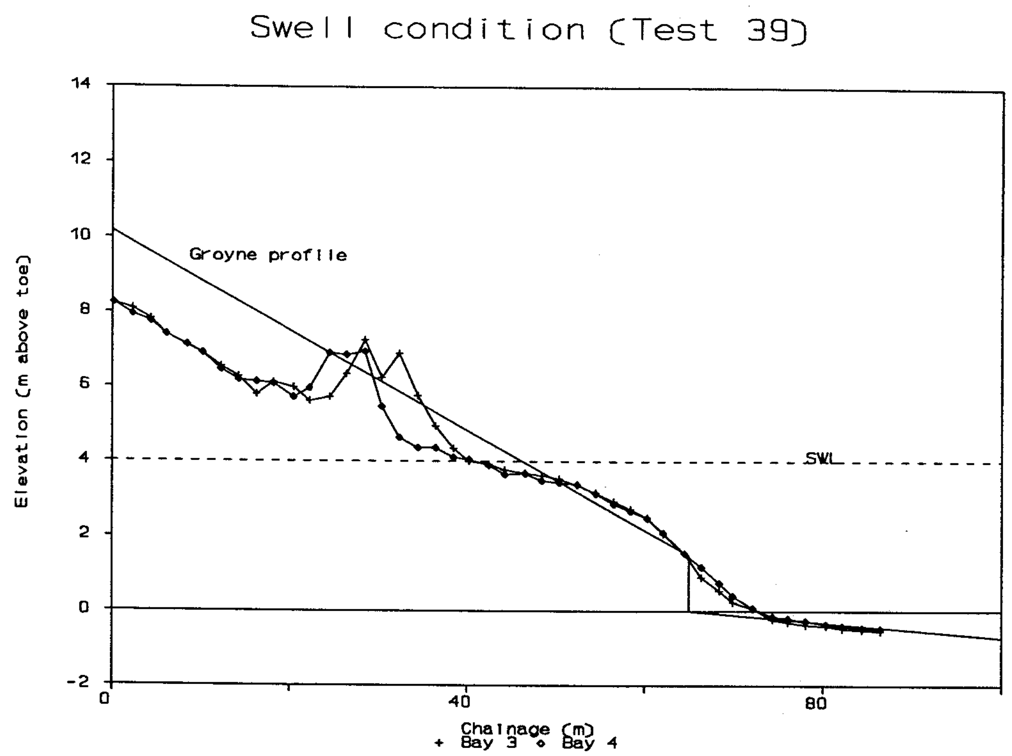


Figure A54 Downdrift beach profiles: High timber groynes, SWL = 4m, spacing = 1.5:1, 15° wave direction



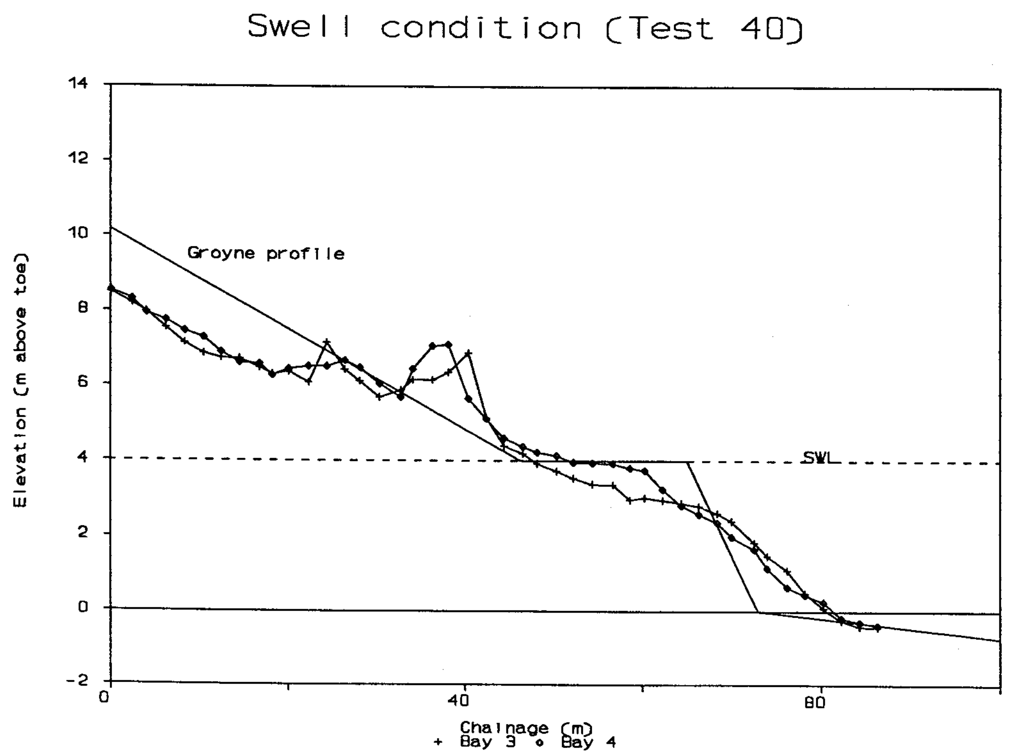
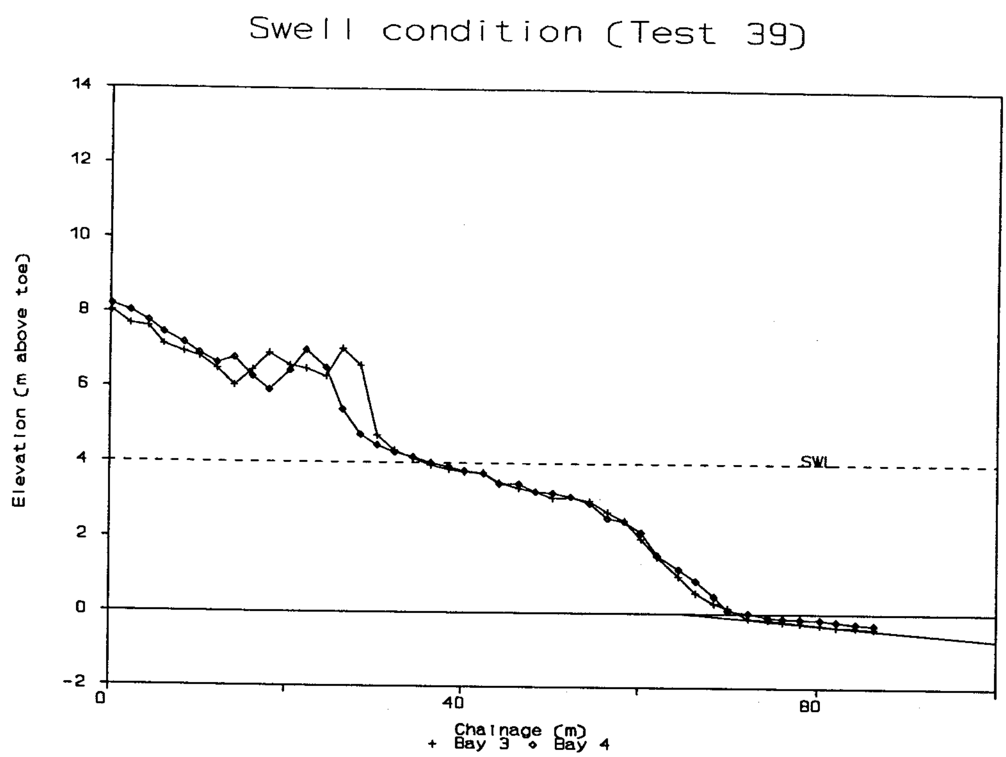


Figure A55 Updrift beach profiles: Low timber and high end rock groynes, SWL = 4m, spacing = 1.5:1, 15° wave direction



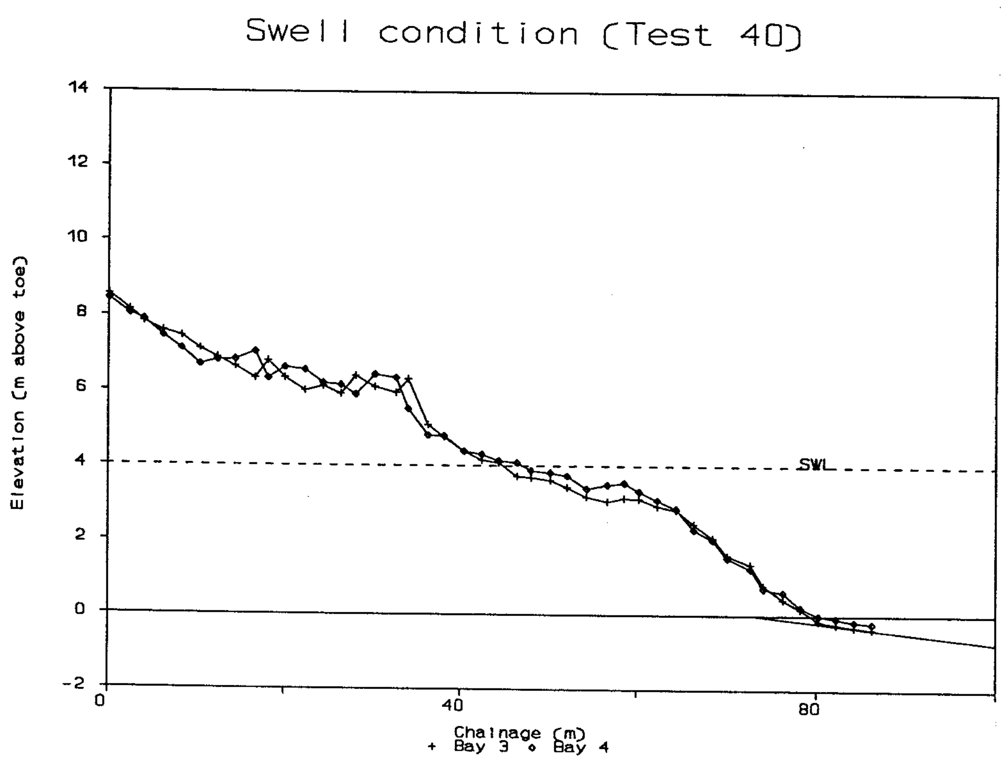
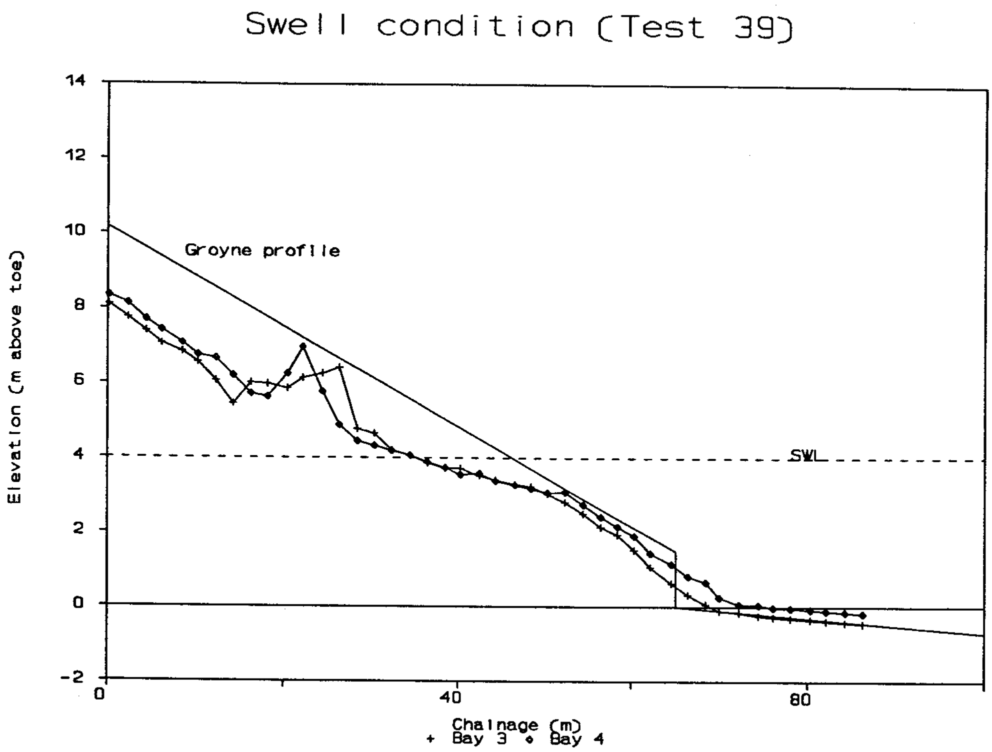


Figure A56 Mid-bay beach profiles: Low timber and high end rock groynes, SWL = 4m, spacing = 1.5:1, 15° wave direction



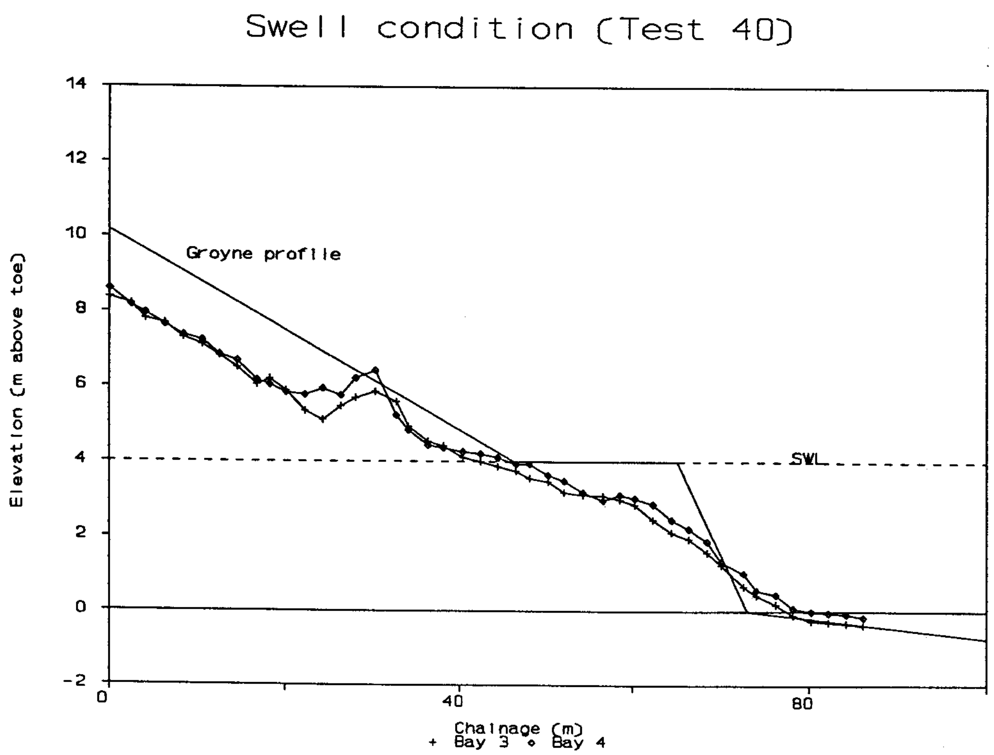


Figure A57 Downdrift beach profiles: Low timber and high end rock groynes, SWL = 4m, spacing = 1.5:1, 15° wave direction

Appendix 3

Efficiency curves - Groynes

Appendix 3 Efficiency curves - Groynes

Contents

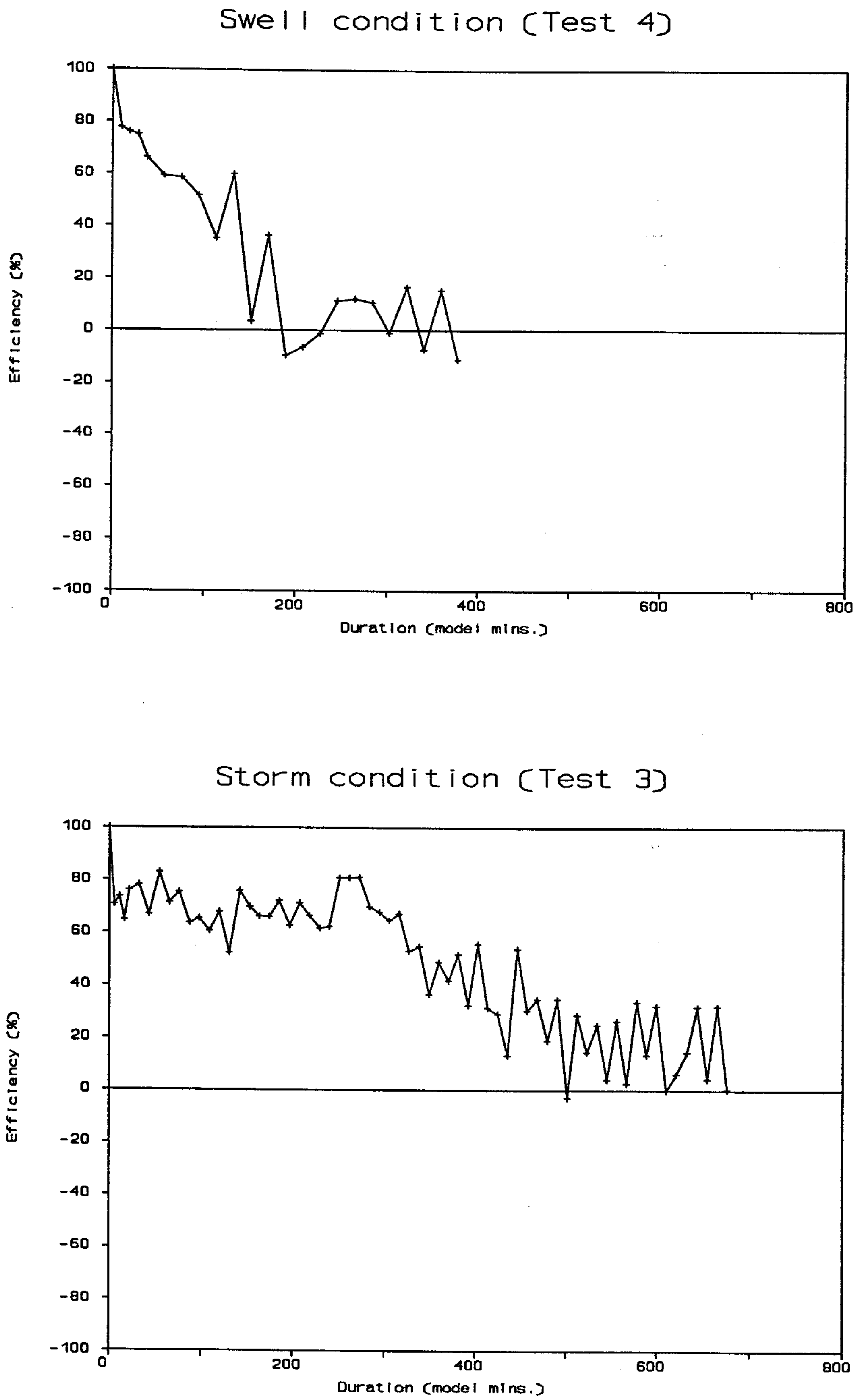
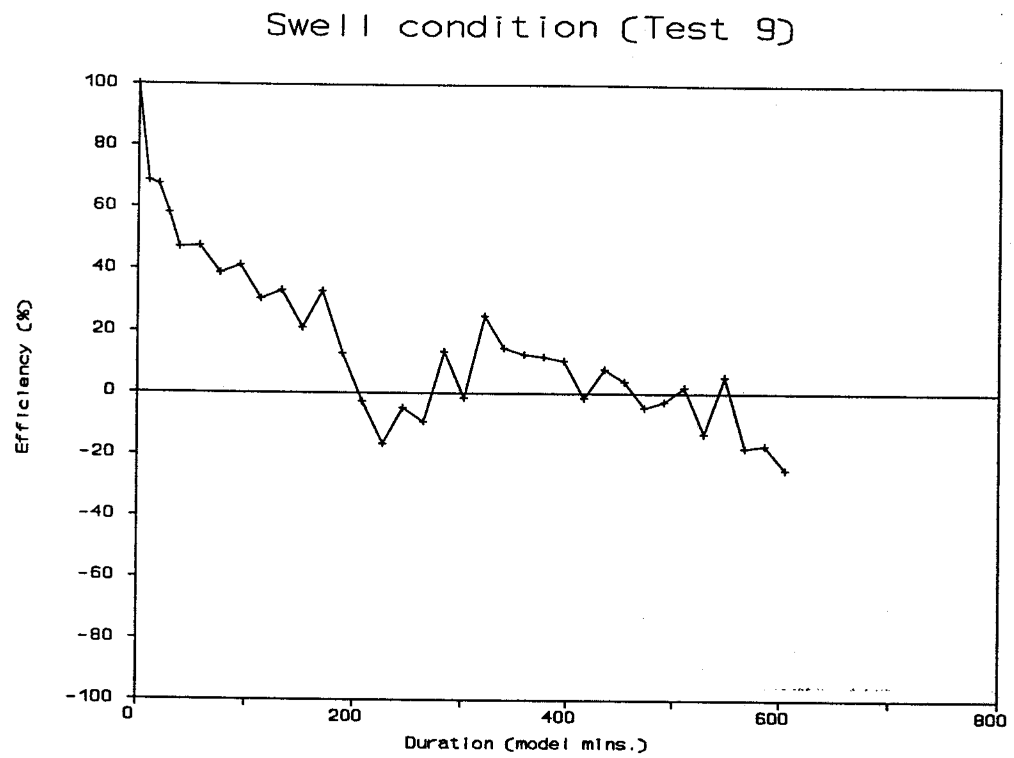


Figure A1 Groyne efficiency: Low timber groynes, SWL = 4m, spacing = 2:1, 30° wave direction



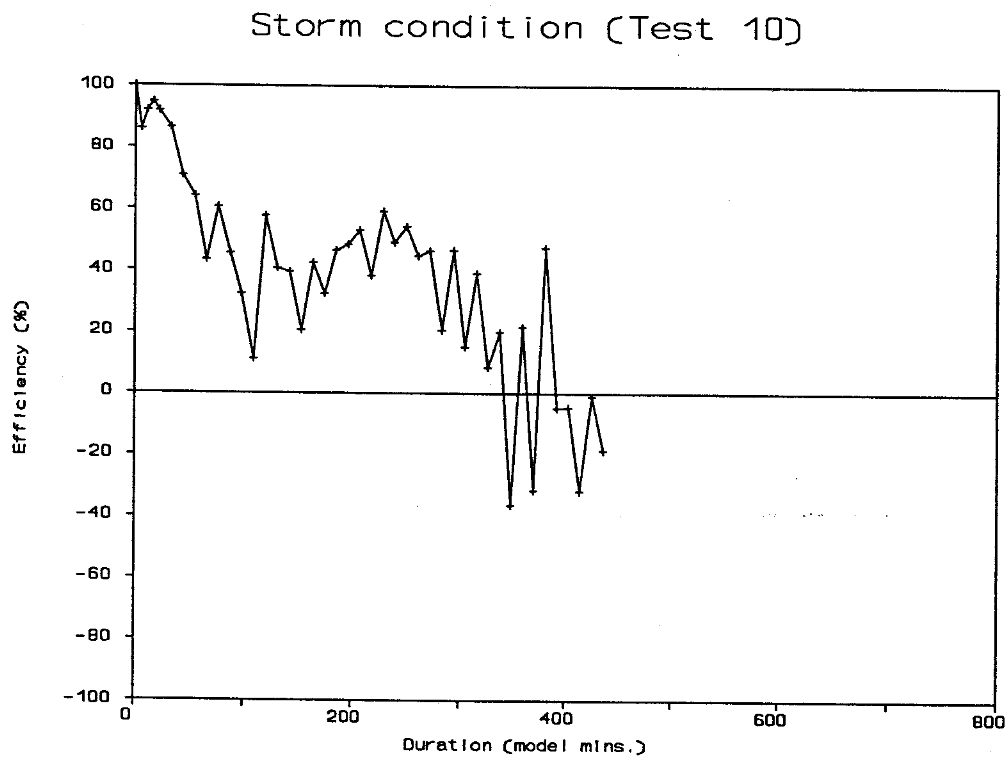
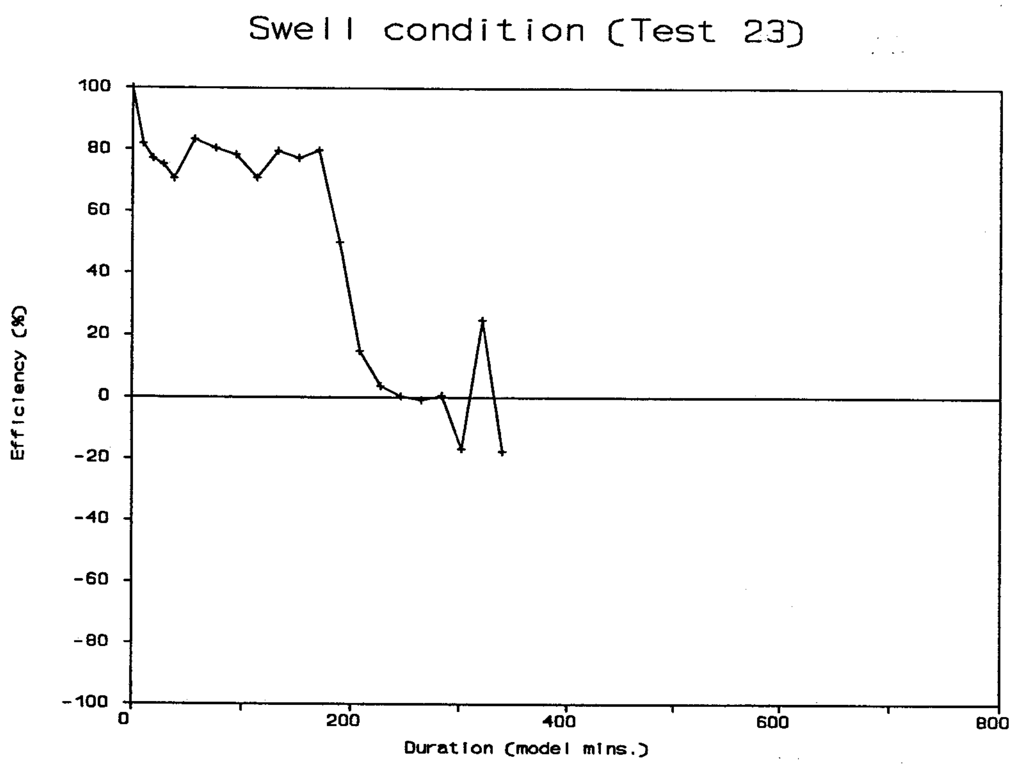


Figure A2 **Groyne efficiency: Low timber groynes, SWL = 3m, spacing = 2:1, 30° wave direction**



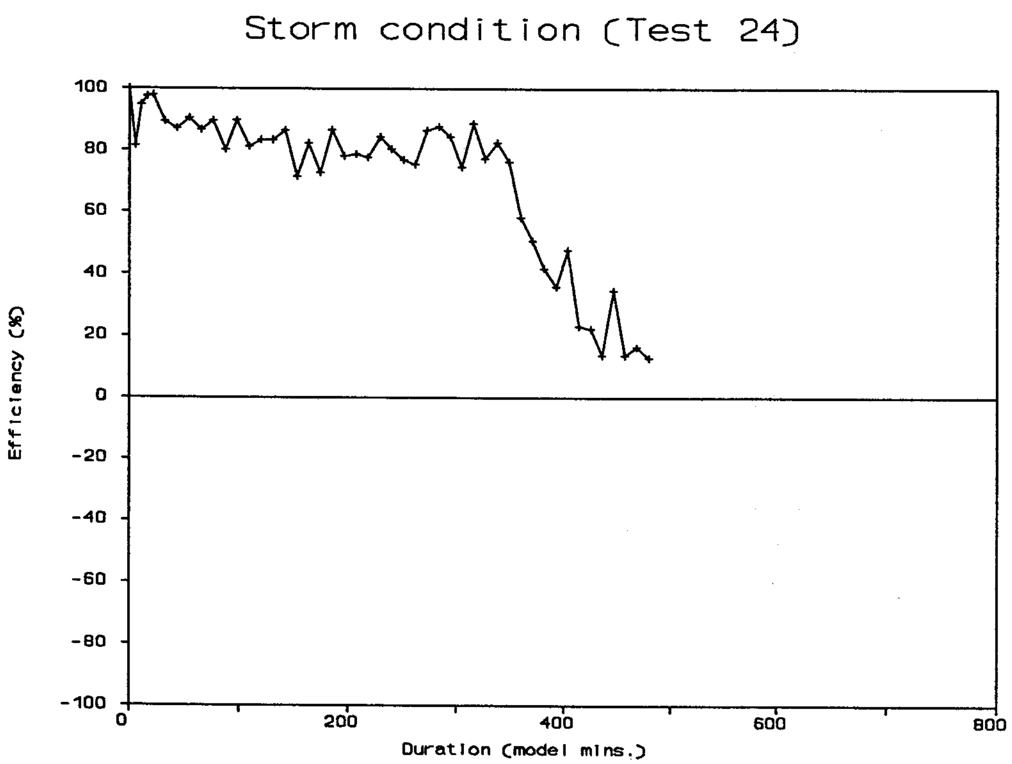


Figure A3 Groyne efficiency: Low timber groynes, SWL = 4m, spacing = 1.5:1, 30° wave direction

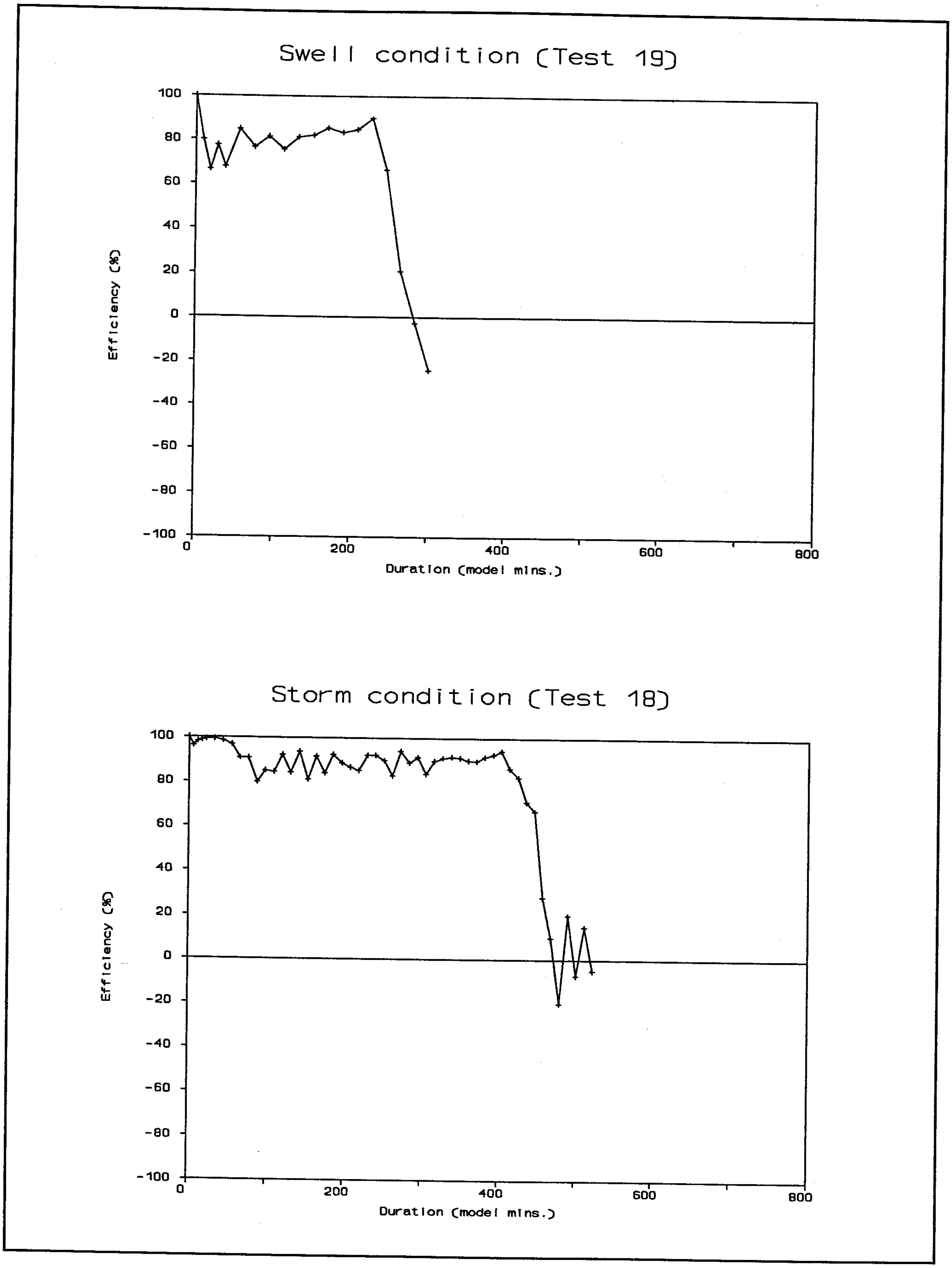
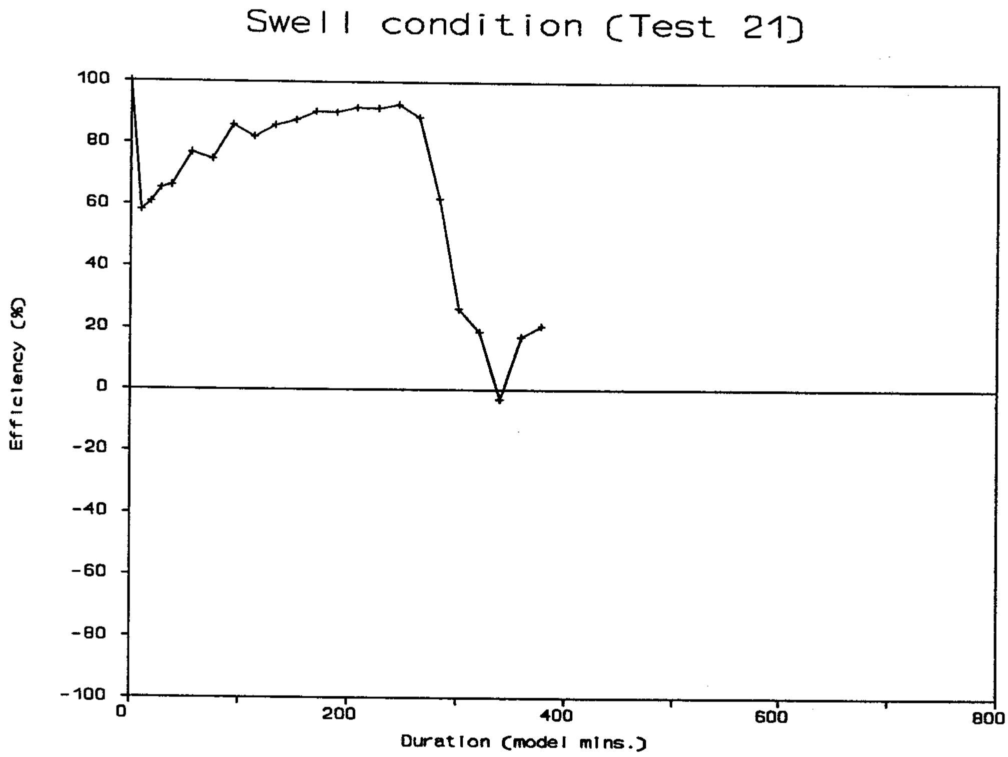


Figure A4 Groyne efficiency: Low timber groynes, SWL = 4m, spacing = 1:1, 30° wave direction



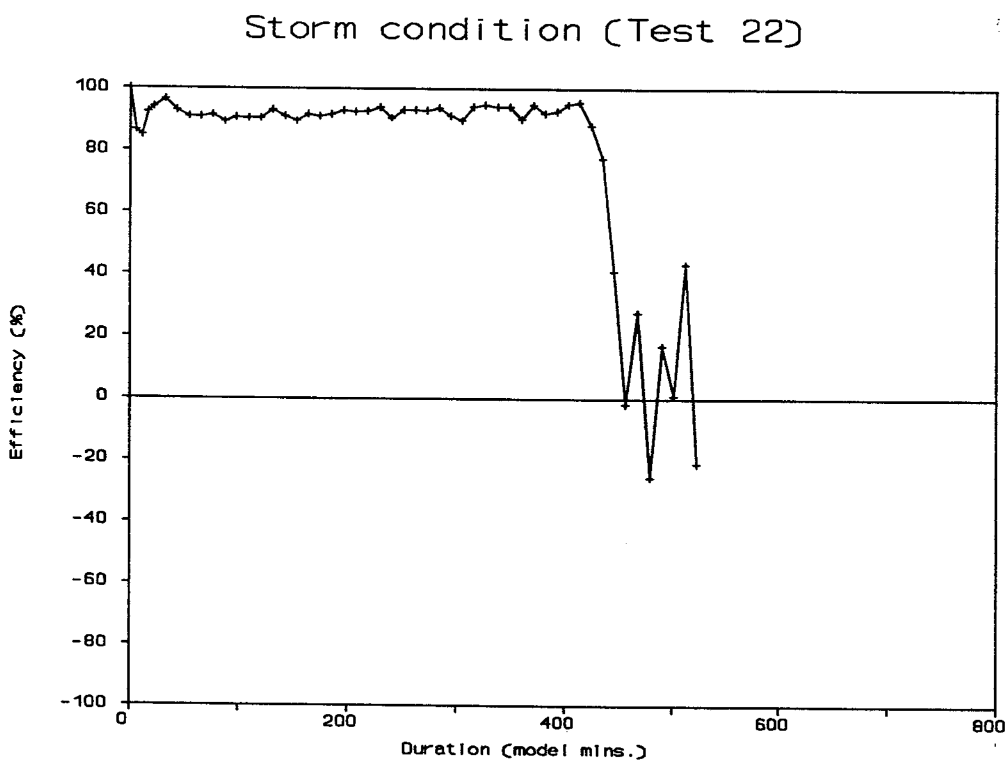
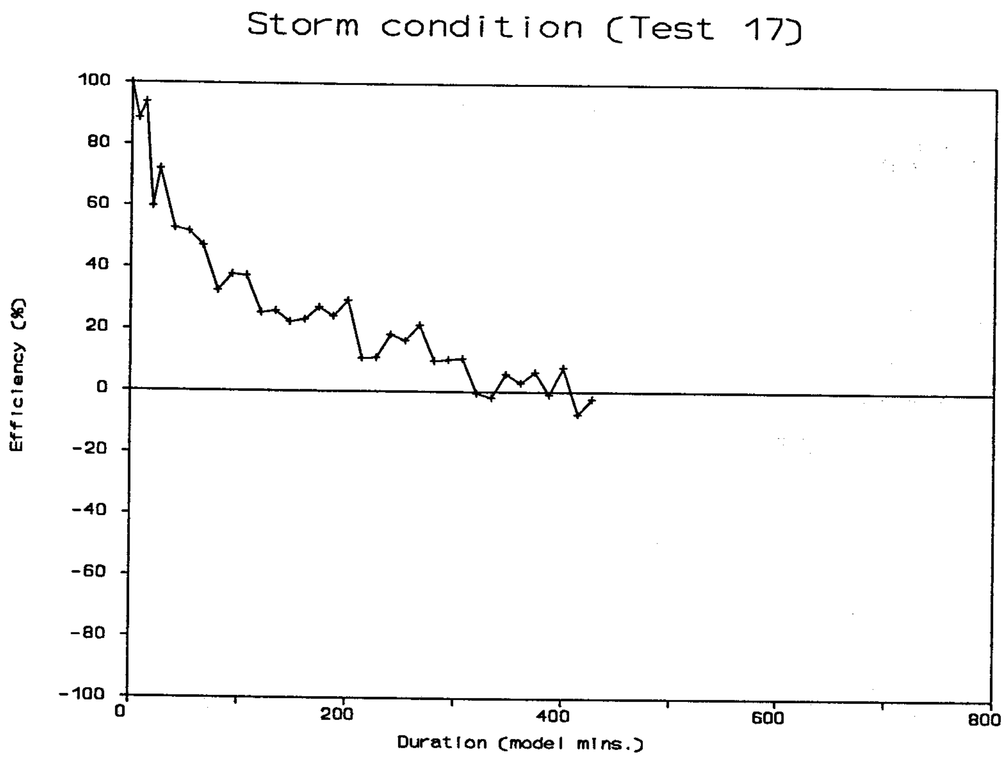


Figure A5 Groyne efficiency: Low timber groynes, SWL = 3m, spacing = 1:1, 30° wave direction



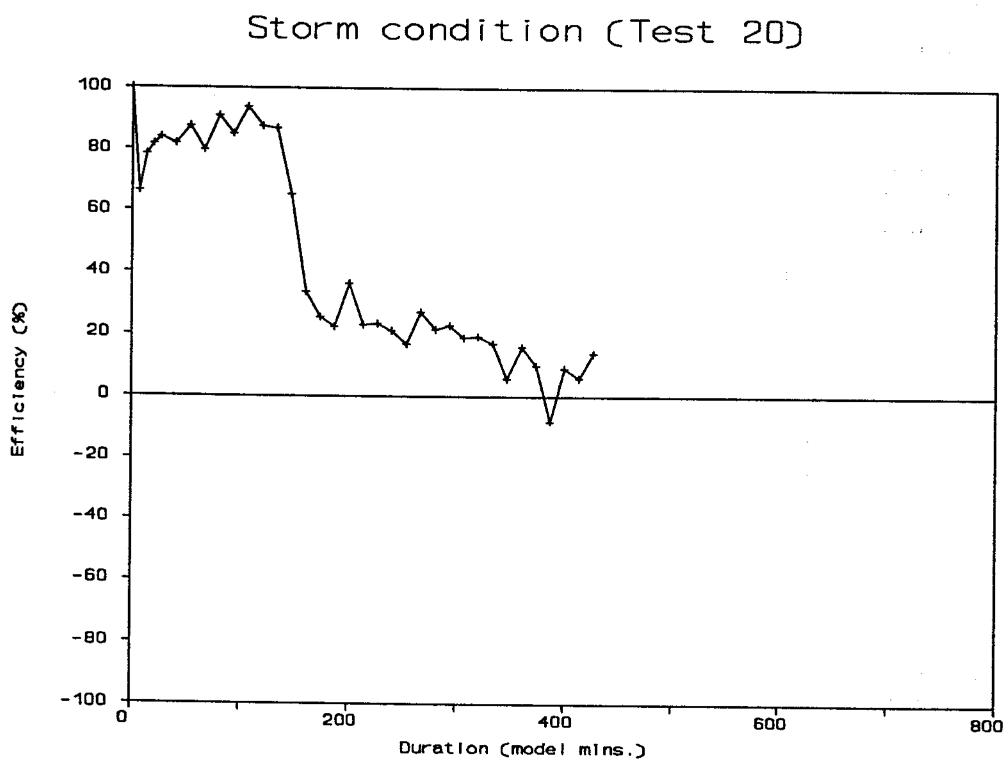
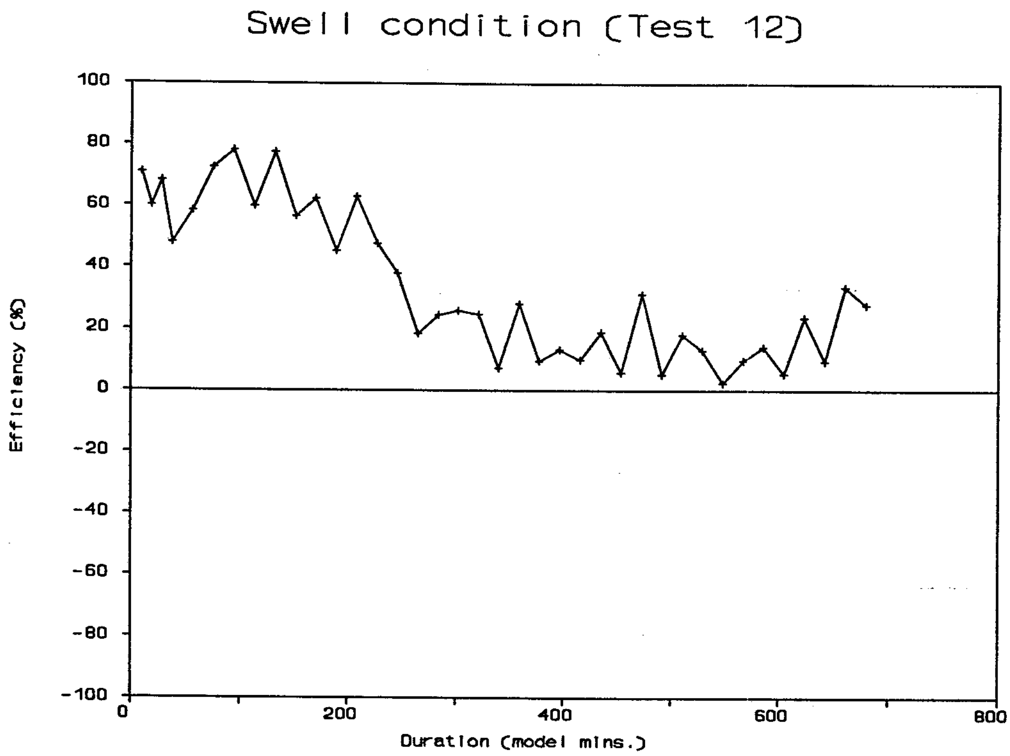


Figure A6 **Groyne efficiency: Low timber groynes, SWL = 4m, spacing = 2:1 and 1:1, 30° wave direction, H_s = 3m**



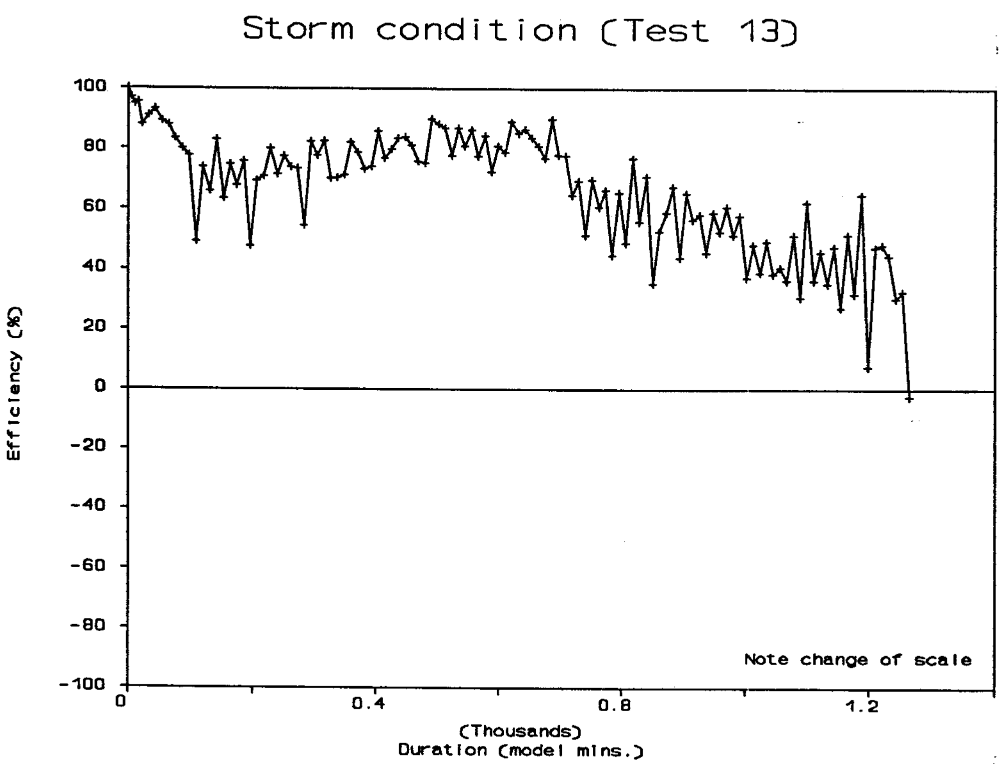
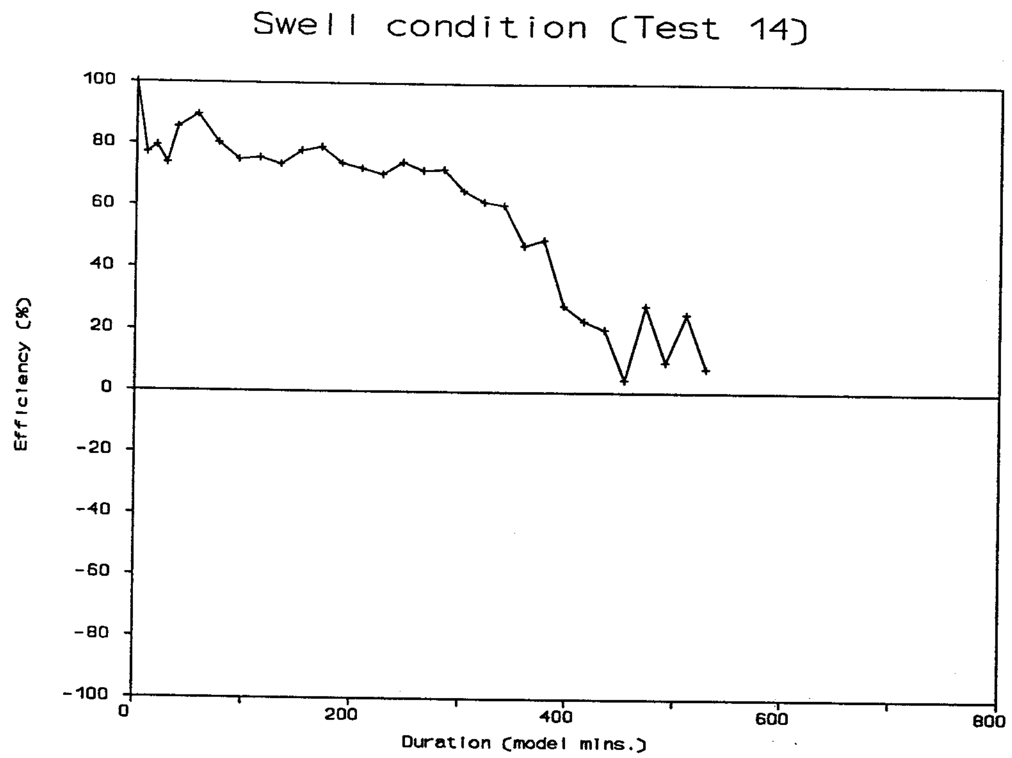


Figure A7 Groyne efficiency: High end timber groynes, SWL = 4m, spacing = 2:1, 30° wave direction



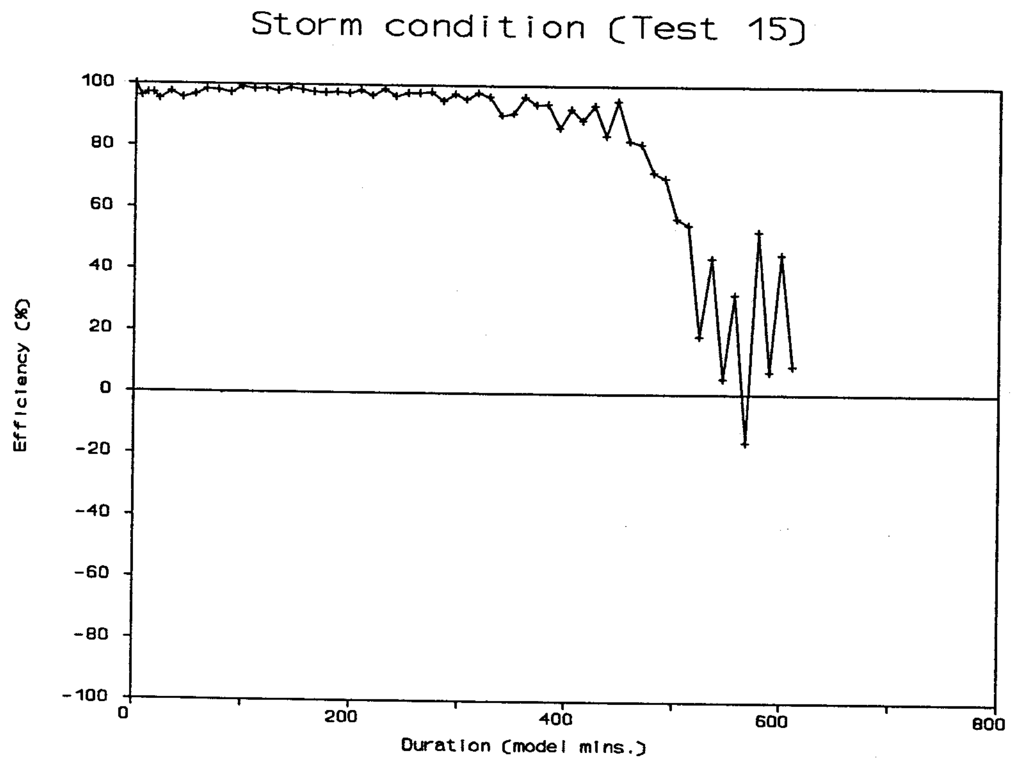


Figure A8 **Groyne efficiency: High end timber groynes, SWL = 3m, spacing = 2:1, 30° wave direction**

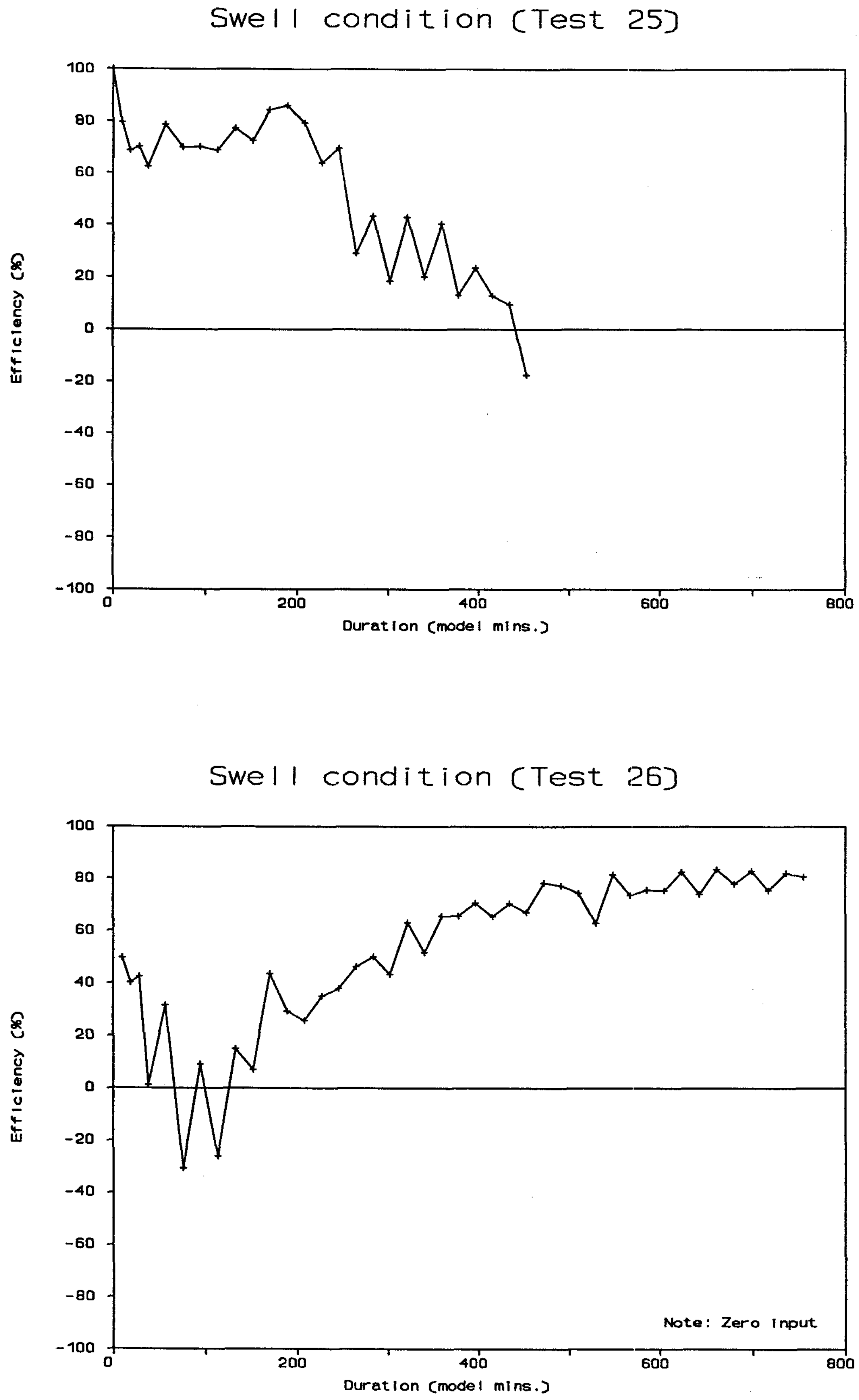
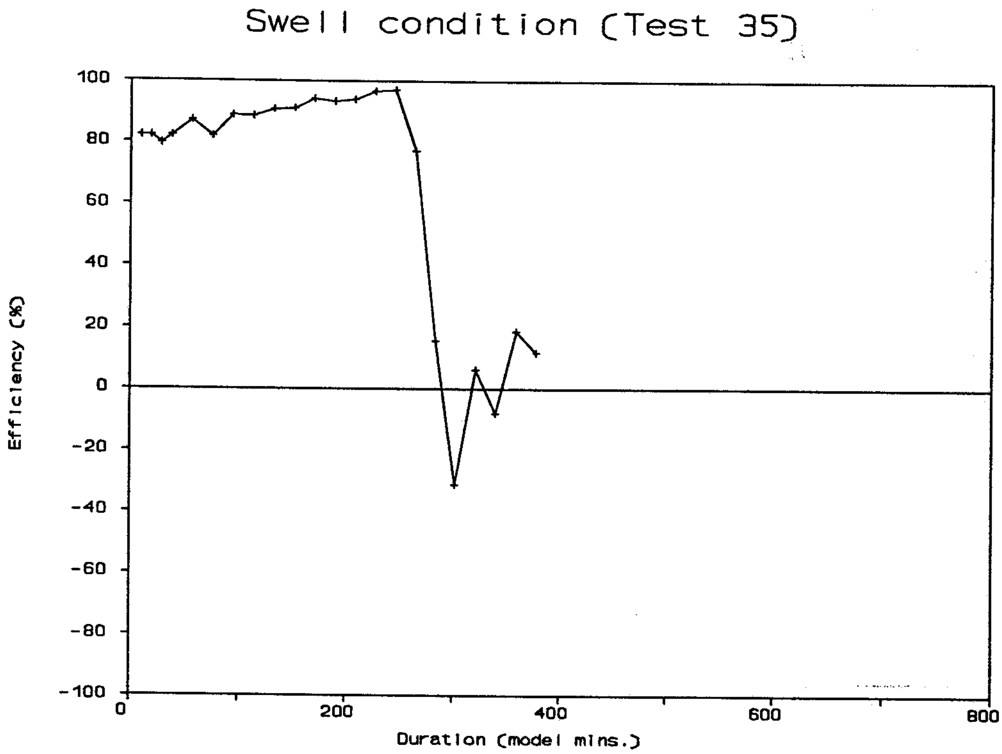


Figure A9 Groyne efficiency: High end timber groynes, SWL = 4m, spacing = 1.5:1, 30° wave direction



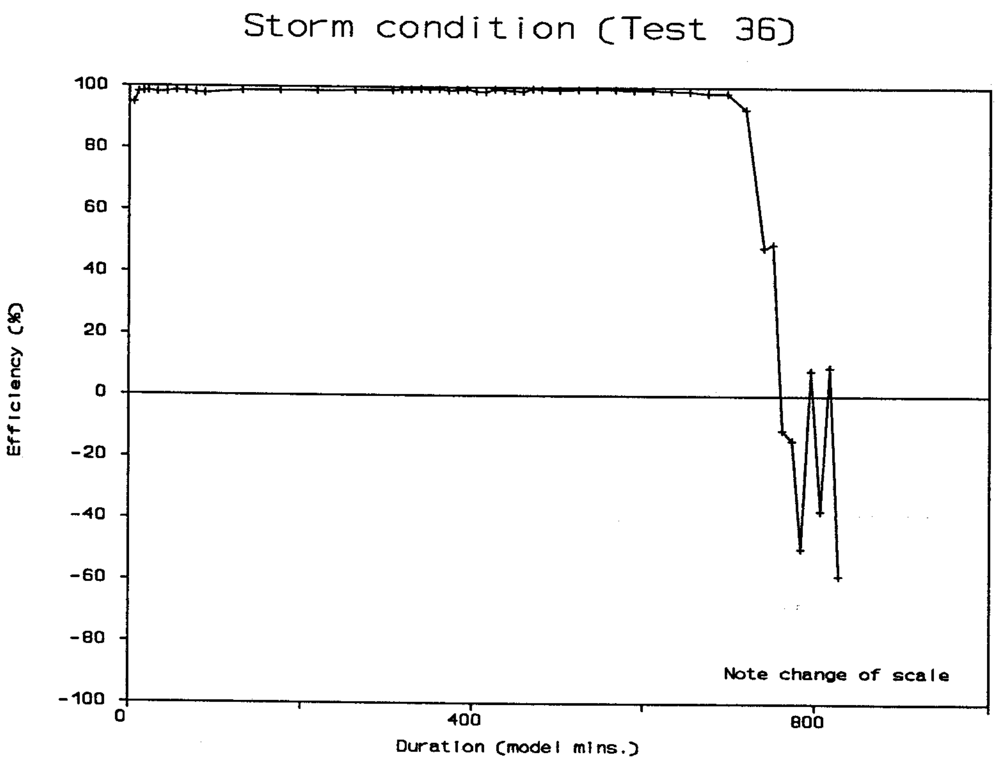
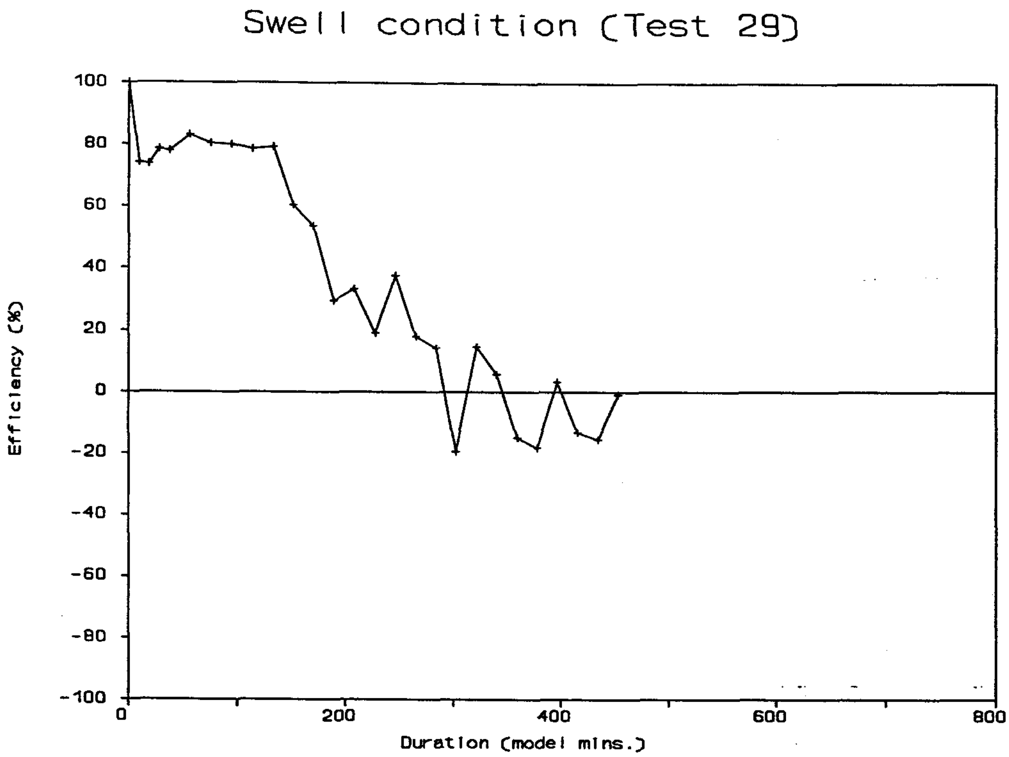


Figure A10 Groyne efficiency: High timber groynes, SWL = 4m, spacing = 1.5:1, 30° wave direction



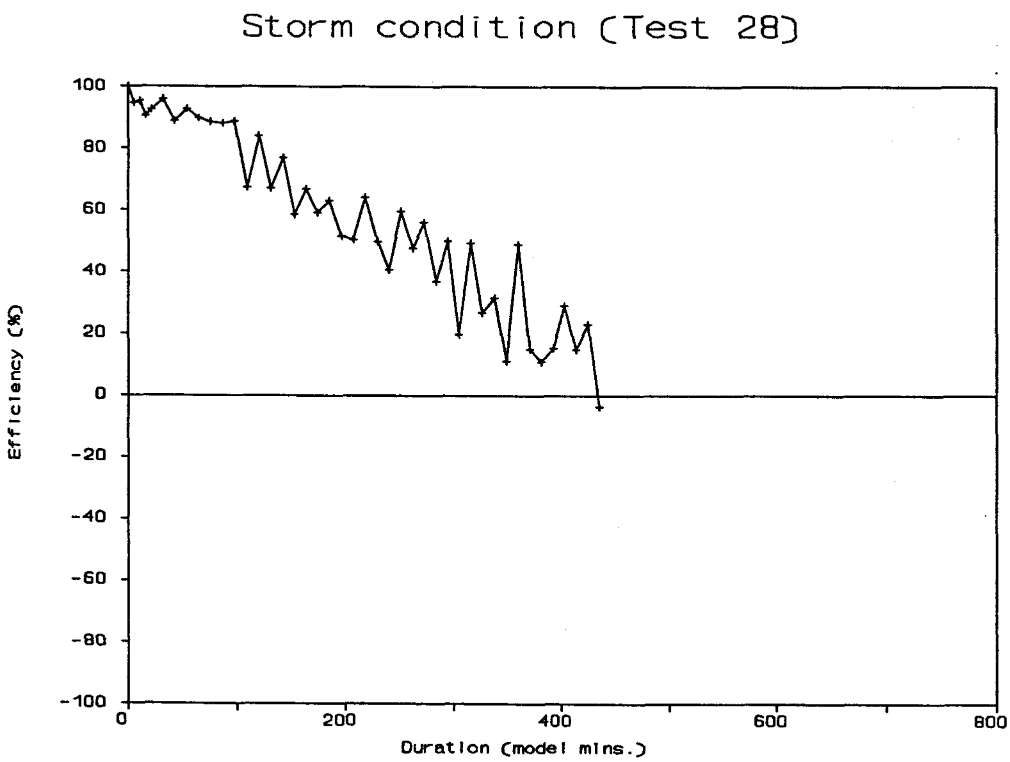
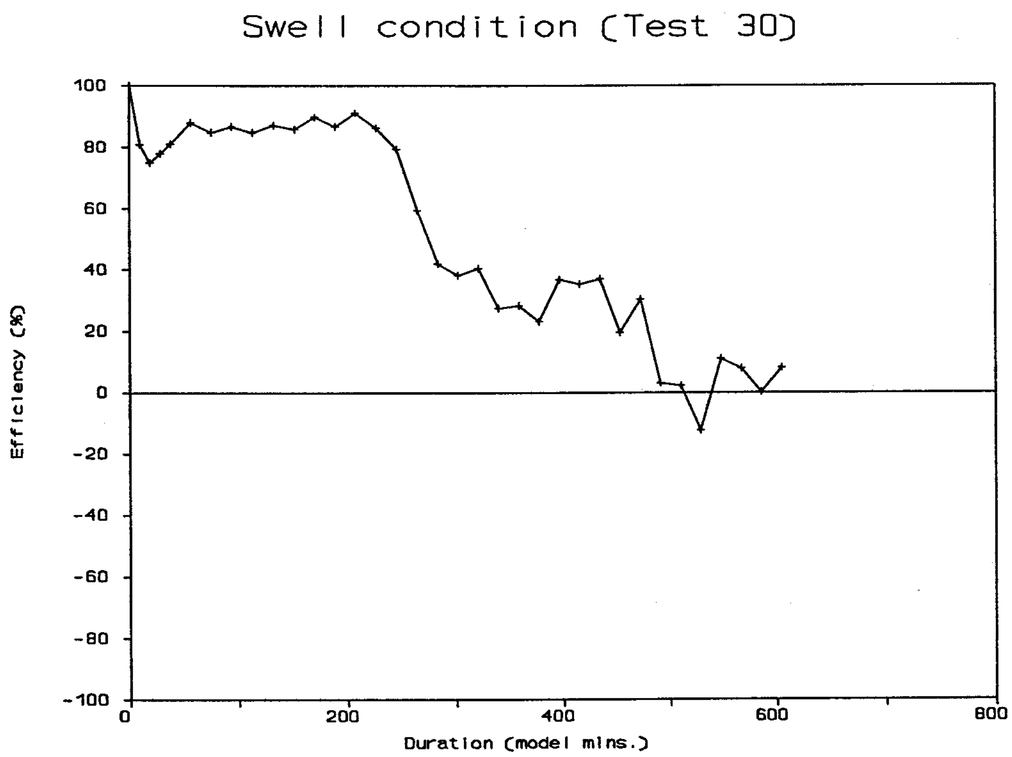


Figure A11 Groyne efficiency: Low rock groynes, SWL = 4m, spacing = 2:1, 30° wave direction



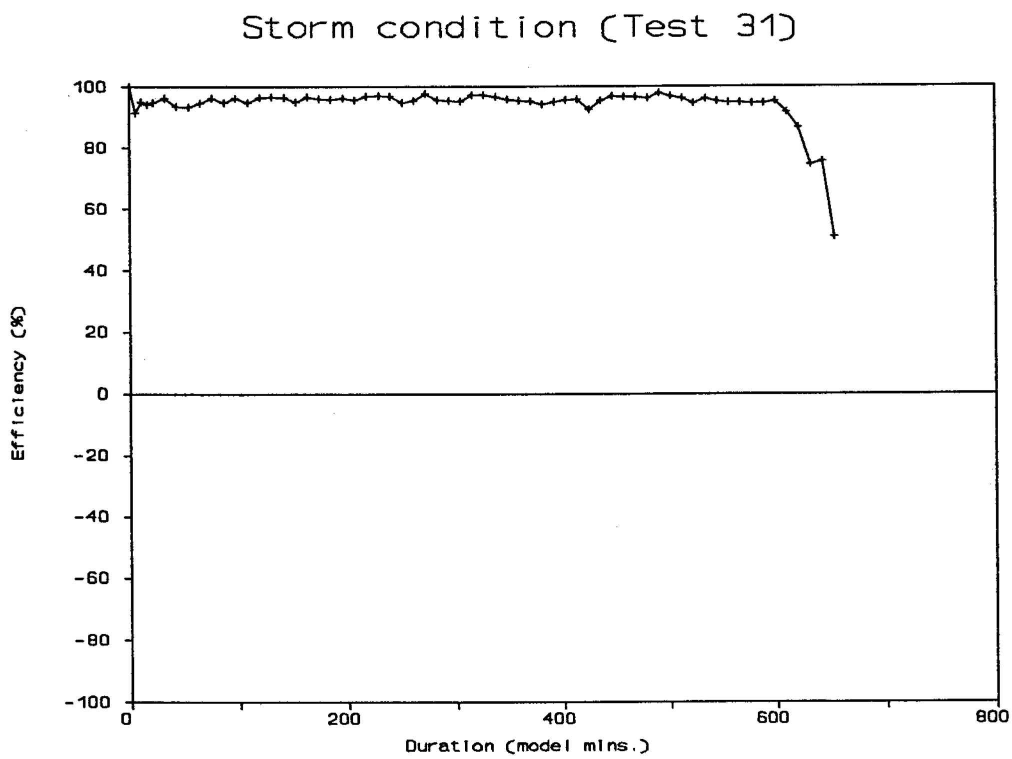
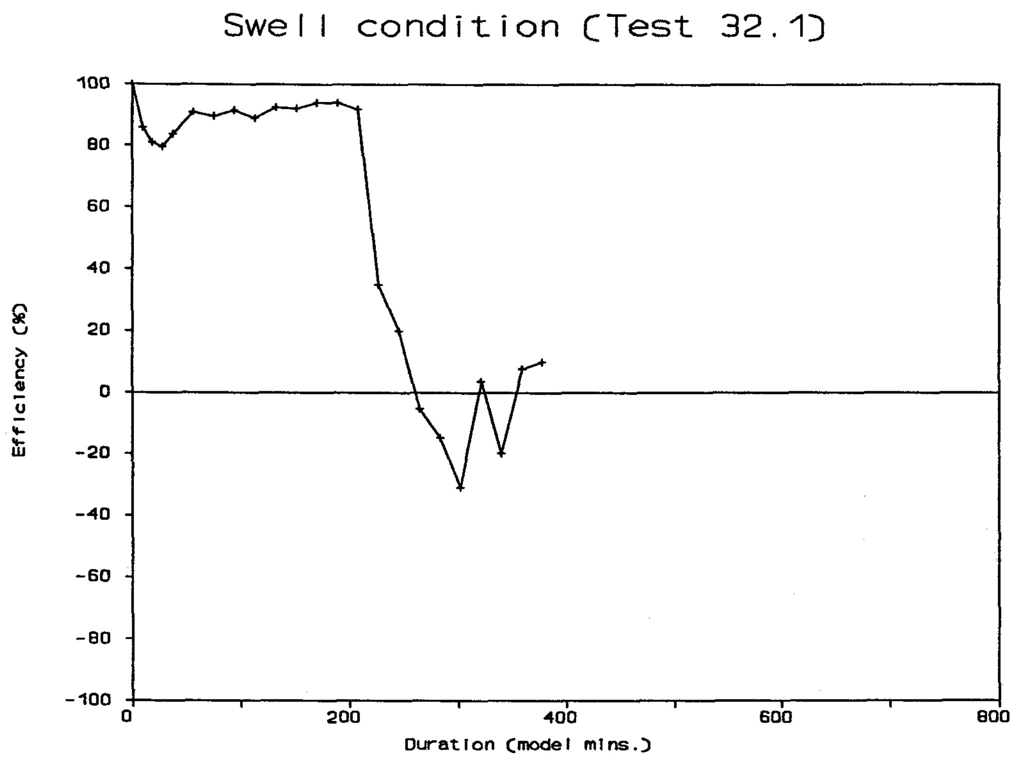


Figure A12 Groyne efficiency: High end rock groynes, SWL = 4m, spacing = 2:1, 30° wave direction



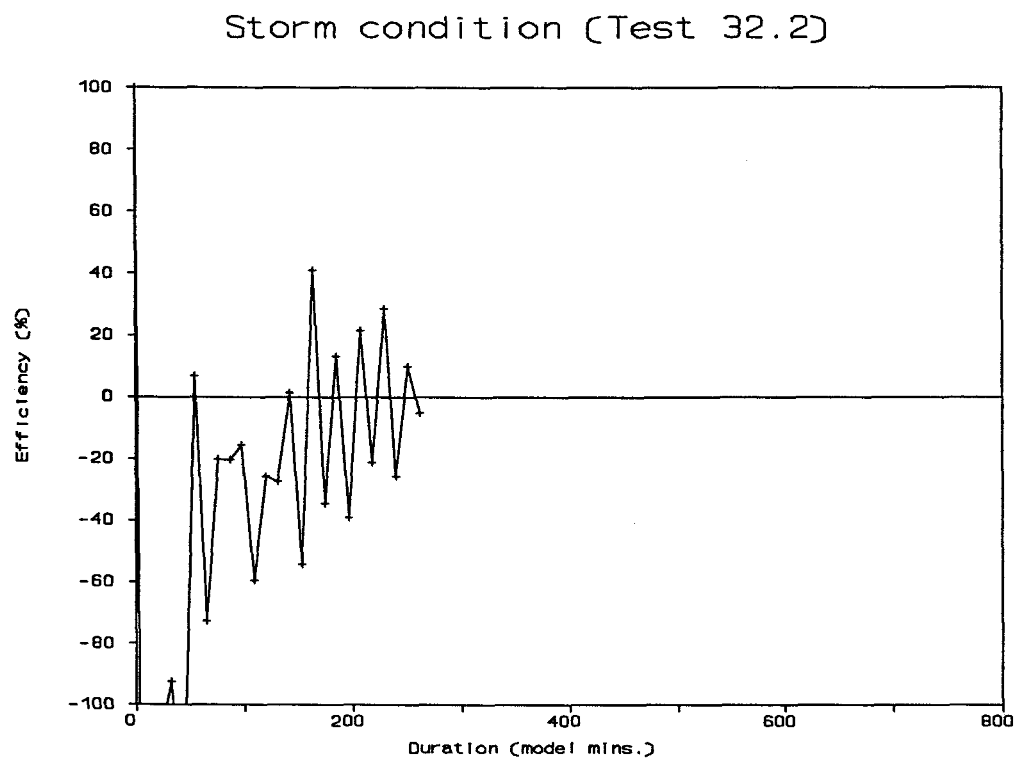


Figure A13 Groyne efficiency: Short rock barriers, SWL = 4m, spacing = 2:1, 30° wave direction

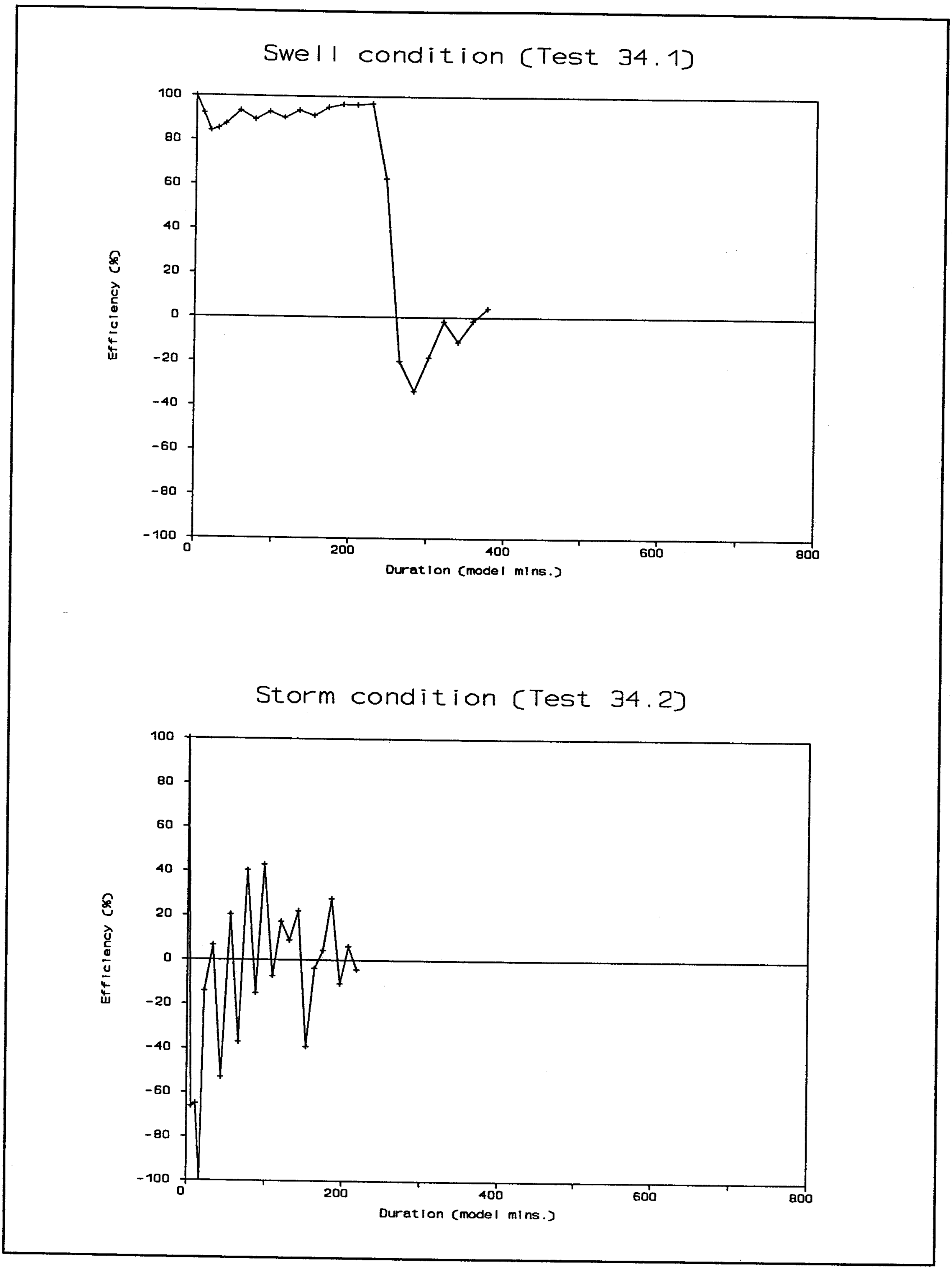
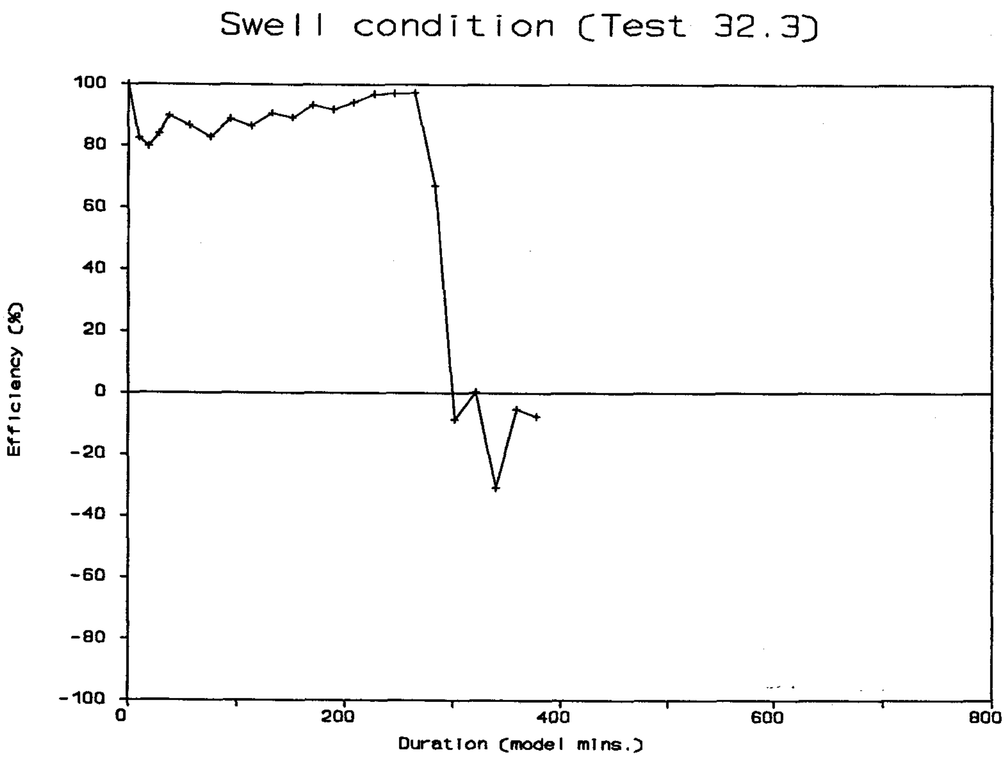


Figure A14 Groyne efficiency: Short rock barriers, SWL = 4m, spacing = 1.5:1, 30° wave direction



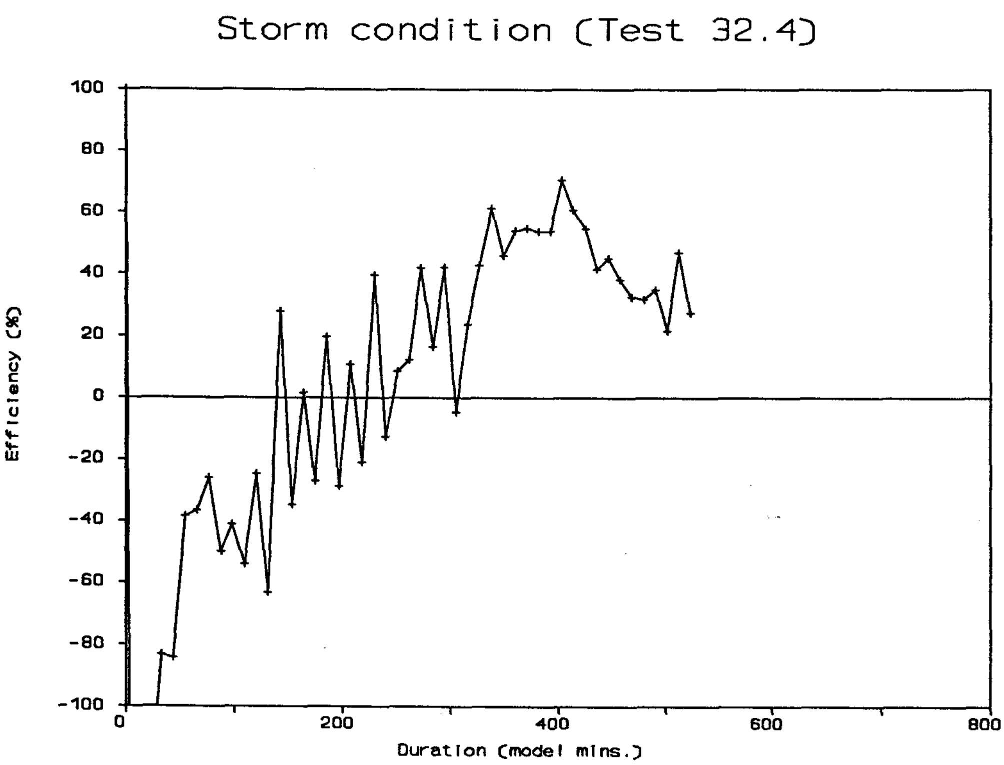
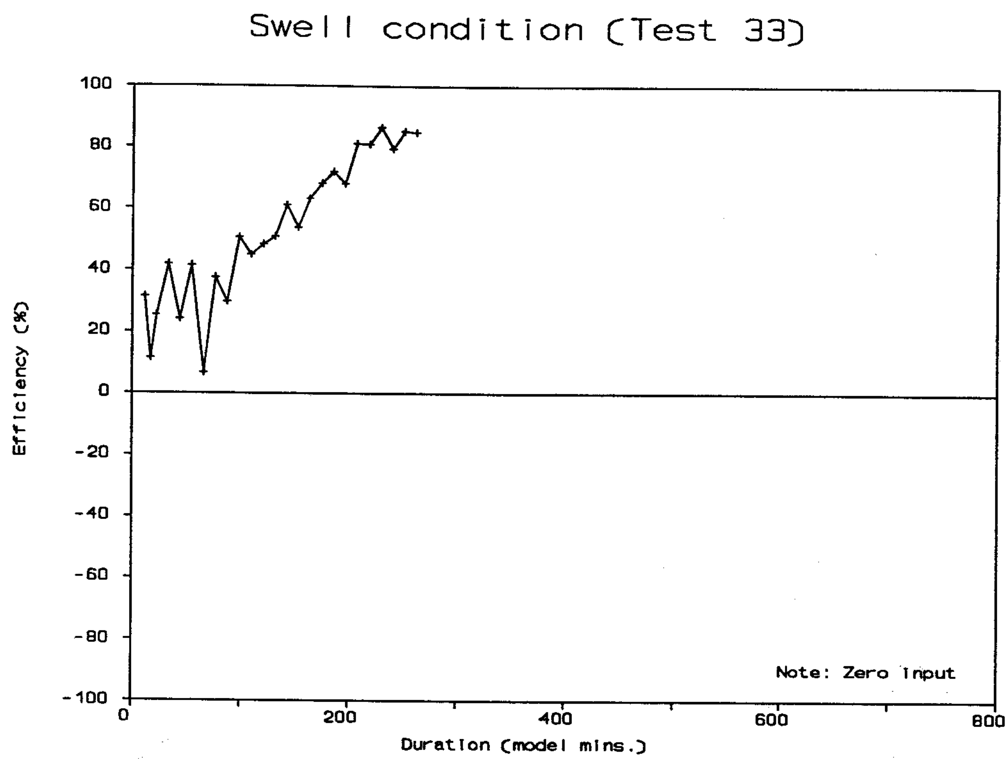


Figure A15 Groyne efficiency: Long rock barriers, SWL = 4m, spacing = 2:1, 30° wave direction



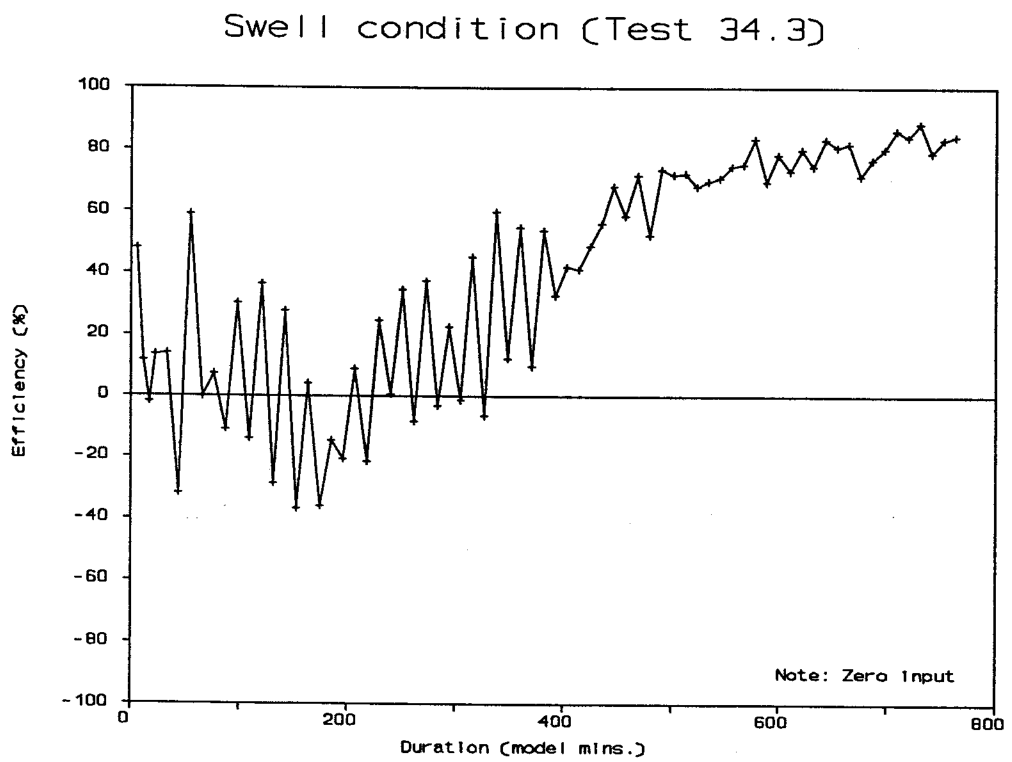
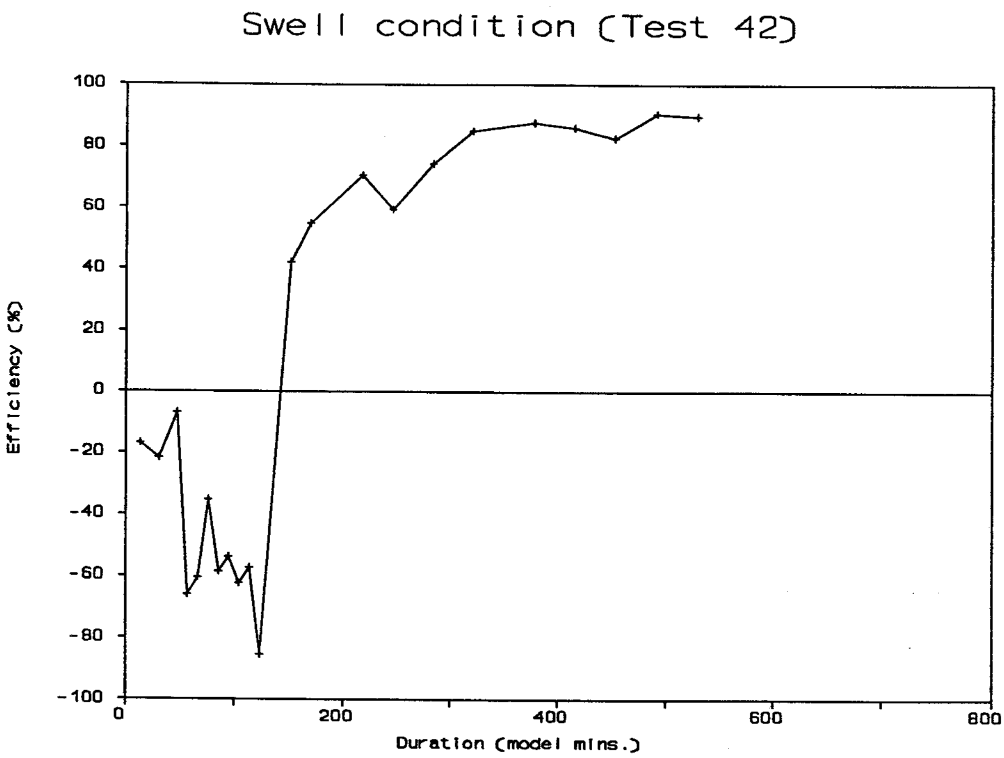


Figure A16 Groyne efficiency: Long and short rock barriers, SWL = 4m, spacing = 1.5:1, 30° wave direction



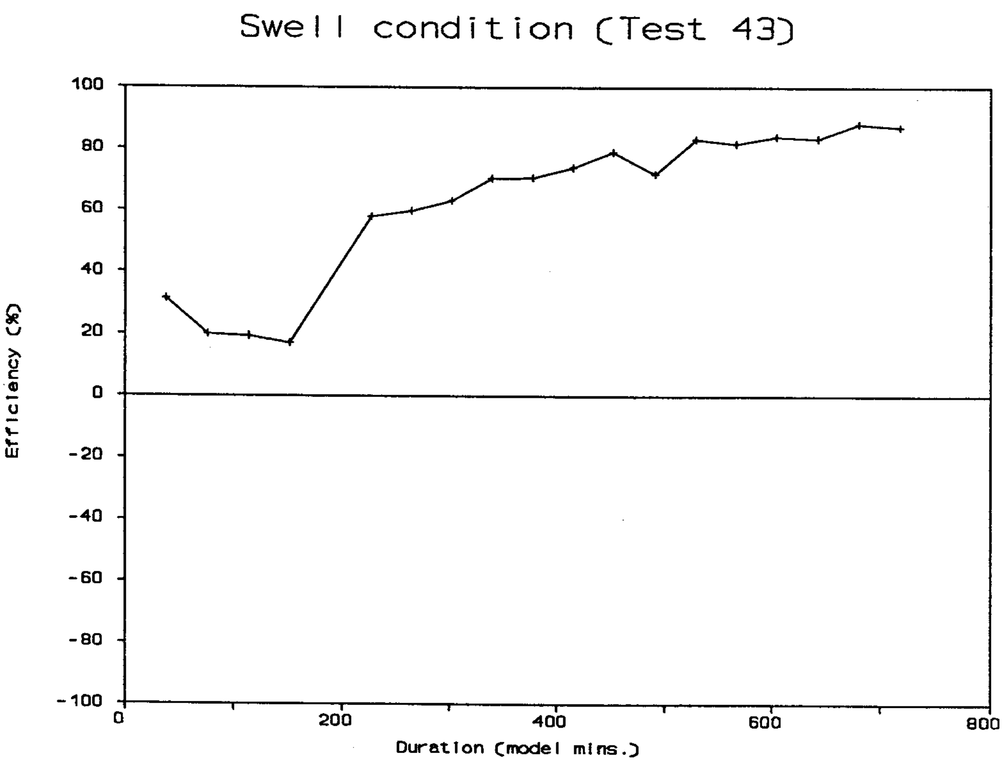


Figure A17 Groyne efficiency: Short rock and high timber, SWL = 4m, spacing = 1.5:1, 30° wave direction

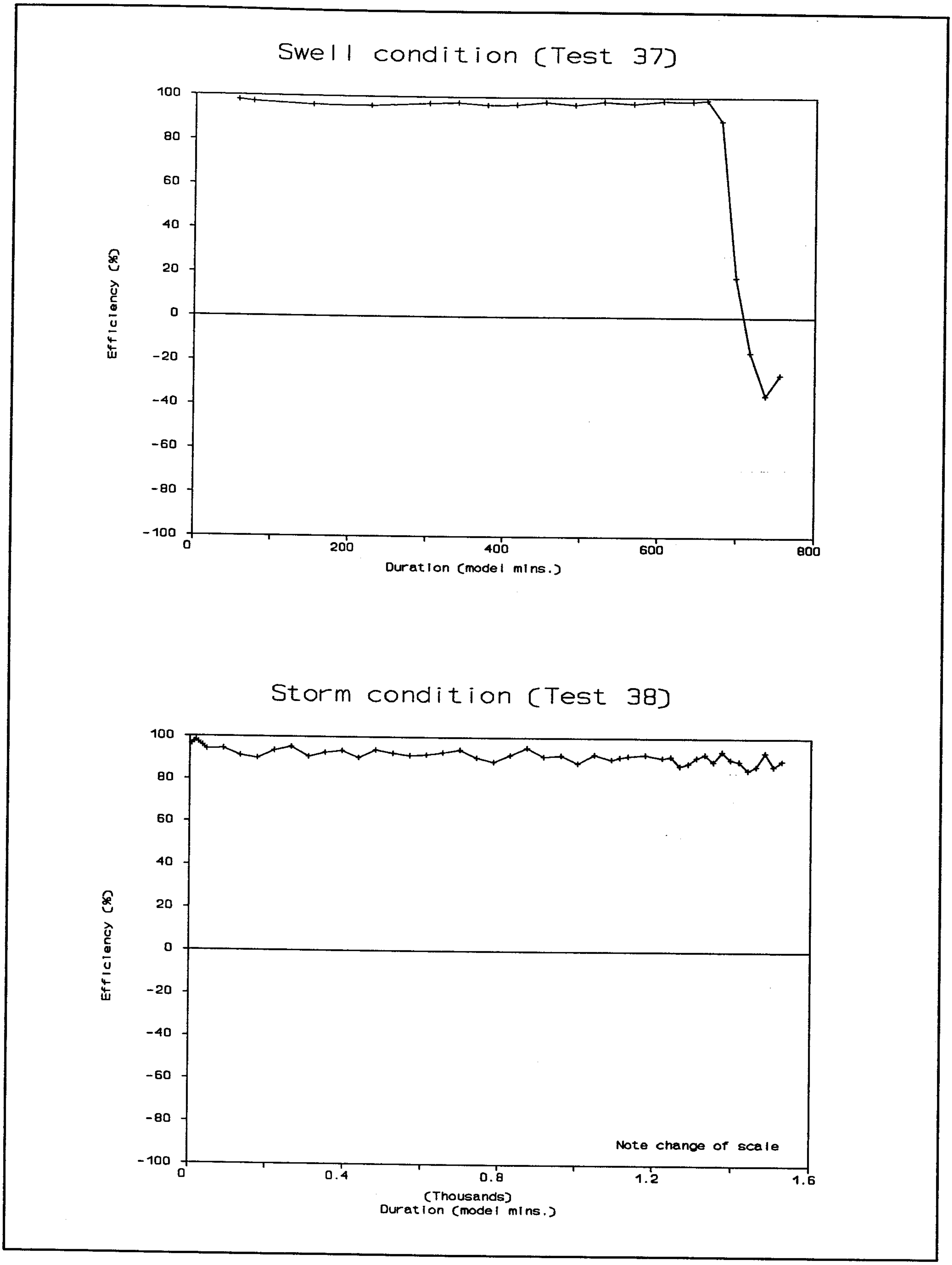


Figure A18 Groyne efficiency: High timber groynes, SWL = 4m, spacing = 1.5:1, 15° wave direction

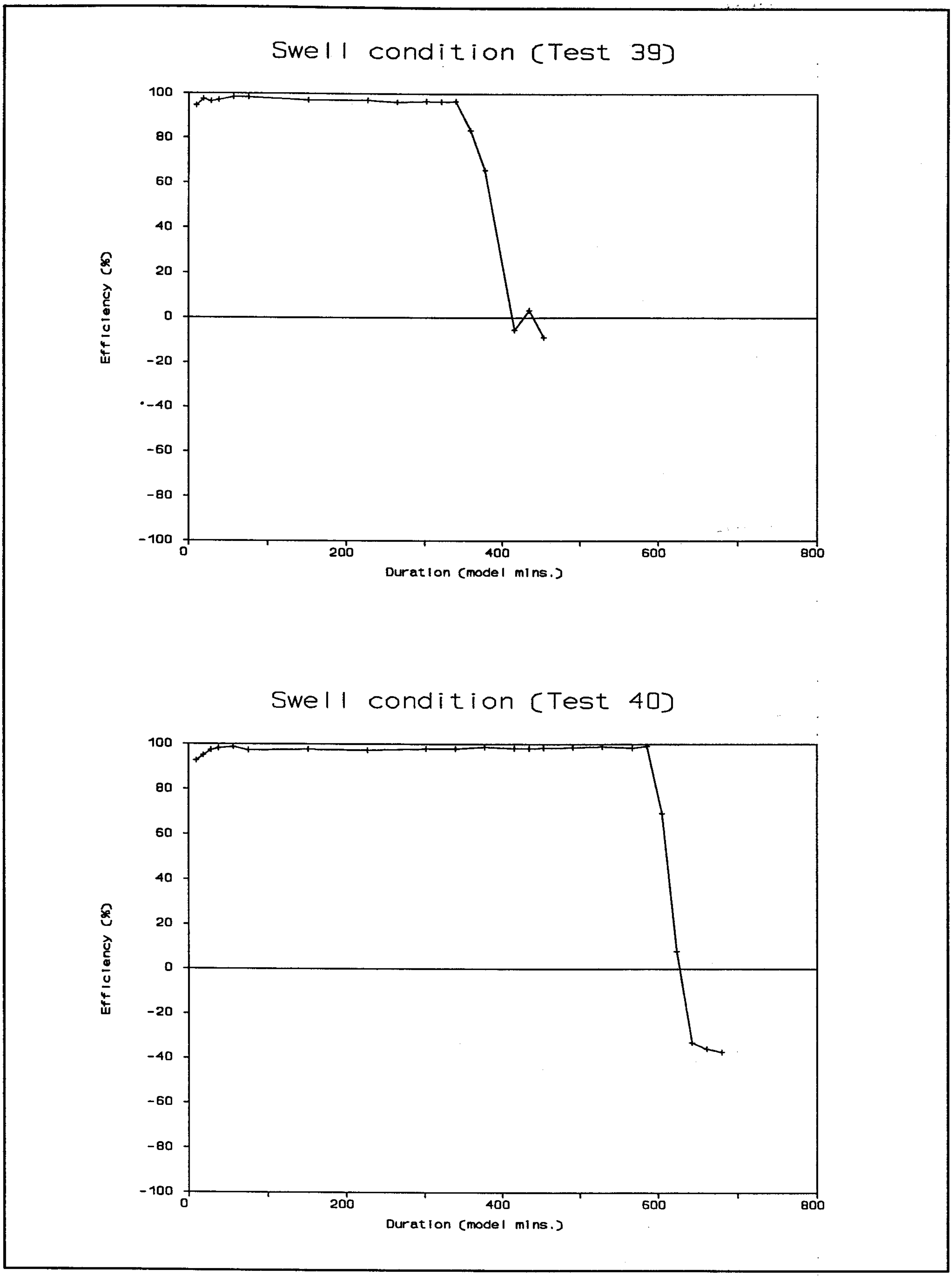


Figure A19 Groyne efficiency: Low timber groynes and high end rock groynes, SWL = 4m, Spacing = 1.5:1, 15° wave direction

Appendix 4

Cross-shore distribution of transport - Groynes

Appendix 4 Cross-shore distribution of transport - Groynes

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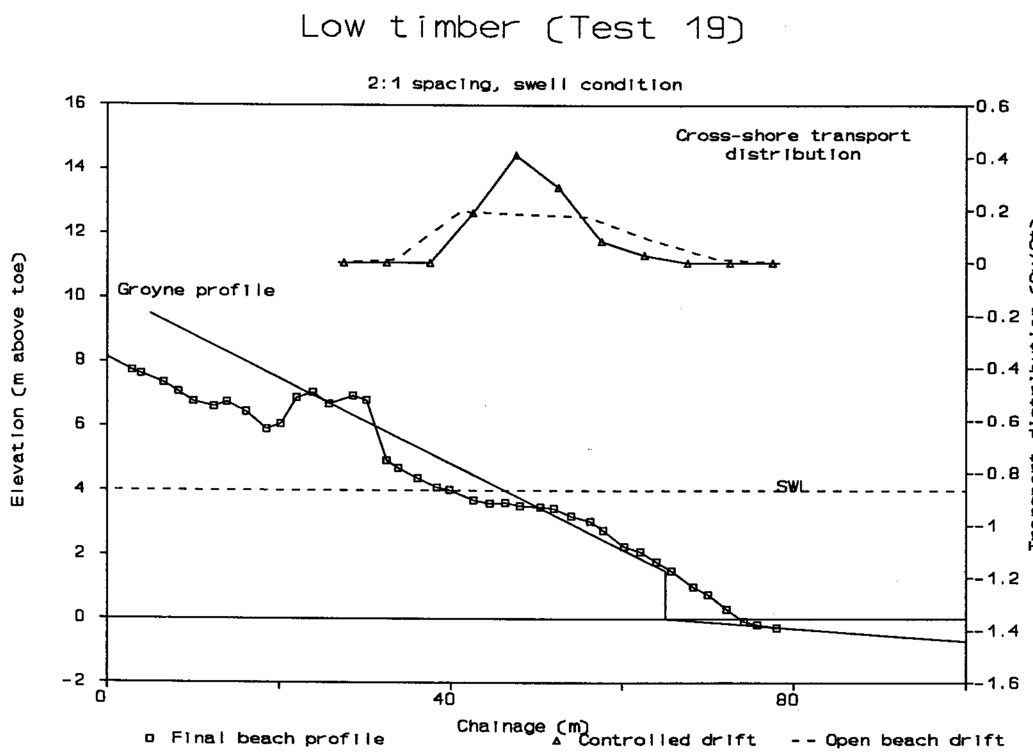


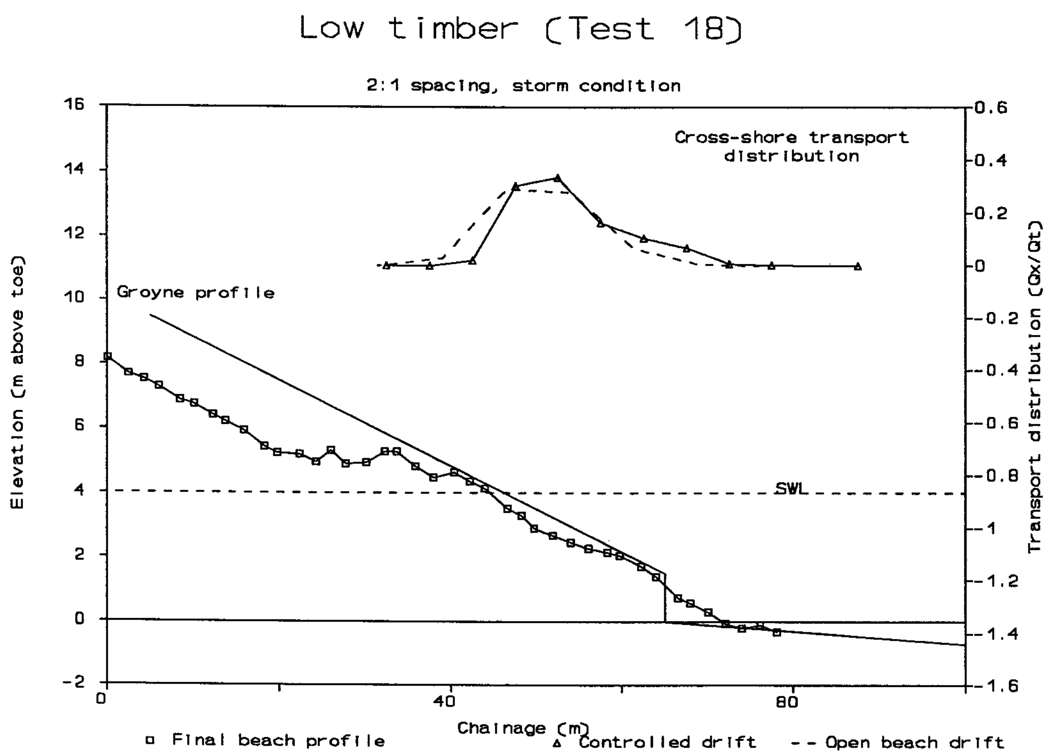
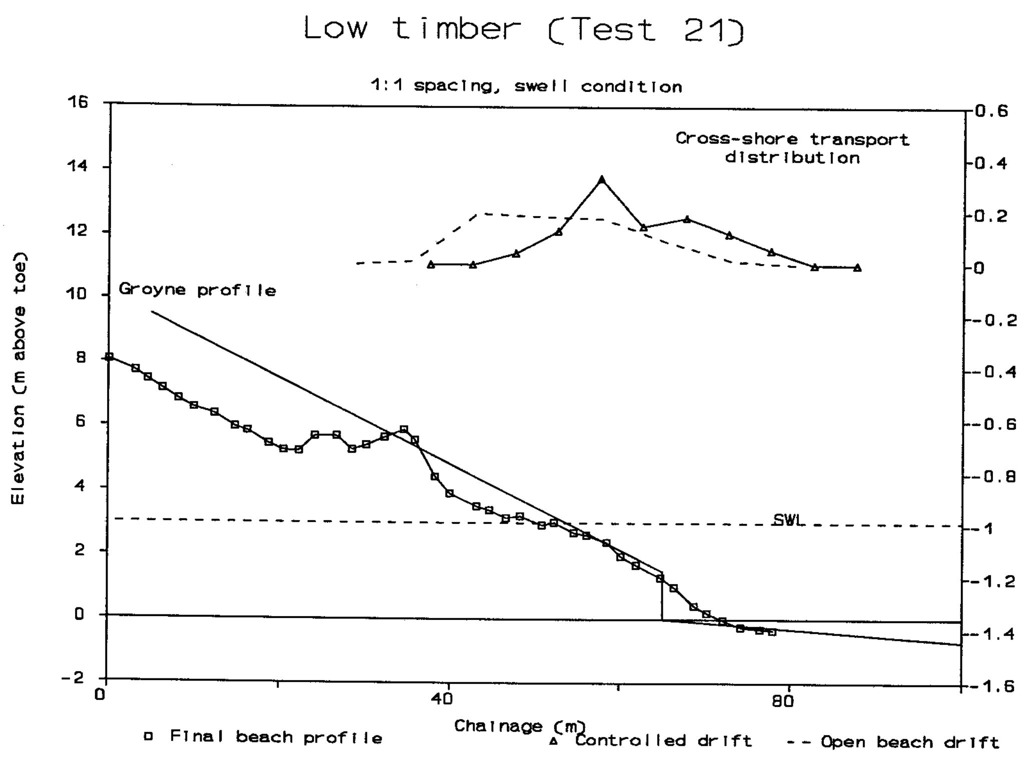


Figure A1 Cross-shore distribution of transport and updrift profile



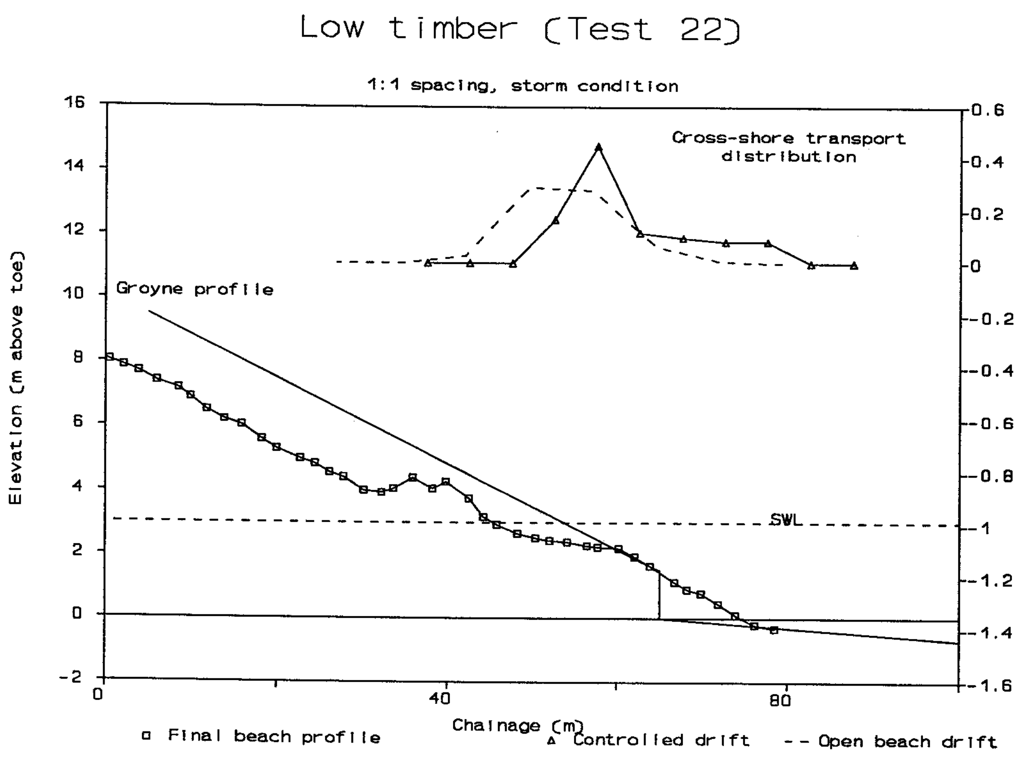
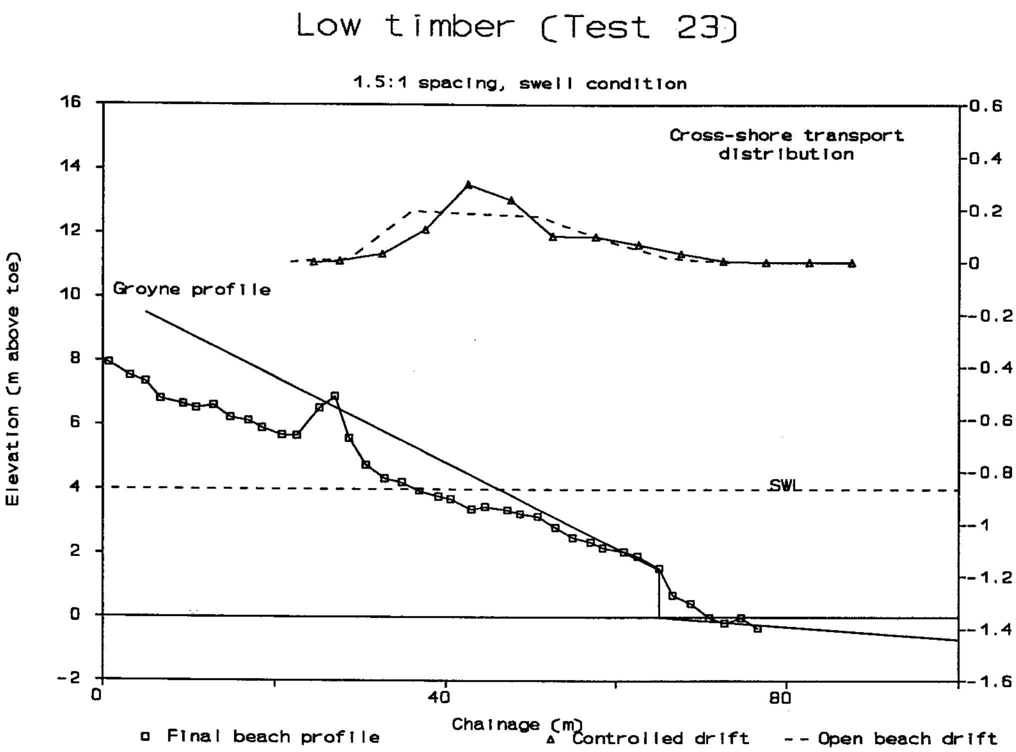


Figure A2 Cross-shore distribution of transport and updrift profile



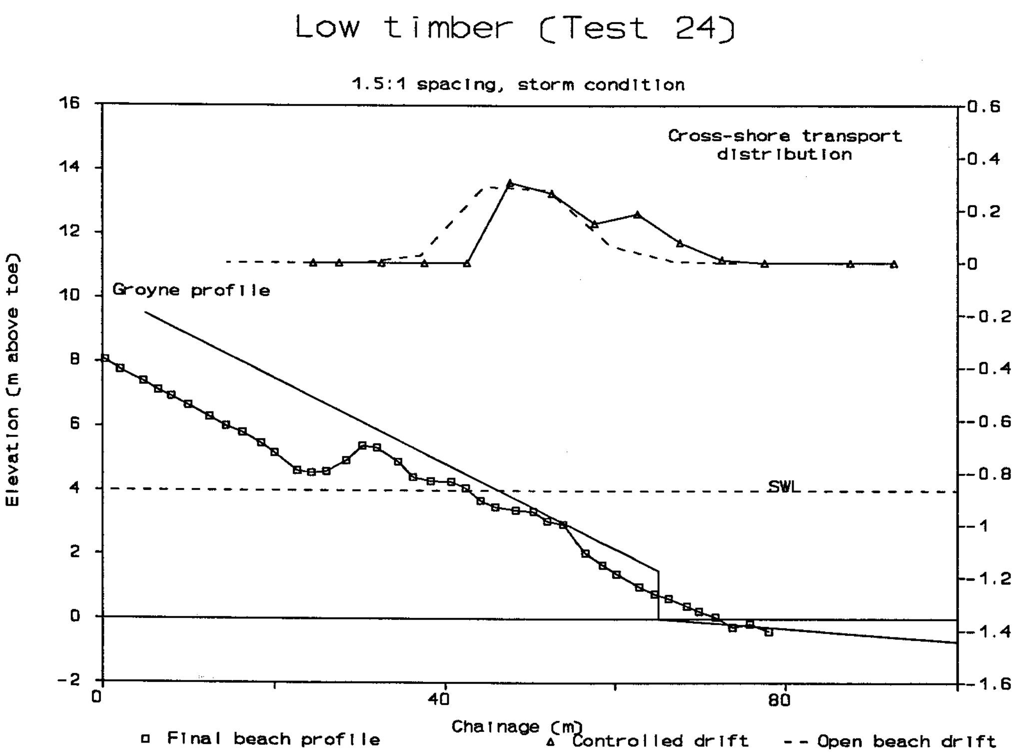
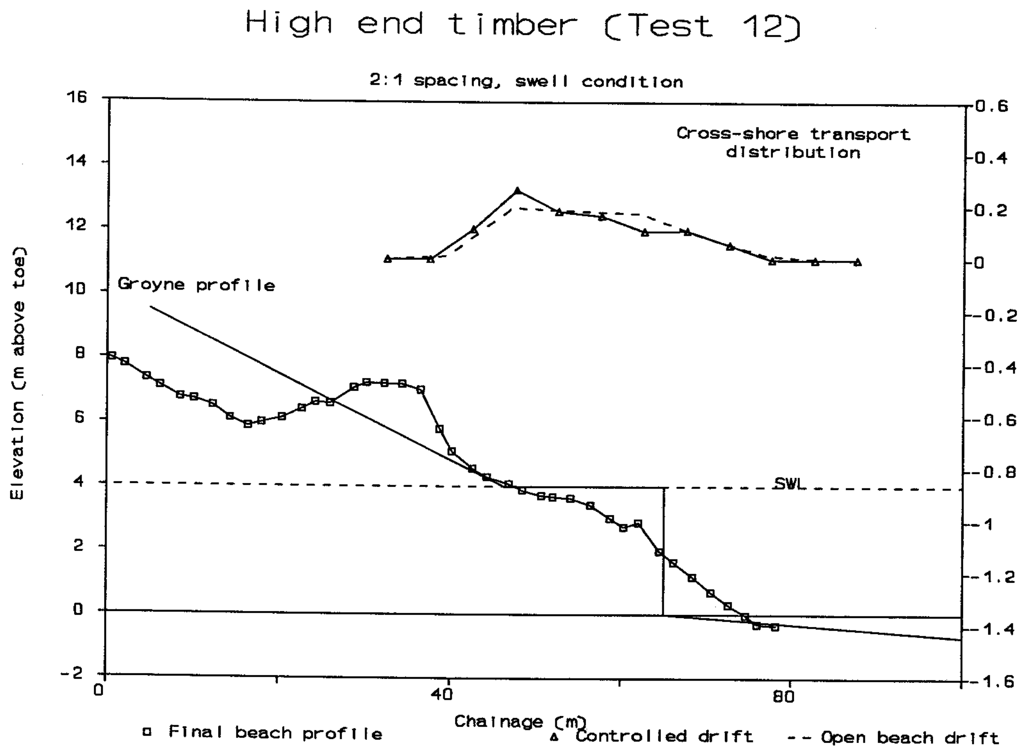


Figure A3 Cross-shore distribution of transport and updrift profile



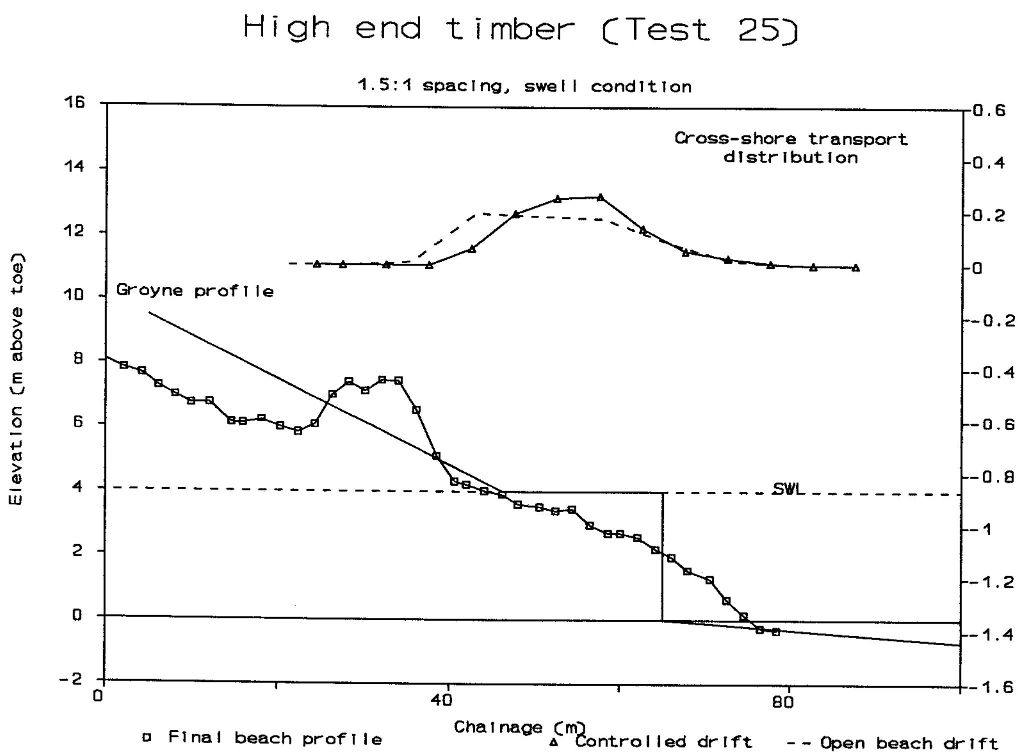
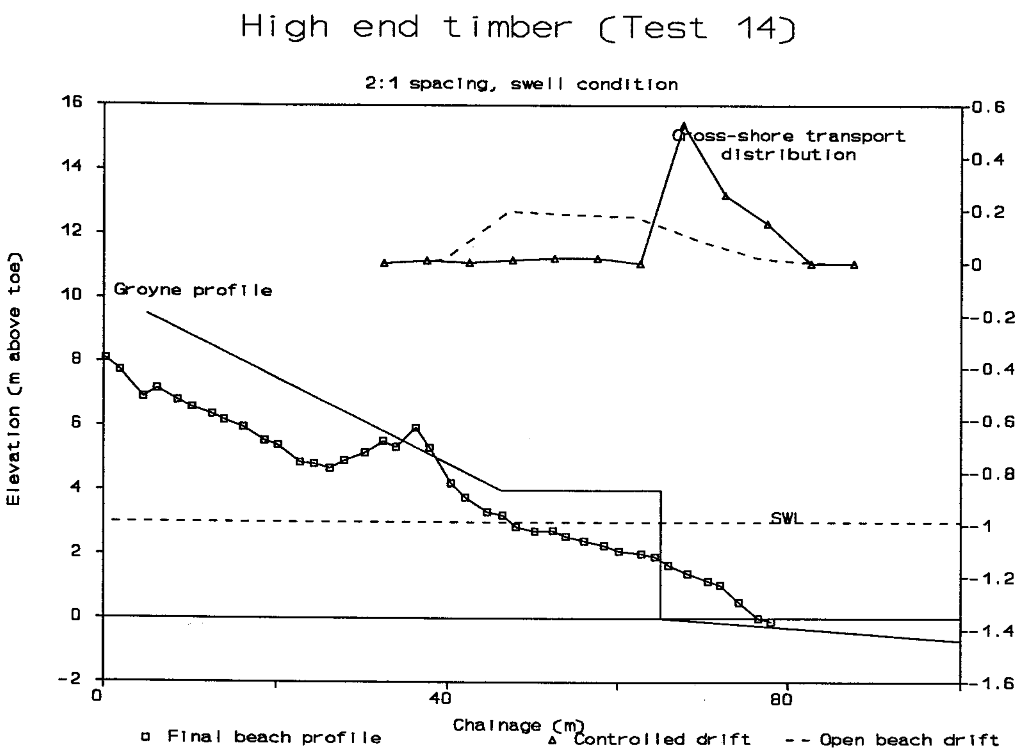
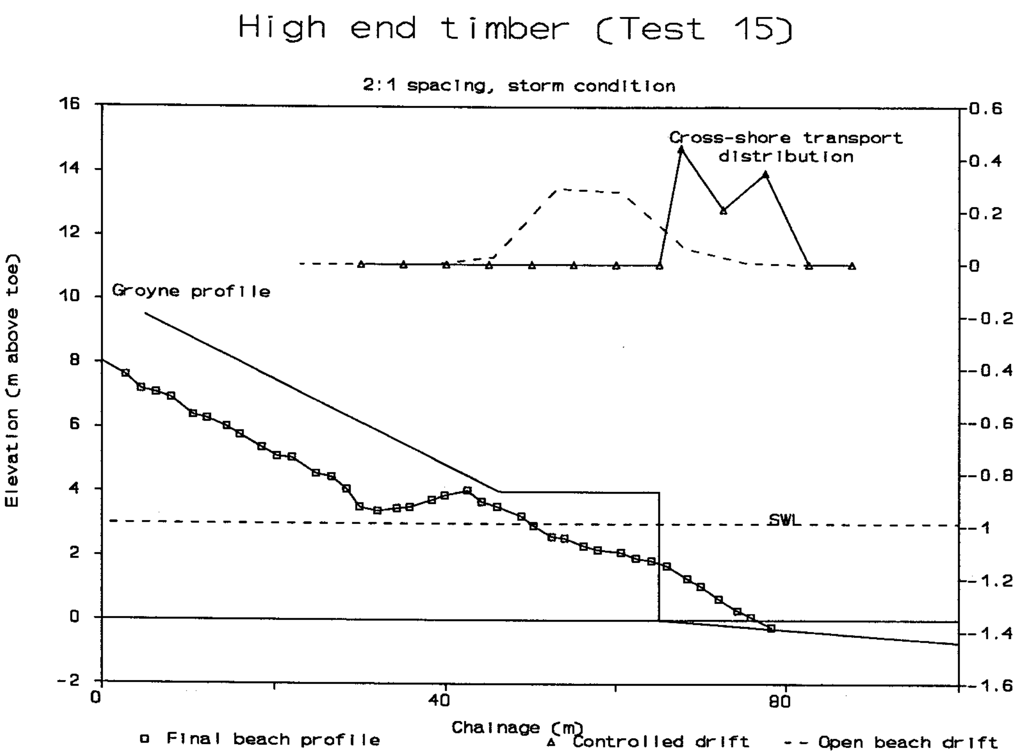


Figure A4 Cross-shore distribution of transport and updrift profile





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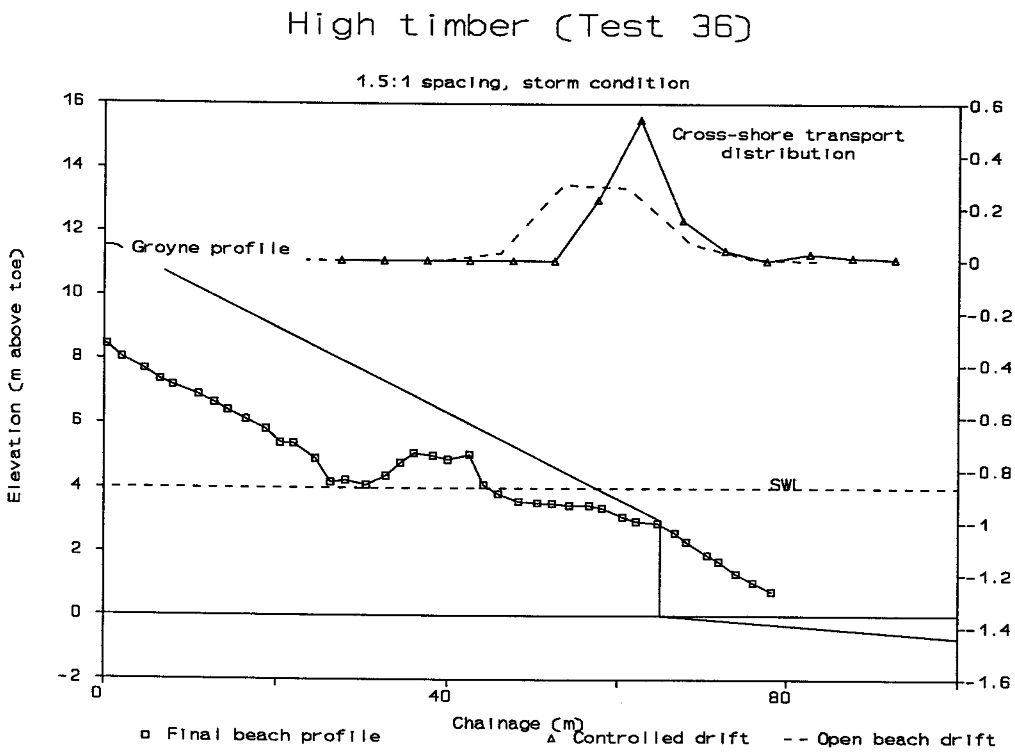
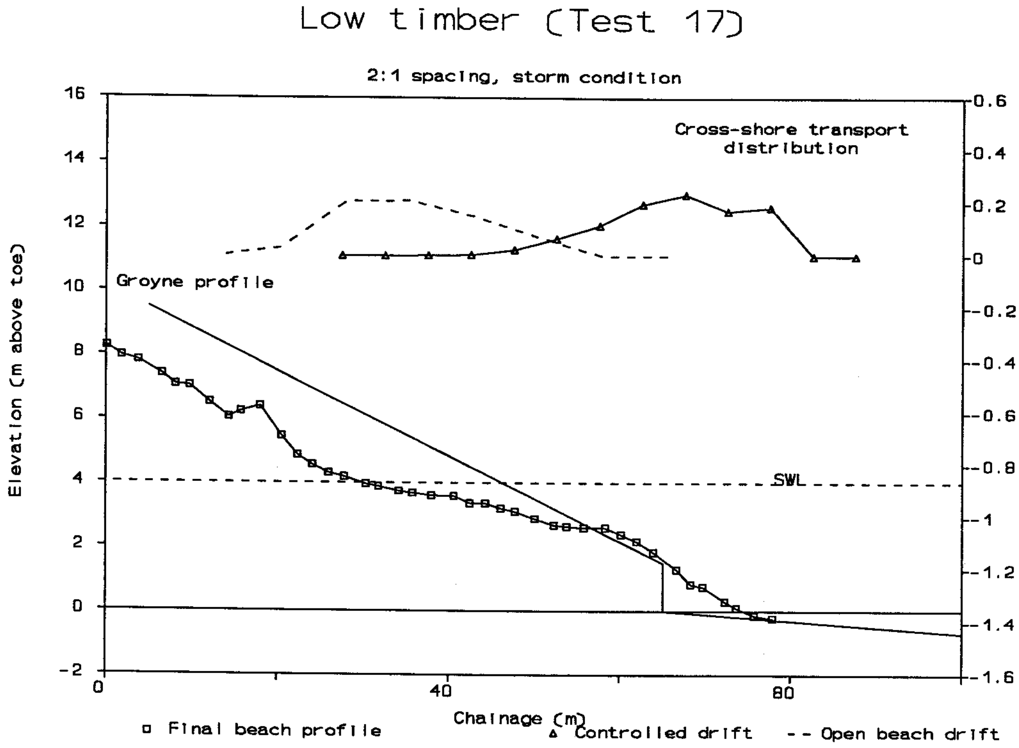


Figure A6 Cross-shore distribution of transport and updrift profile



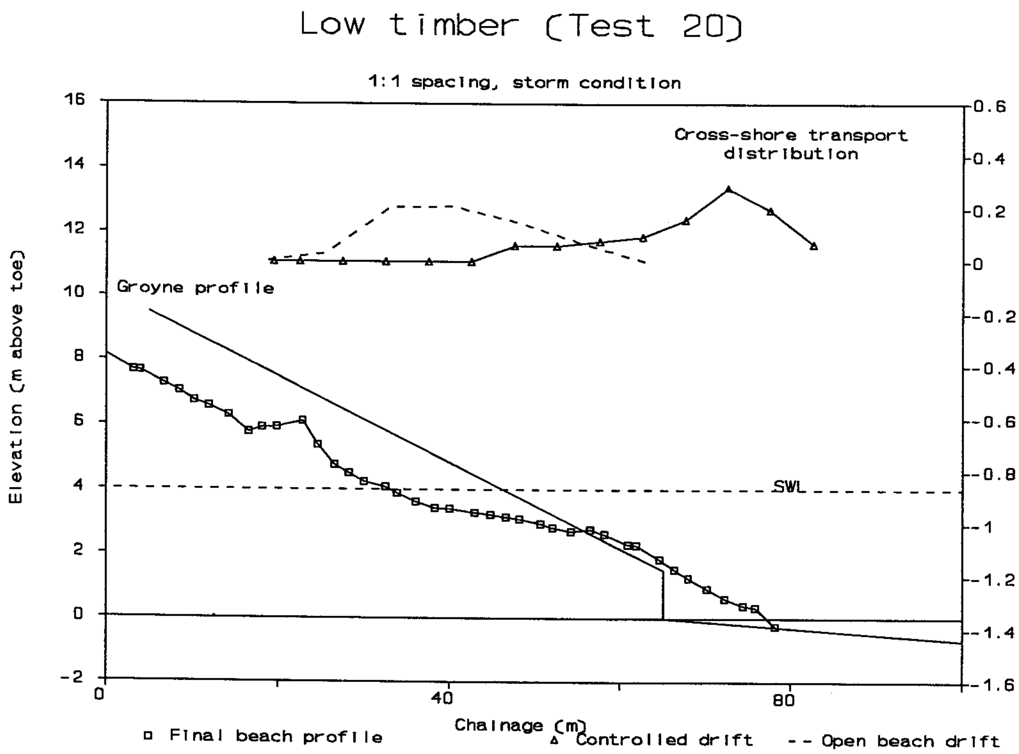


Figure A7 Cross-shore distribution of transport and updrift profile - H_s = 3m

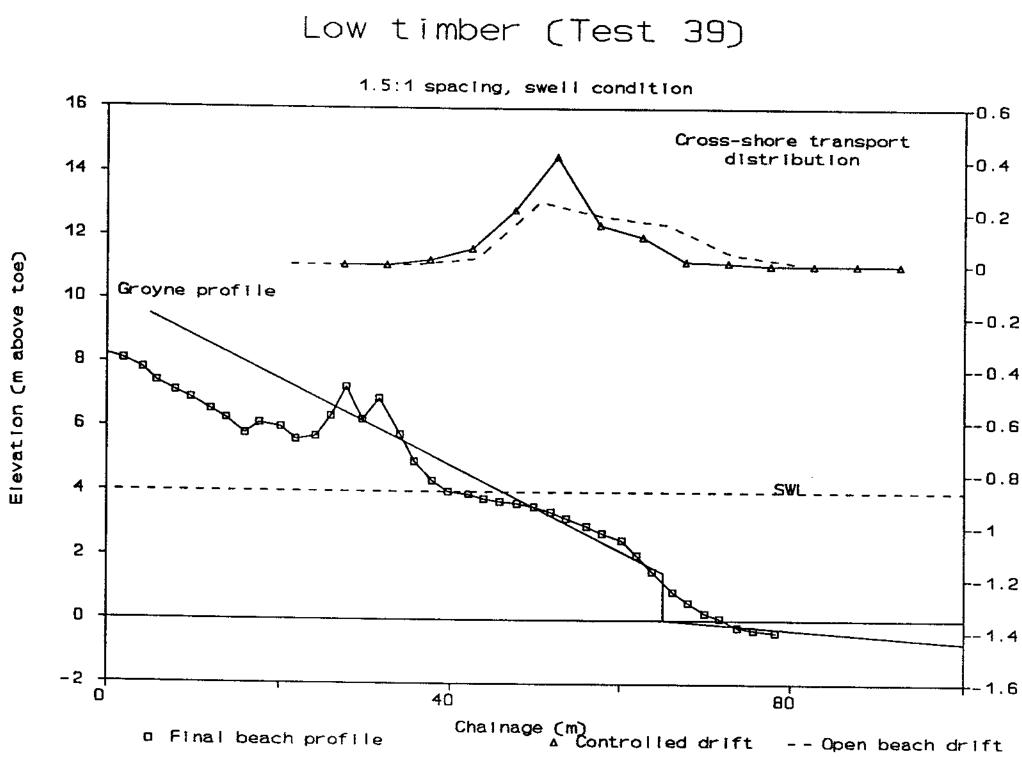


Figure A8 Cross-shore distribution of transport and updrift profile- Offshore direction = 15°

Appendix 5

End of test photographs - Groynes

Appendix 5 End of test photographs - Groynes

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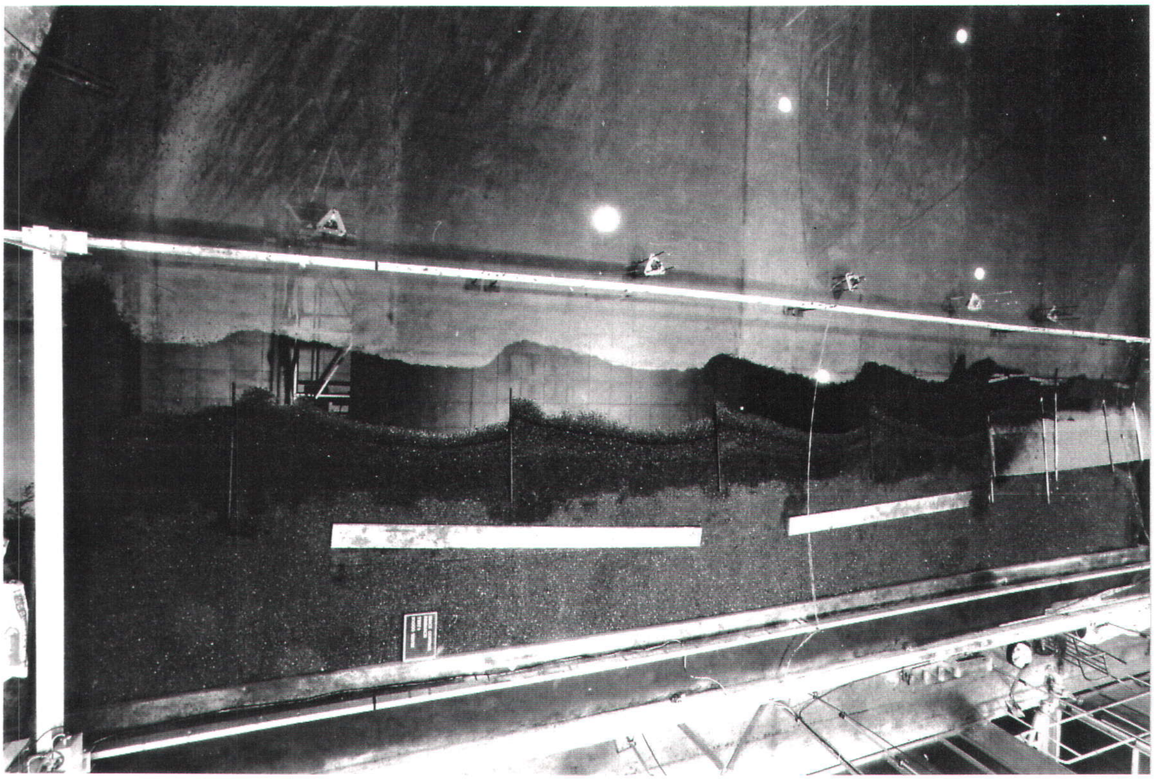

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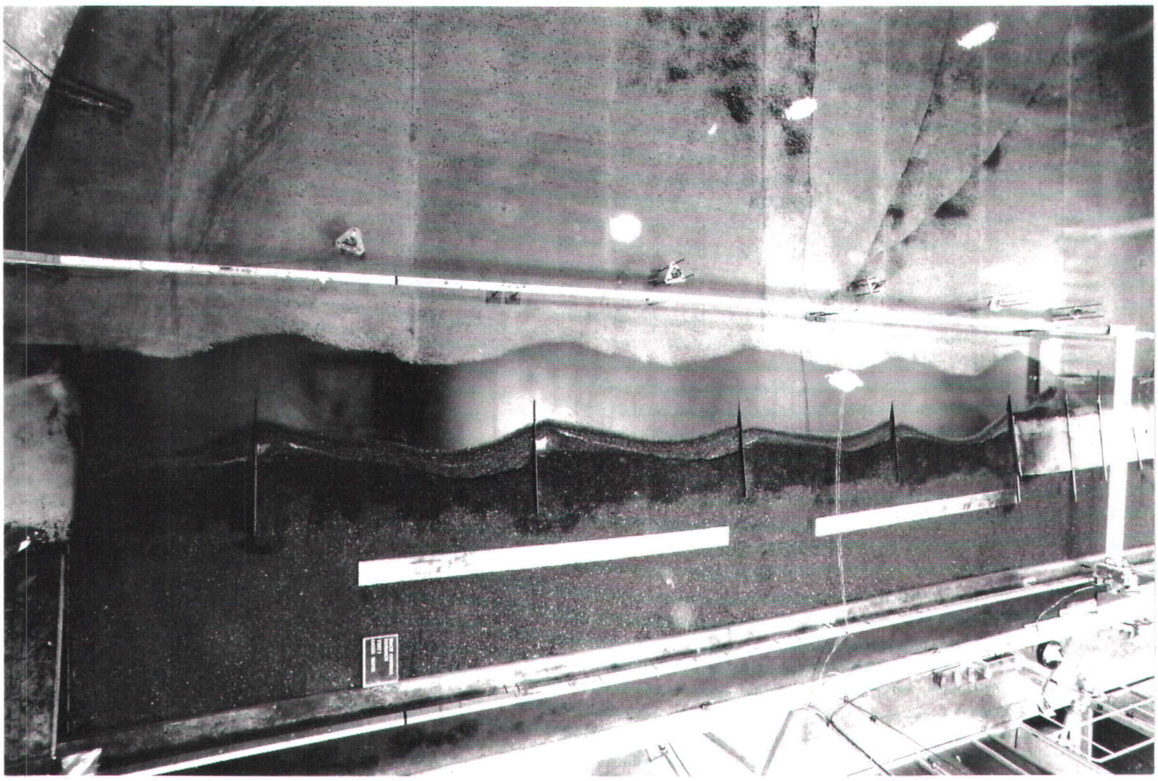

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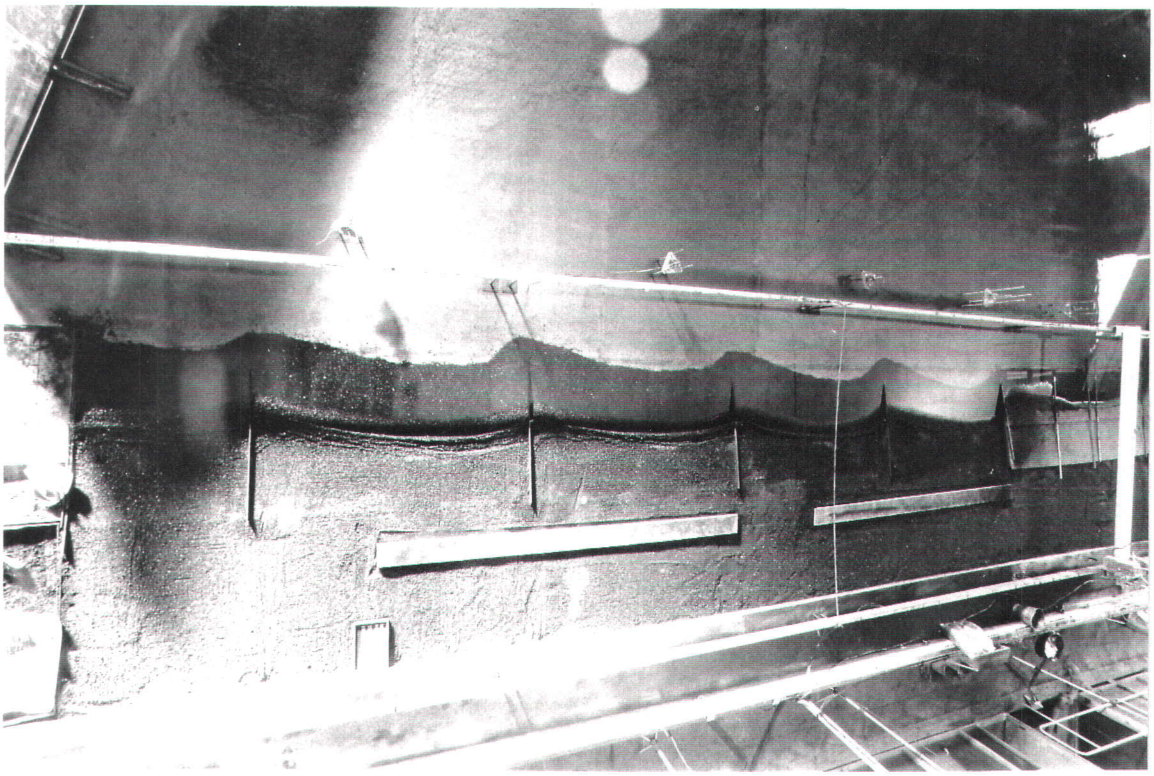

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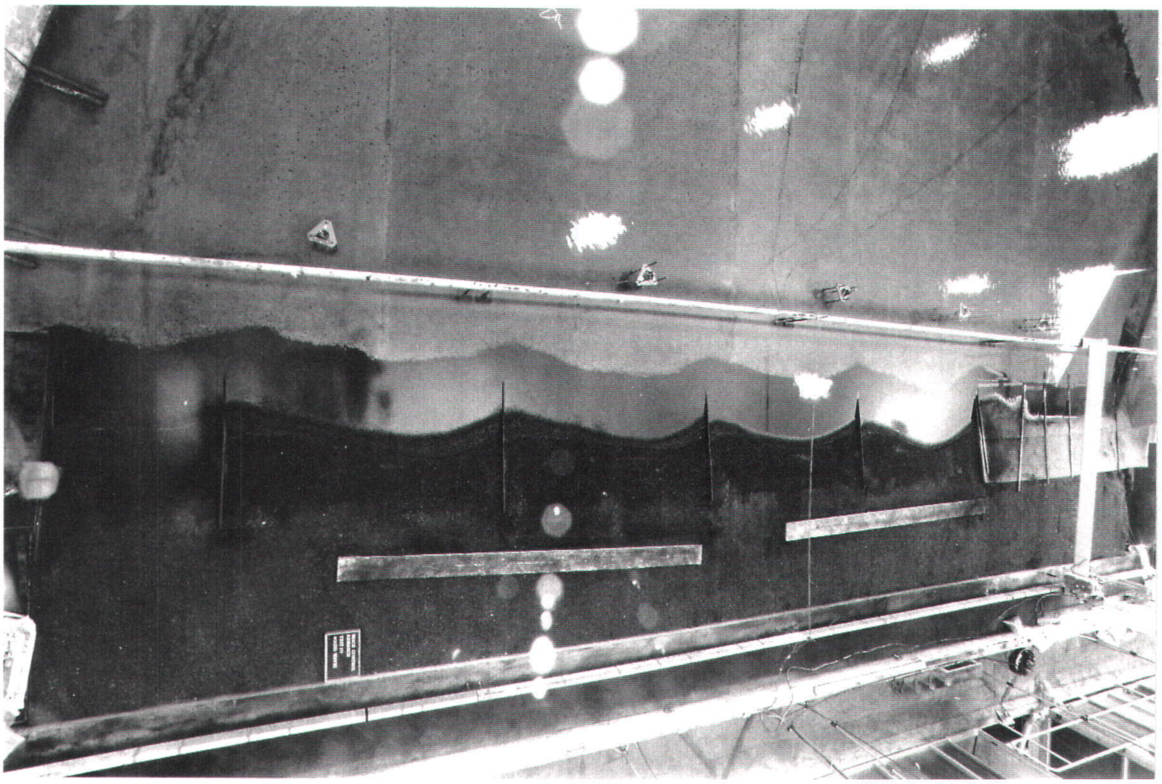

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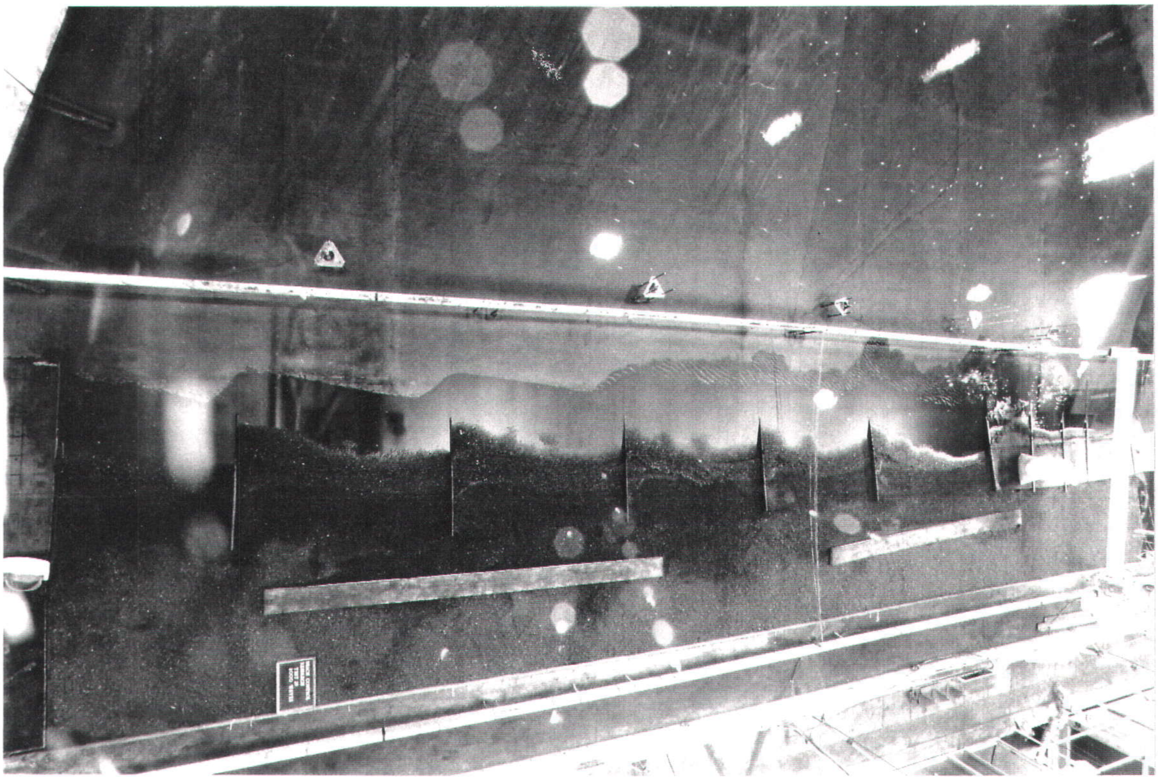

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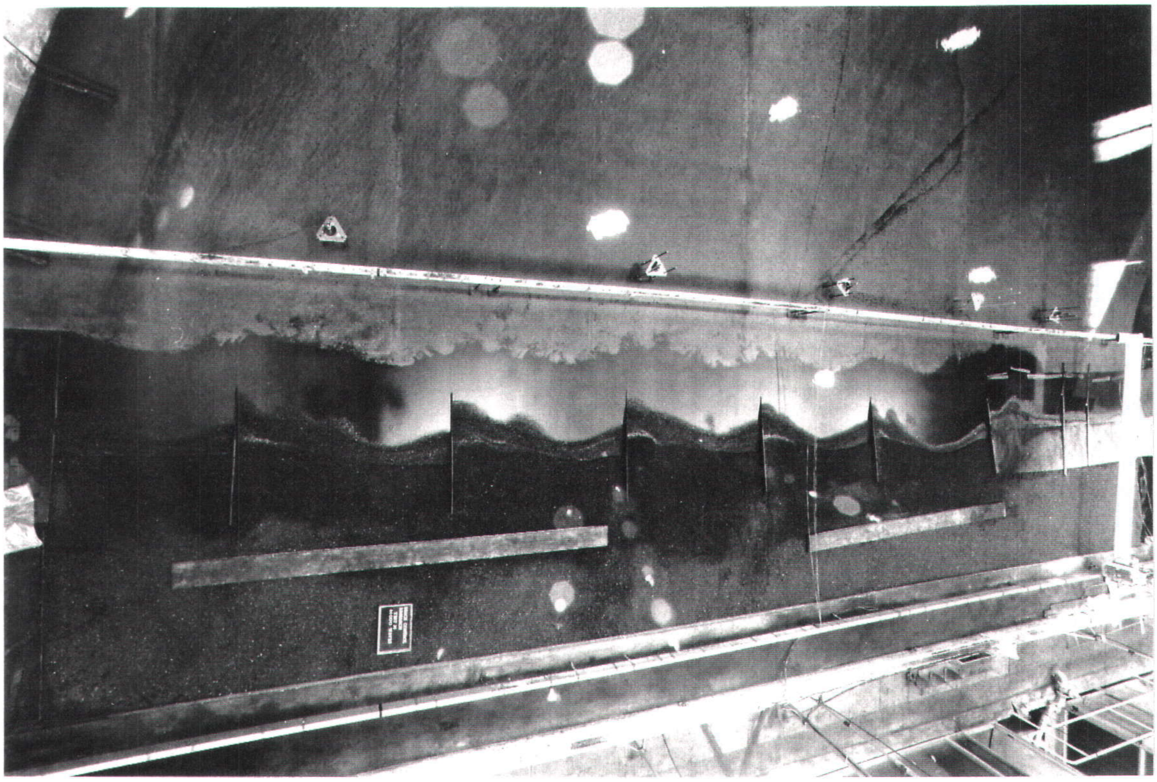

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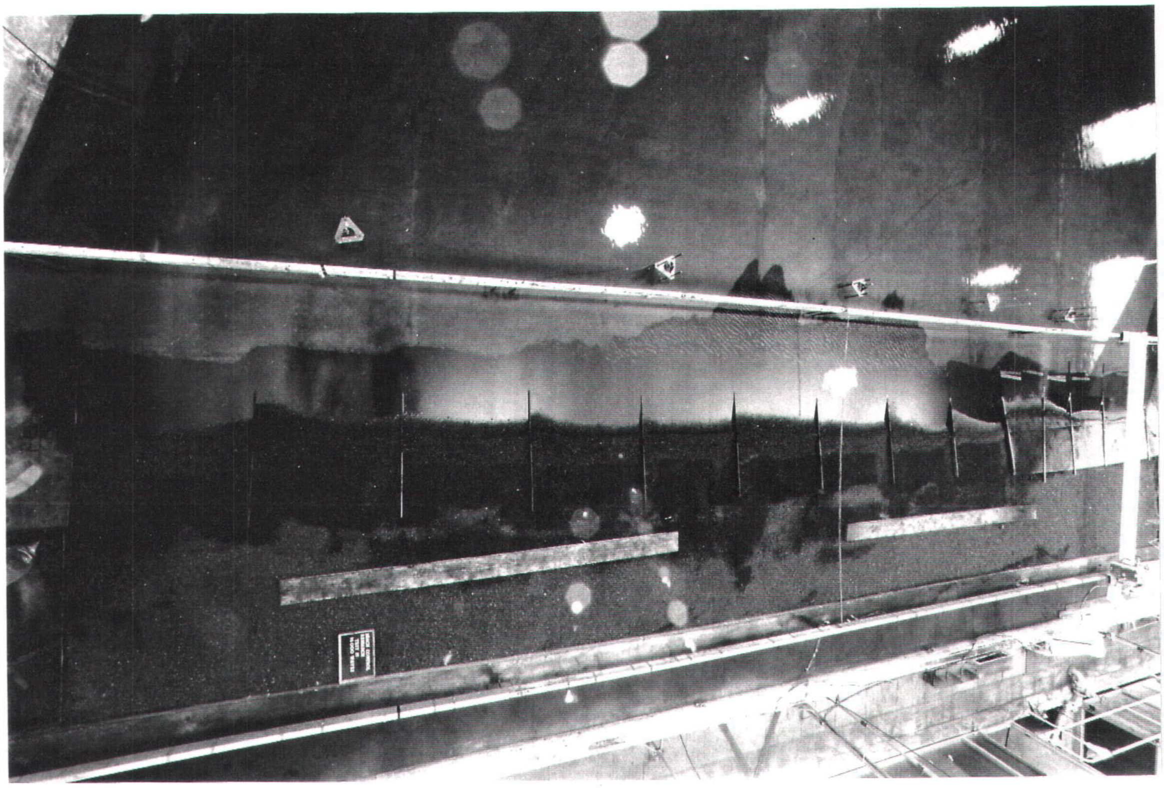

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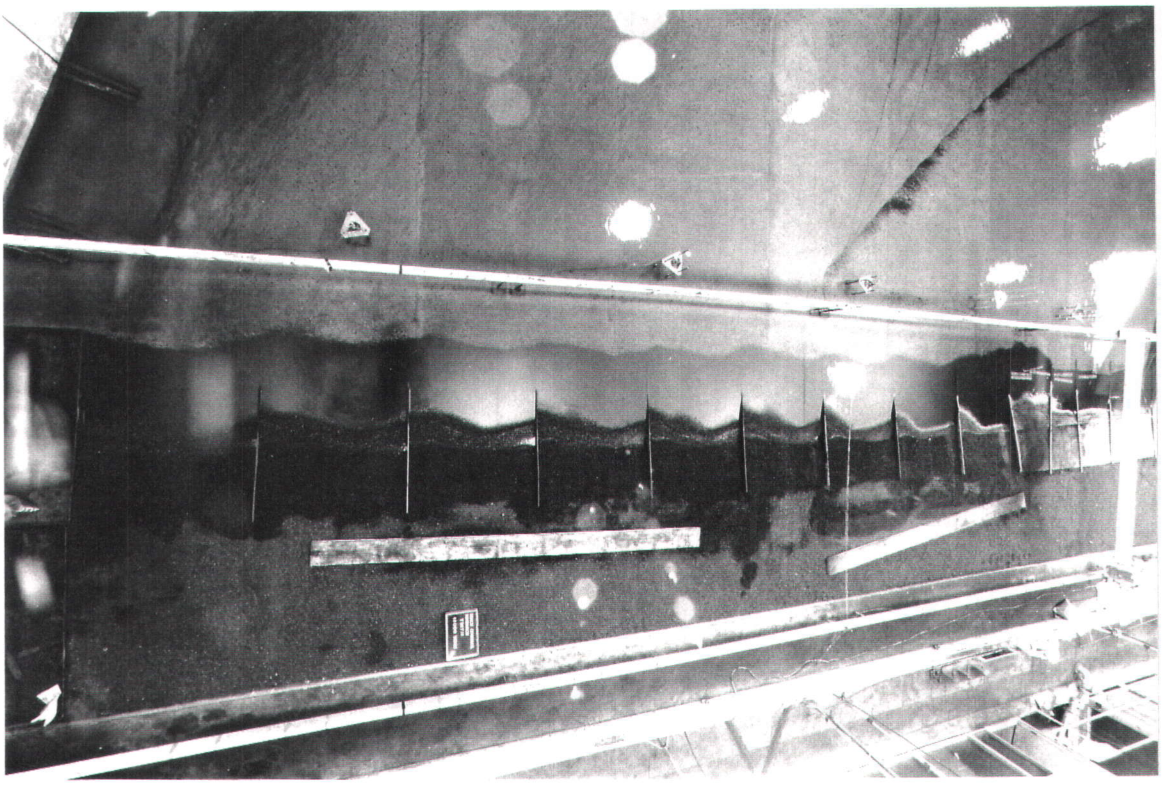

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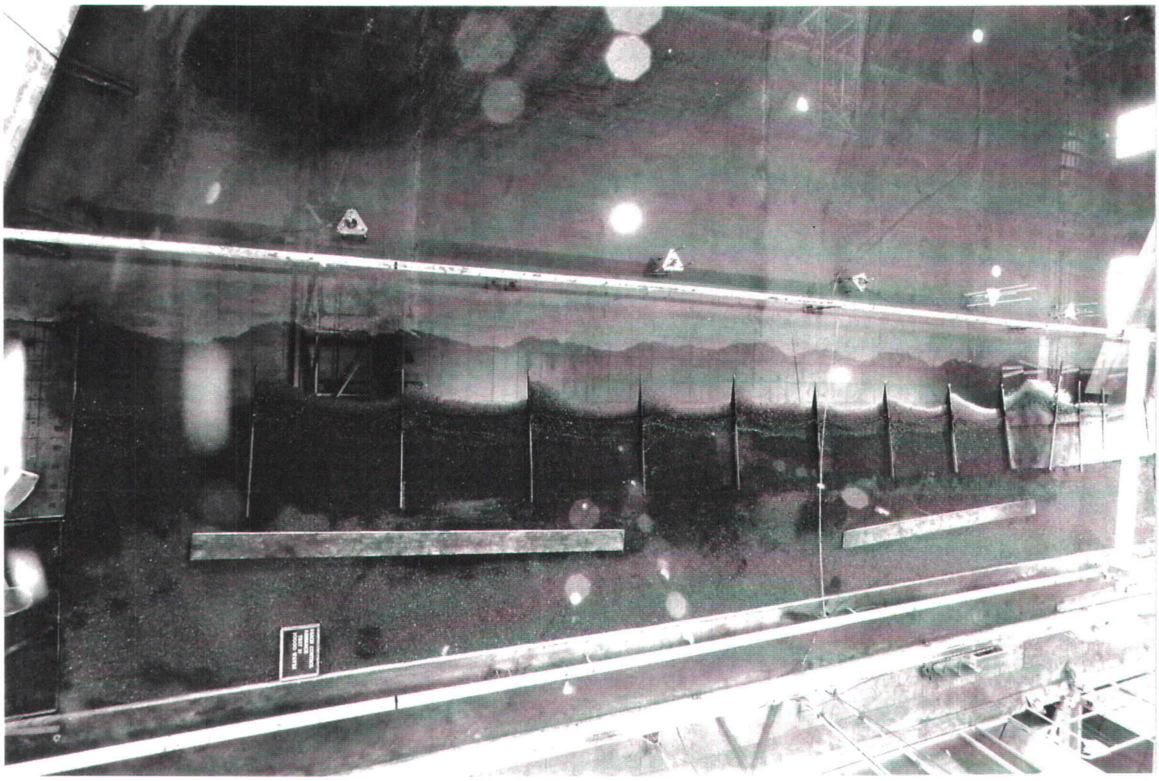

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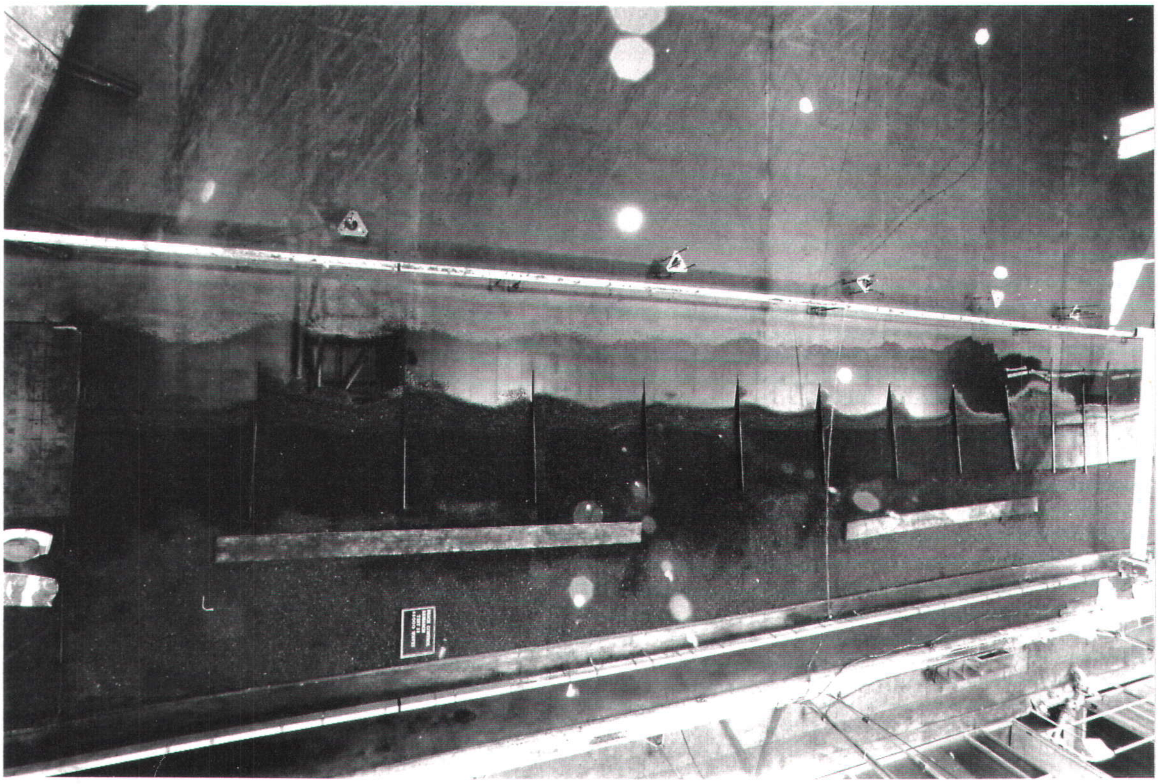

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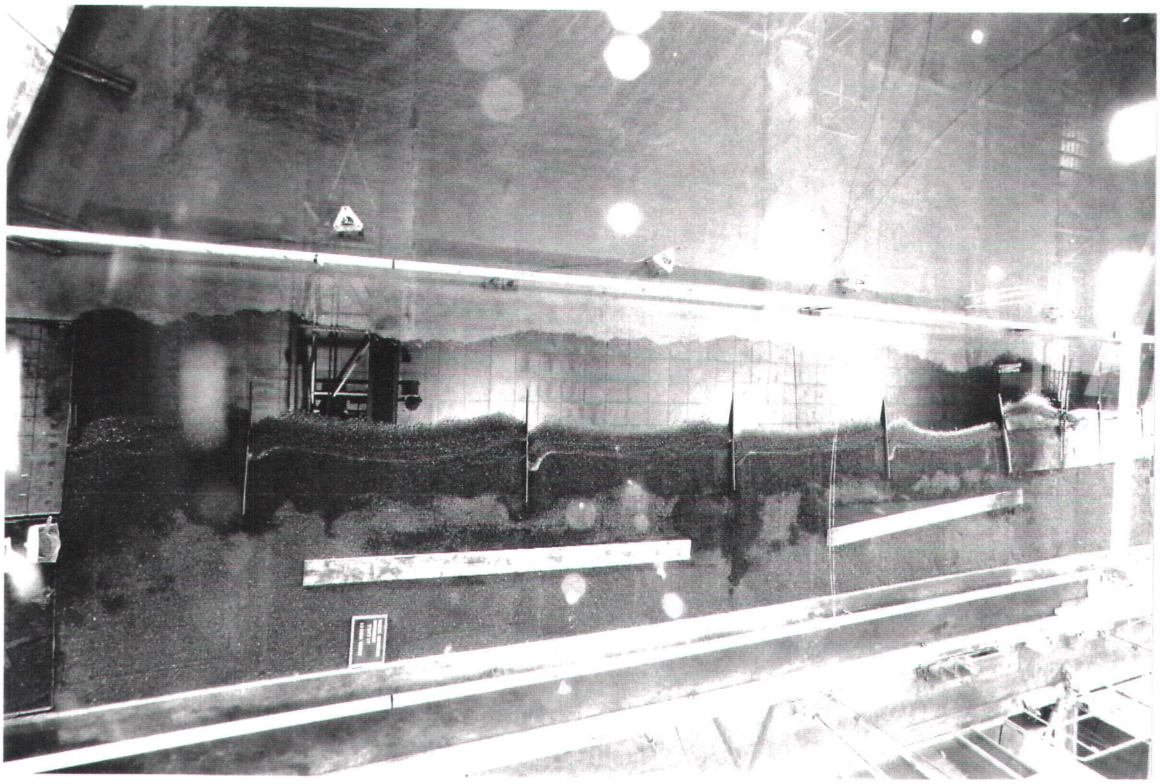

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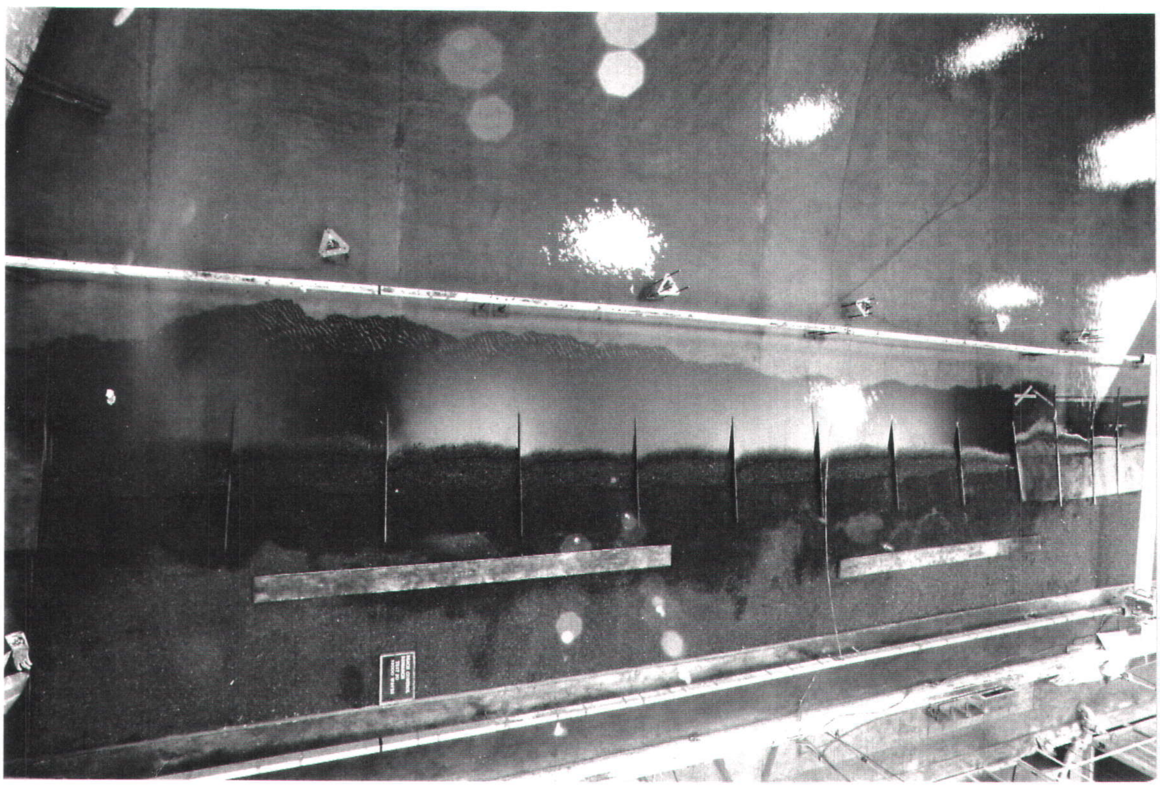

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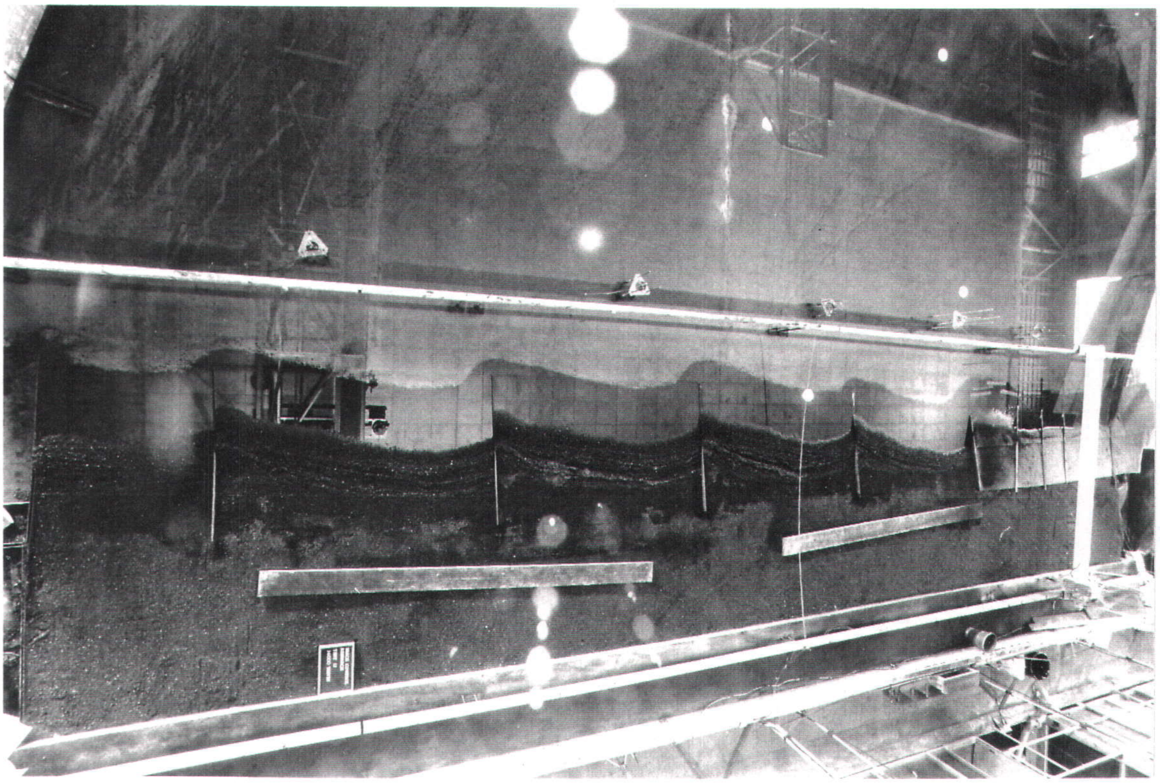

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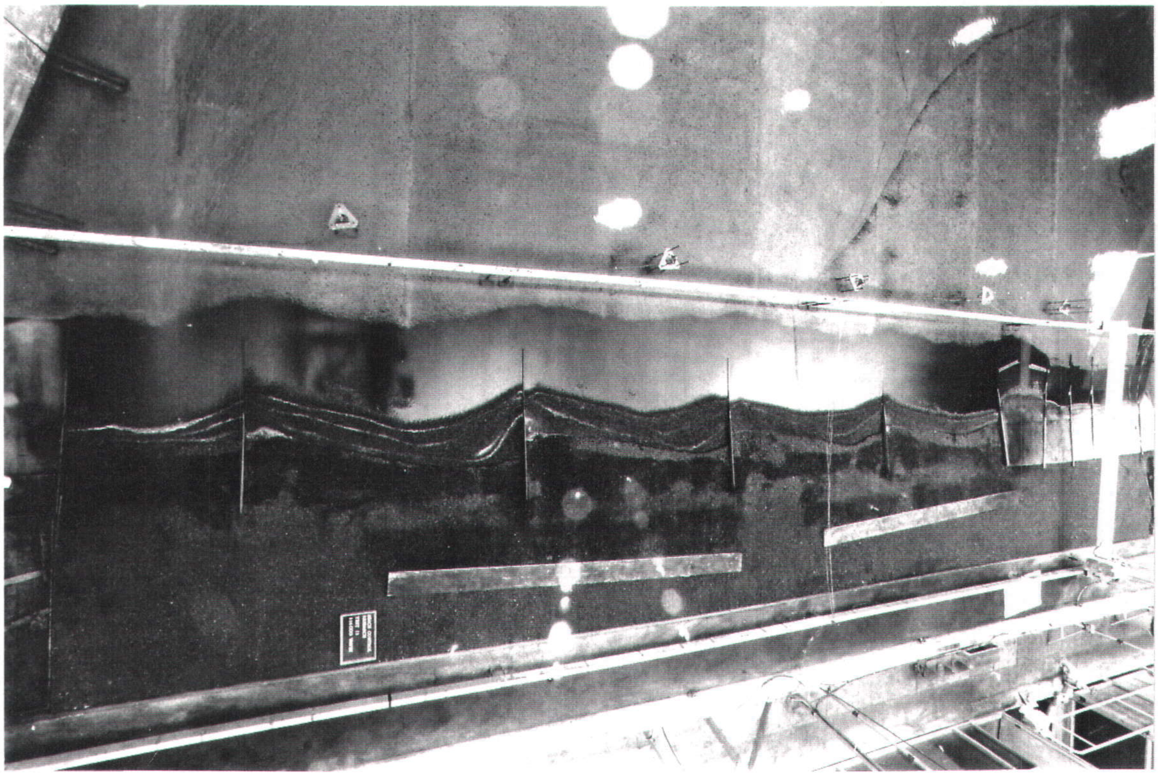

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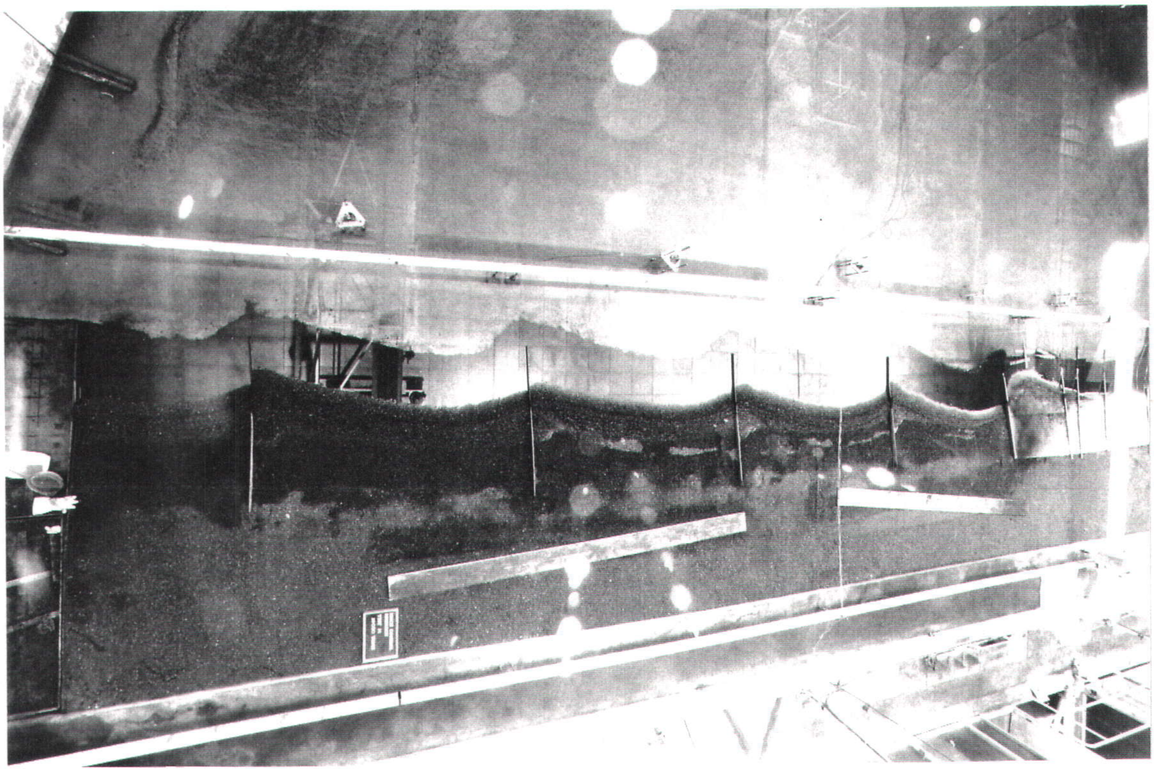

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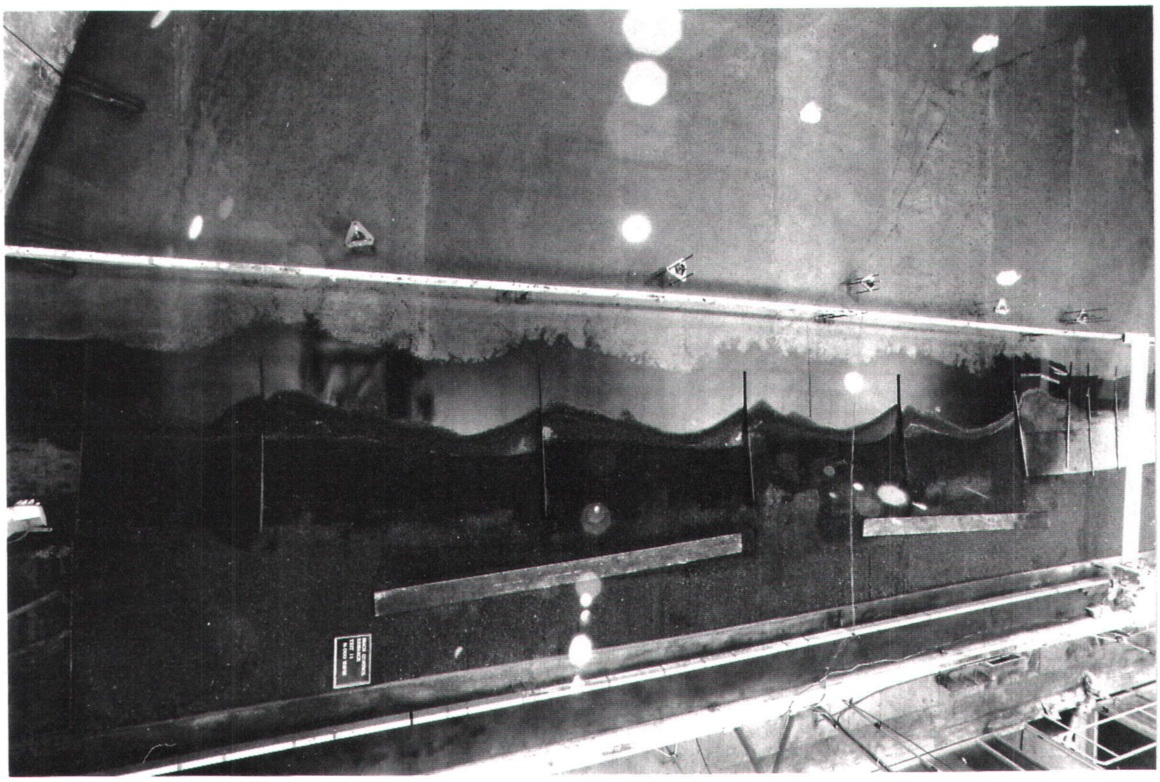

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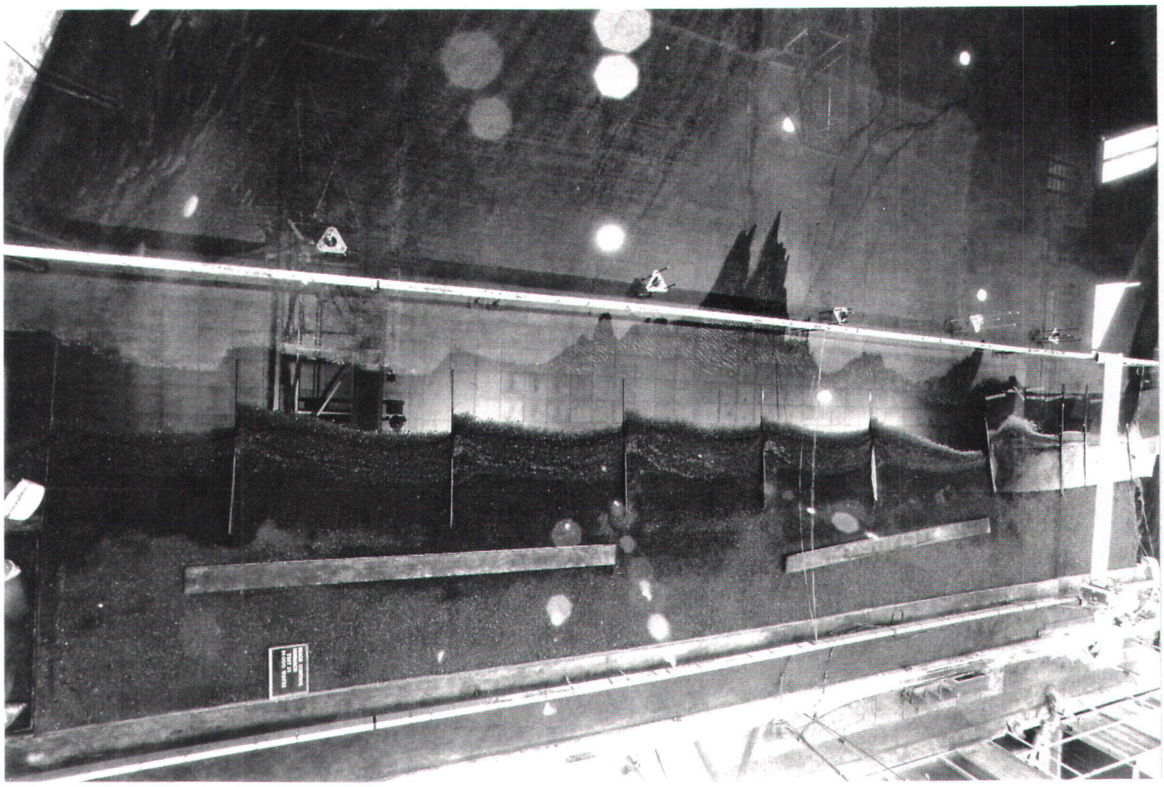

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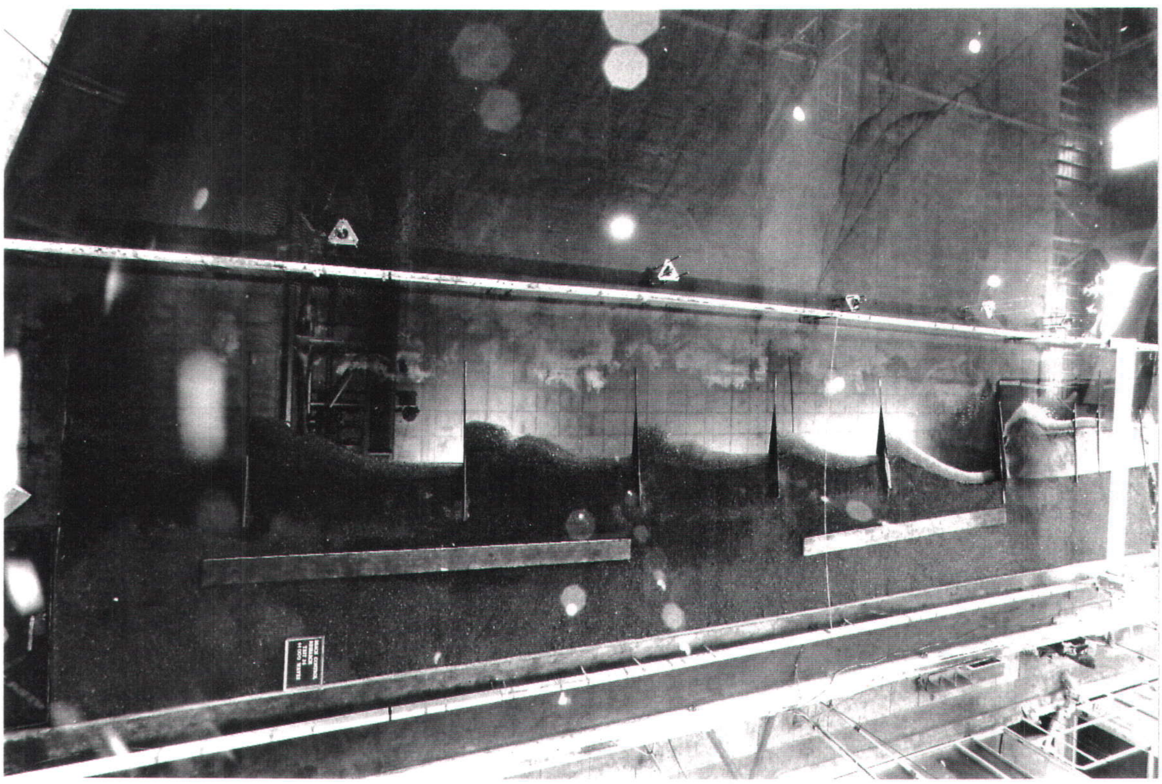

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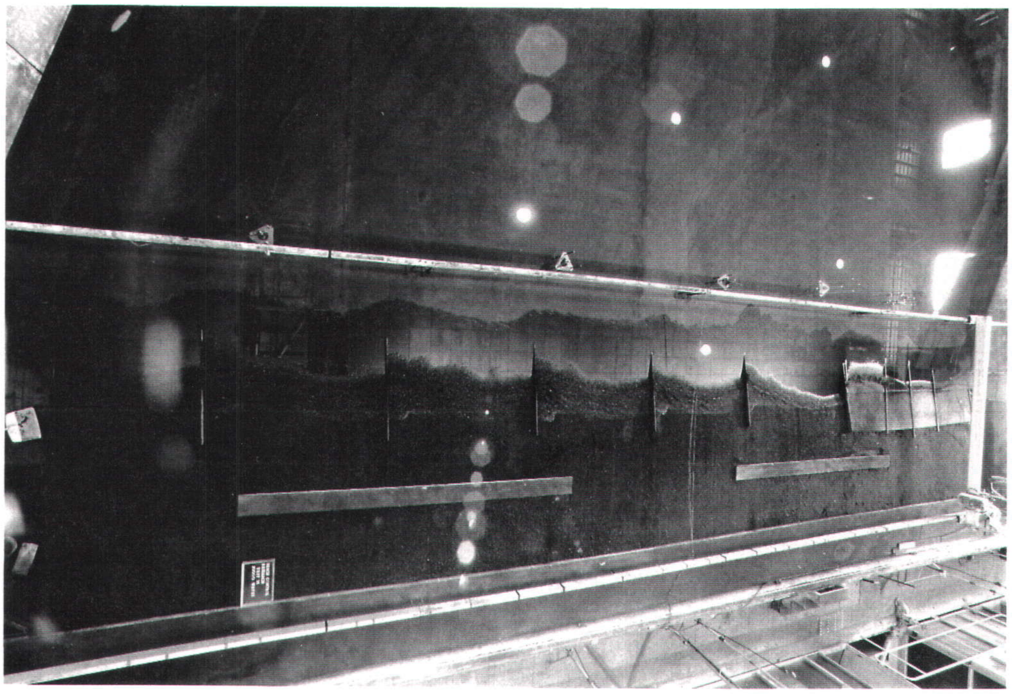

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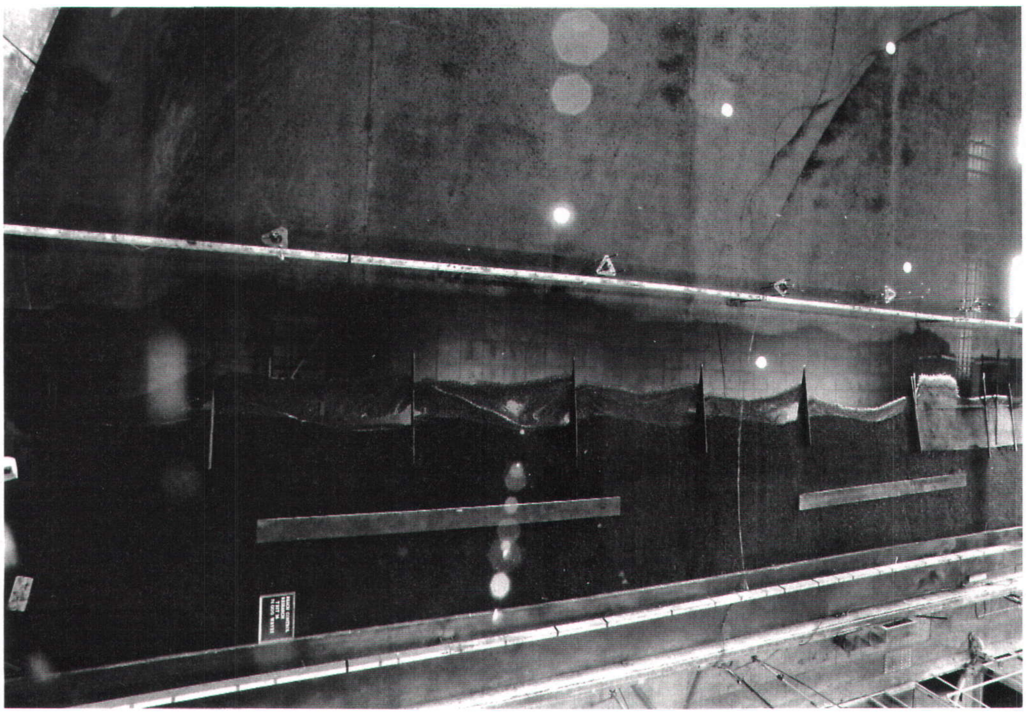

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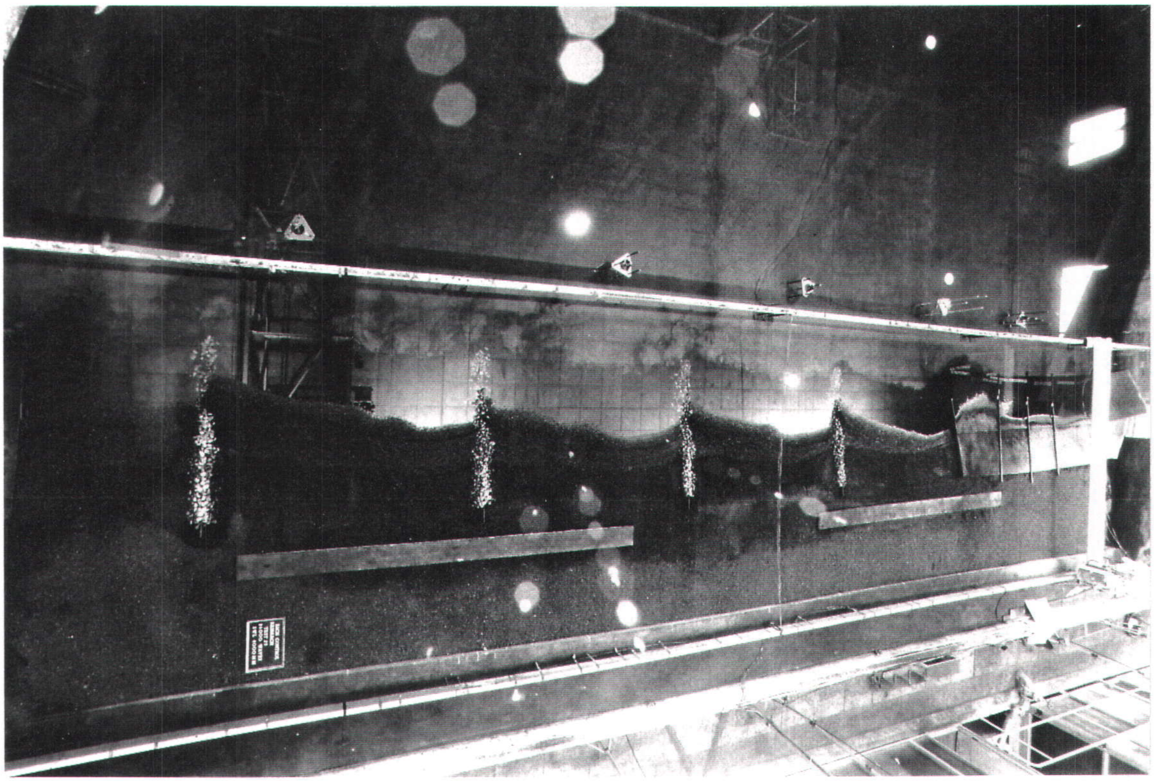


Plate A21 Final beach development (Test 29)

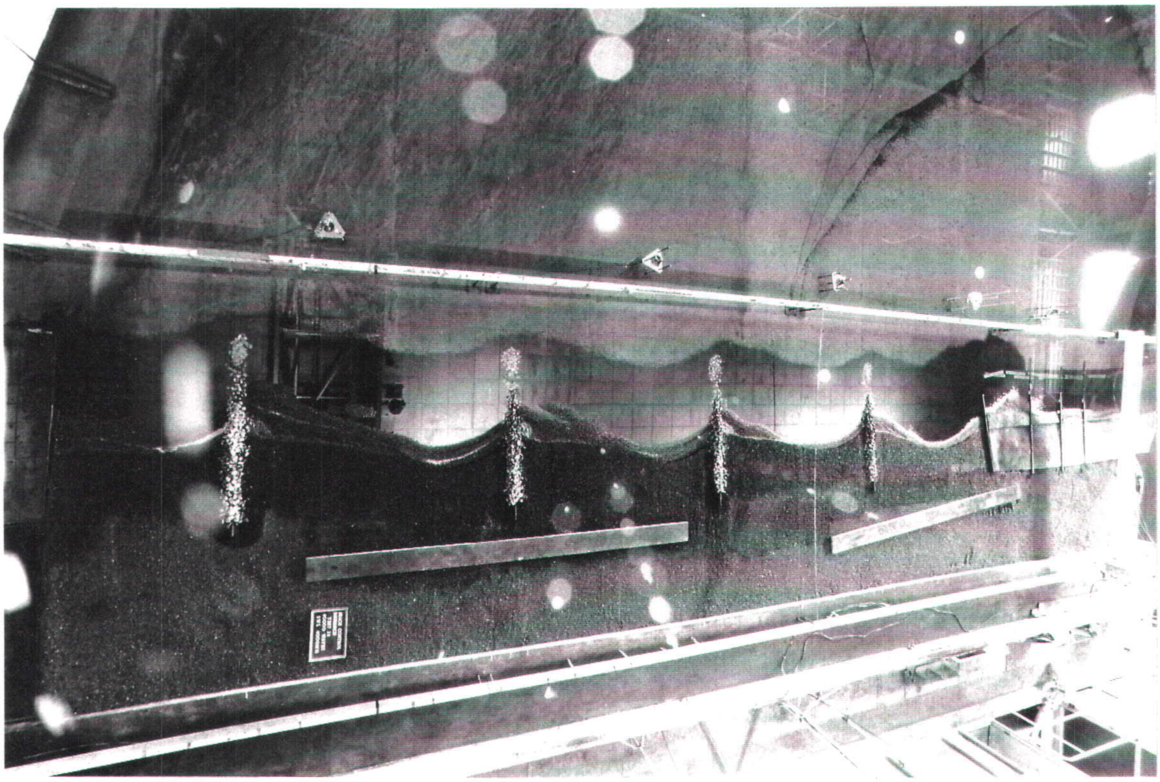


Plate A22 Final beach development (Test 28)

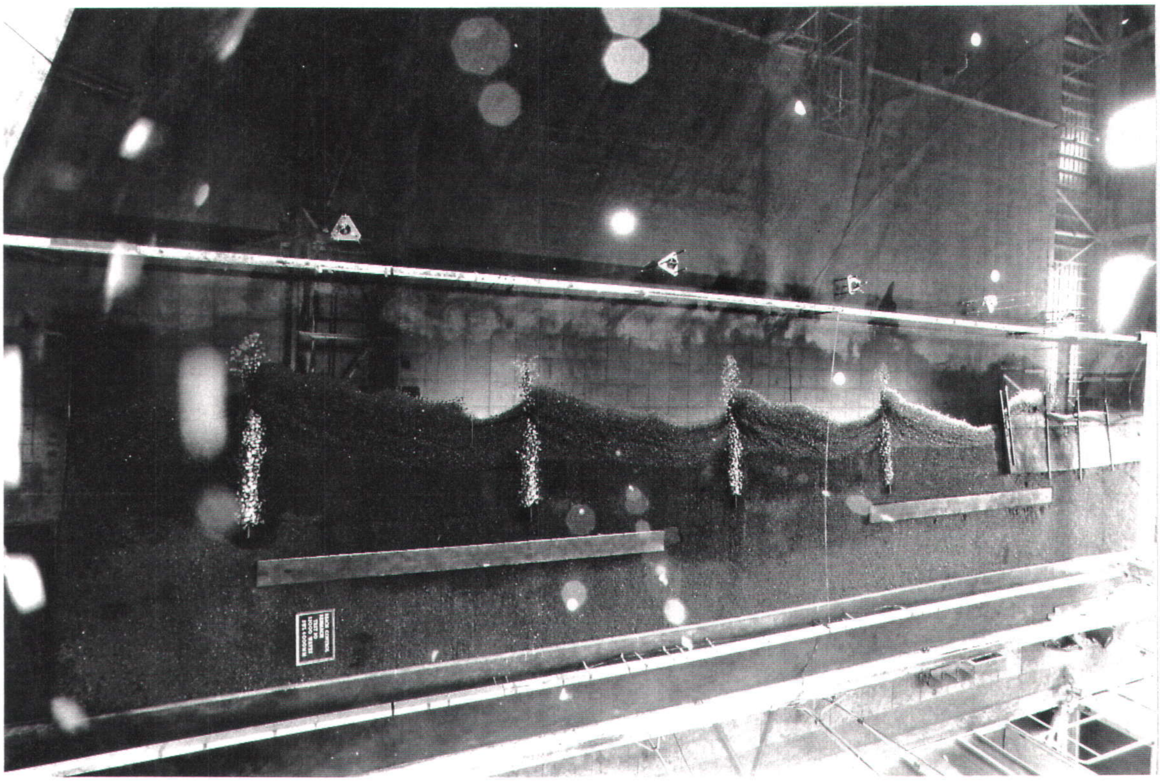
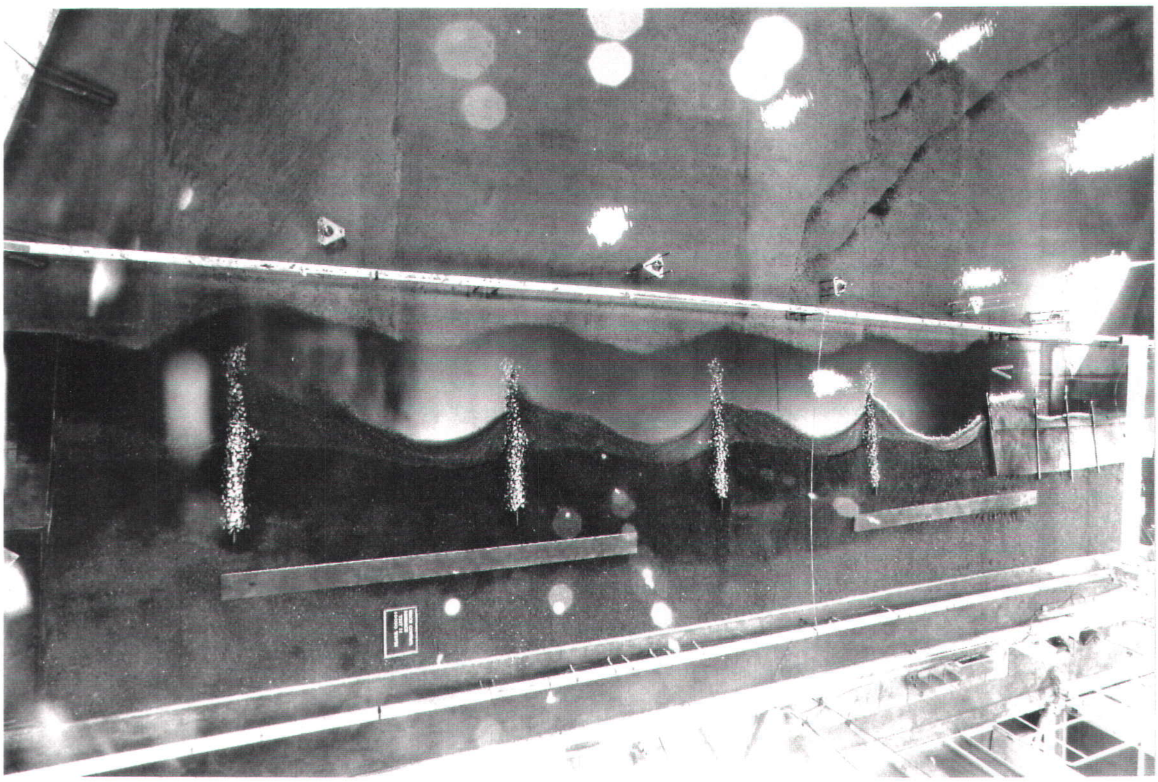


Plate A23 Final beach development (Test 30)



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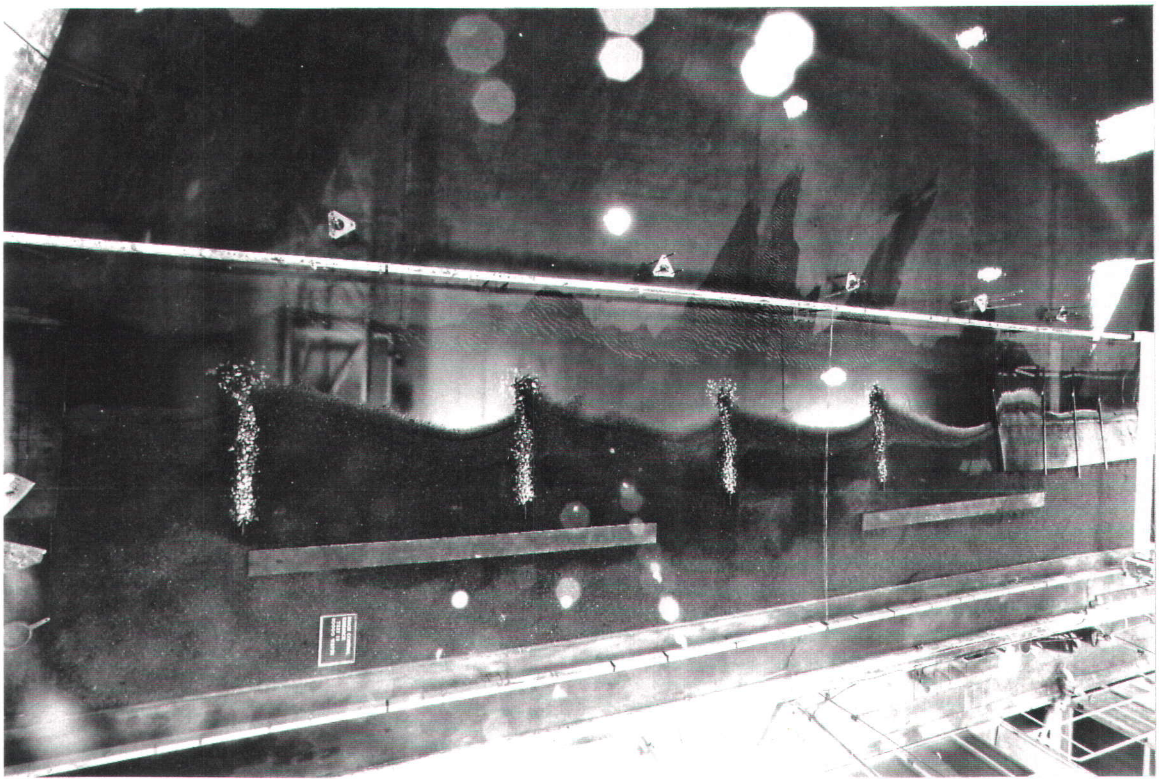


Plate A25 Final beach development (Test 32.1)

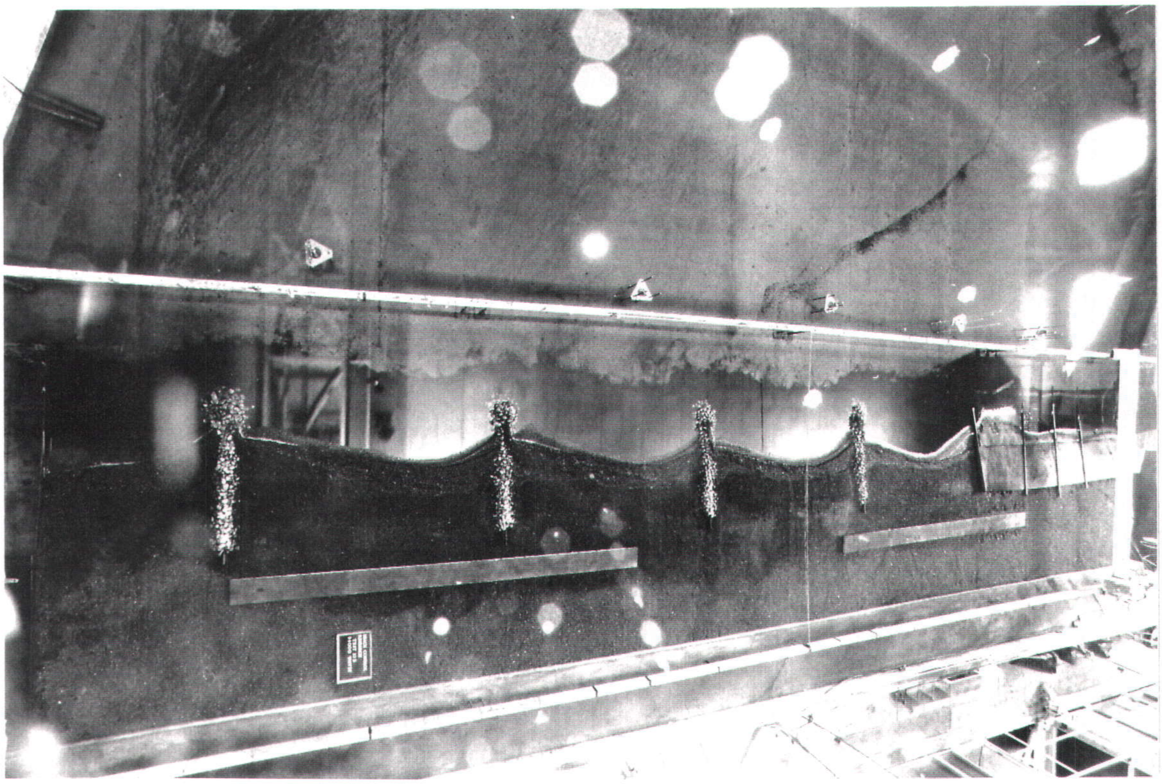


Plate A26 Final beach development (Test 32.2)

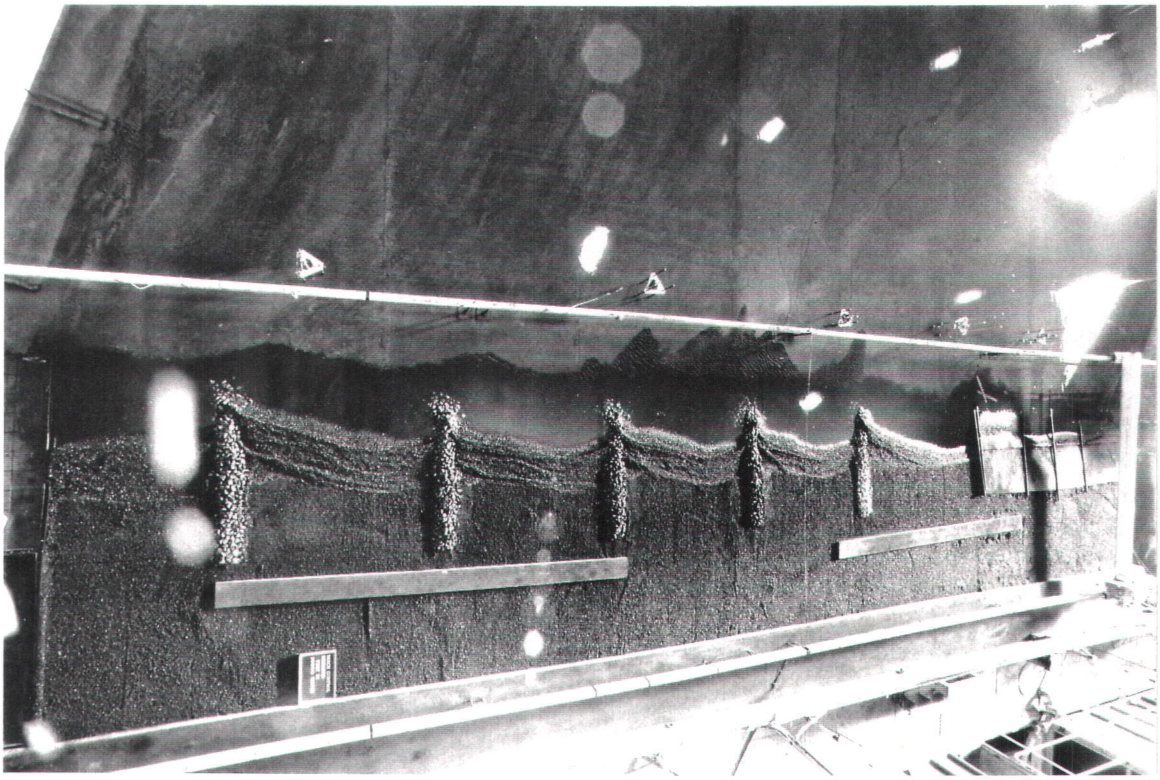
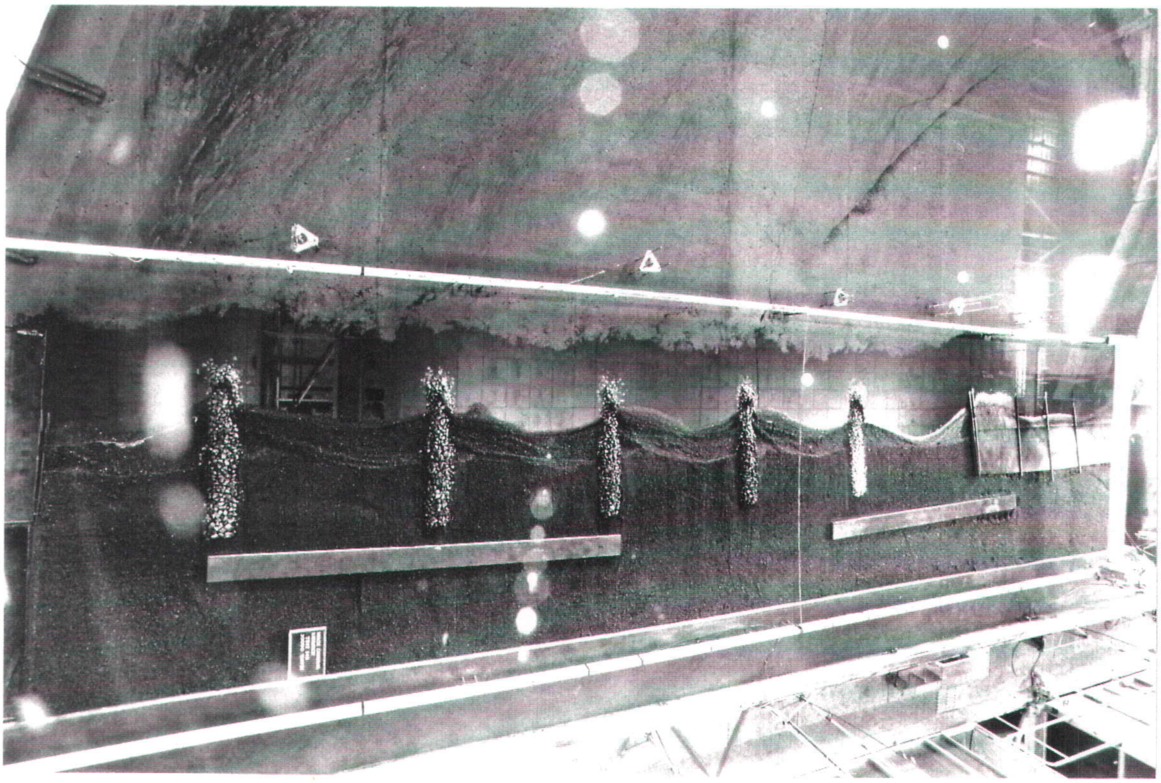


Plate A27 Final beach development (Test 34.1)



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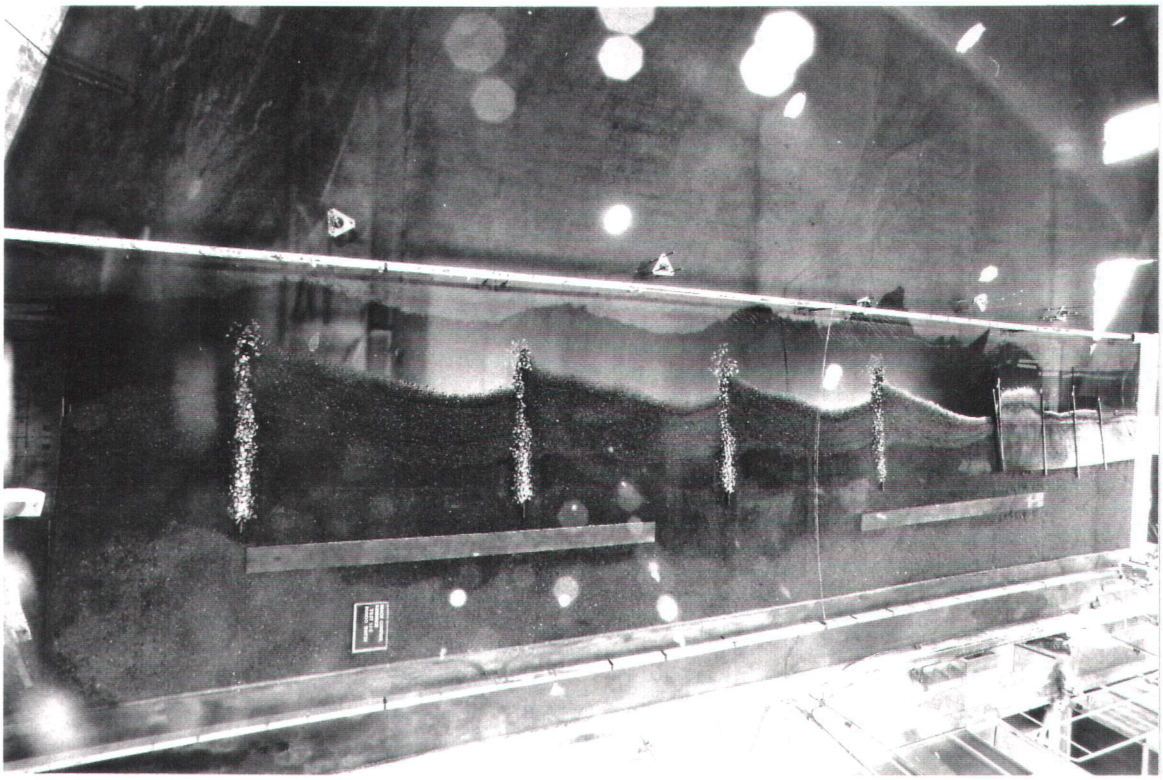


Plate A29 Final beach development (Test 32.3)

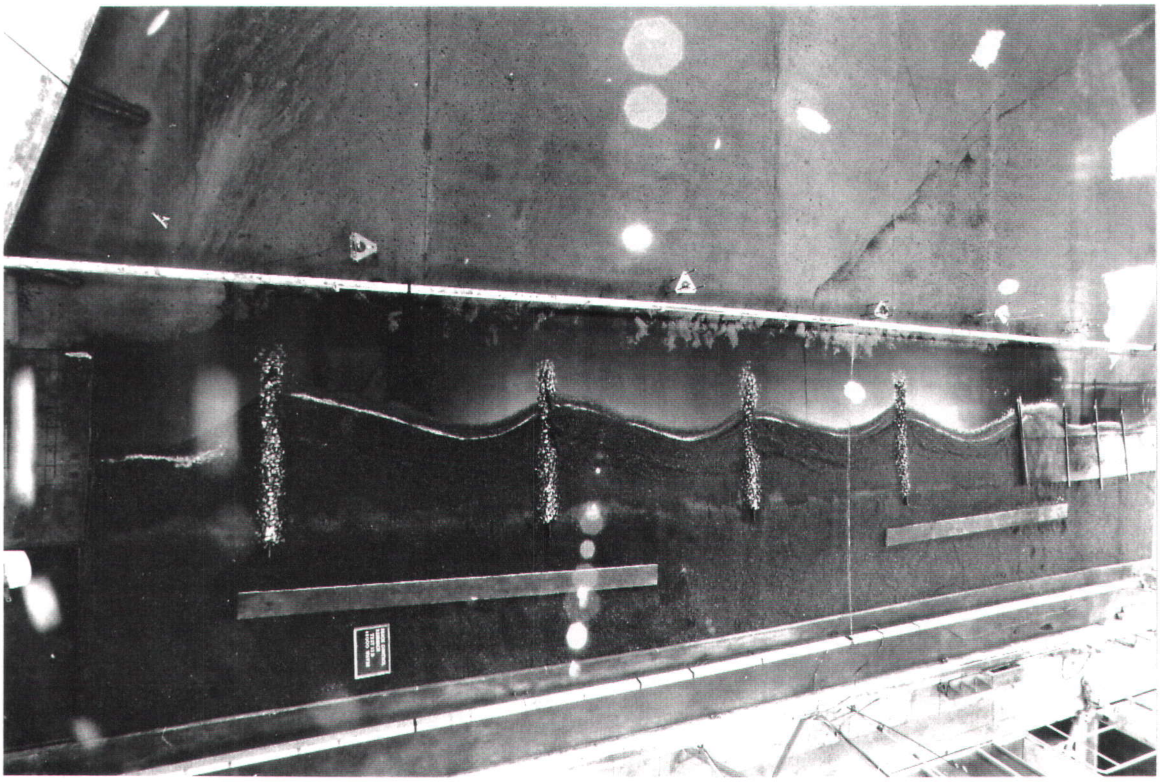
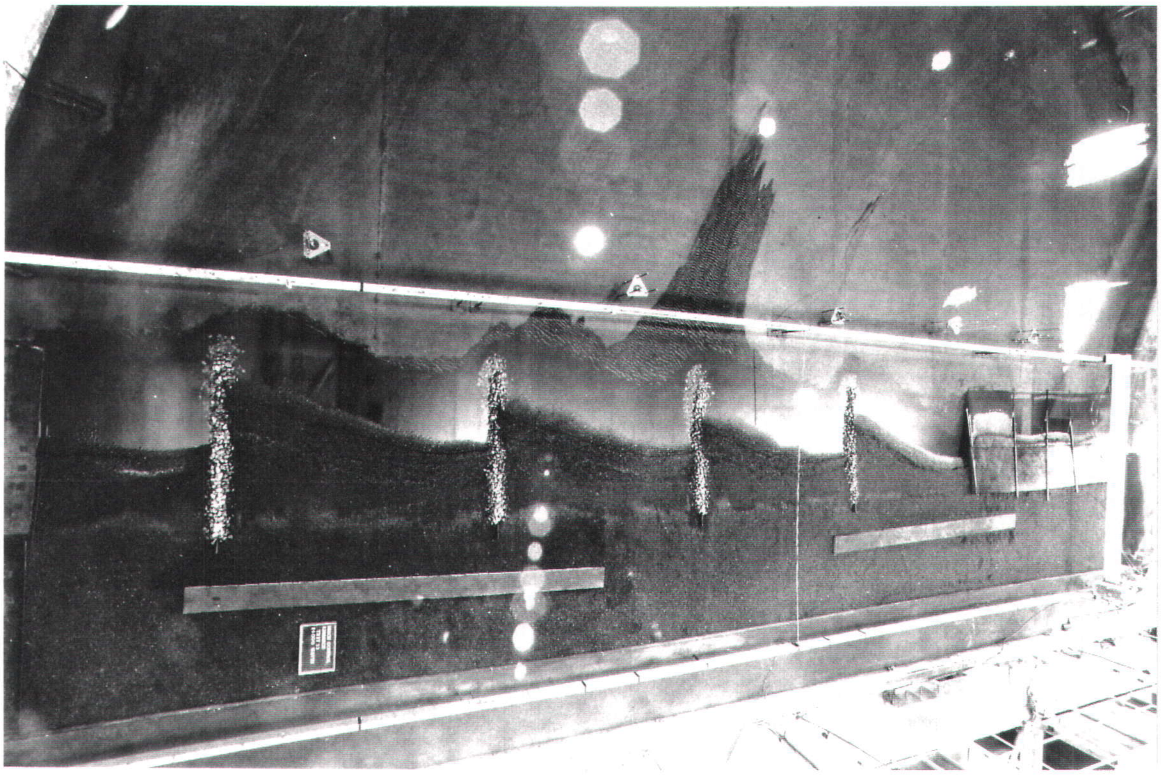


Plate A30 Final beach development (Test 32.4)



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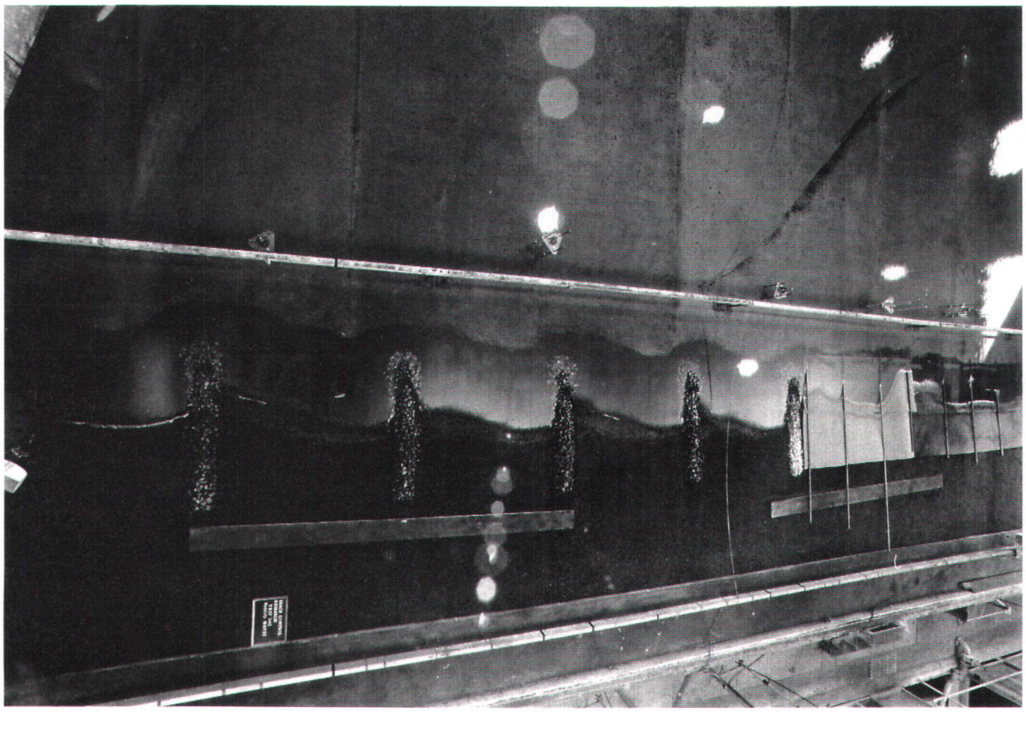

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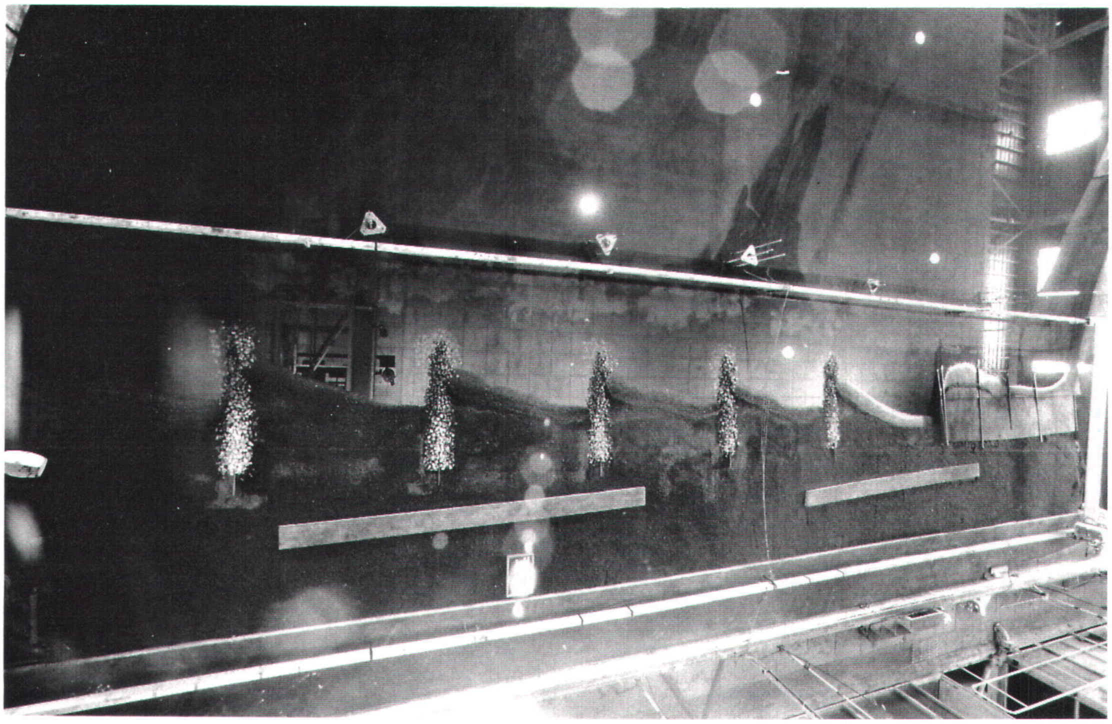


Plate A33 Final beach development (Test 42)

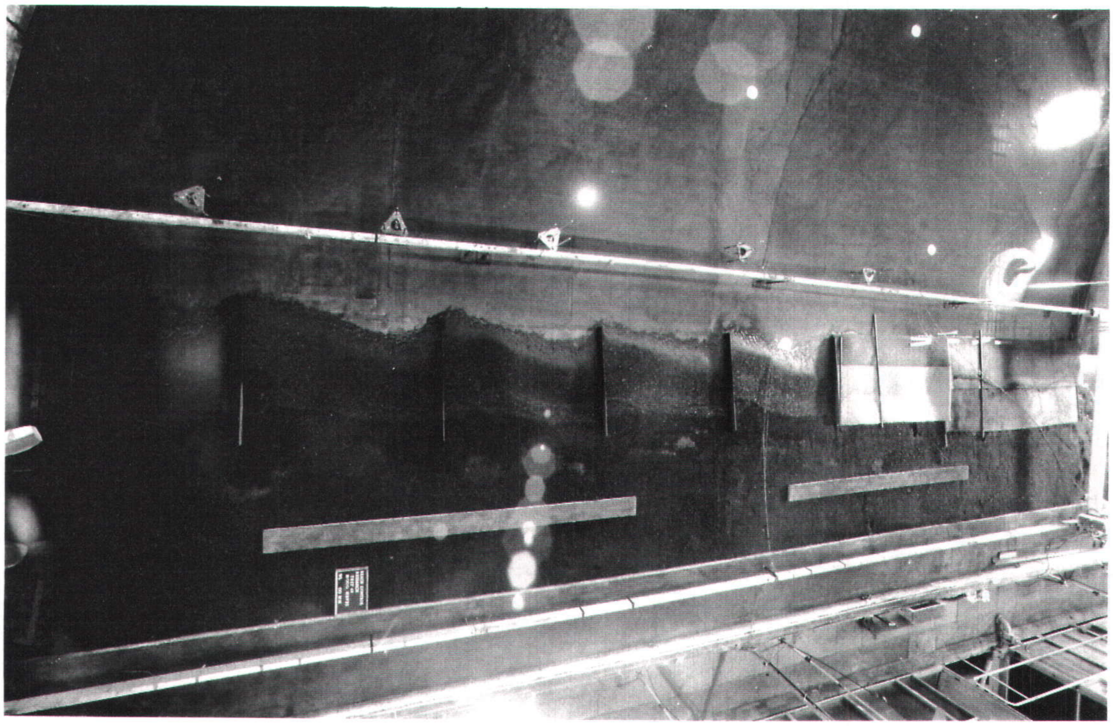


Plate A34 Final beach development (Test 43)

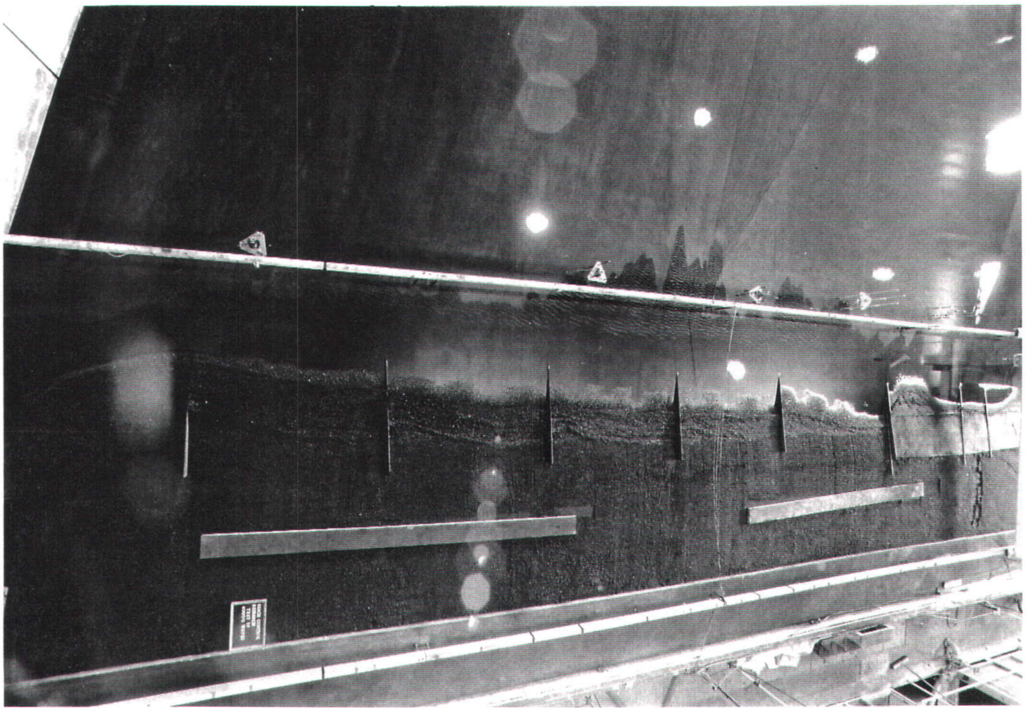
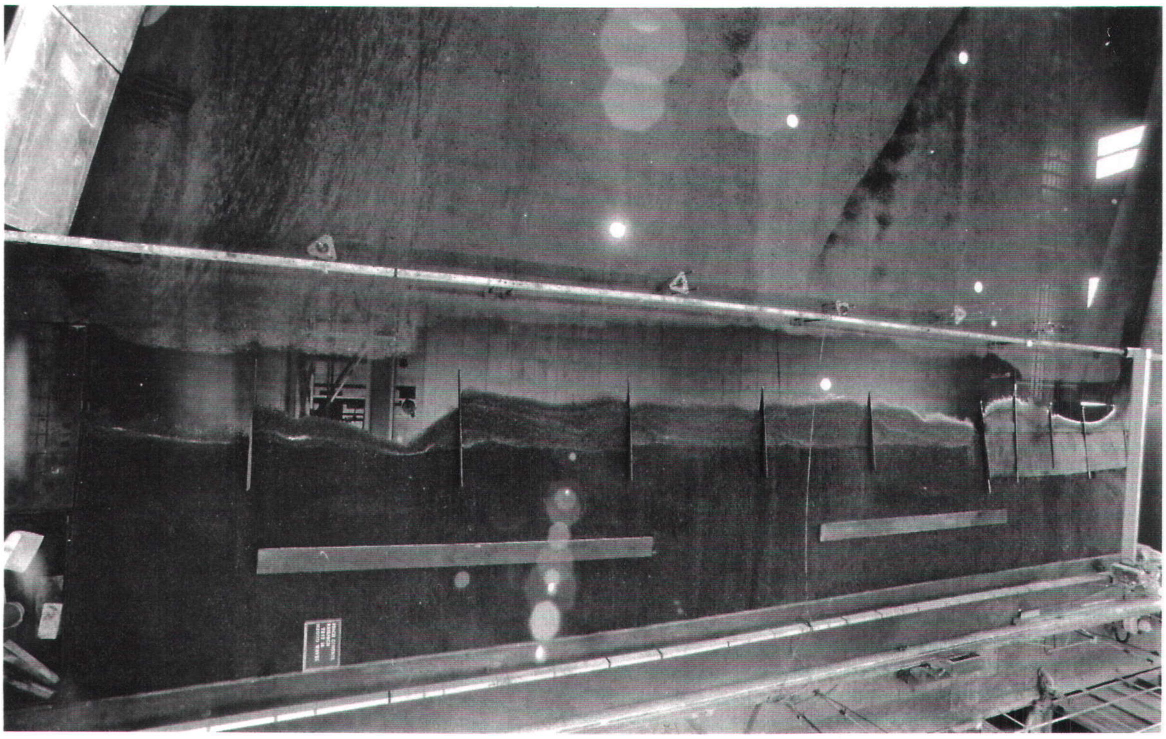


Plate A35 Final beach development (Test 37)



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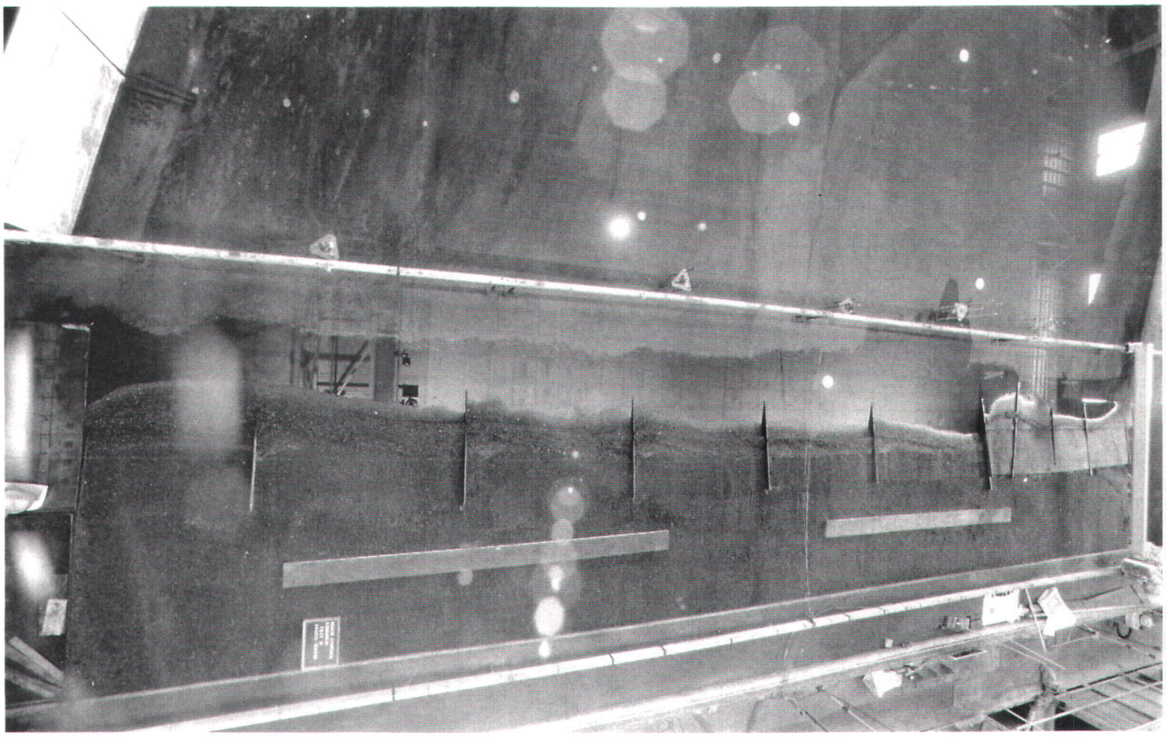

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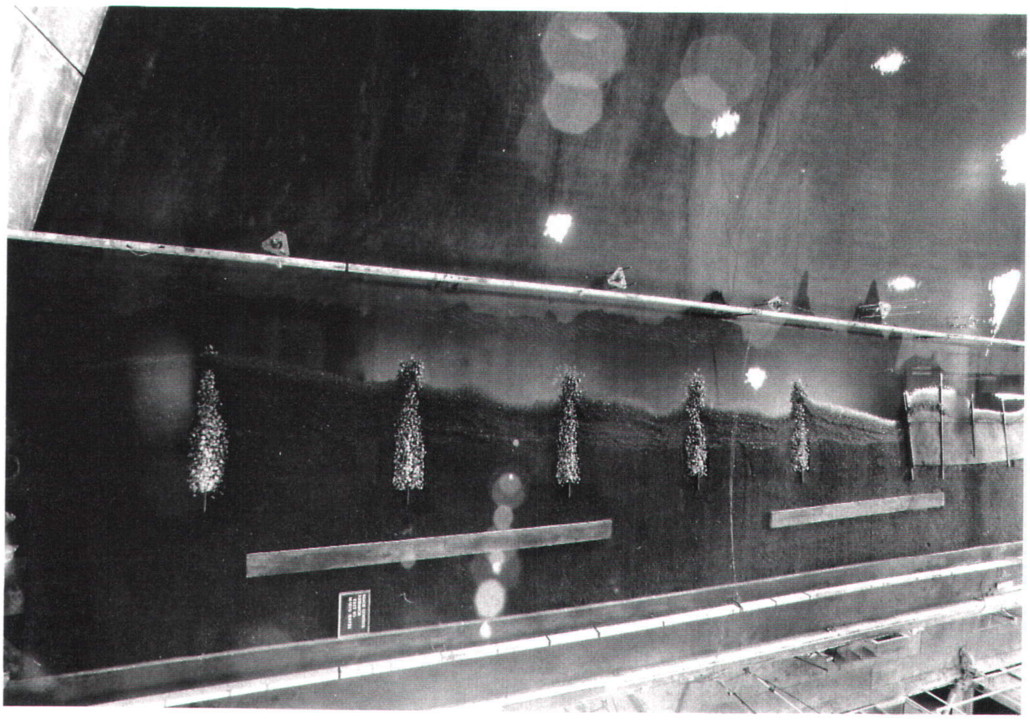


Plate A38 Final beach development (Test 40)

Plate A37 Final beach development (Test 39)

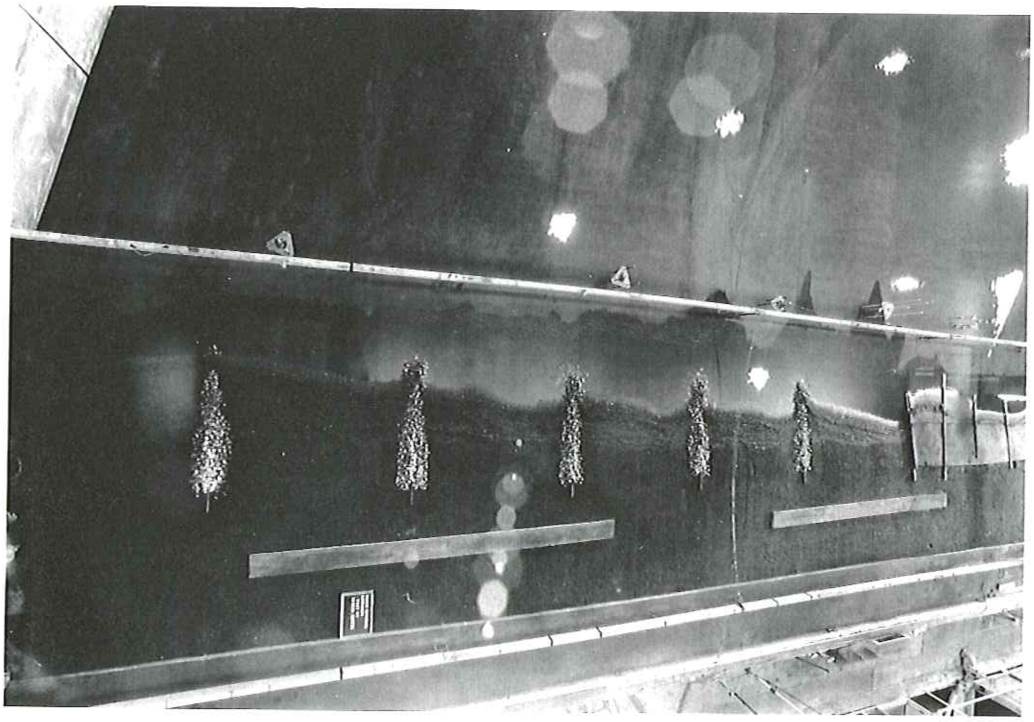


Plate A38 Final beach development (Test 40)

Appendix 6

Beach plan shapes - Breakwaters

Appendix 6 Beach plan shapes - Breakwaters

Contents

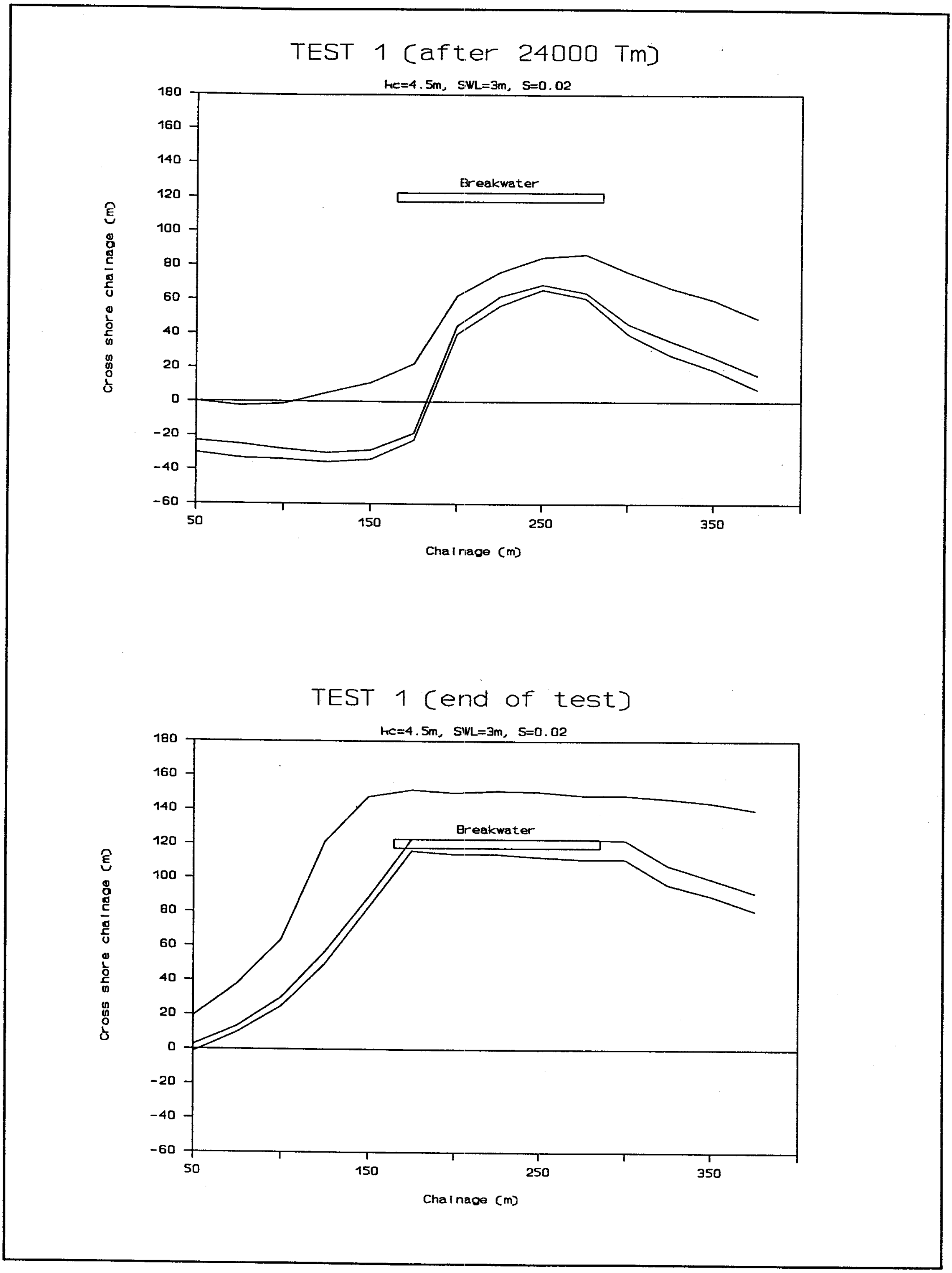
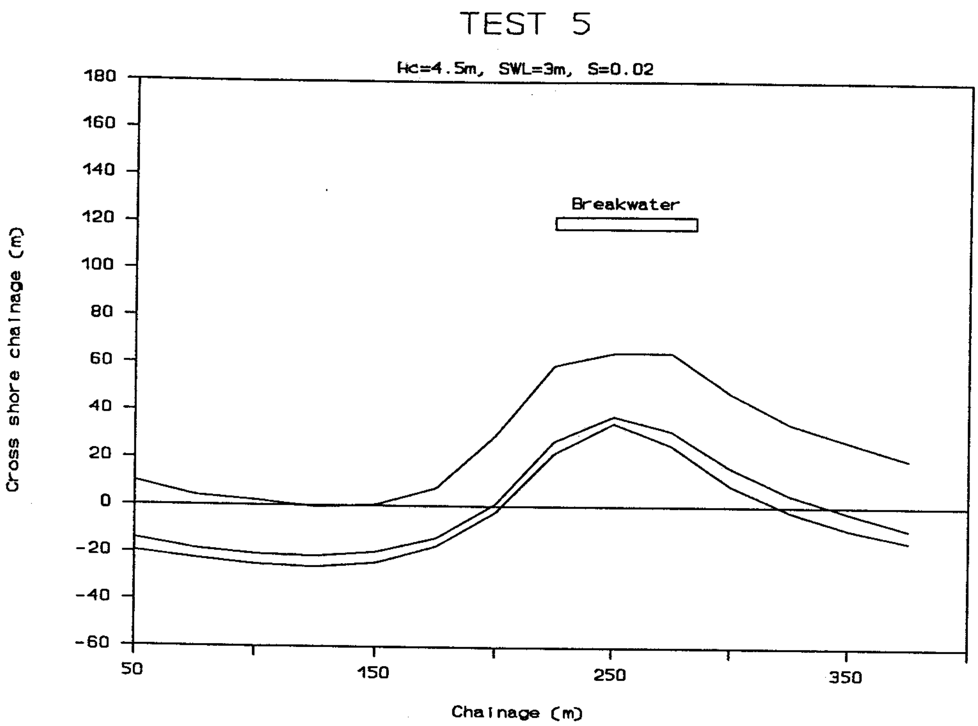


Figure A1 Beach plan shape - Test 1:after 24000 Tm and final



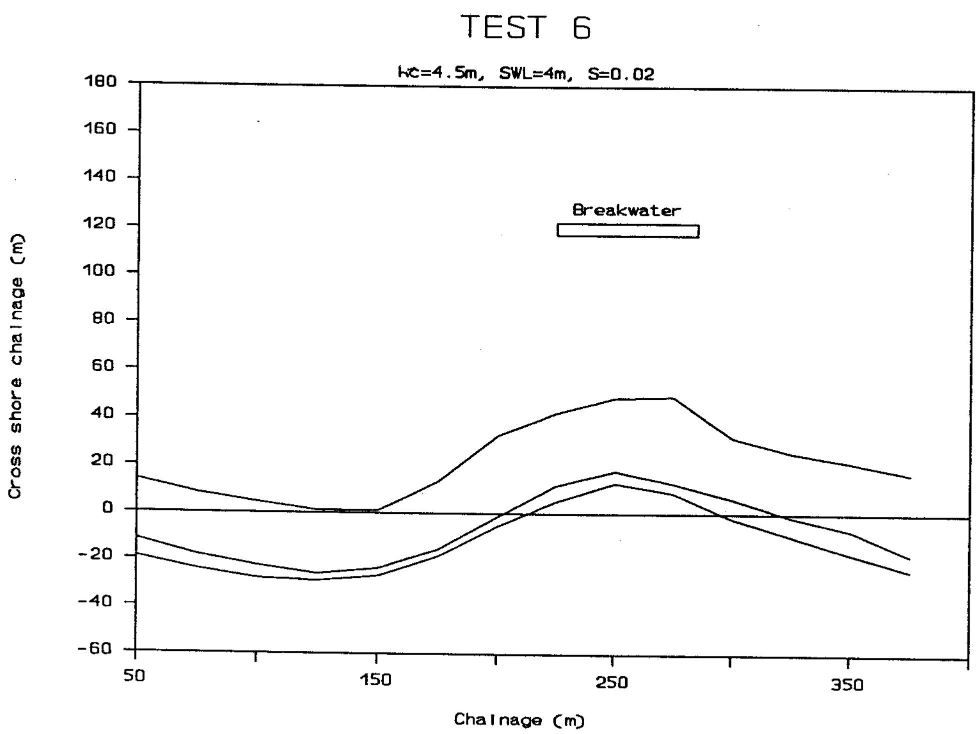


Figure A2 Final beach plan shape - Tests 5 and 6

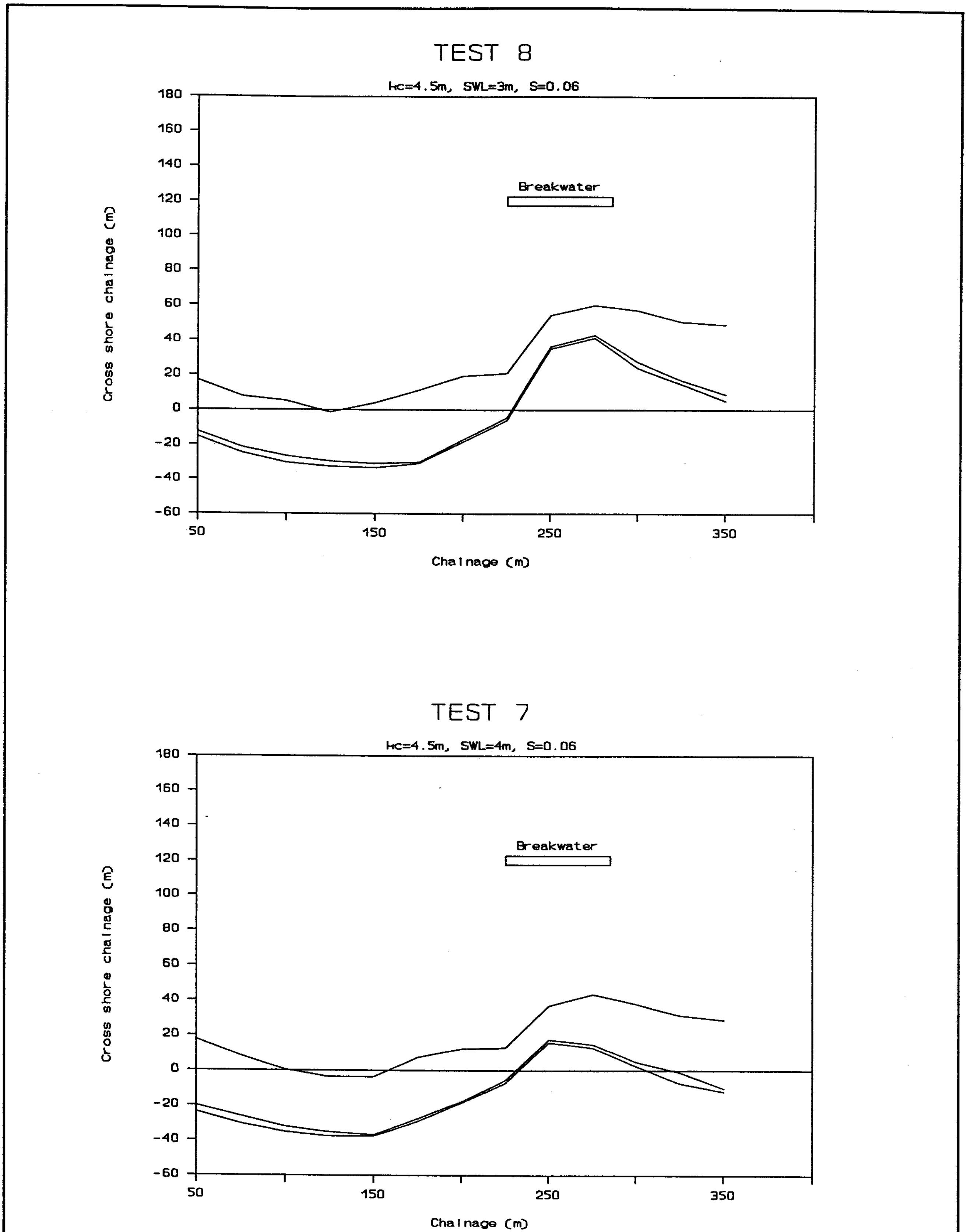
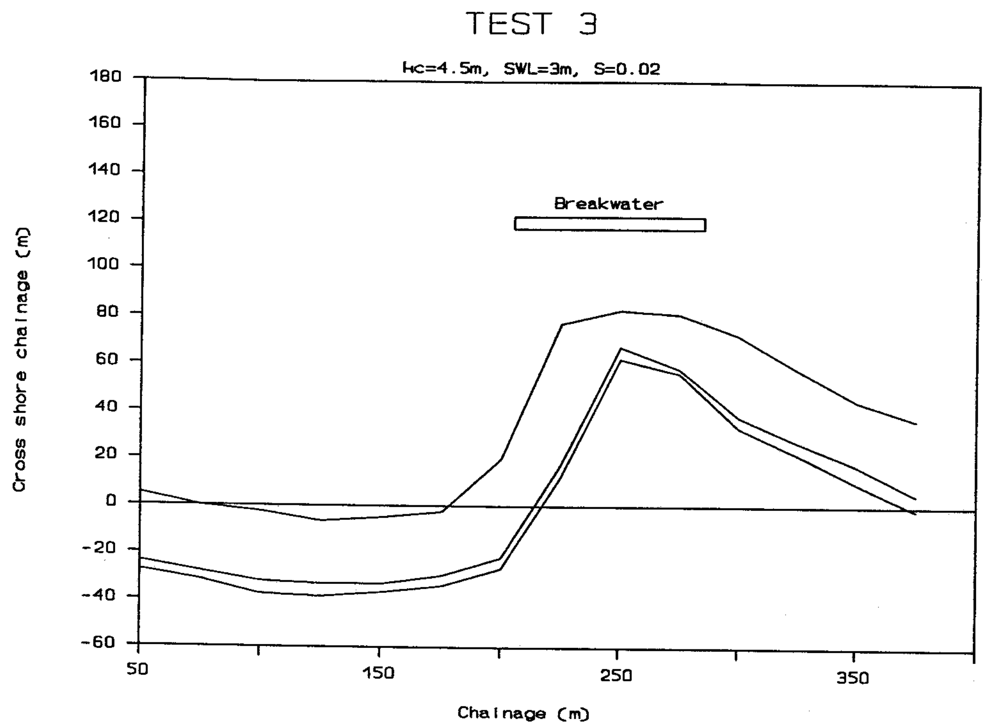


Figure A3 Final beach plan shape - Tests 8 and 7



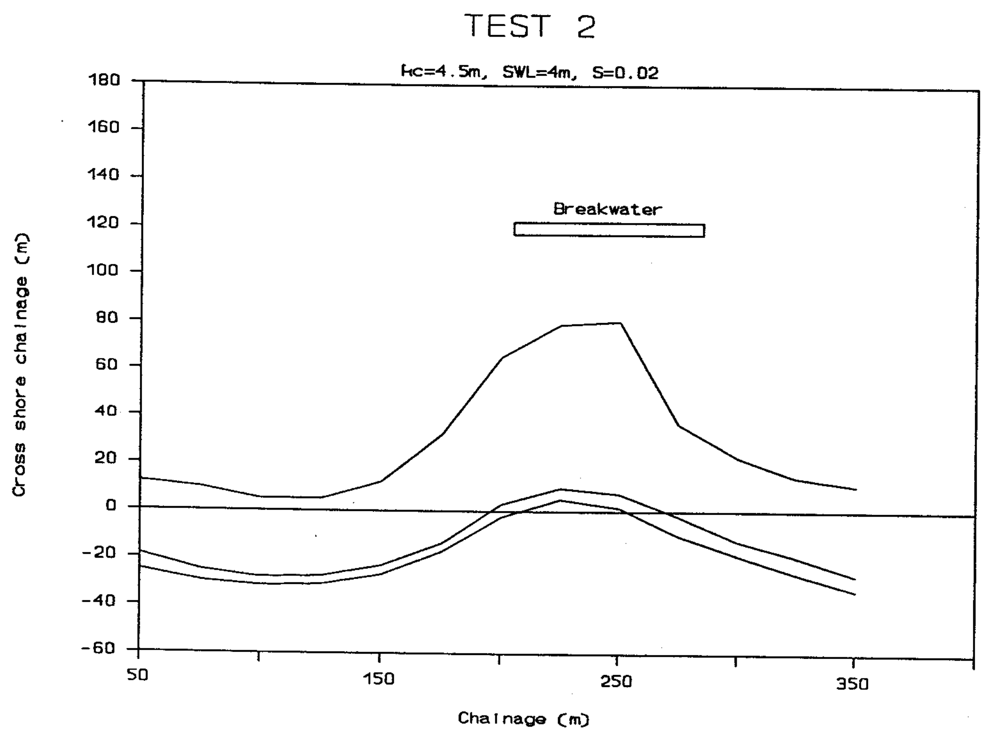


Figure A4 Final beach plan shape - Tests 3 and 2

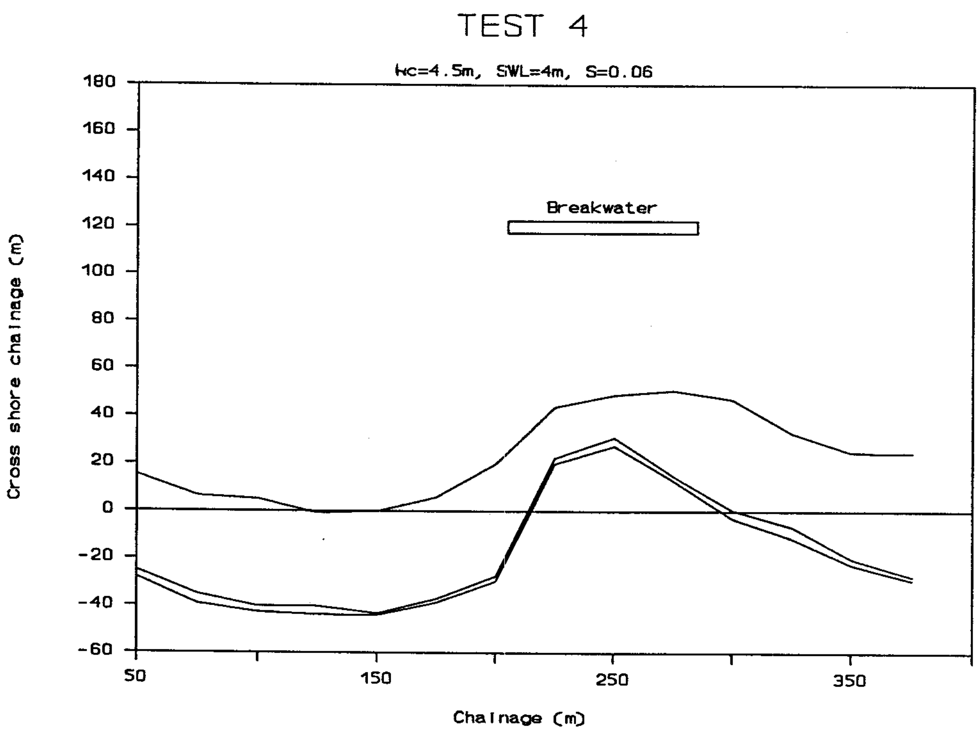
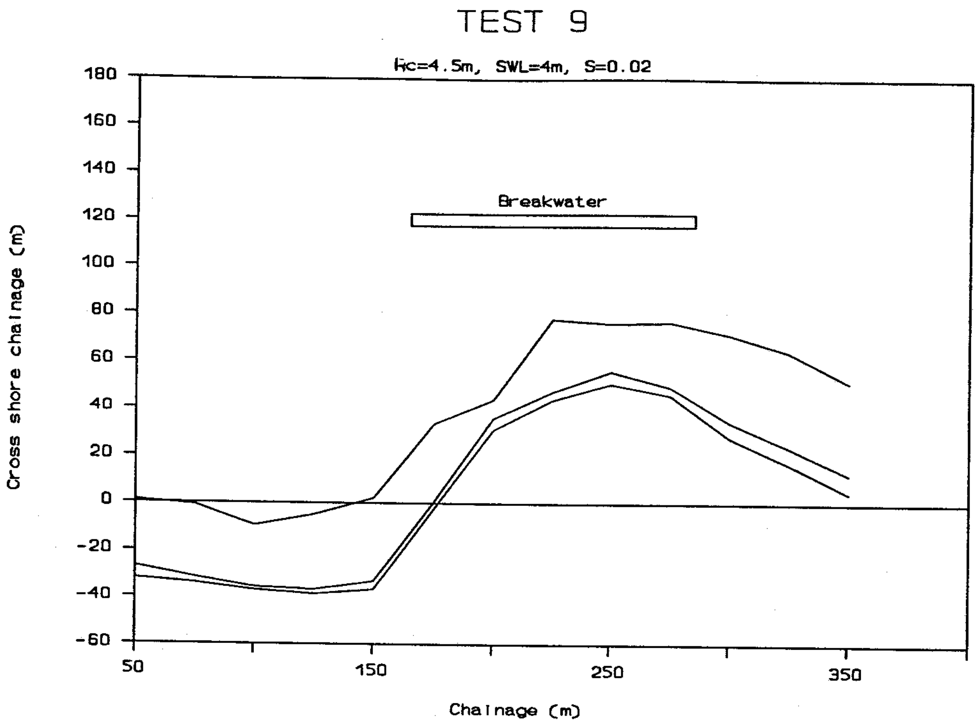


Figure A5 Final beach plan shape - Test 4



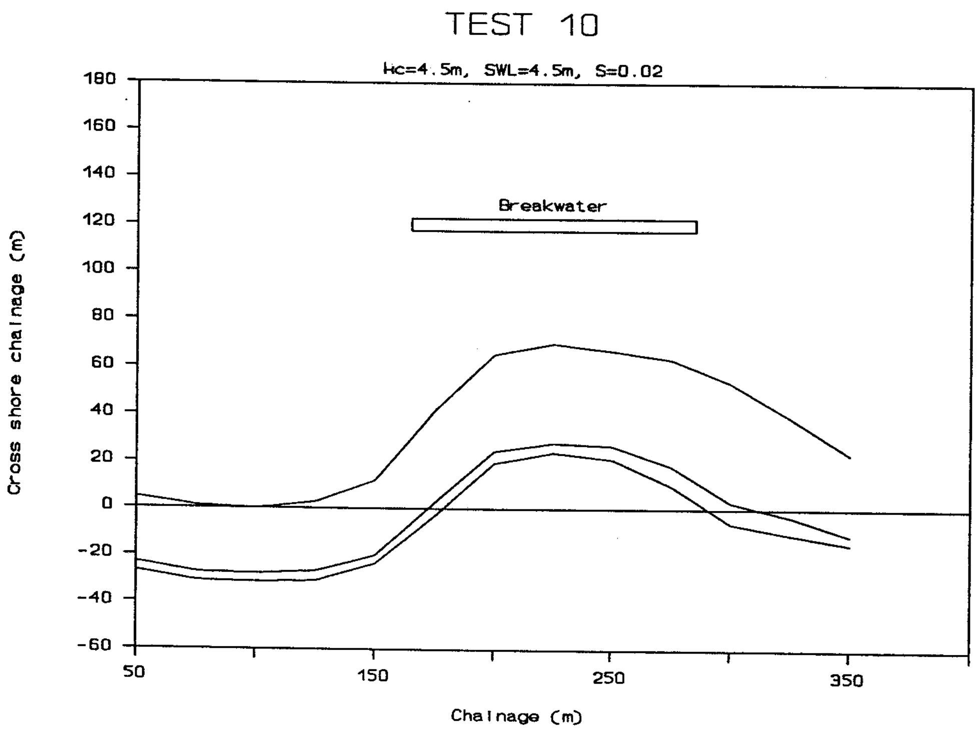
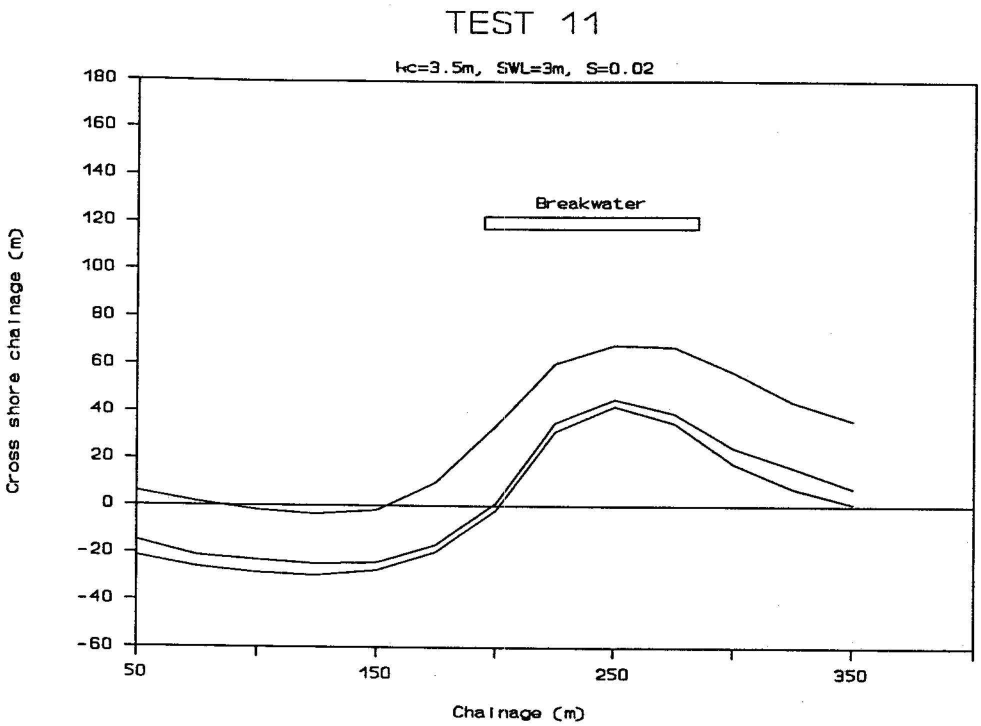


Figure A6 Final beach plan shape - Tests 9 and 10



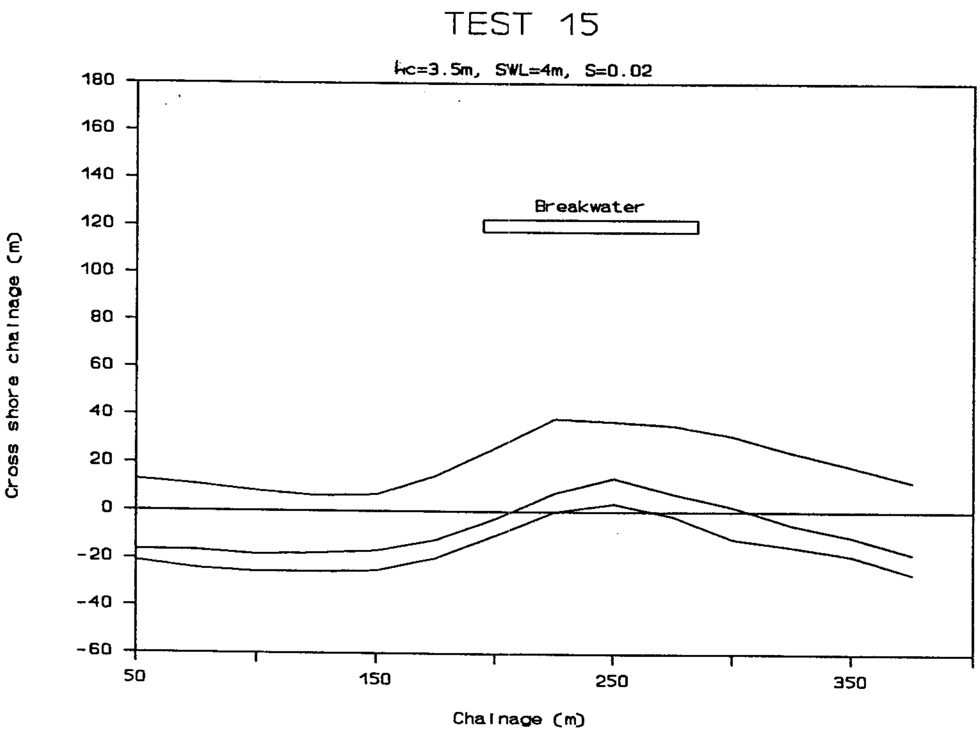
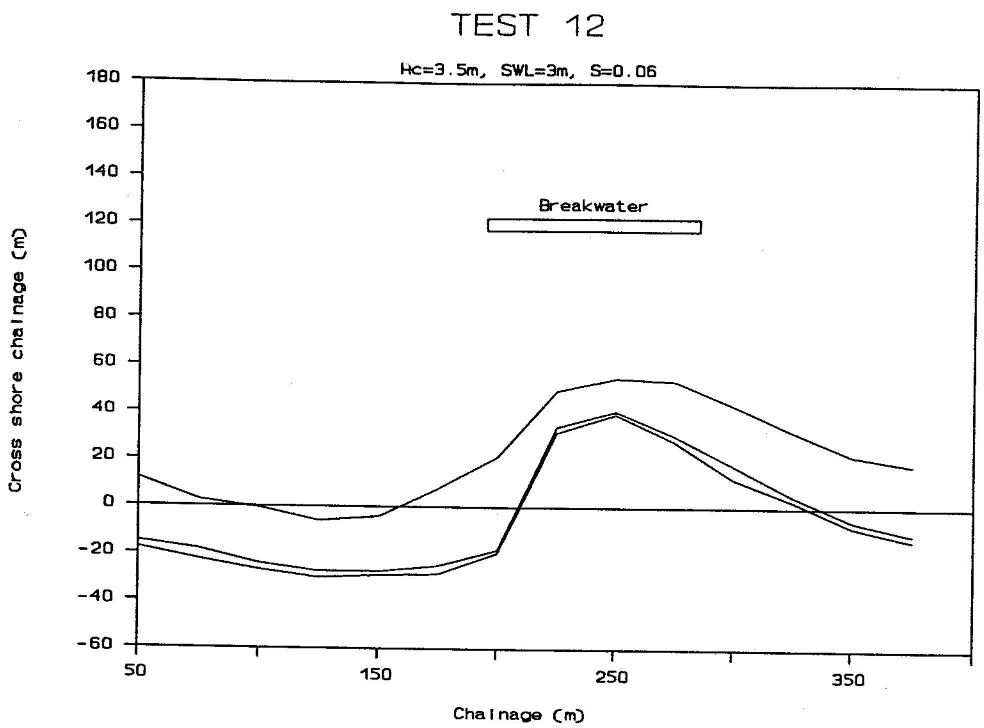


Figure A7 Final beach plan shape - Tests 11 and 15



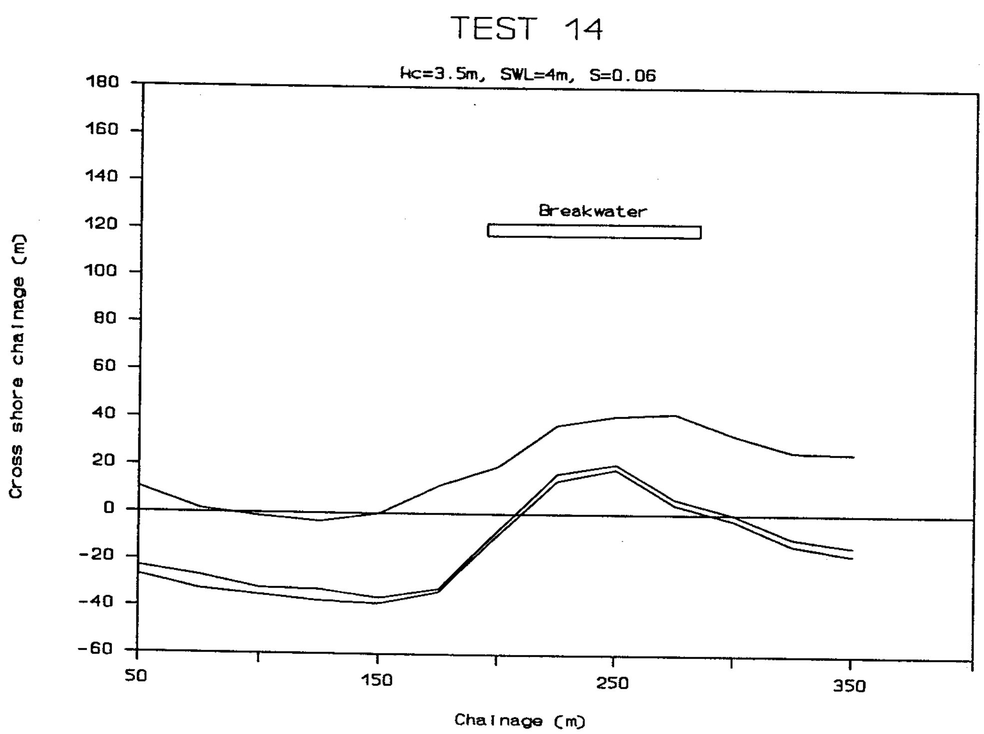


Figure A8 Final beach plan shape - Tests 12 and 14

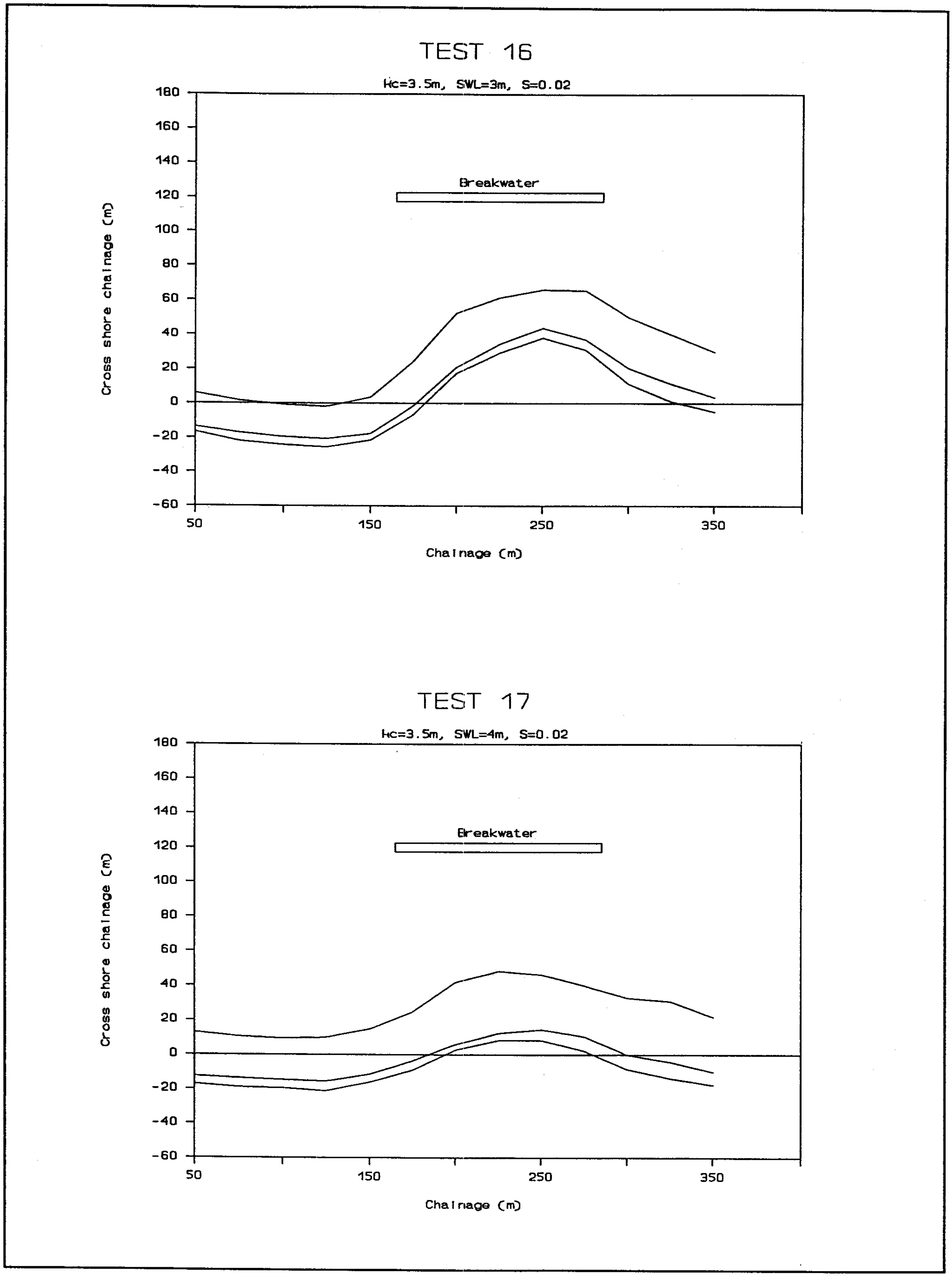
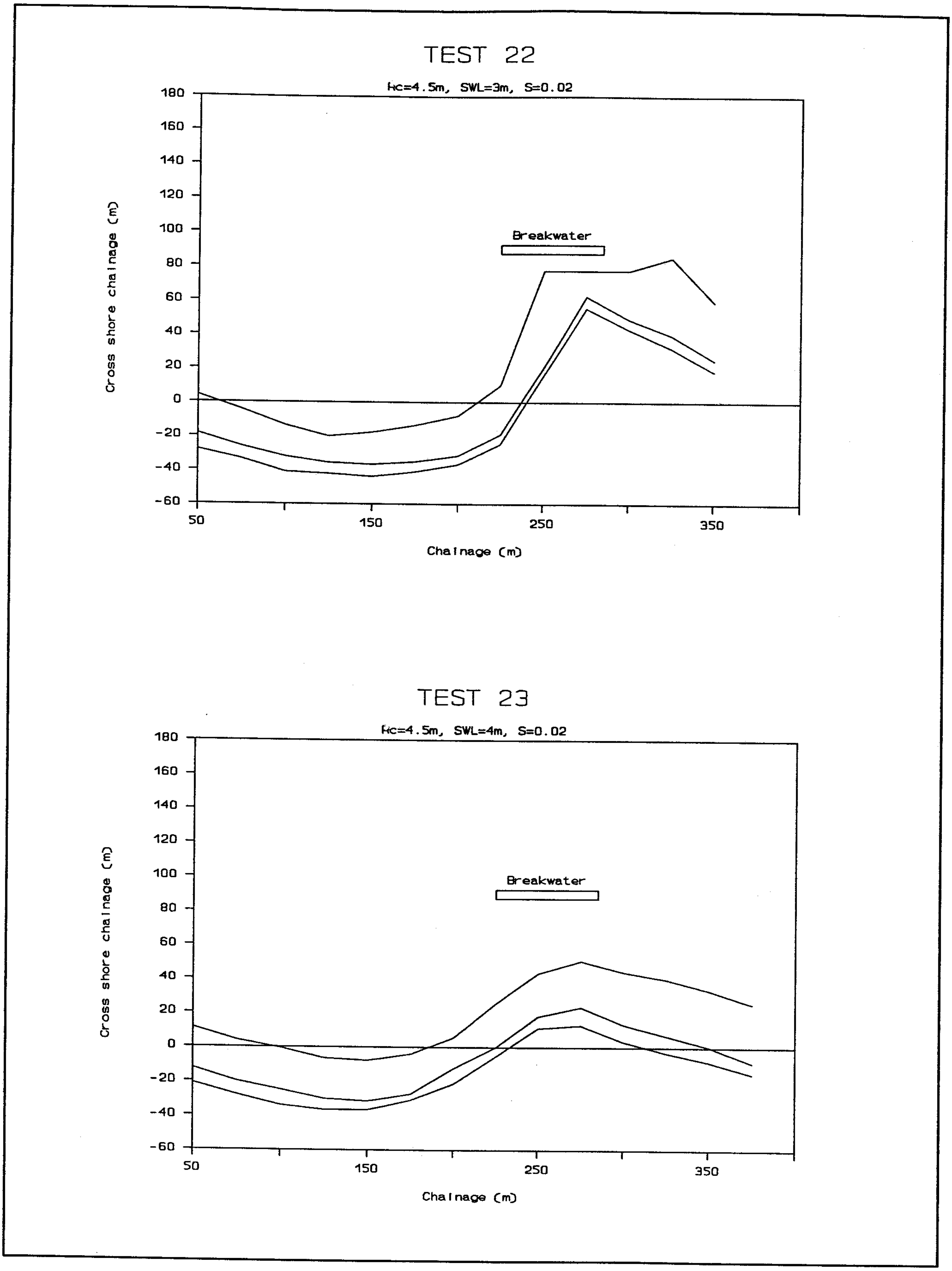


Figure A9 Final beach plan shape - Tests 16 and 17



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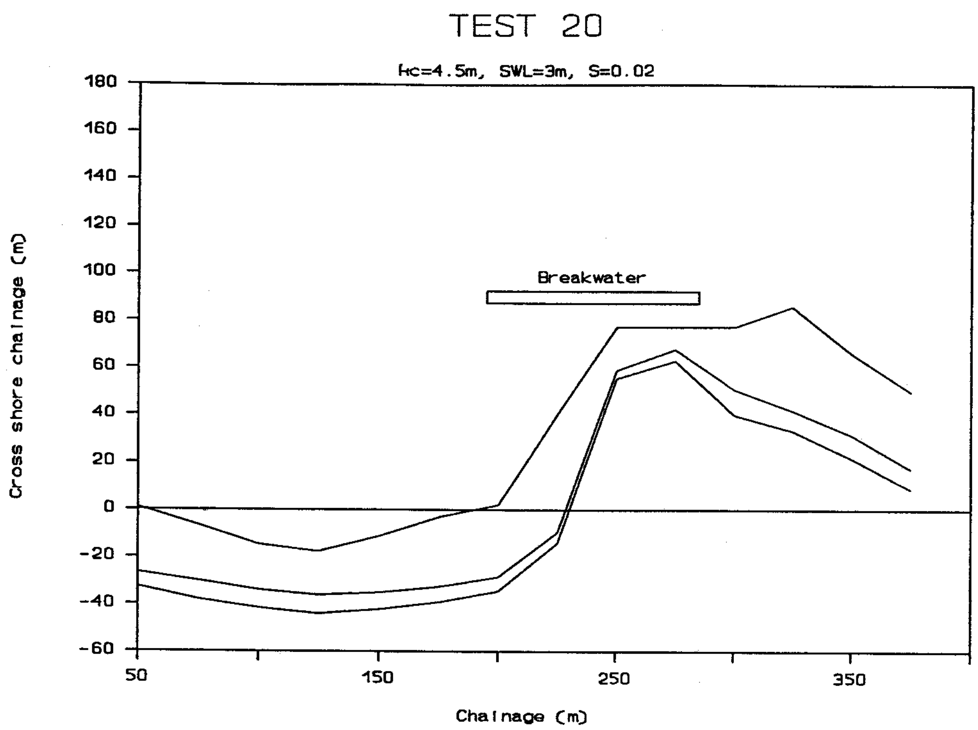


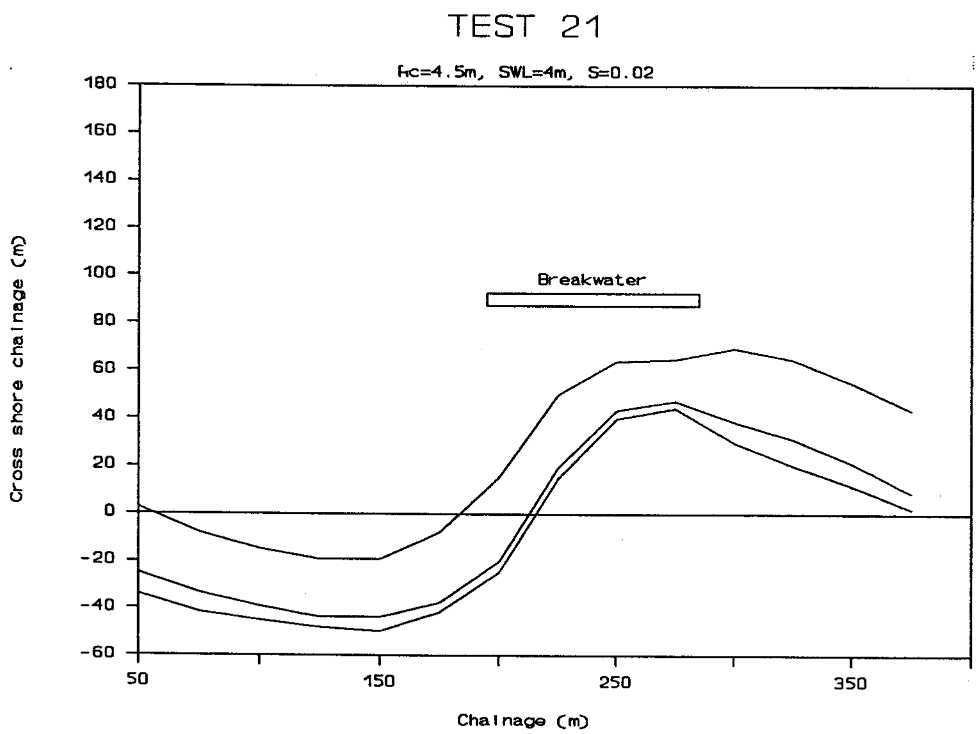
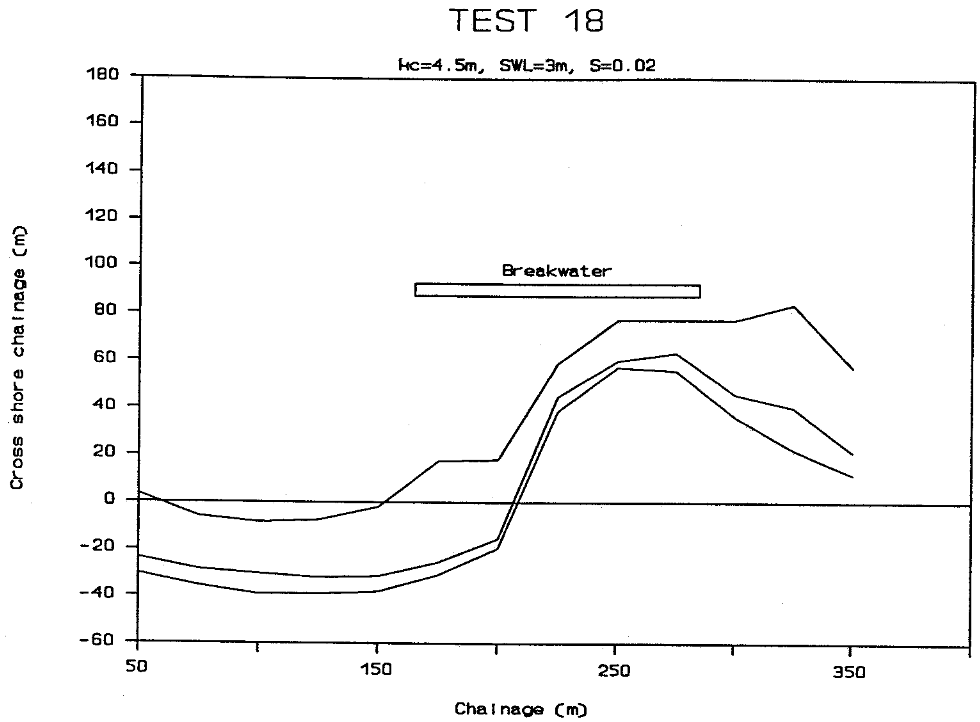
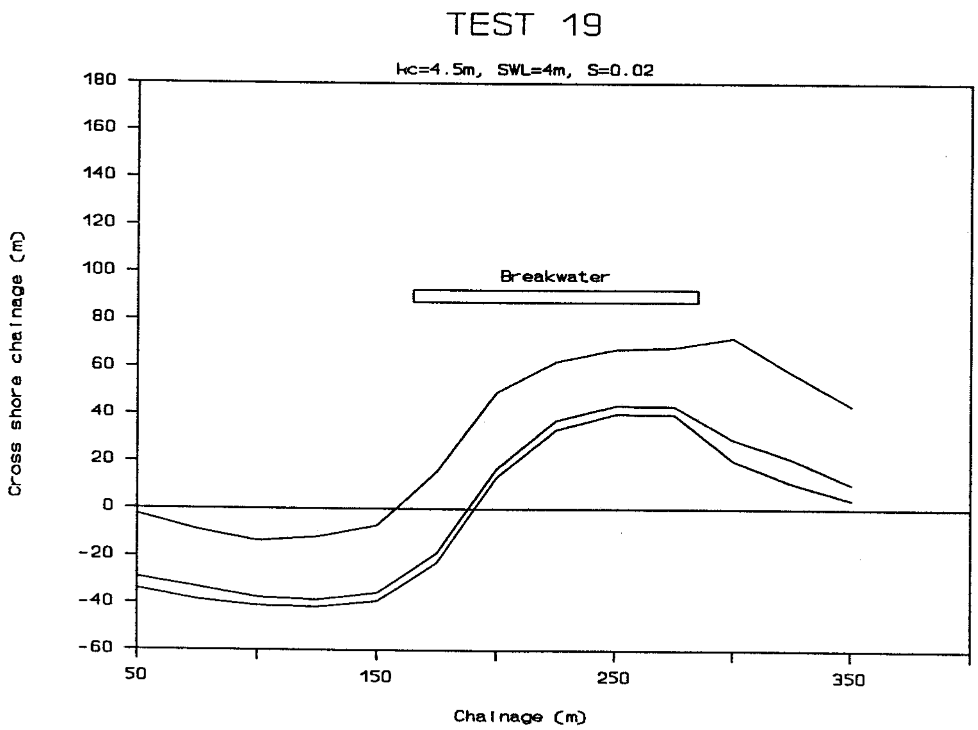


Figure A11 Final beach plan shape - Tests 20 and 21





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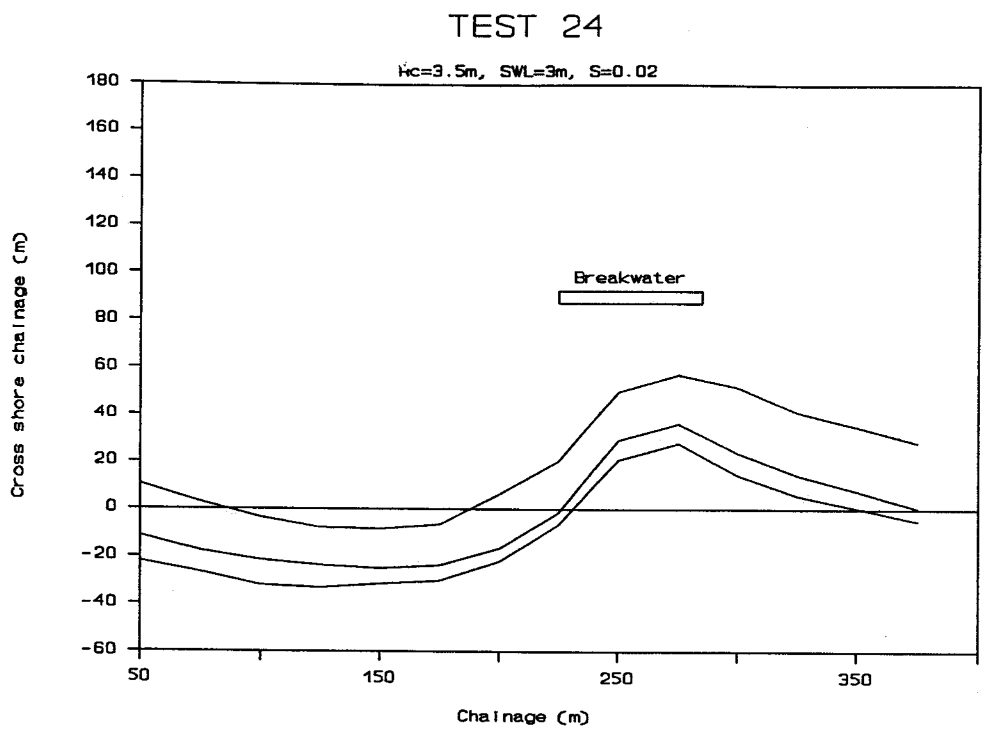


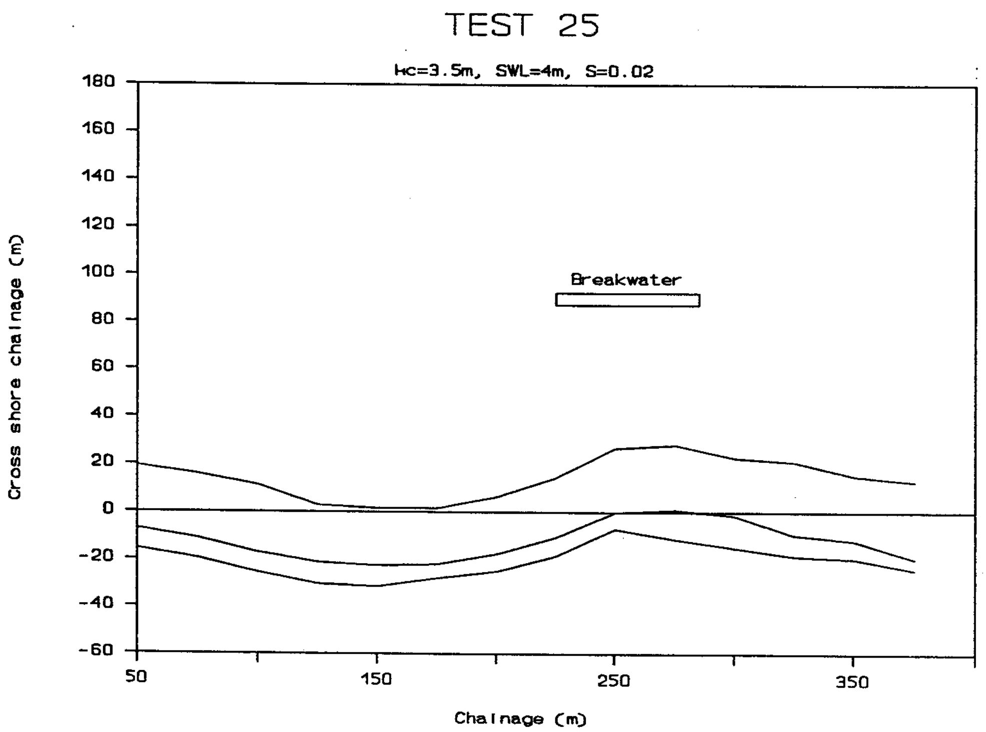
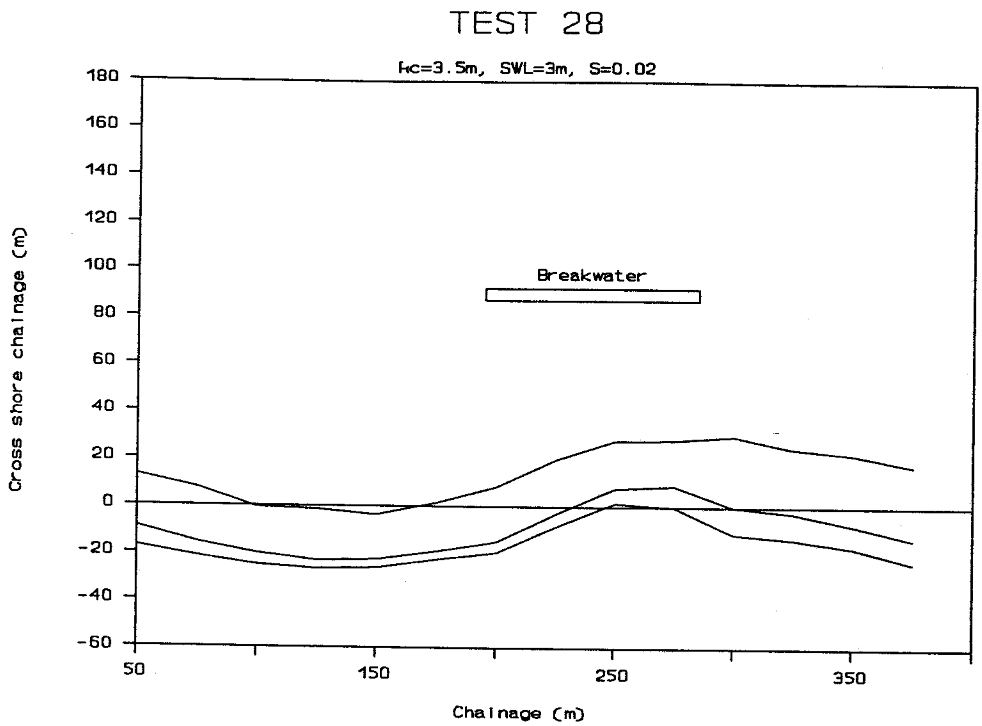


Figure A13 Final beach plan shape - Tests 24 and 25



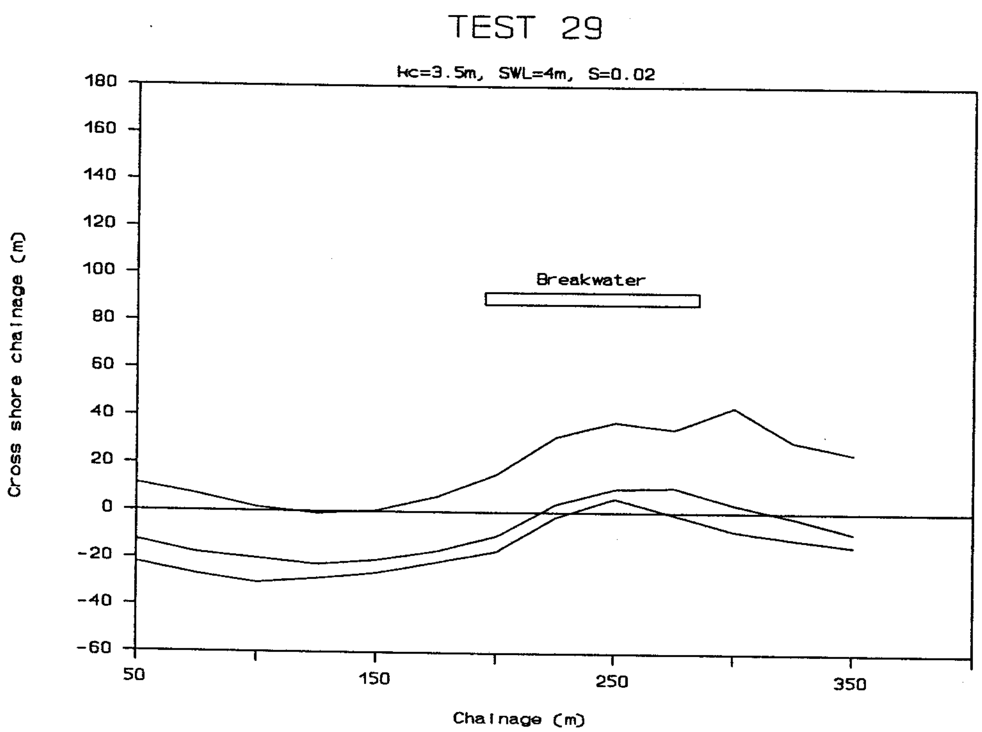
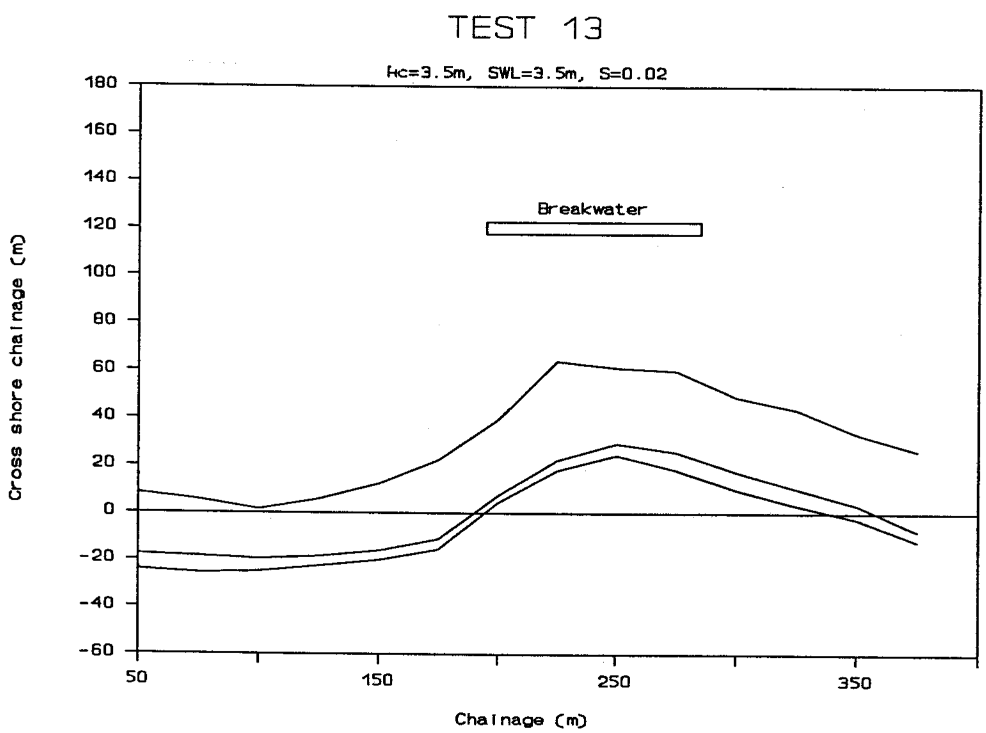


Figure A14 Final beach plan shape - Tests 28 and 29



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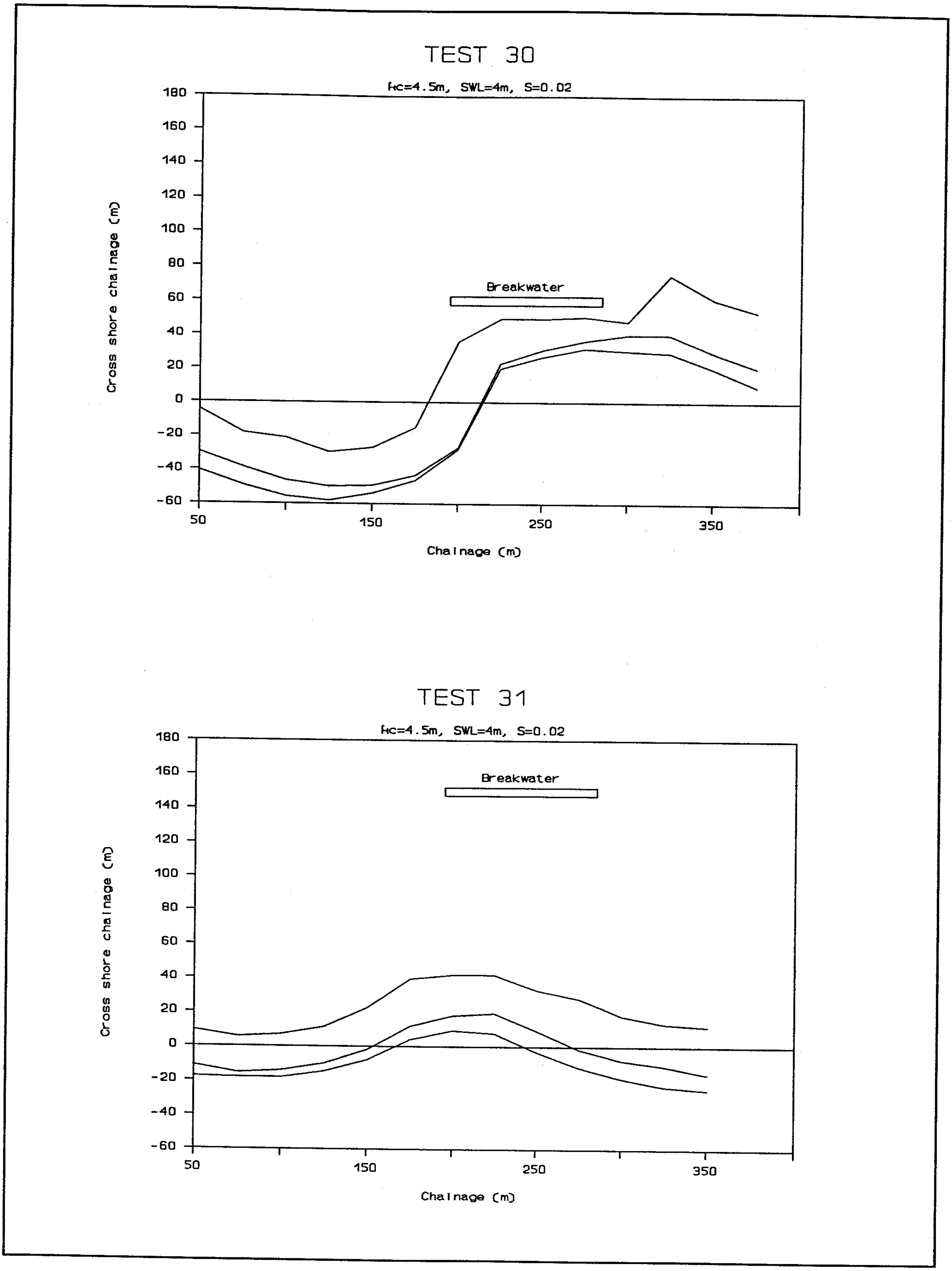
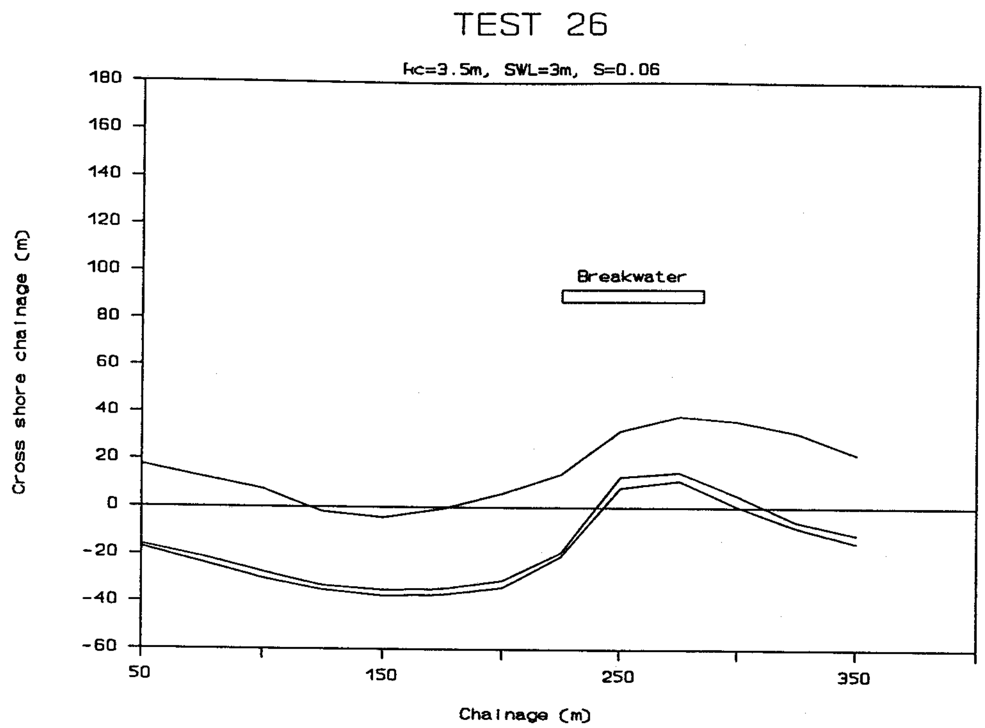


Figure A16 Final beach plan shape - Tests 30 and 31



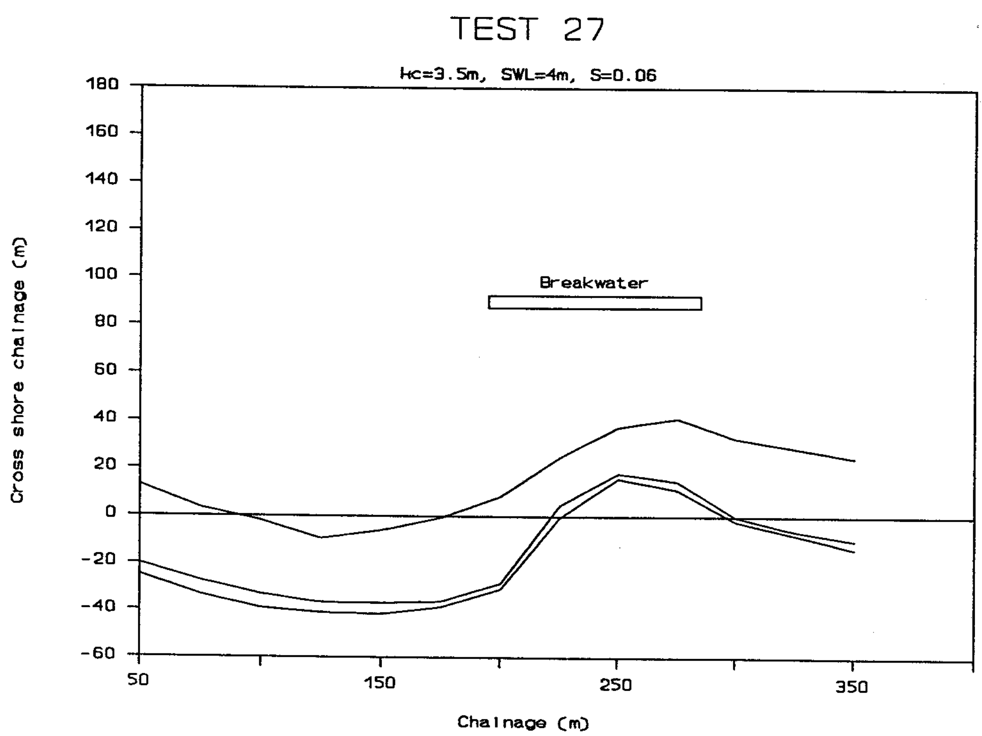
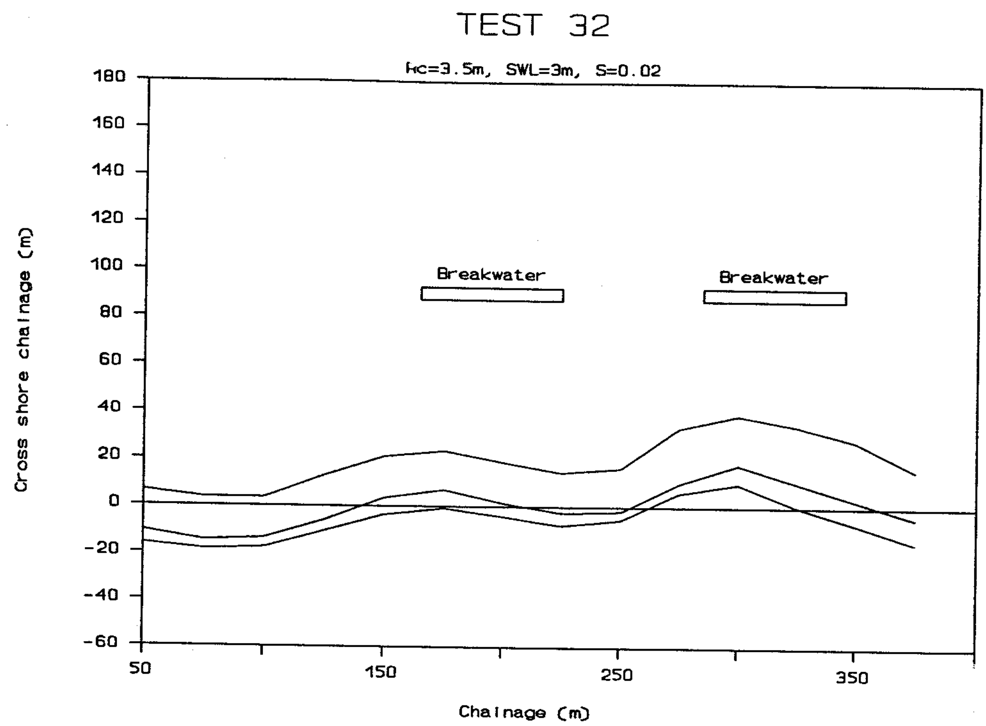


Figure A17 Final beach plan shape - Tests 26 and 27



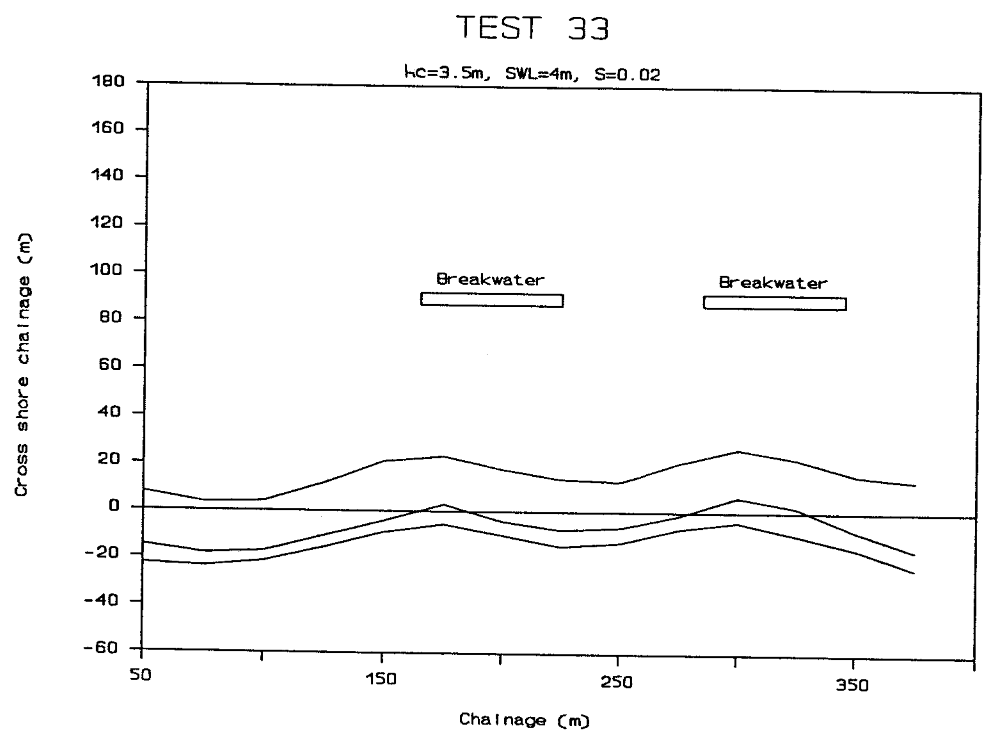


Figure A18 Final beach plan shape - Tests 32 and 33

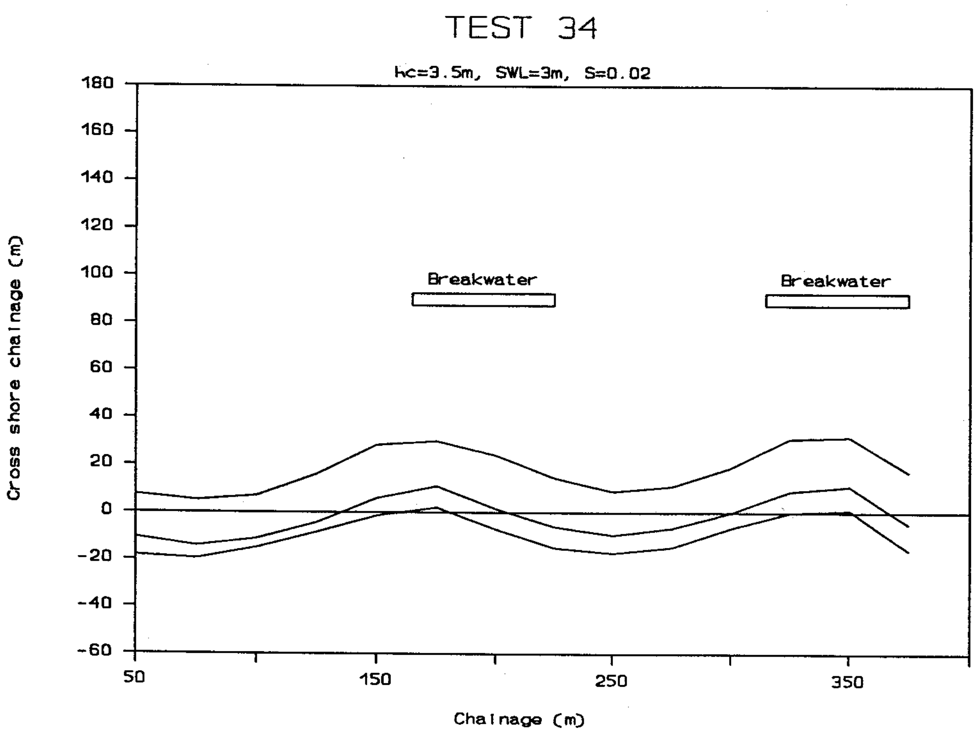


Figure A19 Final beach plan shape - Test 34

Appendix 7

End of test photographs - Breakwaters

Appendix 7 End of test photographs - Breakwaters

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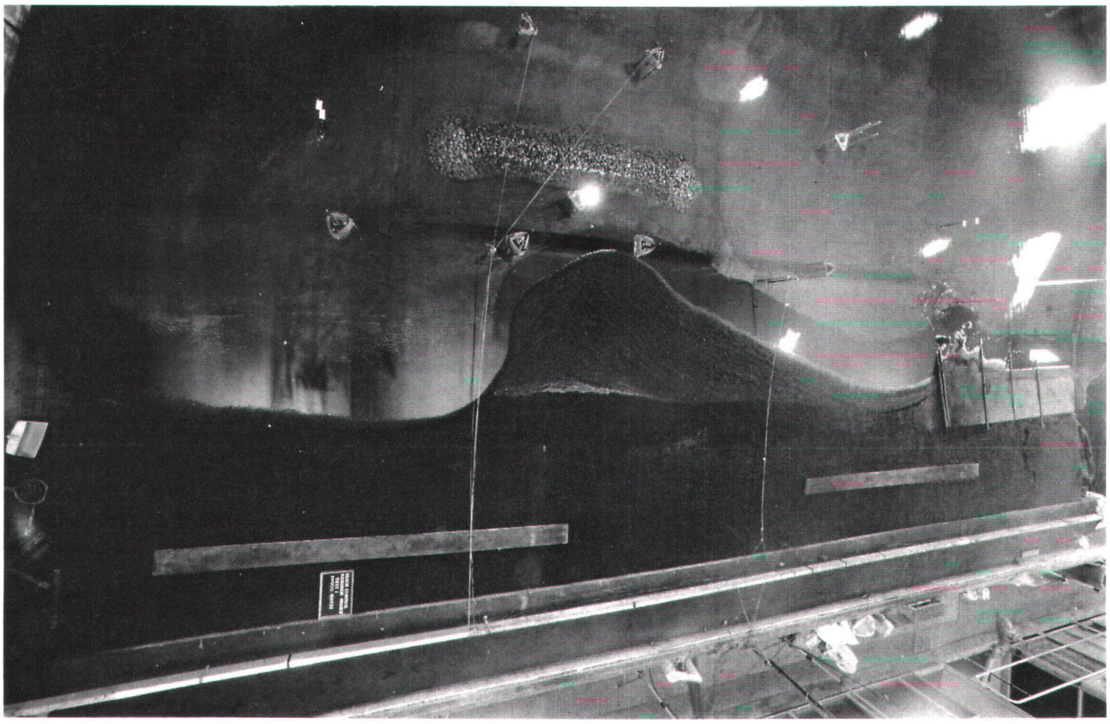


Plate A1a Beach development after 24,000 T_m (Test 1)

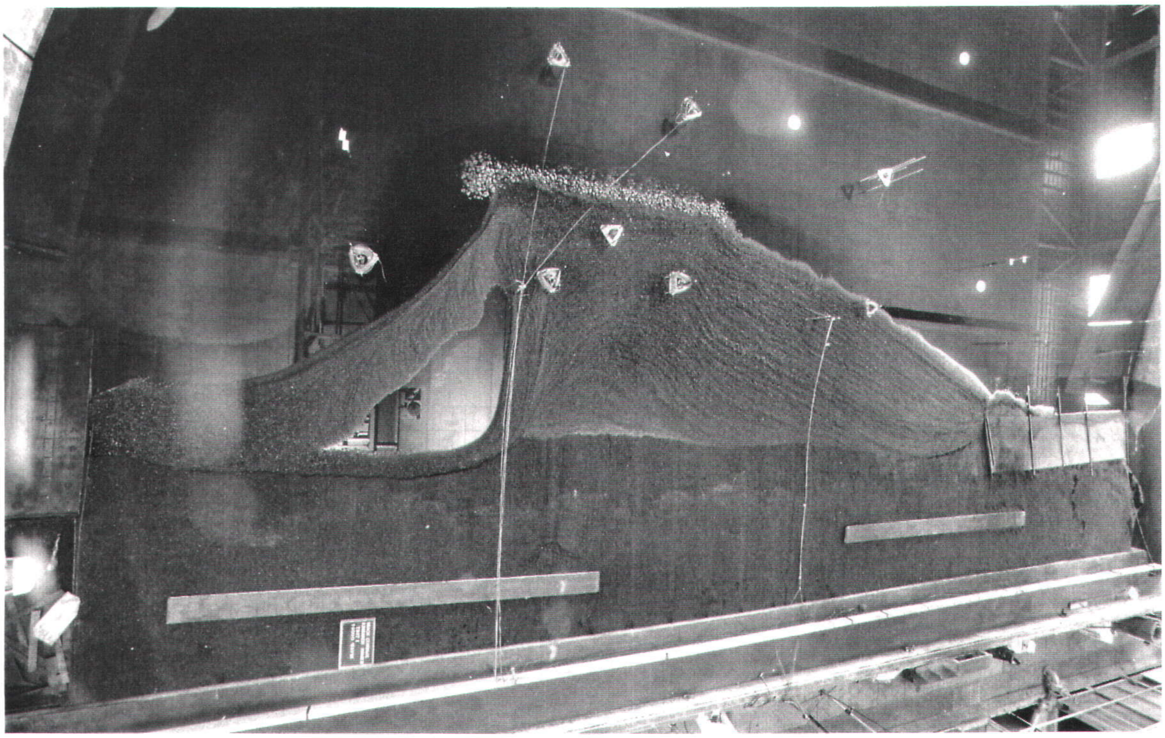


Plate A1b Final beach development (Test 1)

Plate A2   Final beach development (Test 5)

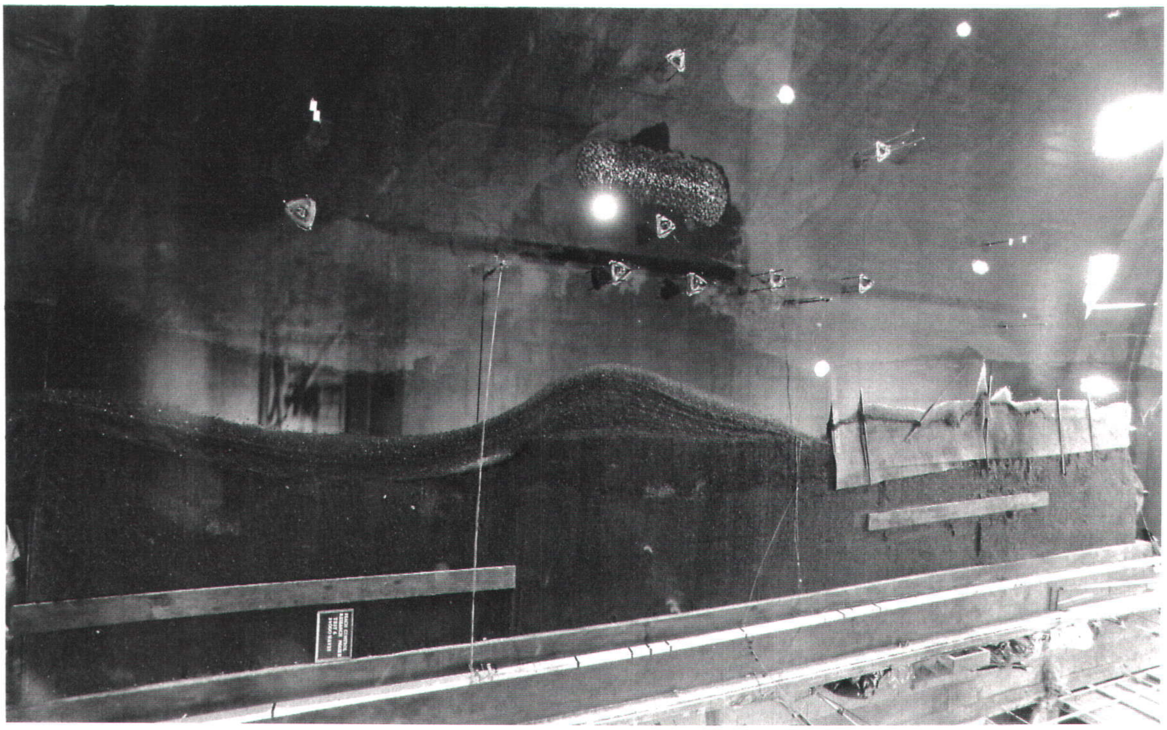
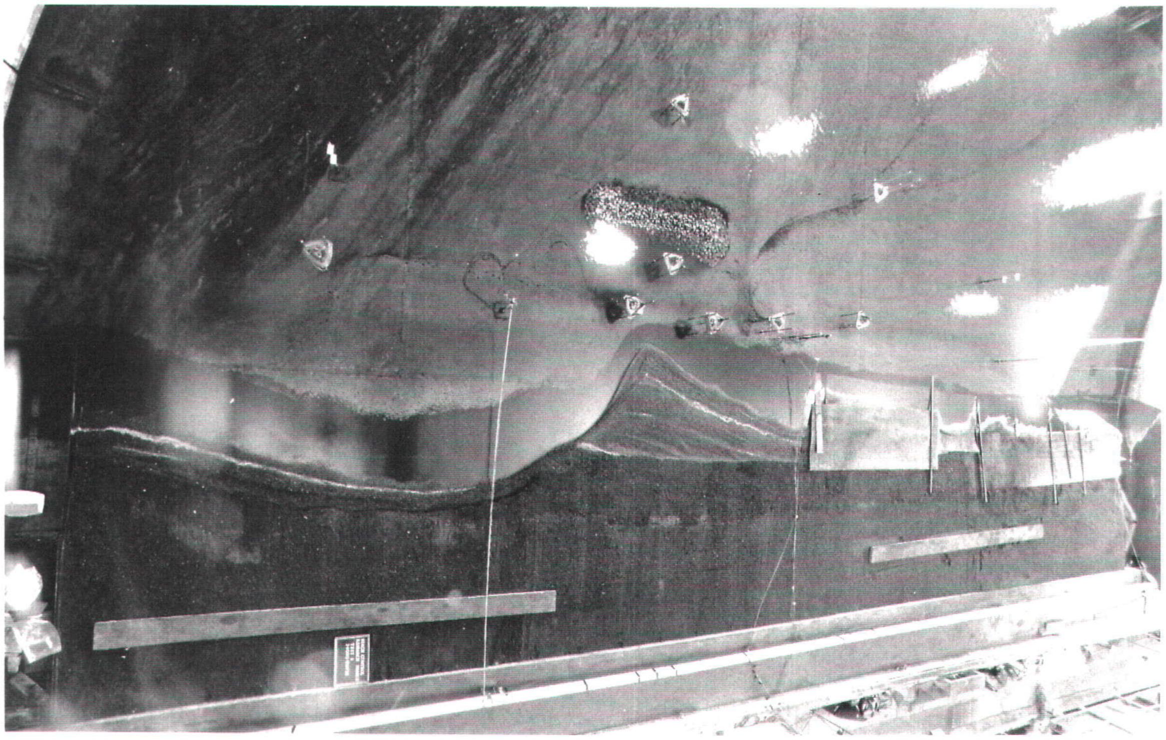


Plate A3   Final beach development (Test 6)



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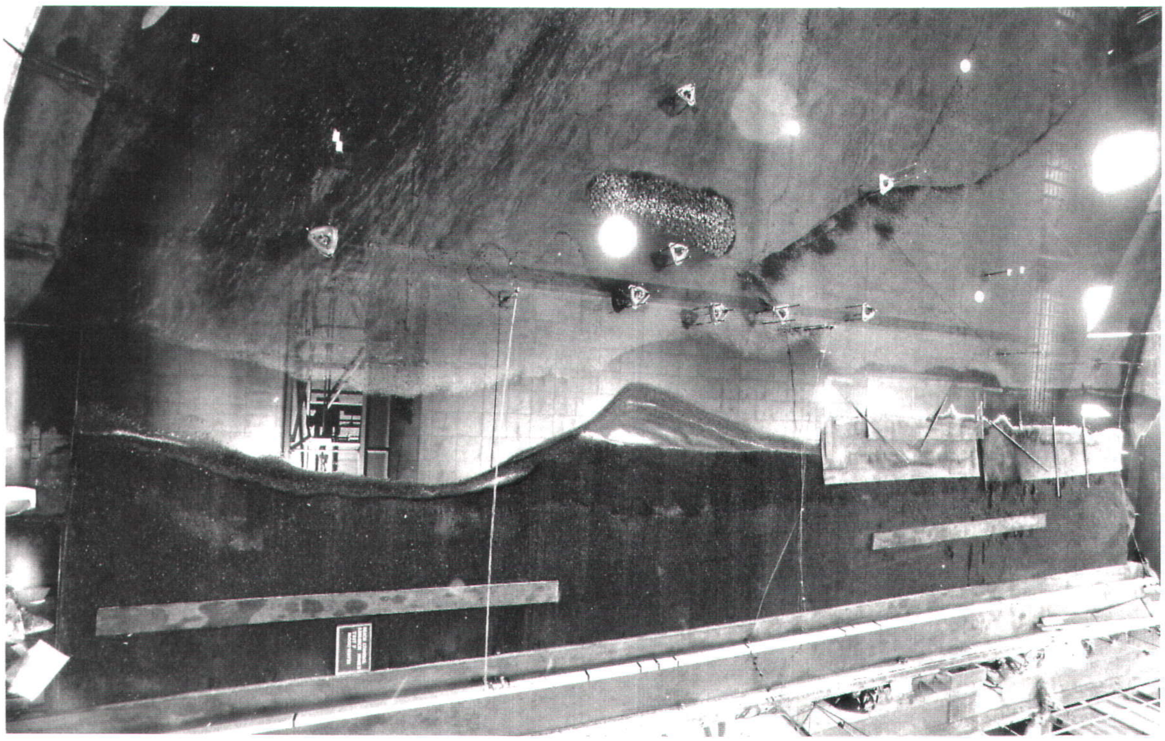


Plate A5   Final beach development (Test 7)

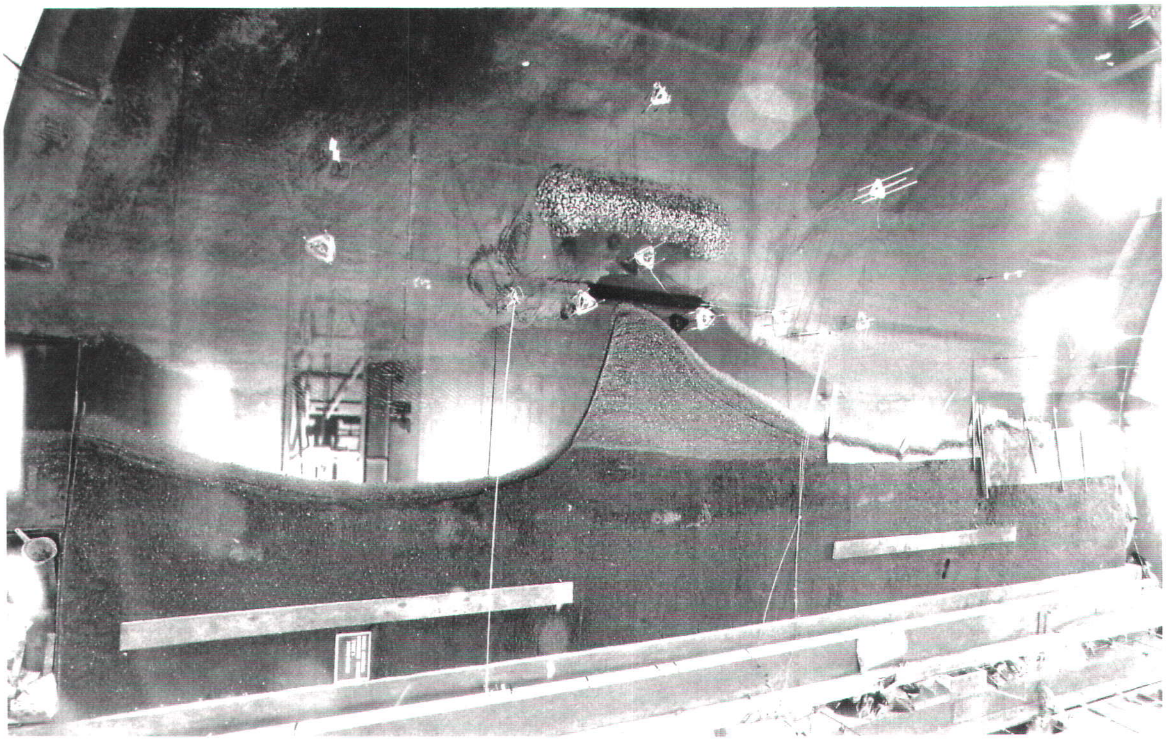


Plate A6 Final beach development (Test 3)

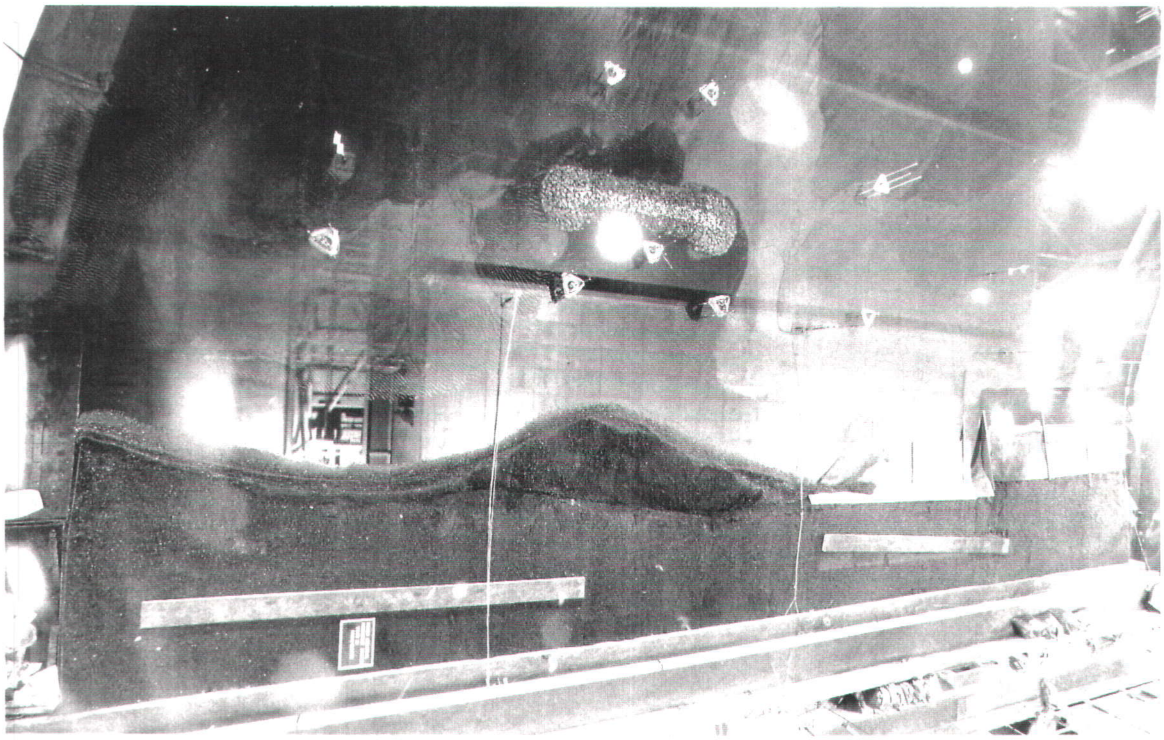


Plate A7 Final beach development (Test 2)

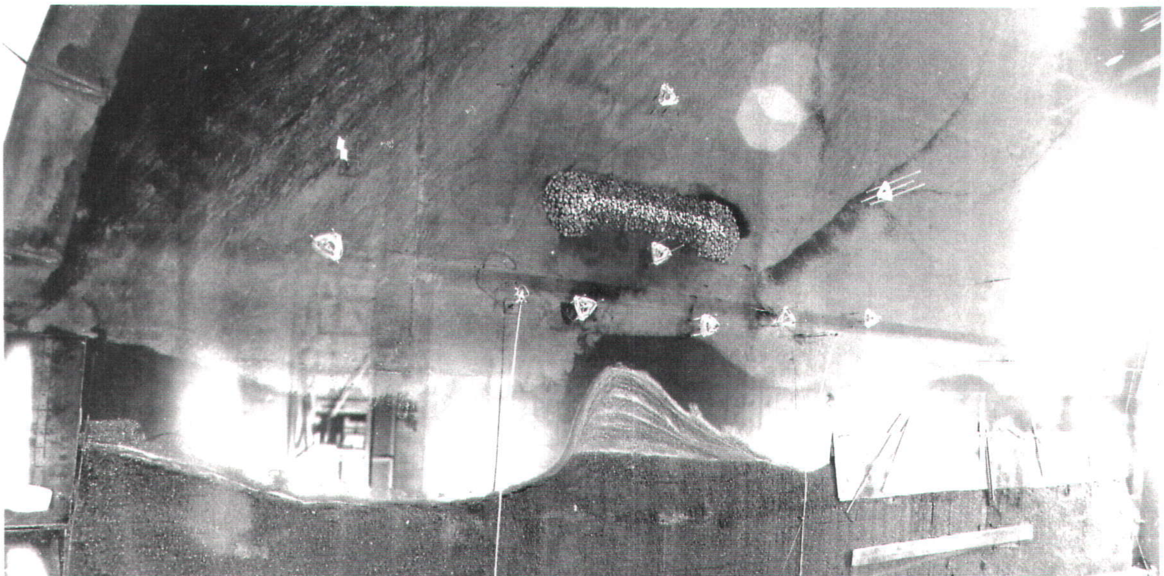


Plate A8   Final beach development (Test 4)

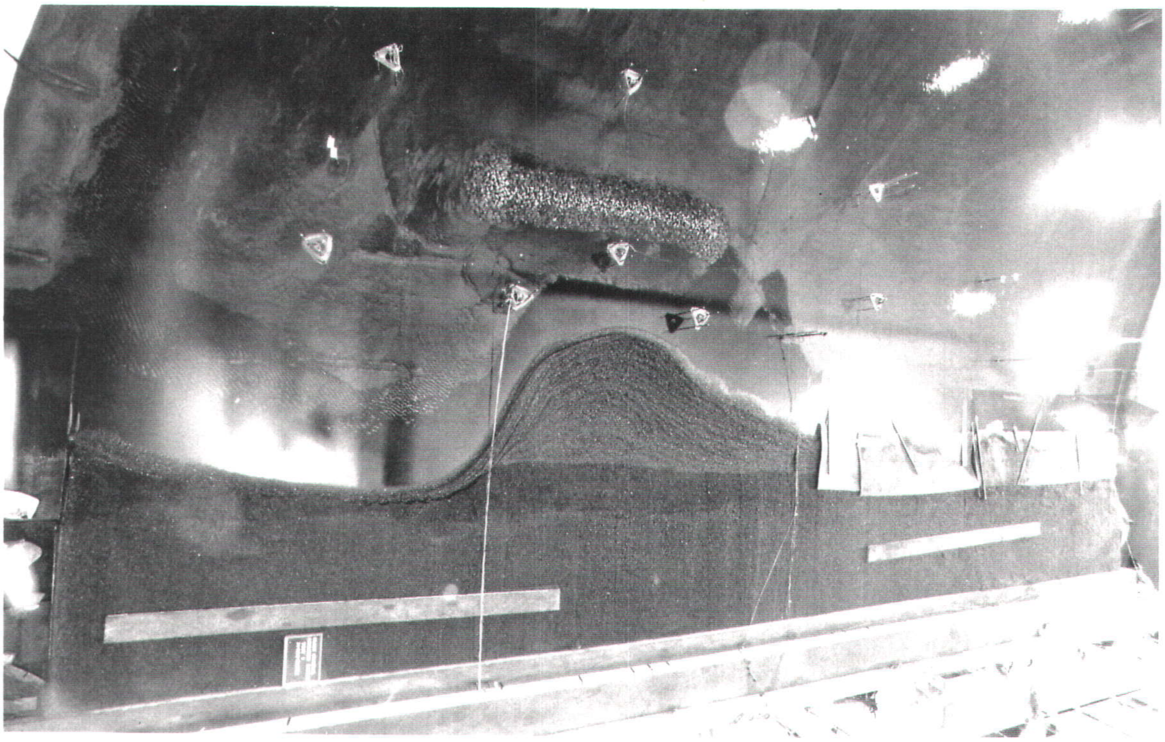
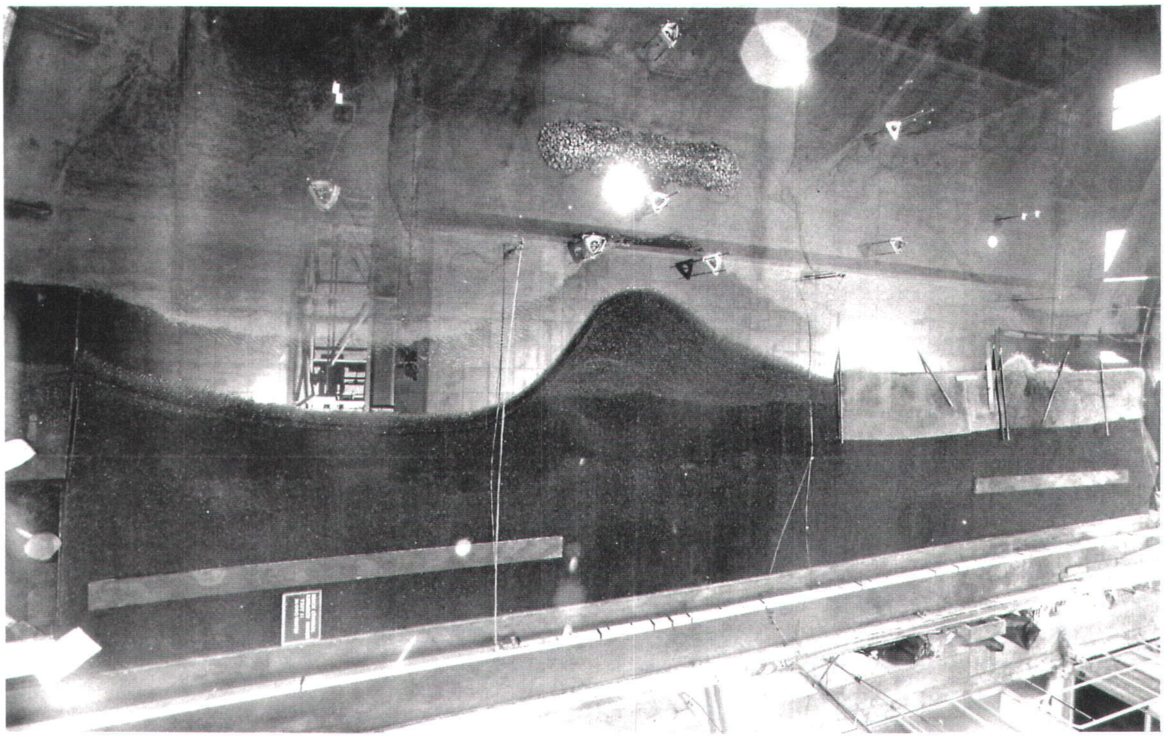


Plate A9 Final beach development (Test 9)

Plate A10 Final beach development (Test 10)



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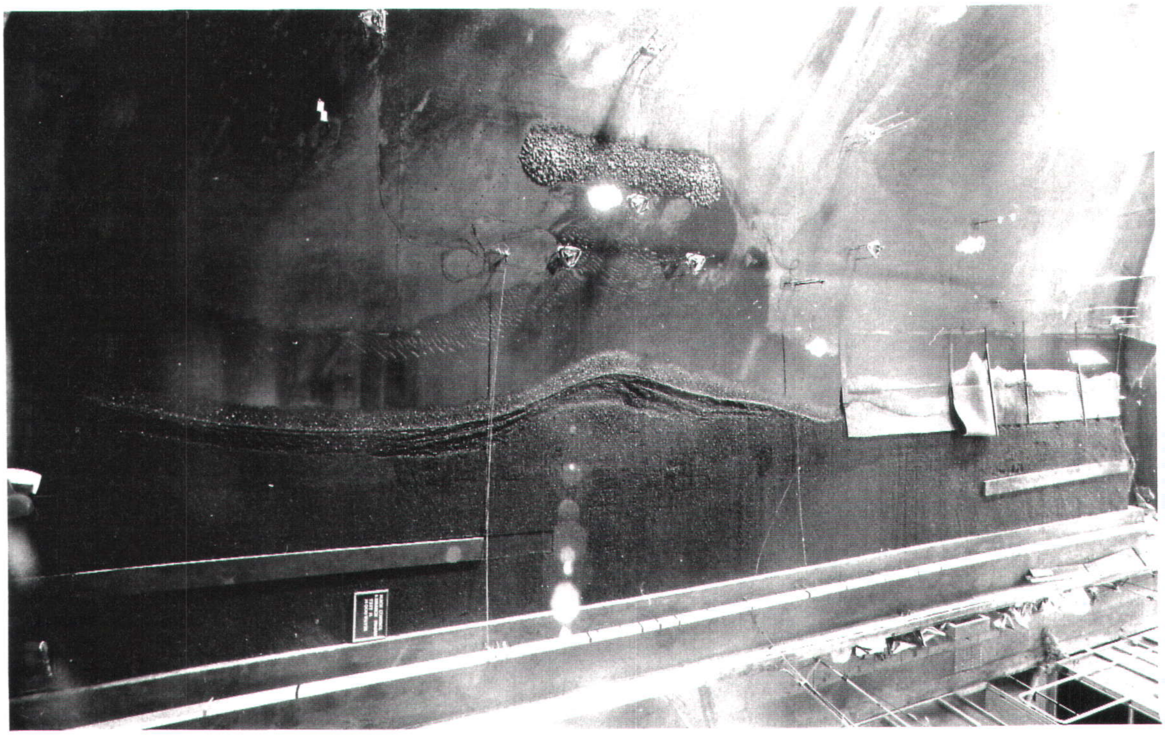

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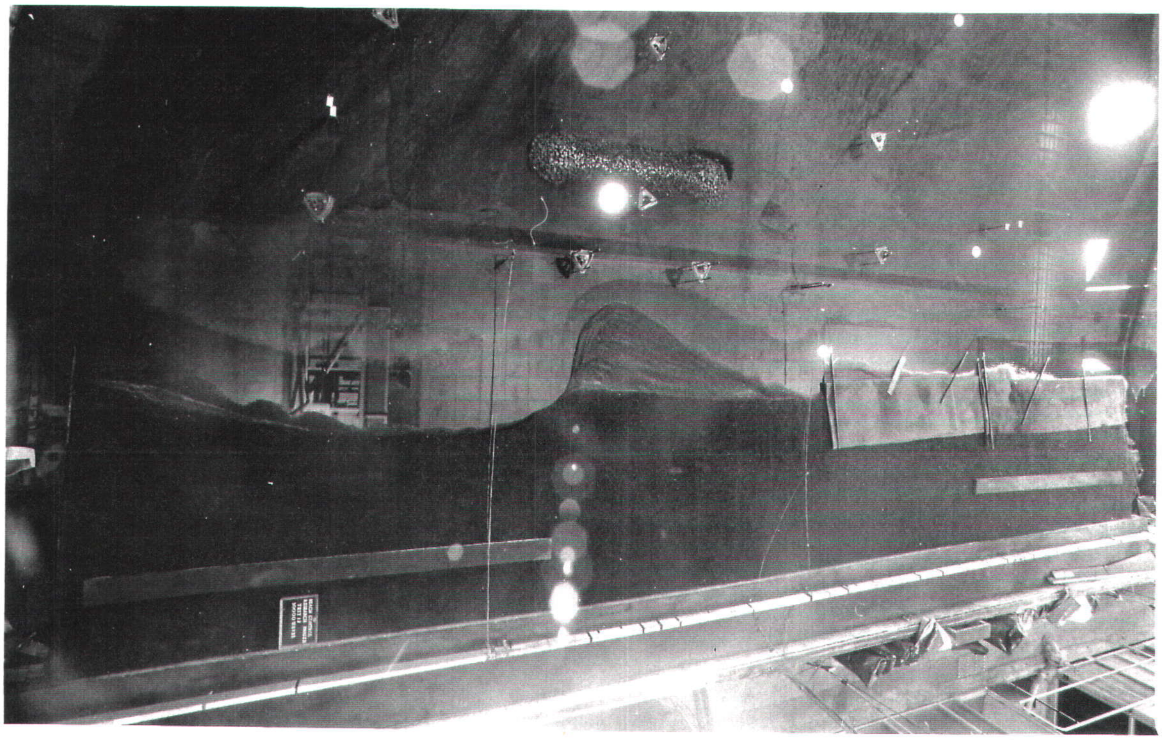

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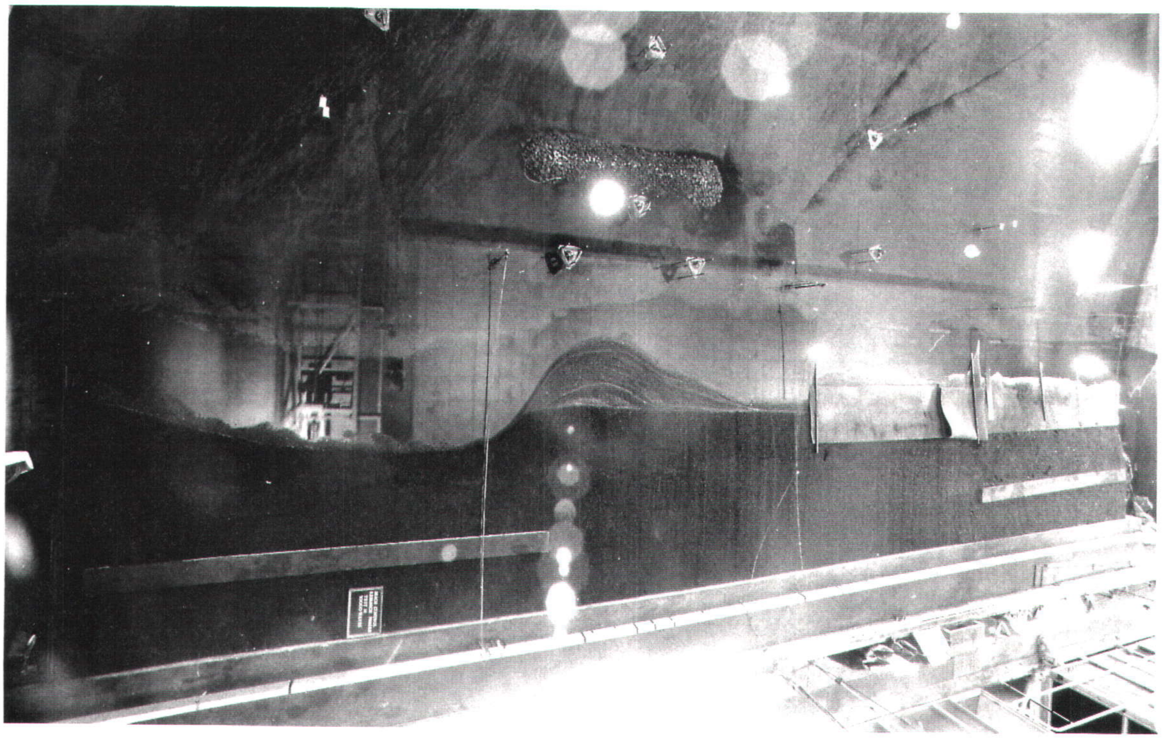

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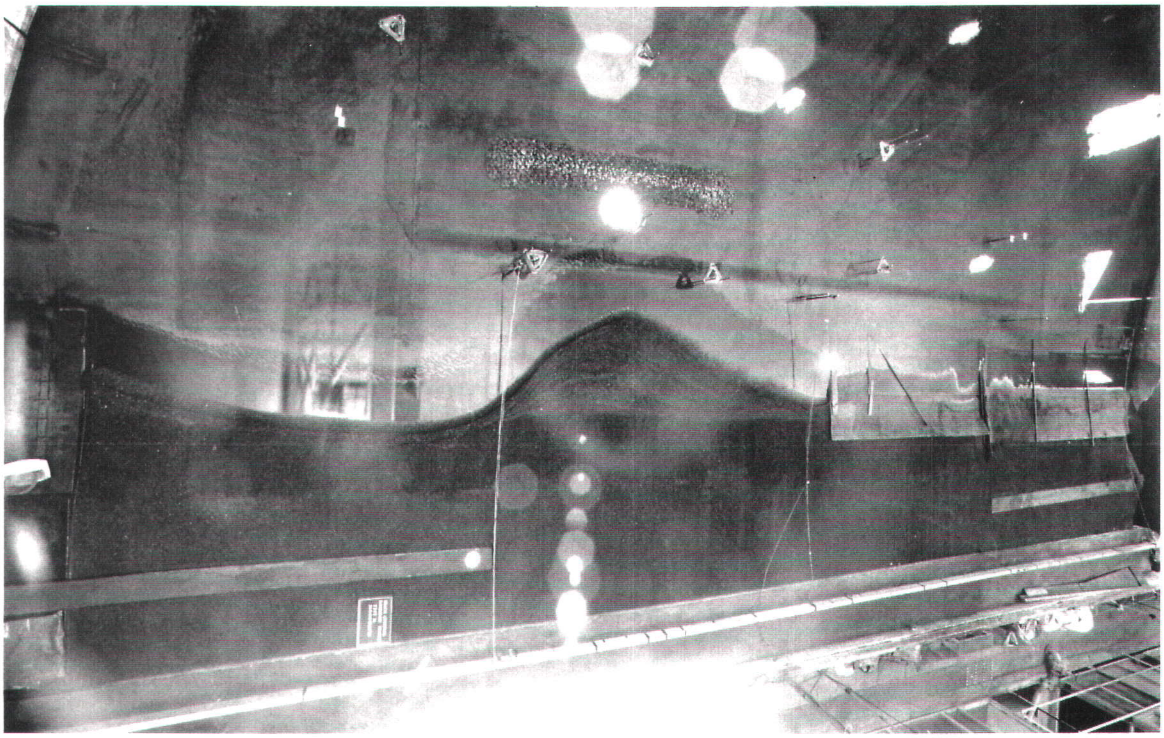


Plate A15 Final beach development (Test 16)

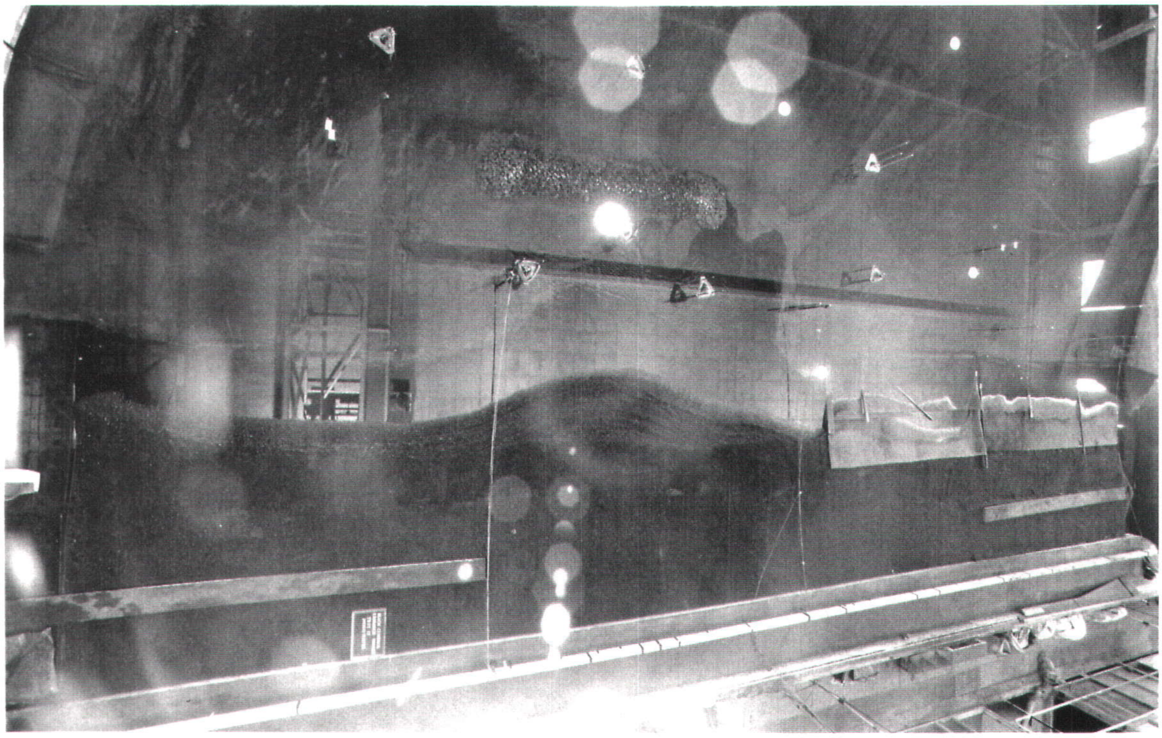


Plate A16 Final beach development (Test 17)

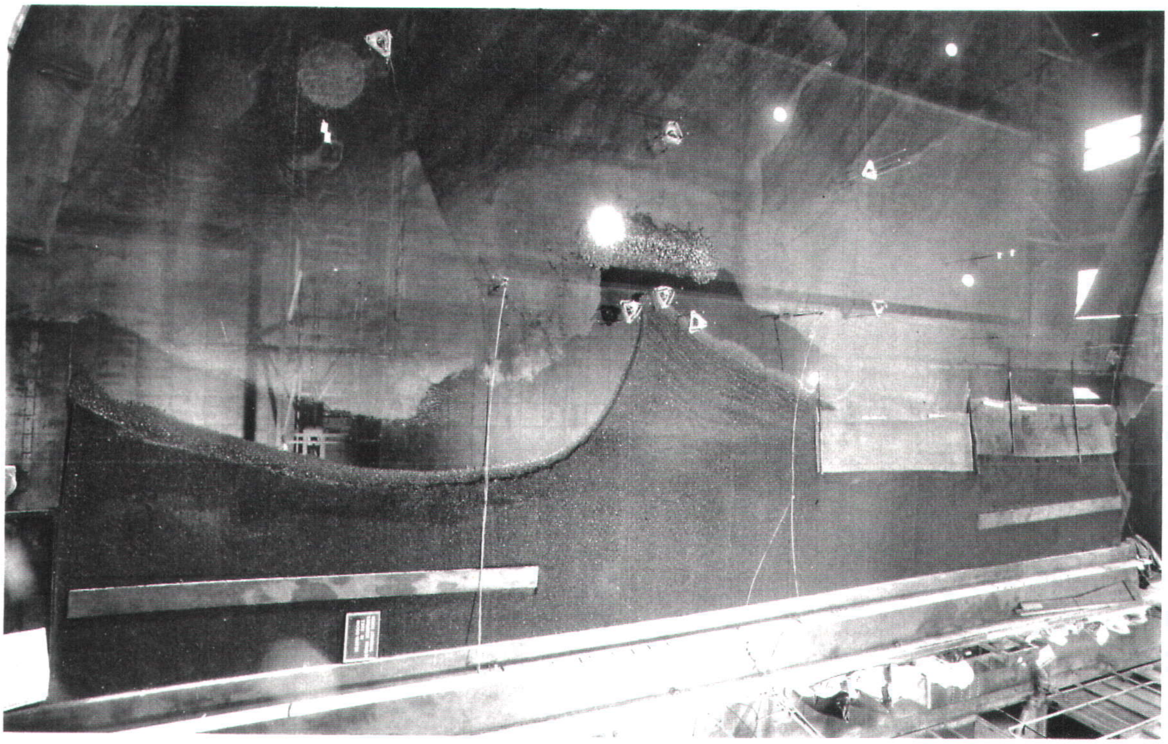


Plate A17 Final beach development (Test 22)

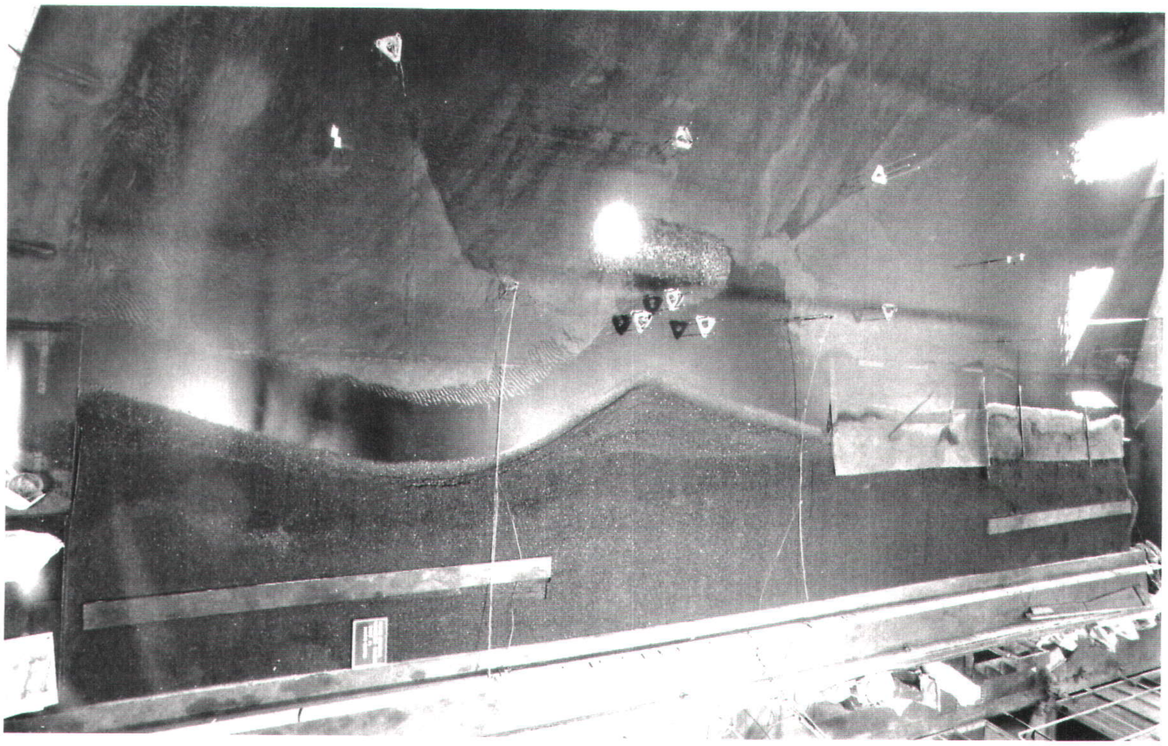


Plate A18 Final beach development (Test 23)

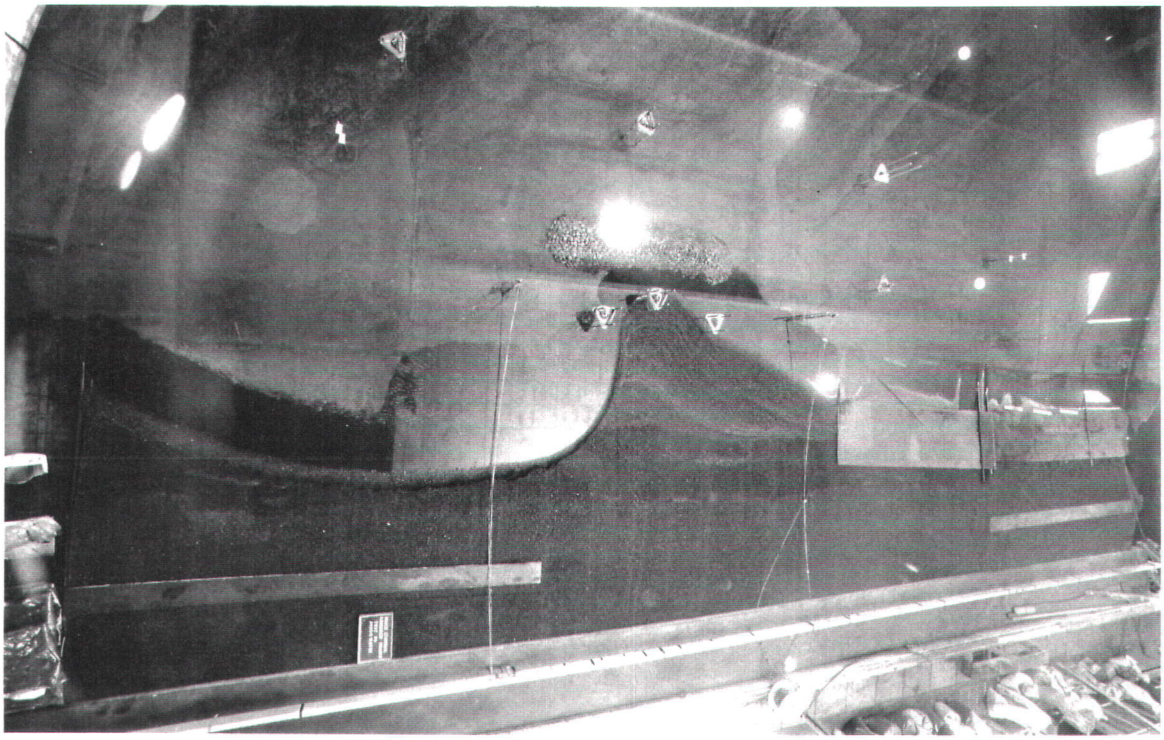


Plate A19 Final beach development (Test 20)

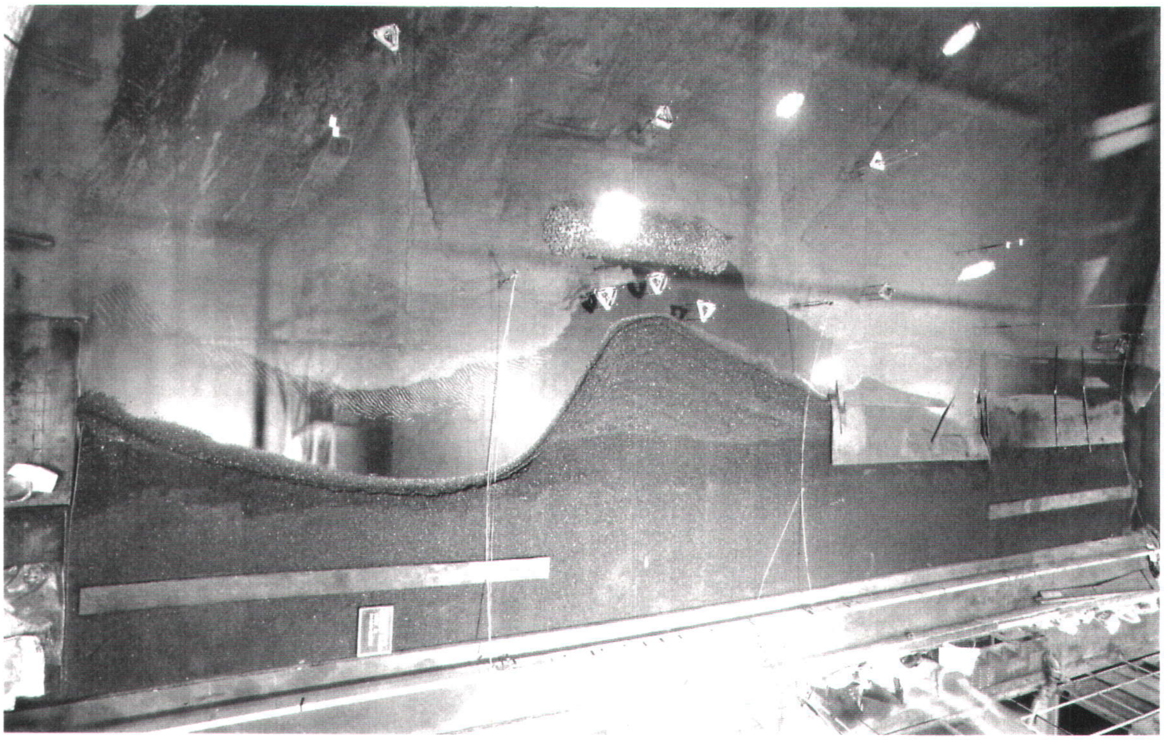
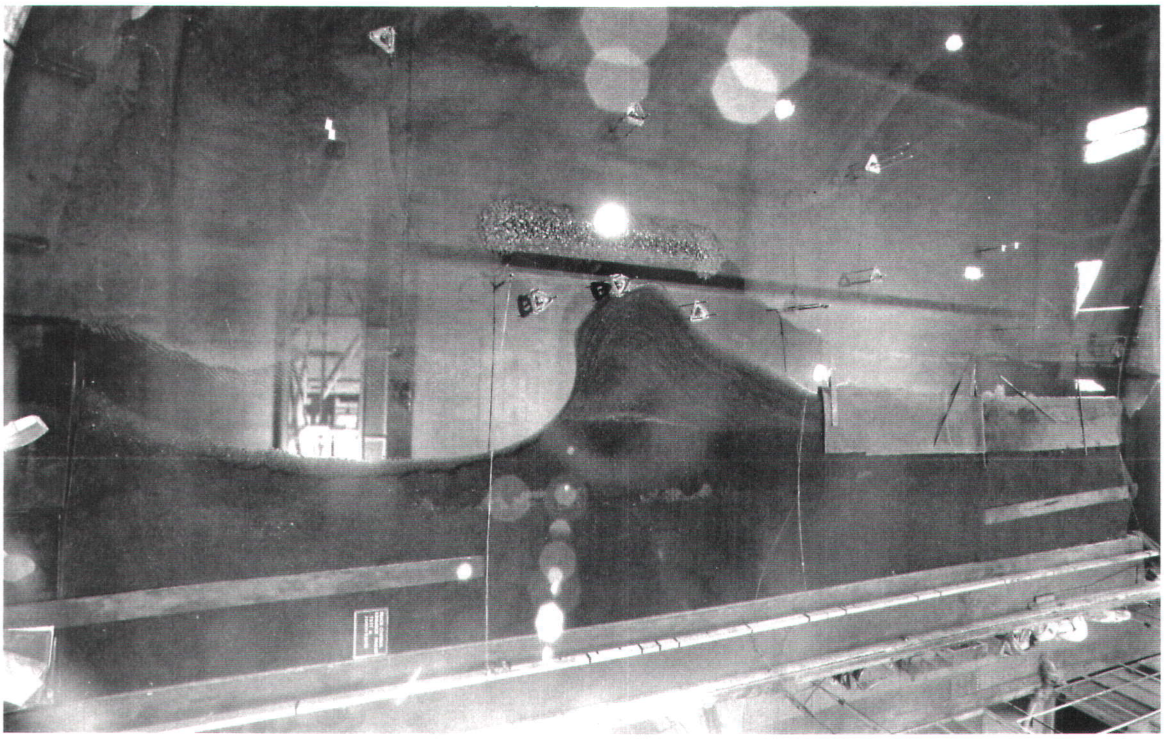


Plate A20 Final beach development (Test 21)



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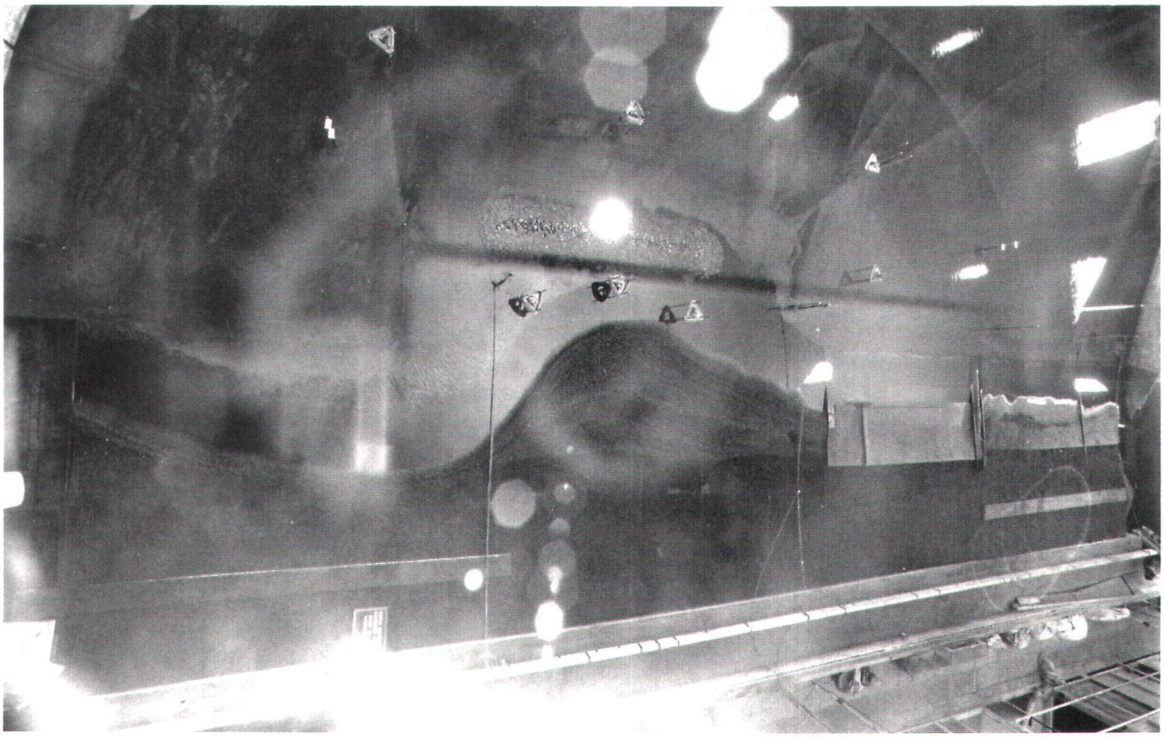

Plate A22 Final beach development (Test 19)

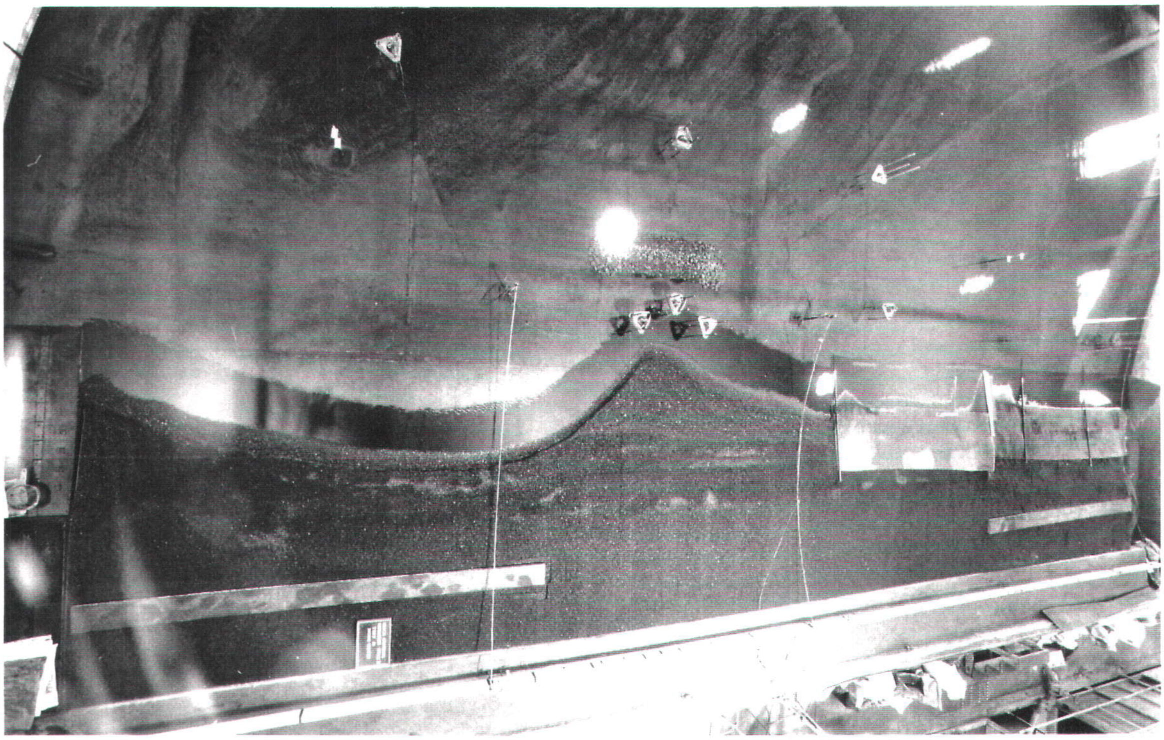

Plate A23 Final beach development (Test 24)

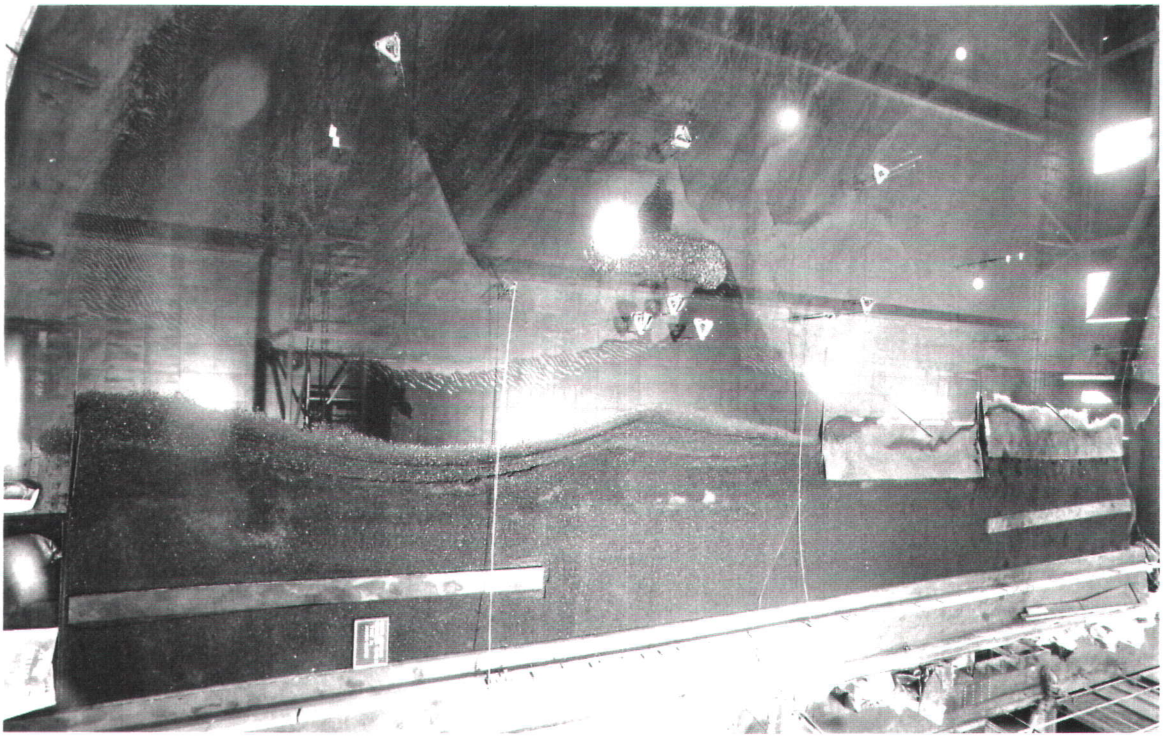

Plate A24 Final beach development (Test 25)

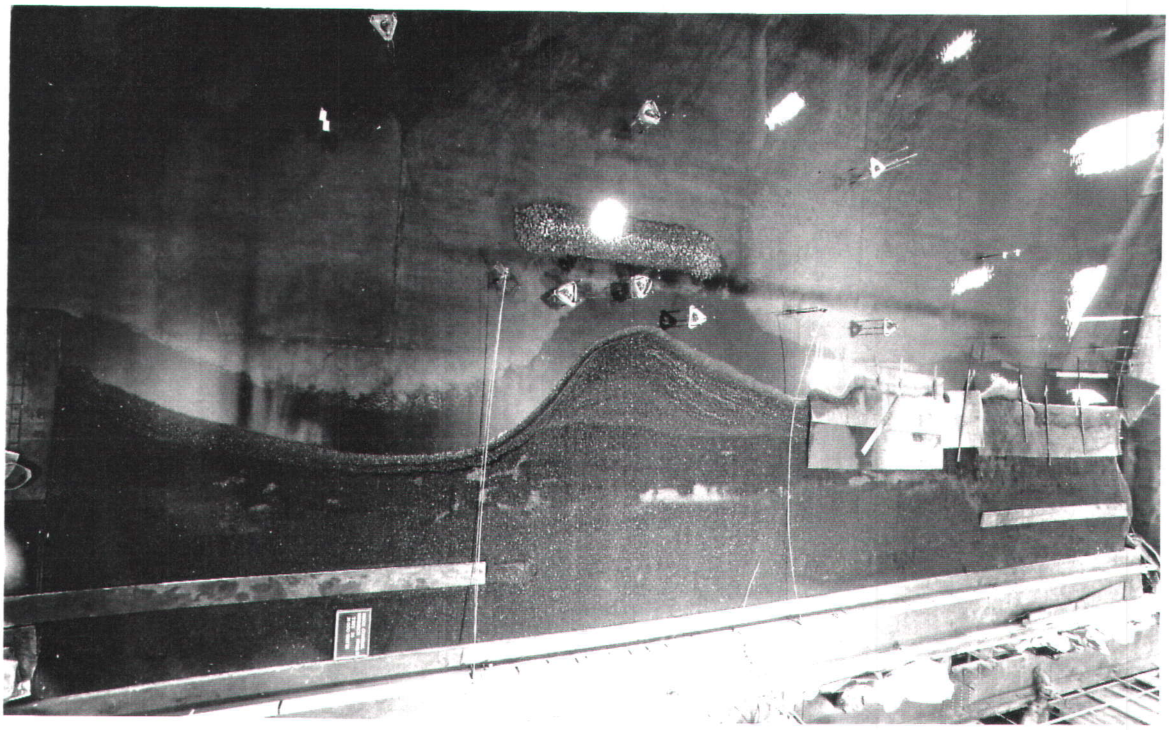

Plate A25 Final beach development (Test 28)

Plate A26 Final beach development (Test 29)

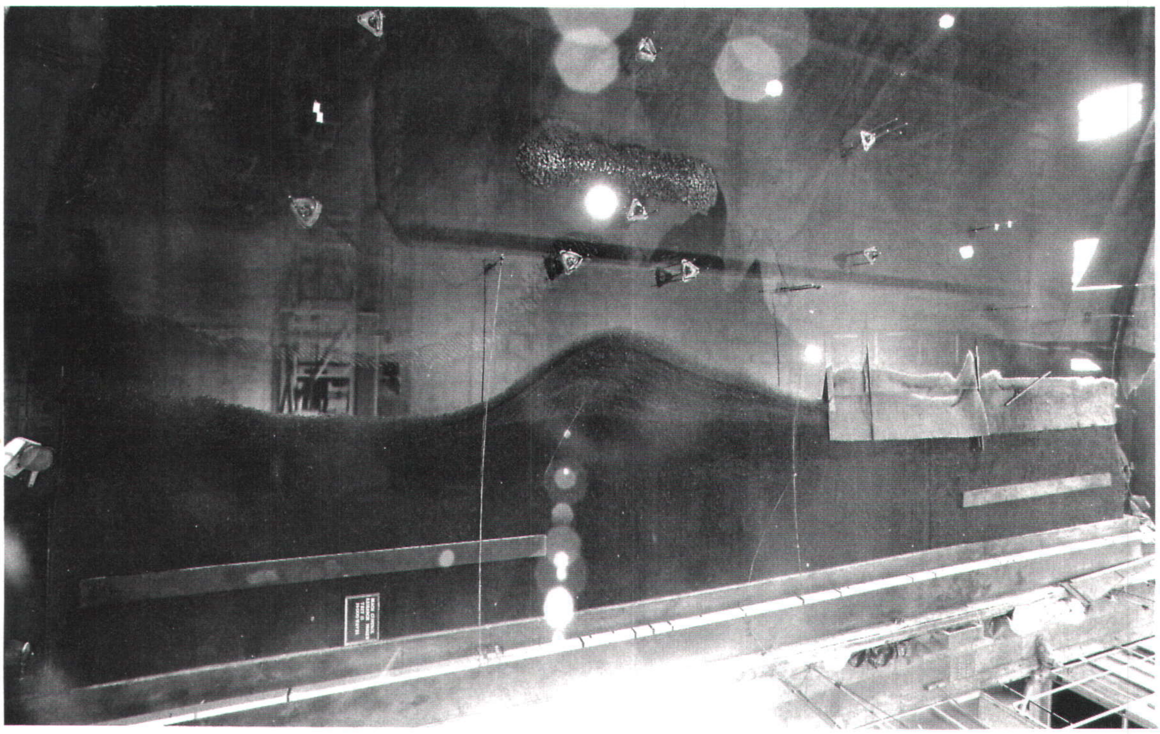

Plate A27 Final beach development (Test 13)

Plate A28 Final beach development (Test 30)

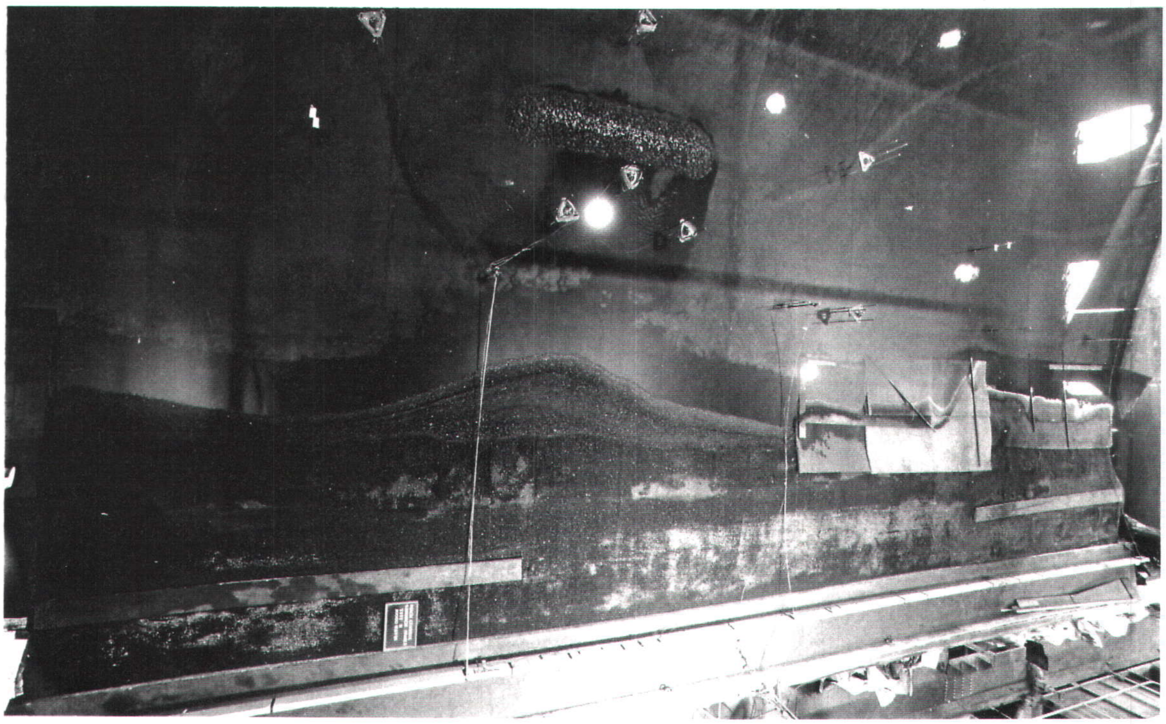

Plate A29 Final beach development (Test 31)

Plate A30 Final beach development (Test 26)

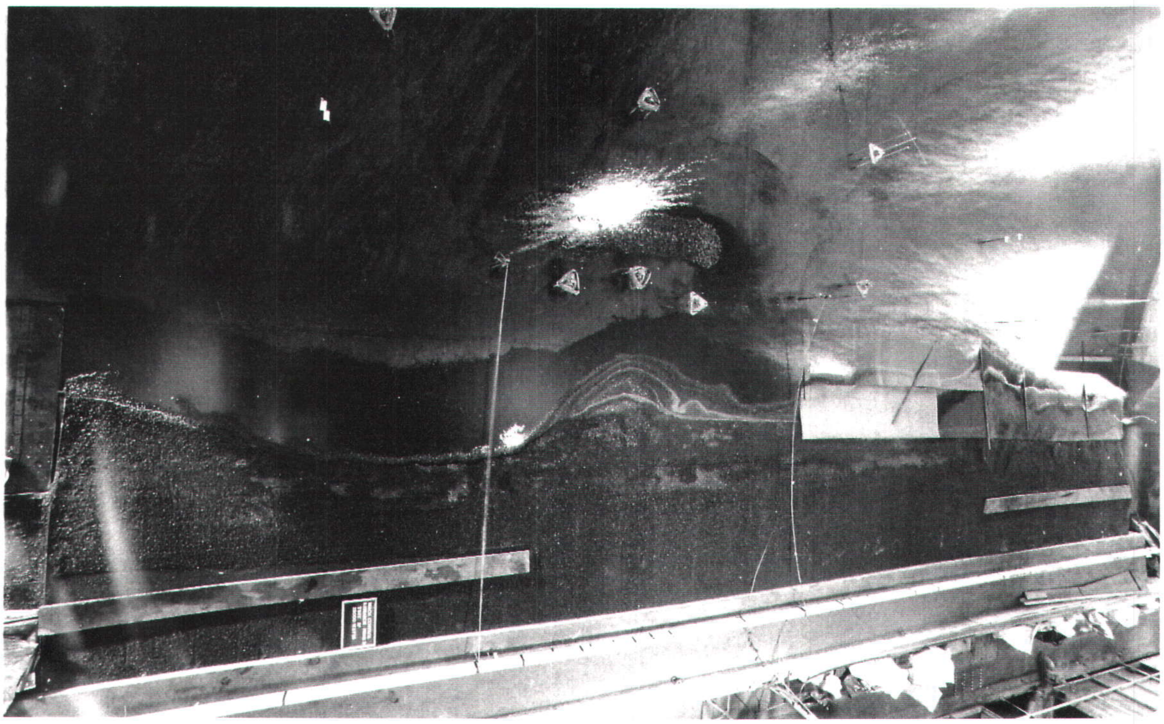

Plate A31 Final beach development (Test 27)

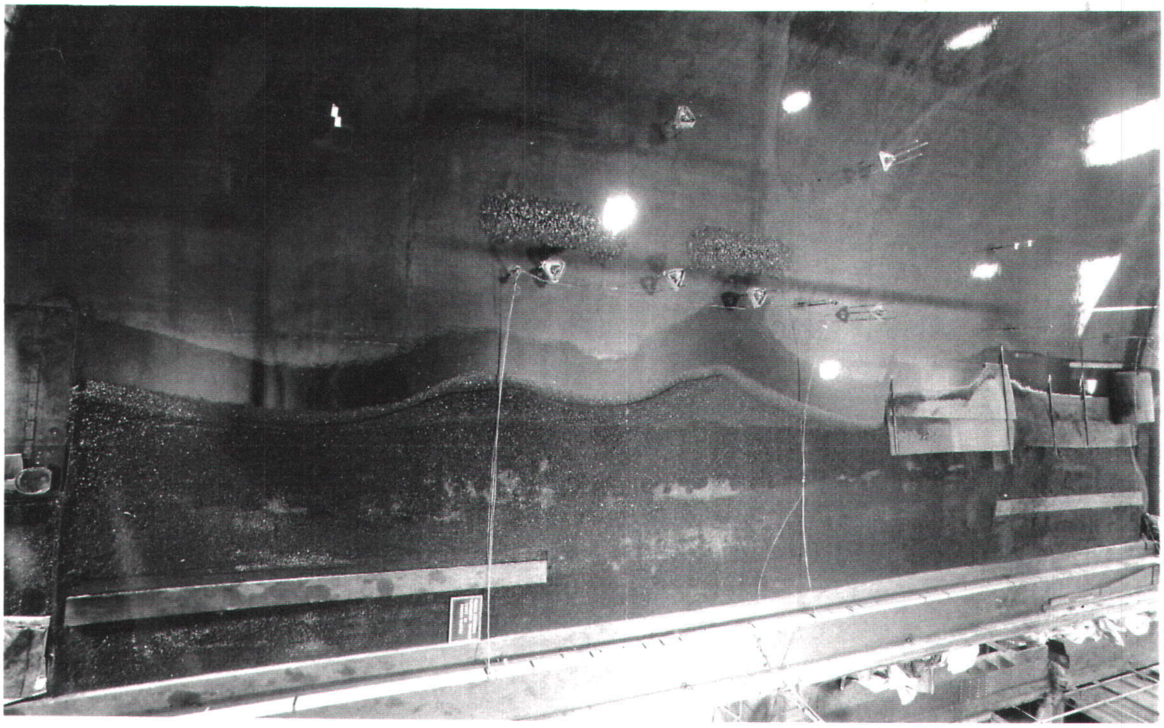

Plate A32 Final beach development (Test 32)

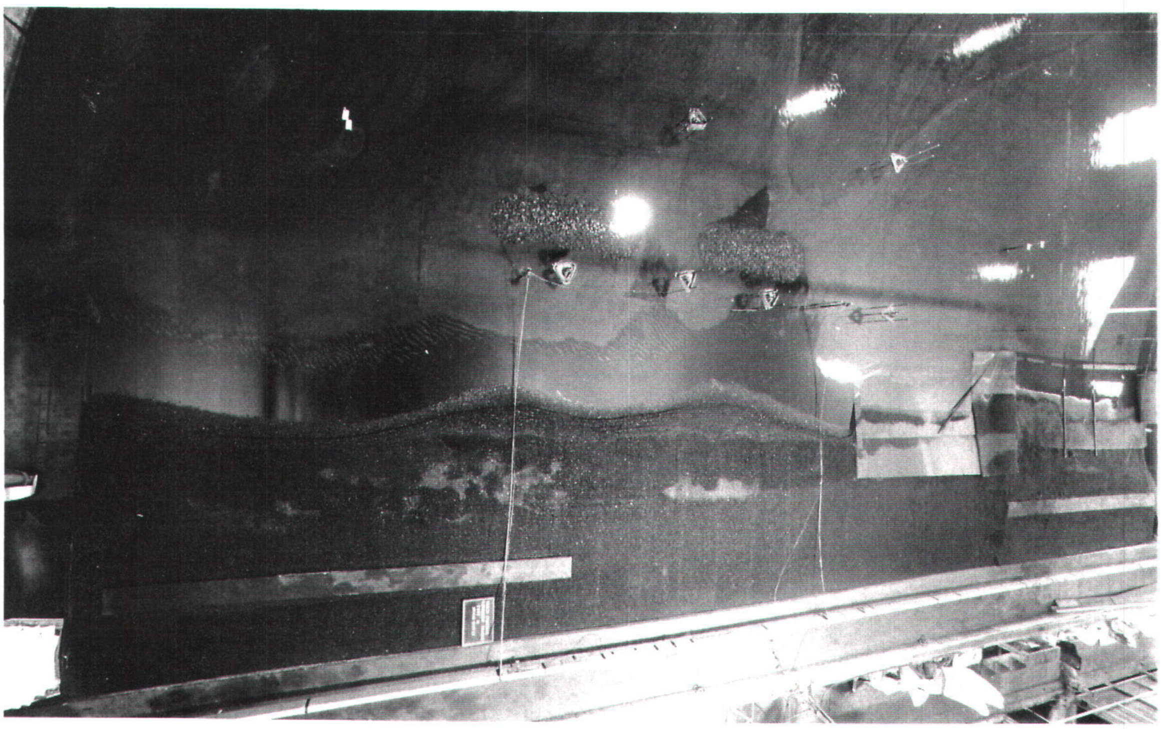

Plate A33 Final beach development (Test 33)

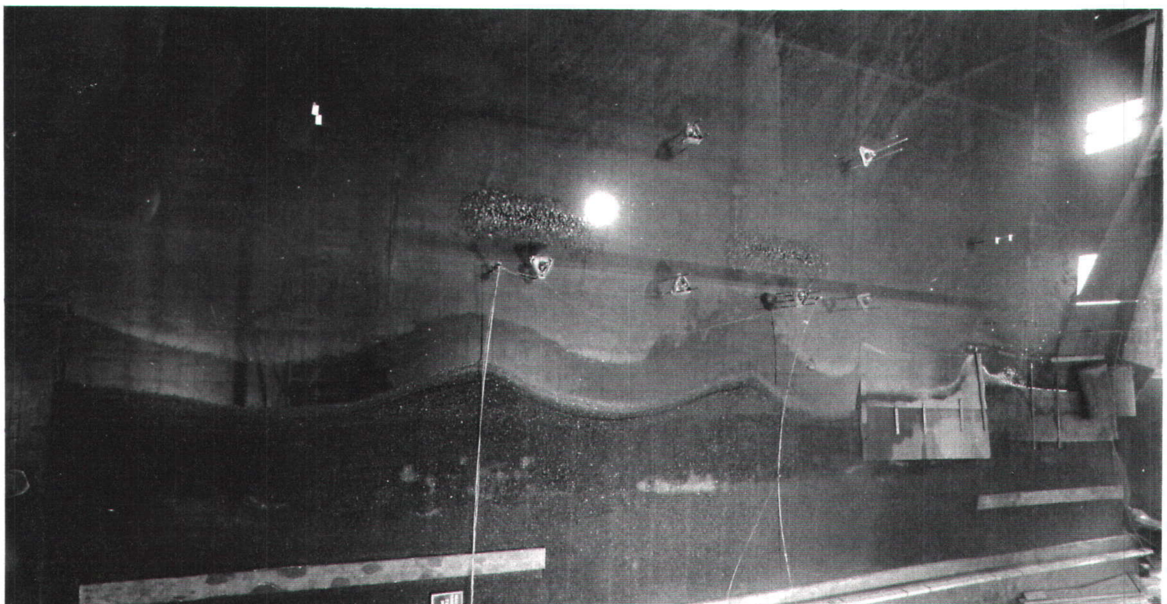

Plate A34 Final beach development (Test 34)

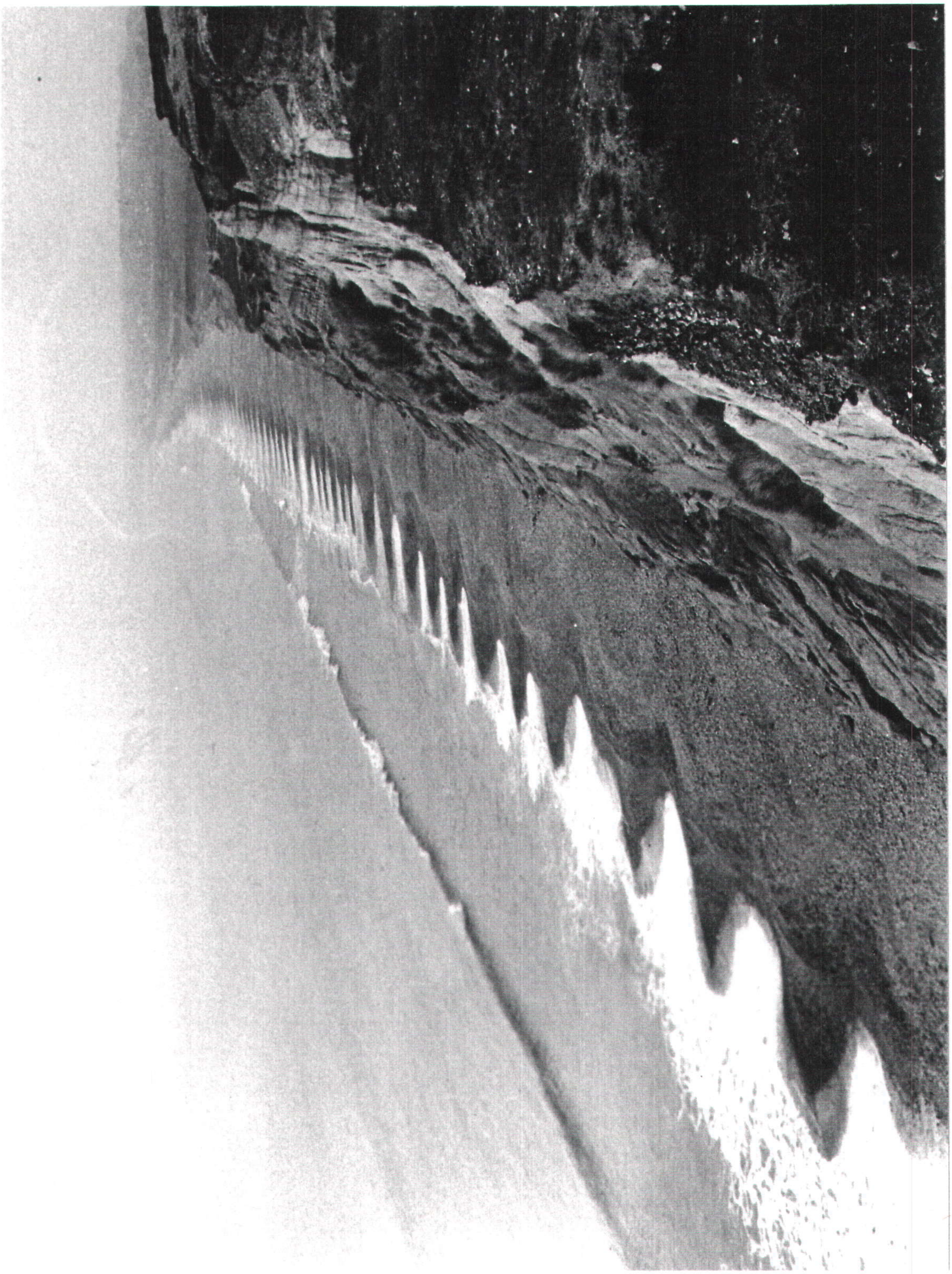

Plate 1 **Beach cusps at Hengistbury Head, Dorset. Reproduced by permission of Bournemouth Borough Council.**

Plate 2 Beach cusps at Dunwich, Suffolk.

Plate 3 Beach cusps at Dunwich, Suffolk.