Dissimilar Sediments:

Model tests of replenished beaches using widely graded sediments

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Summary

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Commercial pressures on existing offshore aggregate reserves make it likely that within the near future beach replenishment schemes will have to be undertaken using material which is dissimilar from that of natural beaches by virtue of either its size or grading. The behaviour of this material in a beach environment and the consequence repercussions for the design of replenishment schemes is, at present, poorly understood. This report describes research carried out to identify the engineering constraints on the use of this material and to provide preliminary guidance on the design of beach replenishment schemes using dissimilar sediments.

The report contains a brief review of previous work and provides details of the experimental investigation undertaken to improve present levels of understanding. A new approach to the design of beach replenishments is outlined. This approach is based on an equilibrium slope concept through which the effects of wave climate and sediment size and grading can be taken into account.

The new method has been validated against field data from UK beaches yielding reasonable agreement between predicted and measured equilibrium slopes. Subsequent comparisons against alternative design methodologies based on 'overfill ratios' suggest that the new method, which incorporates wave height and period dependency, produces a substantial improvement in the prediction of beach replenishment requirements, thereby allowing more cost-effective designs to be adopted.

This report forms part of a continuing study into beach processes which is being carried out with support from the Ministry of Agriculture, Fisheries and Food.

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1 Introduction

1.1 Background

Sand beach replenishment has long been recognised as a viable method of improving coast protection by restoration of the natural defences. One of the first documented replenishments took place in 1927 at Cabrillo Beach in California (Reference 1), while in the UK major replenishments were carried out at Bournemouth in 1970 and Portobello, Scotland in 1972. Subsequently attention in the UK has also focused on the replenishment of shingle beaches with a major scheme at Seaford involving 3 million tons of material being completed in 1987.

As the emphasis in the UK has gradually switched away from hard defences such as sea walls and towards soft engineering schemes, based on the replenishment and maintenance of natural beaches, so the pressure on our natural reserves of aggregate sized material has increased. This is reflected in the results of a recent analysis of demand trends for marine based beach recharge materials (Reference 2) which suggests that average demand has increased from approximately 20,000m^3/year in the early 1970's to 400,000m^3/year in 1990, and may reach a projected figure of around 1.3 million m^3/year by the end of the century. Such pressure on limited reserves is further compounded by a growing demand for concreting aggregate, particularly that dredged from offshore sources, and by the fact that at present the specification for material required by the construction industry is often similar to that of beach recharge schemes. The nett result of these demands is likely to be a shortfall in the volume of coarse material available for beach replenishment projects. As a consequence, coastal managers and engineers will be forced to consider the use of replenishment materials that are finer, or have a wider grading, than naturally occurring beach sediments.

Predicting the future behaviour of these somewhat artificial beaches, in terms of loss of fines and subsequent profile development, will be essential if the replenishment scheme is to fulfil its design objectives. Similarly, to ensure that the available resources are put to the most cost-effective use, designs will need to be optimised with regard to the availability of material, and its natural variability within the dredging area. Only with this ability to predict the behaviour of the replenished beach, both on completion of construction and as it evolves under wave action, will engineers be fully confident in the design of such schemes.

1.2 Scope and purpose of research

1.2.1 Basic hypothesis

It is well known that there is a direct linkage between beach sediment size and beach profile. Sand beaches composed of finer sediments are relatively impermeable to wave action and as a result the volumes of water moving up the beach as wave run-up are largely matched by the volumes returning, under the influence of gravity, as backwash. Consequently the majority of suspended sediment carried by the wave run-up returns in the backwash. As a result there is no hydro-dynamic mechanism for steepening a sand beach and it therefore remains at a relatively shallow slope.

Shingle beaches on the other hand are permeable structures allowing a high proportion of the wave run-up to percolate down into the beach thereby reducing the extent of any run-up and ensuring that the backwash volumes are much reduced. As a consequence material carried up the beach by the wave run-up is more likely to be deposited on the beach slope, and the slope will therefore be correspondingly steeper.

Table 1.1 presents a qualitative summary of sediment size versus beach slope trends gleaned from a number of sources.

Table 1.1 Beach slope - sediment size trends for natural beaches

Median sediment size D_{50} (mm)	Mean beach slope	
	Moderate wave climate	Severe wave climate
0.2	1:50	1:100
0.3	1:25	1:50
0.5	1:20	1:40
5.0	1:8	1:15
10.0	1:7	1:12
25.0	1:4	1:8

The problem now facing beach engineers and managers is that recharge material is likely to be both finer than natural beach sediment and also more widely graded. As a consequence it is to be expected that the permeability of a recharged shingle beach will be less than that of a natural beach, and hence that it will adopt a flatter slope. If this is so, placing widely graded material to a typical (1:7 - 1:8) natural shingle beach slope could result in severe and unexpected erosion as the beach attempts to adopt a more appropriate, and flatter, slope. This erosion will be concentrated in the sub-aerial portion of the beach with material from this region being drawn down to below the mean water line. The nett result will be a marked reduction in the width of the beach and a corresponding lowering in defence standards. This effect is shown schematically in Figure 1.1.

Over a period of time, as the beach is re-worked by wave action, it is likely that the recharge grading will evolve to become more typical of that of a natural beach. One may postulate that this is an exponential process with change occurring very rapidly at first then slowing down as the beach moves towards a more balanced and stable sediment distribution. However, it is also likely that the initial rate of change is strongly dependent on the extent of the difference between the recharge and natural gradings. Therefore whilst it is necessary to ensure that the design profile for a beach recharge is commensurate with the material size and grading being used, it is also important that the designer is aware of the potential for changes in beach profile behaviour as the beach grading evolves towards a more natural state.

1.2.2 Purpose and approach

It is the purpose of this study to establish an appropriate method by which the behaviour of widely graded beach materials in the coastal environment may be predicted, particularly in respect of their profile response to wave action.

Present day understanding of the dynamics of breaking waves, surf zone processes and wave-beach interaction is not yet sufficiently developed to allow a full theoretical treatment of the micro-scale behaviour of uniformly sized sediments, let alone the complex size distributions which typify most natural and replenished beaches. Physical modelling, on the other hand, has been shown to yield reliable information on macro- and micro-scale beach processes provided that both the wave conditions and beach sediment characteristics are correctly scaled (Reference 3). Whilst this information could be obtained under more realistic conditions through an intensive field measurement programme, such an exercise introduces many practical problems, not least that of taking measurements from a complicated interacting system containing many variables, virtually none of which can be controlled. The physical modelling may pose problems with scale effects and create a rather simplistic representation of wave-beach interaction, nevertheless it does allow certain aspects of beach behaviour to be investigated in a systematic manner, so that the general behaviour may be established. Generally the laboratory based approach produces the better data-base and it is therefore the method employed for this study.

The shortage of material for beach replenishment projects affects mainly the coarser, shingle sized, fractions. This, coupled with the generally narrower grading of natural sands, has lead to the study being biased towards the coarser materials. As a result little of the previous published literature is relevant. Nevertheless it is still useful to review the existing methods of beach recharge selection, in part because they are sometimes applied to shingle replenishments (Reference 4), but also because they will provide an initial appreciation of the problem and relevant processes. The study therefore commences with a review of previous work. This, together with an assessment of the sedimentary characteristics of existing UK beaches, is used to develop the physical modelling test programme from which the new design method is developed.

As the final stage in the research, it is useful to compare the performance of the various design methodologies for a range of material gradings and sizes. This allows the limits of applicability to be established and the consequences of using the methods outside their appropriate ranges to be fully demonstrated.

1.3 Natural and dissimilar beach sediments

In any consideration of the performance of beaches composed of dissimilar sediments it is necessary, as a first step, to define the size and grading characteristics of natural beaches which may not be reproduced in any future replenishment. To assist in this, a brief review of the grading and median sediment size of material on a number of UK shingle beaches has been undertaken. The results of this review are summarised in Table 1.2.

From the review, which covered beaches from a variety of sites around the UK, it appears that natural shingle beaches can be considered to have an average median sediment size, D_{50}, of 15.5mm (std. dev. = 6.0mm) with an average grading, D_{84}/D_{16}, of 3.42. Consequently, for the purposes of this study, it has been assumed that replenishment materials with a $D_{50} < 9.5$mm and a grading, $D_{84}/D_{16} > 3.42$ are likely to be dissimilar to the sediments on most natural shingle beaches.

Table 1.2 Natural shingle beach sediment characteristics

Site	Material size (mm)			Grading D_{84}/D_{16}
	D_{10}	D_{50}	D_{100}	
Seaford	6.1	13.7	38.0	2.73
Whitstable	7.6	12.6	50.0	2.41
Chesil (Portland)	23.8	30.0	-	-
Chesil (West Bexington)	8.5	10.0	13.0	1.34
Littlehampton	7.3	13.0	42.0	2.33
Hayling Island	7.0	16.0	64.0	4.00
Hurst Spit	6.0	20.0	63.0	4.25
Pevensey Bay	6.6	14.3	-	3.10
Southwold	6.1	14.0	50.0	4.40
Sidmouth	7.0	22.2	90.0	5.80
Hythe	2.8	5.2	23.2	3.13
Pensarn (N. Wales)	6.9	15.0	50.0	4.13

1.4 Outline of report

Following this introductory section, Chapter 2 summarises existing methodologies for selecting appropriate specifications for beach recharge materials. It is shown that there are three main approaches, denoted as Rule of Thumb, Overfill models and Equilibrium models. Chapter 3 covers the experimental work undertaken as part of this study. It describes the test facility and experimental techniques used, as well as the scope and structure of the test programme. The results obtained from the laboratory work are presented and analysed in qualitative terms in Chapter 4. Detailed analysis of the results leading to the derivation of a new 'equilibrium slope' method for the design of replenishment cross-sections is given in Chapter 5. Chapter 6 considers issues relating to the application of the new method while Chapter 7 compares the results given by this method with those obtained from the existing models, and field data. Finally Chapter 8 draws together the conclusions arising from this work and makes recommendations for further studies to extend the range of the proposed design methodology.

2 Summary of previous work

2.1 General

For many years engineers concerned with beach nourishment have been developing models to predict the behaviour of beaches with dissimilar recharge and native materials.

This section presents a summary of previous work concerned with understanding and resolving the problems which may occur when the specification for recharge material deviates from that of the natural beach sediment.

2.2 Rules of thumb

Before the mid seventies only two major beach replenishment schemes had been carried out in the UK, a 90,000m^3 scheme at Bournemouth in 1970 and a 180,000m^3 scheme at Portobello, Scotland in 1972. These schemes had necessarily been designed with very little engineering understanding of either the behaviour of a replenished beach or the interaction between the recharge and native materials.

Newman (Reference 5) recognised the need to provide guidance on the design of future schemes, and carried out a review of various aspects related to the design and execution of a number of artificial beach nourishment projects worldwide. Based on this evaluation, the following guidelines for planning artificial beach nourishment schemes were proposed:

1. The underlying cause of the beach erosion must be investigated and evaluated.

2. Recharge material must be coarser than the existing material; a mean diameter of 1.5 times the natural beach material is appropriate for the recharge material.

3. Offshore material should not be reclaimed in the nearshore zone, while borrow areas at more than 10km offshore will make the operation too expensive.

4. Beaches exposed to the same wave climate as the recharge area give a good indication of the eventual slope of the nourished beach.

5. Groynes will reduce or avoid material loss due to littoral drift. This is certainly of significance if severe storms occur during the nourishment operation.

6. Regular after-recharge surveying is required to evaluate the beach behaviour and to anticipate new maintenance requirements.

2.3 Overfill methods

The overfill factor (or overfill ratio) methods assume that through a process of sorting and winnowing, induced by wave, tide and current action, finer sediments will be lost from the beach. A number of researchers have developed models to quantify the extent of overfill required to mitigate the additional loss of material due to these processes.

In general, to calculate the overfill requirement (ie the additional placement of recharge material in excess of the project dimensions), the following assumptions are usually made:

1. The native material at any particular site represents the most stable sediment grading for the site specific environmental conditions.

2. That, either:

    - the entire volume of recharge material placed on the beach is sorted out by local processes and adopts a grain size distribution similar to the native material, or

- a minimum portion of the recharge will be sorted to achieve a natural grading.

3. Both the native and recharge material are normally distributed, ie

$$f(\phi) = \frac{1}{\sigma_\phi\sqrt{2\pi}} \exp\left[\frac{(-\phi - \mu_\phi)^2}{2\sigma_\phi^2}\right] \tag{2.1}$$

where

$f(\phi)$ = frequency of occurrence of particle class ϕ
μ_ϕ = mean value of ϕ
σ_ϕ = standard deviation of distribution

and

ϕ = $-\log_2 D$ or $2^{-\phi} = D$

where

D = sediment diameter in mm.

The most well established overfill methods are discussed in Sections 2.3.1 to 2.3.4.

2.3.1 Critical overfill ratio - Krumbein (1957) and Krumbein and James (1965)

Krumbein (Reference 6) and Krumbein and James (Reference 7) established a method for estimating the additional quantity of recharge material required if the recharge and native sediments were dissimilar. The methodology involved multiplying the required volume of beach material, assuming a natural grading, by a critical overfill ratio, $R_{\phi crit}$ to determine the quantity of recharge material over and above that required by the absolute dimensions of the proposed replenishment works. The derivation of $R_{\phi crit}$ is given below:

$$R_{\phi crit} = \frac{\sigma_{\phi r}}{\sigma_{\phi n}} \exp\left[-\frac{(M_{\phi n} - M_{\phi r})^2}{2(\sigma_{\phi n}^2 - \sigma_{\phi r}^2)}\right] \tag{2.2}$$

where

M_ϕ = $(\phi_{84} + \phi_{16})/2$, larger values of M denote finer material
σ_ϕ = $(\phi_{84} - \phi_{16})/2$
ϕ_{84} = 84th percentile in phi units
ϕ_{16} = 16th percentile in phi units
$_r$ = denotes recharge material
$_n$ = denotes native material

The $R_{\phi crit}$ calculation cannot be applied to all the possible combinations of recharge and native sediment gradings. The Shore Protection Manual (Reference 8) provides a table setting out four basic relationships between recharge and native materials, and concludes on the applicability of the $R_{\phi crit}$ method. This table is reproduced in Table 2.1.

Table 2.1 Applicability of the $R_{\phi crit}$ Calculations

Category Case	Relationship of Phi Means	Relationship of Phi Standard Deviations	Response to Sorting Action
I	$M_{\phi r} > M_{\phi n}$ Recharge material is finer than native material	$\sigma_{\phi r} > \sigma_{\phi n}$ Recharge material is more poorly sorted than native material	Best estimate of required overfill ratio is given by $R_{\phi crit}$
II	$M_{\phi r} < M_{\phi n}$ Recharge material is coarser than native material		Required overfill ratio is probably less than that computed for $R_{\phi crit}$
III	$M_{\phi r} < M_{\phi n}$ Recharge material is coarser than native material	$\sigma_{\phi r} < \sigma_{\phi n}$ Recharge material is better sorted than native material	The distribution cannot be matched but the fill material should be stable; may induce scour of native material fronting toe of fill.
IV	$M_{\phi r} > M_{\phi n}$ Recharge material is finer than native material		The distributions cannot be matched. Fill loss cannot be predicted but will probably be large.

2.3.2 *The Dean fill factor, R_D - Dean (1974)*

The models developed by Krumbein and Krumbein and James were based on the premise that a percentage of all the fill material, including that coarser than the native, would be lost through the sorting and winnowing action of waves, tides and currents. Dean (Reference 9) modified this approach so that only recharge material finer than the native would be affected. The nett result of this modification was to reduce the amount of overfill required for any particular scheme if a recharge material containing a significant degree of material coarser than that of the native beach was used.

Dean presented the results of his work as a series of isolines as shown in Figure 2.1. Using this figure the Dean fill factor, R_D (similar to $R_{\phi crit}$ discussed in Section 2.3.1) can be calculated.

Although the model derived by Dean represents an improvement over that devised by Krumbein and James (1965), it is worthwhile noting that the basic underlying assumption - that only material finer than the native sediment will be lost - is not supported by recent field and model data (see Section 2.5).

2.3.3 *Overfill ratio - James (1975)*

James (Reference 10) proposed a third method for defining the overfill factor for replenishment schemes with dissimilar recharge and native materials. The overfill criterion developed by James is illustrated graphically in Figure 2.2.

The methodology presented by James continues the critical sediment size concept developed by Krumbein and James (1965) for the finer materials, whilst taking into account the stability of the coarser fractions by assuming that sediment of diameter greater than $M-\sigma$, where M is the mean diameter and σ is the standard deviation, is stable.

The physical effect of this modification is to provide for a degree of short term stability of the beach under storm events, through the 'armouring' of the surface by the coarser materials.

2.3.4 *Maintenance requirements - James (1975)*

James (Reference 10) recognized that beach replenishment schemes involving dissimilar recharge and native materials would not only require the prediction of the overfill nourishment requirements, but may also require an adjusted rate of periodic renourishment due to the more, or less, rapid loss of recharge material, depending on its properties.

To determine the periodic renourishment requirements James defined a renourishment factor, R_J, as the ratio of the rate at which the recharge material will erode relative to the rate at which the native beach material is eroding. The functional relationship of R_J to the properties of the recharge and native materials is given below:

$$R_J = \exp\left[\Delta\left(\frac{M_{\phi r} - M_{\phi n}}{\sigma_{\phi n}}\right) - \frac{\Delta^2}{2}\left(\frac{\sigma^2_{\phi r}}{\sigma^2_{\phi n}} - 1\right)\right] \qquad (2.3)$$

where

Δ = an empirical dimensionless parameter dependent on the textural properties of the native and recharge sediments. James estimated the typical range to be between 0.5-1.5, generally taken as 1 if unknown.

Using this method James was able to produce the set of curves given in Figure 2.3. These show the relative values of R_J for various combinations of recharge and native materials. The greater the value of R_J the greater the required frequency of renourishment.

2.4 Equilibrium method - Pilarczyk, van Overeem and Bakker (1986)

Pilarczyk, van Overeem and Bakker (Reference 11) observed that as a result of sand transport in the coastal zone, any beach recharge will be reshaped, and possibly migrate away from the area requiring protection. They noted in particular, that recharge material placed on an eroding beach, has a limited lifetime. They then considered the design of a beach replenishment scheme as two distinct problems; the equilibrium cross-shore profile resulting from the cross-shore transport of sediment, and the beach planshape resulting from the longshore transport of material.

In order to estimate required quantities of recharge material two factors were considered:

(i) The shape of the active profile in response to the prevailing hydraulic forces.

(ii) The depth to which the profile will develop.

To establish the profile shape for any proposed replenishment scheme Pilarczyk, van Overeem and Bakker suggested the following approaches:

(i) If the historic beach profile is known before significant erosion took place, the existing beach should be renourished to its former profile.

(ii) If the former profile is not known it may be assumed that the coastal profile will develop according to the present profile up to a certain depth. The relationship representing the effect of the particle diameter on the beach shape (see Figure 2.4a) developed by Dean (Reference 12) and co-workers, may be used to determine this limiting depth.

$$h\ (y) = Ay^{m} \approx By^{2/3}\ D^{1/2} \qquad (2.4)$$

where

h = water depth
A = a site specific constant roughly proportional to $D^{1/2}$
D = particle diameter
y = distance from shoreline
m = empirical exponent (usually = $2/3$)
B = unknown empirical factor

To take into account the influence of dissimilar native and recharge materials Pilarczyk, van Overeem and Bakker used the graph of the effect of sediment size on beach slope (shown in Figure 2.5) derived by Dalrymple and Thompson (Reference 13) and the relationship for profile steepness as derived by Vellinga (Reference 14), and shown in Figure 2.4b, to estimate the new beach slope assuming dissimilar materials. This relationship is of the form:

$$\ell_2 = \left(\frac{w_1}{w_2}\right)^{0.56} \ell_1 \qquad (2.5)$$

where

w = fall velocity
ℓ = distance offshore of a given depth contour
1 = denotes native material
2 = denotes recharge material

If the recharge material is coarser than the native material, ie $w_2 > w_1$, then the profile of the replenished beach is steeper than the original profile. The converse effect applies for recharge materials finer than native sediments.

They concluded from Figure 2.4c that it is important to know the depth to which the coastal profile will develop, especially in the case of fine recharge materials where the profile may develop in an almost unlimited fashion in the seaward direction. Pilarczyk, van Overeem and Bakker used the Birkemeier (Reference 15) re-evaluation of Hallermeier's work (Reference 16) to define the cutoff depth of the active beach for practical purposes as:

$$d = 1.75\ H_s \tag{2.6}$$

where H_s corresponds to nearshore wave conditions and is the local significant wave height with a frequency of occurrence of 0.137% (ie 12 hours per year).

For recharge material coarser than the native, the profile of the beach fill was assumed to intersect the original profile at a level inshore of the active profile limit as defined above. In this situation the coastal profile is completely fixed. However, in many cases the required intersection was considered to take place at a lower level. In order to obtain this intersection a transition zone was defined, with a seaward limit given by:

$$d_i = (1.5 \text{ to } 2.0)\ d_1 \approx 3\ H_s \tag{2.7}$$

The thickness of the beach fill between the depths d_1 and d_i was then assumed to decrease linearly as shown in Figure 2.4c.

2.5 Critical review

When considering the use of any of the models outlined in the previous section it is important to bear in mind four fundamental points.

(i) The models and rules of thumb apply almost exclusively to sand-sized beach sediments, and should not be directly applied to shingle or sand/shingle mixtures.

(ii) The models are theoretically based and, with the possible exception of the equilibrium method proposed by Pilarczyk, van Overeem and Bakker (Reference 11), have not been adequately validated against field data.

(iii) The overfill methods produce information only on the possible volumes of material required for a replenishment scheme. The design slopes etc then have to be determined by other methods, if these are available. If the hypothesis put forward in Section 1.2.1 is correct then the subsequent performance of a replenishment is critically linked to both sediment characteristics and design beach slope.

(iv) None of the methods include an allowance for site specific variations in design wave conditions. Thus if the recharge and native materials are similar for two sites the overfill volumes will also be similar. This is despite the fact that one site may experience a much more severe wave climate, and therefore a greater degree of beach sorting and more extensive cross-shore profile changes.

A further, more specific difficulty, arises with Dean's model (Reference 9) as a result of his assumption that only the finer fractions of the sediment grading are lost by selective sorting under wave action. This assumption, whilst appearing logical, is not supported by recent field observations of shingle beach replenishments (References 17, 18 and 19), which appear to show a preferential sorting and removal, in an offshore direction, of the coarsest fraction of the beach sediment. This effect is also prevalent in physical model studies, and may be implicit in the widely held views that material on the UK's shingle beaches is now much finer than it used to be.

It is clear, from the foregoing, that there is a need to develop a model with specific application to shingle beach replenishments. This should allow direct calculation of design beach slopes, including the effects of wave climate, from which recharge volumes can be determined. It is the purpose of this study to develop such a model.

3 *Experimental investigation*

3.1 Physical model

In order to formulate an understanding of the linkages between wave conditions, sediment characteristics and beach response an extensive programme of physical model testing was undertaken. The modelling was carried out in a random wave basin at HR Wallingford. Generally, wave basins are used for testing structures or beaches under oblique wave action. However, for this study a basin was used, because by sub-dividing it into ten separate, 1 metre wide flumes (Figure 3.1), and running only orthogonal waves, a number of different sediment sizes and gradings could be tested simultaneously. This ensured that all the beaches would be subject to exactly the same wave conditions, sequences and durations, thereby ensuring conformity of the results. The basin used was 22 metres long by 18 metres wide, with an operating water depth of 0.4 metres. It was equipped with:

- a random wave generating system comprising one 15 metre electro-hydraulically driven paddle controlled by a micro-computer.
- wave probes for calibrating and monitoring the required wave conditions.
- wave guides to prevent lateral loss of energy.
- a computer controlled 2D bed profiler providing survey coverage at a vertical resolution of 1mm over the test beach sections.

Sub-dividing the basin into the ten separate flumes (Plate 1), each 1 metre wide and 5 metres long, allowed concurrent testing of ten different beaches. The material used for the beaches was crushed anthracite which was selected on the basis of proven scaling laws (Reference 3) to ensure that the model beaches accurately reproduced prototype behaviour. The dimensions of the test facility, coupled with the capabilities of the wave generation system, suggested a nominal model scale of 1:40. The beaches were scaled on this basis and laid at an initial slope of 1:7 down to the basin floor. Figure 3.2 shows a typical cross-section through a model beach.

3.1.1 Design of model beaches

When modelling any beach sediment the three main requirements are to reproduce the beach permeability (and hence slope), the threshold of sediment motion, and the relative magnitudes of onshore/offshore movement.

However, although there are three requirements to be satisfied the model sediment particles have only two main characteristics, that is their size and specific gravity. It is therefore very unlikely that all three modelling requirements can be achieved simultaneously. Indeed some compromise is almost always necessary in the selection of the theoretical characteristics for the model material. These complications in the modelling of beach sediment are further compounded by the fact that there is only a limited range of specific gravities amongst the readily available material. Frequently, therefore, the selection of the model sediment is governed as much by availability as by theoretical considerations.

Many commercial and research studies for shingle beaches have been undertaken by HR Wallingford using crushed anthracite as the model material. The anthracite has the advantage of usually providing a good fit to the theoretical requirements and also of being available on the commercial market in a range of sizes and gradings. This means that it is relatively easy to produce specific model gradings by simply blending appropriate quantities of the commercially available sizes.

For this particular study ten different mixes were prepared, the details of which are given in Table 3.1, below, in prototype (full scale) terms.

Table 3.1 Summary of test beach sediment characteristics

Beach	Sediment Size (mm)			Beach Grading D_{84}/D_{16}
	D_{50}	D_{84}	D_{16}	
1	7.2	10.6	5.5	1.9
2	7.5	15.0	5.4	2.8
3	8.2	21.2	5.3	4.0
4	8.0	27.7	4.9	5.7
5	6.8	16.5	4.4	3.7
6	14.8	32.2	7.1	4.5
7	4.0	12.3	2.7	4.6
8	5.0	6.6	3.1	2.1
9	6.7	18.3	2.2	8.3
10	11.2	31.4	4.5	7.0

The choice of gradings was made following the review of natural shingle beaches presented in Section 1.3. It assumes that three situations are likely to be encountered in future shingle beach replenishments:

(i) The recharge material will be finer than the natural beach sediment but will be of similar grading.

(ii) The recharge material will be finer than the natural beach sediment and will be more widely graded.

(iii) The recharge material will be of similar median size to the natural beach sediment but will be more widely graded.

The selection of the test beaches set out in Table 3.1 is intended to cover these three basic situations, though for completeness and to provide a wider range of data for analysis, and thus reduce any future need for extrapolation of trends, some extension beyond these limits is included. Although other scenarios exist, and are indeed covered in some of the earlier work, it is unlikely that they will be commonly encountered in most future UK shingle beach replenishments. Therefore, these subsequent combinations have not been considered in detail in this study.

Although the hypothesis put forward in Section 1.2 suggests that finer and more widely graded sediments should be laid at flatter slopes to prevent excessive beach erosion, a single design beach slope of 1:7 was adopted for the model tests. This avoided the need to pre-empt the eventual beach slope development, simplified model construction and the test programme, and allowed a direct measure of the extent of any eventual erosion. It also rather more closely mimics present day design practice and therefore allows the consequences of that practice to be fully identified.

3.1.2 Selection of test conditions

Earlier work on shingle beaches (Reference 20) has shown that both wave height and wave period are influential in determining beach profile response. Spectral shape, on the other hand, has been concluded (Reference 21) to have only minor influence on beach profiles provided that the average zero-crossing period, T_m (and not the peak spectral period, T_p) is used to compare the profiles. Wave duration is also an important parameter but only during the very early stages of profile development (Reference 20). Since shingle beach profiles evolve very rapidly (approximately 80% of the total profile change occurring during the first 500 waves of testing), tests run for periods of 1000-5000 waves or greater will display little dependence on wave duration.

For this study twelve test conditions were selected and calibrated. These conditions were chosen to cover a range of sea states with characteristic steepnesses (H_s/L_m) ranging from 0.01 to 0.06. The conditions used are tabulated below in Table 3.2 in prototype (full scale) terms. In each case a JONSWAP spectrum was employed.

Table 3.2 Wave conditions for laboratory tests

Spectrum no.	Significant wave height H_s (m)	Mean wave period T_m (sec)	Characteristic sea steepness (H_s/T_m)
1	2.98	5.70	0.060
2	2.68	4.47	0.057
3	2.37	5.20	0.056
4	2.11	4.85	0.058
5	2.51	5.67	0.050
6	1.99	5.66	0.040
7	1.81	5.37	0.040
8	1.60	5.10	0.039
9	1.40	4.71	0.030
10	1.49	5.63	0.030
11	1.02	5.65	0.020
12	1.00	7.87	0.010

Throughout testing the mean water level was kept constant. This follows common practice and recognises that gradually varying water levels, on the time scale of tidal variations, do not affect the shape or slope of beach profiles (Reference 20). Their effect, for deepwater beaches, is restricted to determining the location of the profile on the beach face.

3.2 Test programme

Following calibration of the test conditions the model beaches were constructed to the required 1:7 plane profile in each of the flume sections. The test conditions were then run for 5000 waves starting with the least severe condition and working up to the most severe. In this way profiles were built up on top of each other thus limiting the need for substantive re-moulding between tests.

Measurements of beach profile were taken at the outset of testing and then at 1000 and 5000 waves. At the end of each test, beach samples were collected from the crest, shoreline, wave breaking zone and offshore regions of the beach profile. The samples were collected both from the surface and from approximately 1 metre (full scale) depth within the beach. All samples were subsequently size graded using standard sediment laboratory techniques. To support the measurement programme continuous visual observations were made and recorded by the modelling staff.

4 Presentation and discussion of test results

4.1 Data presentation

Data was collected from the model tests in two basic formats:

- Computer generated beach profile results
- Sediment size distributions for pre-defined locations on the beach profile.

In total 250 beach profiles were recorded and nearly 1000 sediment samples collected. It is not possible to present all this information within this report.

Instead, therefore, typical results have been presented where they assist understanding of points made in the text or where they are necessary to illustrate trends. A summary of profile statistics, relevant to the analysis set out in Chapter 6, is provided in Appendix 1. The statistics include:

- Beach crest location and height
- Transition point location and depth
- Beach toe location and depth.

All dimensions are referenced to the still water level shoreline, while the terminology adopted is that used in Reference 20 and reiterated in Figure 4.1.

4.2 Data trends

4.2.1 Profile response

Typical results for beaches 1-4 are shown in Figures 4.2 and 4.3 for, respectively, a severe storm condition (Spectrum 1: H_s = 2.98m, T_m = 5.7s) and more moderate seastate (Spectrum 12: H_s = 1.0m, T_m = 7.87s). From these figures it is evident that under storm conditions the wider beach grading (Beach 4) experiences a far higher degree of erosion than the more natural grading. Furthermore, under constructive swell wave conditions, beaches with wide sediment grading are less inclined to accrete than those with narrow grading. This tendency manifests itself both in the shoreline positions, which are located further seaward for the narrower grading, and in the reduced volumes of material contained in the beach crests formed by the more widely graded sediments. Analysis of the results for all test conditions also shows that the wider gradings continue to experience erosion even when the more natural beach gradings are showing accretionary behaviour.

This trend is illustrated in Figure 4.4, where the still water level retreat relative to the initial profile shoreline is plotted as a function of beach grading for three test conditions (Spectra 1, 6 and 10). Figure 4.5 shows a similar result for beach crest retreat. In both plots the most severe wave condition produces a retreat for the widest grading that is approximately 15 metres more severe than that recorded for the more natural narrow beach grading. Clearly, if this trend towards more extensive erosion is not taken into account during the design of a replenishment scheme using dissimilar sediments, the protection afforded by that scheme will be significantly impaired.

The most likely reason for the increased erosion of the widely graded laboratory beaches is that they were placed at too steep an initial slope. Whilst 1:7 may be a reasonable slope for more natural shingle beach gradings it appears that the reduced permeability of the wider grading necessitates the beach being placed at a flatter slope. In this respect there is, as previously mentioned, a considerable analogy between widely graded shingle beaches and beaches composed of finer sediments. In both cases if the initial slope is too steep the beach will strive towards a shallower gradient. In the process material will be drawn from the upper part of the beach and deposited on the lower profile, causing the extreme erosion observed in the laboratory study.

In terms of the response of the individual beaches to changes in the incident wave conditions, the results are very much in accordance with earlier research findings (Reference 20). Generally the steeper storm waves (H_s/L_m = 0.06) produce erosion, with the severity of the erosion decreasing with reducing wave height and period (Figure 4.6). Erosion is also directly related to sea steepness with more moderate conditions, characterised by lower steepnesses, producing at first less erosion and then gradually promoting build-up of the beach. This trend is clearly shown in Figures 4.7 and 4.8 for narrow and widely graded sediments respectively. The delayed transition from erosion to accretion, referred to earlier, is also evident.

4.2.2 Sediment distributions

It is recognised that under wave action there is a sorting, or re-working, of beach sediments which results in the finer material moving down into the core of the beach leaving a coarser armouring layer on the surface. In many respects this process produces results similar to the actions of a sieve, and indeed it has analogies with a number of aspects of powder mechanics. The coarser surface layer is however in itself subject to cross-shore sorting under the action of waves, which results in a variable distribution of coarse material across the surface of a beach profile. Figures 4.9 and 4.10 show typical sediment size distributions across the surface of eroding and accreting beach profiles respectively. Clear differences can be seen between the two sets of distributions. These are in part related to the level of wave energy incident on the beach, and therefore the extent of the vertical sorting of material, but in part are also a function of the different modes of sediment transport associated with accreting and eroding beaches.

The most pronounced changes in sediment size occur on the storm wave (eroding) profile (Figure 4.9) where the much higher levels of wave energy have resulted in a more extensive vertical sorting of the sediments and therefore a surface layer composed of much coarser material than that of the basic background population. The median size of the surface sediments varies across the profile in a manner similar to that reported by other researchers (References 20 and 22), being coarsest in the wave breaker region and at the beach crest but much finer in the area around the shoreline. Within the results there is a clear dependency on the initial beach sediment grading with the wider grading, which implicitly includes a higher proportion of coarse sediment, exhibiting surface distributions consistently coarser than those of the narrower beach grading.

The results of the sediment analysis suggest that whilst erosion occurs over the whole sub-aerial part of the storm profiles, the size of the sediments moving offshore is determined by location on the profiles. Thus, at the beach crest the finer fractions of the surface layer are moved offshore while closer to the shoreline there is a general offshore migration of the entire surface layer leaving a surface sediment distribution similar to that of the background beach population.

The surface sediment distributions associated with more moderate wave conditions (Figure 4.10) are generally less well developed than those of storm profiles because of the lower levels of wave energy and hence restricted vertical sorting. This effect can clearly be seen in the ratio's of sample to population D_{50} which range from $0.3 < D_{50s}/D_{50p} < 2.0$ for the accreting profiles to $1.0 < D_{50s}/D_{50p} < 6.0$ for the eroding profiles.

As shown in Figure 4.10, despite the poorly developed vertical sorting, the accreting beach profiles exhibit a steady coarsening of the surface layer from the beach toe up to the crest. This is indicative of the preferential onshore movement of surface sediment leaving the finest fractions of the beach material exposed at the foot of the profile whilst producing a deeper coarser layer at the beach crest. As with the eroding beach profiles the results suggest that given beach materials with similar median sizes, the coarsest surface layers occur with the most widely graded sediments.

The sediment distribution results outlined above provide further evidence of a tendency for preferential offshore movement of coarse sediments during storm action, which appears to be more pronounced, or perhaps just more obvious, for wider beach gradings. The findings support tentative field and model observations (Section 1.2) that widely graded sediments evolve towards a more typical beach grading through the preferential loss of the coarsest fractions.

5 Derivation of design methodology for beach replenishment

5.1 General

It is becoming apparent that when considering any shingle beach replenishment the design slope should be matched to the sediment characteristics to ensure that erosion (due to cross-shore sediment transport) under the design storm is minimised. To do this it is necessary to find some means of relating an equilibrium beach slope to the sediment characteristics, in particular median sediment size and grading, and to the incident wave conditions. This chapter presents the results of a detailed analysis of the test results leading to the derivation of an appropriate design methodology.

5.2 Analysis of test data

In real life beaches develop relatively complex profiles which may include both convex and concave sections, and on which transient features will develop, grow and disappear. The prediction of these types of profiles is covered in a relatively simplistic fashion in Reference 20, with more detailed results being available, mainly for sand beaches, through the use of sophisticated morphodynamic process models. For the purpose of this study it is, however, more appropriate to consider beach profiles as being approximated by a single 'equilibrium' slope, defined as the straight line between the beach crest and the lower limit of profile deformation (the wave base).

As the beach crest erodes under storm conditions, and the wave base extends seawards, so the 'equilibrium' beach slope will flatten out to a more gentle gradient. We may therefore make use of the qualitative results outlined in Chapter 4 to relate the beach slope to the sediment characteristics, defined as D_{84}/D_{16} and D_{50}, and to the wave conditions defined as H_s and T_m.

The previous analysis has shown that beach slope decreases as the characteristic sea steepness, H_s/L_m, and the sediment grading, D_{84}/D_{16}, increase. Additionally, we know that finer sediments are associated with flatter slopes and that therefore a function of the form H_s/D_{50} is inversely related to the beach slope.

Denoting the 'equilibrium' slope by Sin θ (where θ is the angle between the slope and a horizontal line) we may therefore propose that:

$$\text{Sin}\ \theta = f(H_s/L_m,\ D_{84}/D_{16},\ H_s/D_{50})^{-1} \qquad (5.1)$$

Using modified regression analysis the precise dependency of Sin θ on each of the listed variables can be determined, yielding an expression of the form:

$$\text{Sin } \theta = f\left[(H_s/L_m)^{-0.220},\ (D_{84}/D_{16})^{-0.394},\ (H_s/D_{50})^{-0.306}\right] \tag{5.2}$$

From this it would appear that Sin θ is most strongly correlated with sediment grading as denoted by D_{84}/D_{16}. The influence of sediment size and sea steepness on the beach slope is less pronounced.

Plotting this expression against the test data and applying a regression analysis to the resulting distribution allows the final form of the relationship to be established as,

$$\text{Sin } \theta = 0.206\left[(H_s/L_m)^{-0.220}\ (D_{84}/D_{16})^{-0.394}\ (H_s/D_{50})^{-0.306}\right]^{0.567}$$

$$\text{or Sin } \theta = 0.206\left(\frac{H_s}{L_m}\right)^{-0.124}\left(\frac{D_{84}}{D_{16}}\right)^{-0.223}\left(\frac{H_s}{D_{50}}\right)^{-0.174} \tag{5.3}$$

Figure 5.1 shows the correspondence between the measured data and the values predicted using equation 5.3, together with the 90% confidence limits. It can be seen that Equation 5.3 accurately reflects the trends in the measured results.

Equation 5.3 can also be reproduced in graphical terms as shown in Figures 5.2 to 5.8. In the figures, beach slope (in degrees) is plotted against median sediment size, D_{50}, for a range of possible beach gradings. In each case the significant wave height is held constant at 2.0 metres.

For the typical shingle beach values of D_{50} = 15.5mm and D_{84}/D_{16} = 3.42 (as derived in Section 1.3) the resultant equilibrium slopes under waves with H_s = 2.0m range from 1:7.6 (7.4°) for a sea steepness of 0.005 through to 1:10.4 (5.5°) for storm seas with a characteristic steepness of 0.06. These values seem realistic and inspire some confidence in the application of Figures 5.2-5.8 and Equation 5.3 to other beach situations. The plots also confirm that beach slope increases with increasing material size and decreases with increasing sediment grading, though the trend is less pronounced for finer sediments than it is for coarse material.

The effect of wave height, H_s, on the beach slope/sediment size relationship is shown in Figure 5.9 for a sediment grading, D_{84}/D_{16} = 3.5 and a characteristic sea steepness, H_s/L_m= 0.06. From the figure it can be seen that the 'equilibrium' beach slope decreases with increasing wave height and that the rate of decrease is greatest for the coarser sediments. This is much as expected since it is well known that severe storms act to flatten a beach, thus increasing its surf zone volume and ensuring a constant level of wave energy dissipation per unit volume of surf zone. Similarly, more moderate wave conditions build up beaches, reduce the surf zone width, and hence produce a steeper beach slope.

5.3 Physical background

Quick (Reference 23) linked the cross-shore movement of sediments to the stress produced by the momentum flux of a breaking wave. He assumed that the amount of sediment movement depends on the stress intensity (seastate characteristics) and the roughness of the bed (sediment characteristics), and that in most equilibrium situations the nett offshore stress is balanced by an onshore stress produced by beach permeability. Equating these two concepts Quick derived the following expression for the 'equilibrium' slope,

$$\mathrm{Sin}\ \theta \propto \lambda^{-\frac{1}{2}} \frac{D_{10}^{0.7}}{D_{50}^{0.4}} H_s^{-0.25} \qquad (5.4)$$

where λ is the beach surface friction coefficient.

Assuming that λ is a constant for shingle beaches we can write

$$\mathrm{Sin}\ \theta = \frac{D_{10}^{0.7}}{D_{50}^{0.4}} H_s^{-0.25} \text{ or, } \mathrm{Sin}\ \theta \approx C \left(\frac{D_{50}}{D_{10}}\right)^{-0.7} \left(\frac{H_s}{D_{50}}\right)^{-0.3} \qquad (5.5)$$

where C is a constant.

Comparing this expression to equation 5.3 we can see that there is reasonable correspondence in terms of the relative importance of the sediment size parameter, H_s/D_{50}, but that Quick's theoretical arguments have elevated the importance of the grading parameter (D_{84}/D_{16} or D_{50}/D_{10}) by a factor of nearly 3. This discrepancy is in part due to the different definition of the grading parameter since Quick's definition would result in smaller values than those given by the D_{84}/D_{16} term used in this study. The final point is that for full qualitative agreement between the two equations the 'constant', C, in Equation 5.5 would have to be a function of sea steepness. This apart, the general similarity between the two expressions suggests that Quick's theoretical reasoning does provide an insight into the physical background of Equation 5.3

6 Application of the equilibrium slope method

For engineers faced with designing a beach replenishment scheme the implications of the findings presented in the previous chapters are particularly relevant. A beach will need to be placed at an initial construction slope close to its 'equilibrium' slope, under the design wave conditions, if it is not to suffer severe erosion during storms. In practice, it is probably advisable to use Equation 5.3 to determine the 'equilibrium' slope for a range of wave return periods, and for a range of sediment gradings and sizes representative of those likely to be obtained from the proposed dredging area. A compromise slope can then be adopted based on a full understanding of its sensitivity to possible variations in wave conditions or sediment characteristics.

There are two further aspects that also need to be borne in mind when designing replenishments using dissimilar sediments. Firstly, because wave run-up remains nearly constant for the range of sediment sizes usually considered for shingle beach replenishment schemes, the adoption of wider gradings and finer materials will result in a considerable increase in the replenishment volumes required. This is illustrated in Figure 6.1 where the increase in beach volumes relative to that required for a typical shingle beach replenishment (D_{50} = 15.5mm, D_{84}/D_{16} = 3.42) is shown as a function of sediment size and material grading. The results clearly demonstrate that while the use of materials not required by the aggregate industry may result in a cost saving for a beach replenishment, it is likely that if those materials are significantly finer or more widely graded than the natural beach material, then any cost saving may be partly offset by the increased volumes required.

The second point to bear in mind is that in time the replenished beach will gradually evolve a grading more closely allied to that of the natural beach. In doing so, however, it will automatically adopt a steeper 'equilibrium' slope. The implication of this is that the beach will maintain a natural balance between grading and 'equilibrium' slope; thus provided losses in the longshore direction are minimised, the standard of defence provided by the beach is likely to be maintained.

6.1 Recommended procedure

In applying the results of this research to the design of shingle beach replenishments using dissimilar sediments it is recommended that the following approach to the selection of the optimum beach profile is adopted.

(i) Identify possible sources of replenishment material and undertake surveys in sufficient detail to establish the extent of variations in material size and grading within the resource area.

(ii) Determine wave climate at the replenishment site and calculate design wave conditions corresponding to a range of appropriate return periods.

(iii) Use Equation 5.3 to establish an 'equilibrium' slope envelope using all likely combinations of wave conditions and sediment characteristics.

(iv) Select an appropriate slope from within the derived envelope. The selection procedure may make use of probabilistic techniques to assess the most cost-effective or lowest risk option, or may simply result in a mean or extreme lower slope being employed. If the latter option is adopted it should be recognised that if the design 'equilibrium' slope is too shallow it will result in a very costly scheme and one which may, under mild conditions, also suffer a pronounced onshore movement of material, resulting in a beach that is much wider at the shoreline than originally intended.

(v) Calculate a design crest level which will usually be set at or above the 2% wave run-up exceedance level. This level could be obtained by the application of suitable formulae such as those set out in Reference 20.

(vi) Select an appropriate beach width either to provide the standard of defence required taking account of maintenance commitments or to meet amenity or recreational requirements.

(vii) Calculate and cost the volumes of material required per metre run of scheme.

The application of this type of approach in the selection of cross-sections for future beach replenishments will assist the cost-effective usage of the available recharge material, and will also improve understanding of the likely sensitivity of the selected design profile to eventual variations in material size or grading, or incident wave conditions.

6.2 Recommendations for use of the equilibrium slope model

The recommended approach for the design of optimal replenishment profiles is set out in Section 6.1 of this report. A major part of this procedure should involve the assessment of the sensitivity of the selected 'equilibrium' slope to,

(i) slight changes in wave conditions occurring either as a result of future climate change or in response to spatial variations in coastal exposure or bathymetry within the recharge frontage.

(ii) the selection of the design seastates ie. do more frequently occurring or more severe conditions significantly alter the equilibrium slope?

(iii) variations in the size and grading of the recharge material either as a result of the winning and placement techniques adopted or because of natural variability within the dredging area/quarry.

Additionally, the beach grading will gradually evolve under the action of waves and currents. This may be a result of preferential sorting of the sediment sizes or due to mixing between the recharge and existing materials. Either way it will often be useful to have some idea of the potential longer term variation in the 'equilibrium' slope, though it is always likely that the worst case (ie. the flattest slope) will exist early on in the design life of the scheme, when the recharge material is at its most dissimilar.

In certain situations the equilibrium slope model will predict beach slopes which require unacceptably large volumes of material to construct. Where this is the case there may need to be some relaxation of the required defence standards or an increase in the frequency / volumes of maintenance replenishments. In other situations the model may predict a recharge volume less than that required by a replenishment using equivalent material. Whilst it may be tempting to accept this reduced volume, in practice other influences, such as recharge layer thickness and the relative permeability of the native and recharge materials, become important and may have a detrimental impact on the recharge stability. It is therefore recommended that the recharge volume is never less than that which would be required for recharge material equivalent in size and grading to that of the existing beach.

7 Comparison of equilibrium slope method against field data and alternative models

As almost the final step in the development of any new design model or methodology it is necessary to validate the model results against available field data, and to compare predictions against those provided by other models. This process ensures that the new model is providing reliable results and helps to establish realistic ranges of applicability.

7.1 Comparison against field data

For the comparison against field data, measurements were sourced from both published reports and site specific studies undertaken by HR Wallingford. Data was obtained for a number of shingle beaches, with differing median sediment sizes and gradings, and also for a range of typical sand beaches plus the rather unique beach at Carlyon Bay in Cornwall which is composed of sand from the China Clay workings but behaves much as a shingle structure. Full details of the field data used in the comparison are given in Table 7.1.

Table 7.1 Field data used in model validation

Beach	Sediment type	Sediment size, D_{50} (mm)	Sediment grading D_{84}/D_{16}	Typical beach slope (1:n)
Milford on Sea, Hampshire	Shingle	17.2	1.9	5-6
Seaford	Shingle	13.7	2.73	7.5
West Bexington, Chesil	Shingle	10.0	1.34	8.8
Hythe, Kent	Pea shingle	5.2	3.13	10
-	Sand	0.30	1.6	30-50
-	Sand	0.15	1.6	50-100
Carlyon Bay, Cornwall	China Clay sand	1.71	2.73	7

Equation 5.3 was used to calculate an equivalent 'equilibrium' slope for each of the field data sets for a range of wave conditions. The resulting comparisons are plotted in Figure 7.1 It is immediately obvious that for each of the test cases there is a range of possible predicted 'equilibrium' slopes depending on the precise wave conditions utilised in Equation 5.3. In practice, there will also be a day-to-day variation in the measured beach slopes as they respond to natural variations in wave climates. However, information on this natural beach slope variation is not readily available and therefore the measured slopes plotted in Figure 7.1 must be treated as averaged values. A further point to bear in mind is that beaches located in different parts of the country will, depending on their relative exposure, experience different wave climates. Some beaches may be located in swell dominated environments and have a correspondingly steep 'equilibrium' slope, while others will be in storm dominated environments and will exhibit flatter slopes. Within the scope of this comparison it has not been possible to take into account the different climates affecting the various test sites, and this will therefore automatically introduce some errors into the results.

Despite the limitations described above, Figure 7.1 shows generally good agreement between the measurements and predictions for the shingle sized materials. The results are less encouraging for the sand beaches which generally have measured slopes much flatter than those predicted by the 'equilibrium' slope method. This suggests that the method cannot be applied to sand sized sediments.

Although the results of the comparison between predicted and field data have yielded encouraging results for shingle sized sediments, a more extensive comparison including site specific wave climates and temporal variations in natural beach profiles is advisable to improve confidence in the model results, and to assist in providing a clearer definition of the limits of applicability. This more detailed comparison will require good quality field data; a requirement which should be satisfied by the study of replenished beaches currently being funded at HR Wallingford by the Ministry of Agriculture, Fisheries and Food under Research Commission FD0702.

7.2 Comparison between the equilibrium slope and alternative models

In order to provide a clearer picture of the predictive capabilities of the 'equilibrium' slope model compared to previously advocated sand beach design methodologies (see Section 2) a number of cases were set up against which the models could be tested. The test cases assumed that it was required to increase the crest width of an existing shingle beach (D_{50} = 15.5mm, D_{84}/D_{16} = 3.42) by 5 metres. Four different replenishment scenarios were investigated based on criteria set out in Reference 8 (see Table 2.1). These were:

- Type 1 Recharge finer and more widely graded than native material ie. D_{50r} = 8mm, D_{84r}/D_{16r} = 10.
- Type 2 Recharge coarser and more widely graded than native material ie. D_{50r} = 20mm, D_{84r}/D_{16r} = 10.
- Type 3 Recharge coarser and more narrowly graded than native material ie. D_{50r} = 20mm, D_{84r}/D_{16r} = 2.
- Type 4 Recharge finer and more narrowly graded than native material ie. D_{50r} = 8mm, D_{84r}/D_{16r} = 2.

The volumes of material predicted for each of the replenishment scenarios were calculated using the overfill ratio method of James (Reference 10), as advocated by the Shore Protection Manual (Reference 8), the model proposed by Pilarczyk, van Overeem and Bakker (Reference 11) and the equilibrium slope model derived in this study. The equilibrium slope model includes the influence of wave height and sea steepness, and this was therefore included in the comparison by running the model for four different design sea states as tabulated below. This allowed the relative importance of wave climate in the calculation of replenishment volumes to be assessed. Since the models of James and Pilarczyk et al are insensitive to wave climate they provide only a single result for each of the replenishment scenarios.

Table 7.2 Design seastates used in model comparisons

Design seastate	Wave height H_s (m)	Sea steepness H_s/L_m
1	2.0	0.005
2	1.0	0.06
3	2.0	0.06
4	4.0	0.06

The results of the calculations are presented in Figures 7.2 to 7.5 in terms of a relative volumetric increase, which is defined as,

"The volume of recharge calculated by a model divided by the volume required had the replacement been undertaken using material equivalent to that of the existing beach."

Each figure contains results for one particular replenishment scenario under each of the design seastates.

The results for replenishment type 1 (Figure 7.2) show that the James and Pilarczyk methods are in good agreement suggesting that despite the dissimilarity between the recharge and native materials little increase in volume is required. The equilibrium slope method contradicts this, suggesting that an increase in volume is required and that the increase is strongly dependent on the design wave conditions. A similar dependency is also apparent in Figures 7.3 to 7.5.

For replenishment type 2 (Figure 7.3), the overfill ratio presented by James suggests that a 50% increase in replenishment volume is required whilst the model of Pilarczyk et al again suggests little increase. The equilibrium slope model produces results that are again dependent on seastate but which are in general agreement with the overfill ratio results.

Results for replenishment type 3 (Figure 7.4) have only been plotted for the Pilarczyk et al and equilibrium slope models. This is because the James method predicts that the replenishment will be unstable, requiring volume increases in excess of 1000%. The Pilarczyk model again shows little change in volumes required whilst the equilibrium slope model predicts a general reduction in beach volume requirements. In practice, it would be unwise to reduce the recharge volume below that required for an equivalent sediment recharge because other factors such as the thickness of the replenishment layer and the relative permeability of the two materials become increasingly influential in determining the beach profile behaviour.

Finally, replenishment type 4 (Figure 7.5) shows trends that are very similar for all of the design methodologies. For this replenishment scenario the equilibrium slope model shows only a minor dependency on wave conditions.

In attempting to analyse the results it is necessary to emphasise that both the James and Pilarczyk models are primarily intended for use with sand-sized sediments. These sediment generally have a much narrower range of gradings than those considered for the shingle beaches used in this study, and a more restricted range of median sediment sizes. The models are therefore much less sensitive to changes in these parameters than they ought to be for application to dissimilar shingle recharges. The apparent inability of the Pilarczyk model to predict volumetric changes in beach recharge requirements is largely due to its use of the sediment fall velocity to describe the influence of changes in median sediment size. Whilst fall velocity is strongly correlated with sediment size for fine sand-sized particles it is much less well correlated for coarser shingle. Typically an increase in shingle size from 8mm to 20mm corresponds to a 47% increase in fall velocity, while an increase in sand size from 0.15mm to 0.3mm corresponds to a 400% increase in fall velocity. Models which utilise only fall velocity to describe sediment characteristics are therefore much less able to account for the influence of size variation amongst coarser sediments. This appears to severely restrict the application of Pilarczyk's model to shingle beach replenishments.

7.3 Summary

The results of the various comparisons have shown that the equilibrium slope model has application to shingle beach replenishments using dissimilar sediments. However, for sand-sized sediments the model underpredicts the 'equilibrium' slope. For the time being, therefore, selection of the optimum cross-section for these types of replenishment is best done using either the James or Pilarczyk models, though these approaches are limited by their inability to account for the influence of wave climate. In later stages of research it may be possible to extend the equilibrium slope concept to sand beaches thereby allowing a more complete evaluation of profile response. Further research is also justified to improve the validation of the equilibrium slope model against field data.

8 Conclusions and recommendations

8.1 Conclusions

A comprehensive series of physical model tests has been undertaken to explore the behaviour of shingle beaches composed of sediments which are dissimilar from those of natural beaches by virtue of either size or grading. The results have allowed the development of a model for optimising the design profile for beach replenishments based on an 'equilibrium' slope concept. The model has been validated against available field data and compared with alternative design approaches.

The major conclusions arising from the research are summarised below:

(i) Natural shingle beaches may be considered to have an average D_{50} of 15.5mm (std dev = 6.0mm) and an average grading D_{84}/D_{16} of 3.42.

(ii) The profile response of shingle beaches is a function both of the incident wave conditions, described by significant wave height H_s, and mean sea steepness, H_s/L_m, and the beach sediment characteristics, in particular median sediment size, D_{50}, and grading, D_{84}/D_{16}.

(iii) The effect of sediment grading on the beach slope is analogous to that of sediment size, in that, by virtue of their lower permeability, finer or more widely graded sediments produce flatter slopes, while coarser or more narrowly graded sediments produce steeper slopes.

(iv) Previous methods of assessing recharge volumes for beach replenishment schemes are restricted to sand-sized sediments and take no direct account of the important influence of wave climate and design wave conditions. They are not appropriate for the design of shingle beach replenishments.

(v) The placement of widely graded sediments at steep, but typical, shingle beach slopes will result in severe erosion of the beach under storm conditions as it attempts to adopt a profile more in keeping with its sedimentary characteristics. The extent of this erosion may significantly reduce the anticipated level of protection provided by the scheme.

(vi) Under constructive swell wave conditions, beaches with a wide sediment grading are less inclined to accrete than those with a narrow grading. Wider gradings also result in a delayed transition from beach erosion to accretion compared to that of more narrowly graded beach sediments.

(vii) The model results support previous speculation conceming a preferential offshore movement of coarse sediments during storm action. This process appears to be more pronounced for wider beach gradings.

(viii) In keeping with previous research findings the model results show that distinctly different surface sediment distributions exist for erosive and accretionary beach profiles. Erosion profiles show the coarsest material occurring in the wave breaker zone, and to a lesser extent at the beach crest, with finer material located at the shoreline. On accretion profiles there is a steady increase in mean sediment size from the toe to the crest of the profile.

(ix) Equation 5.3 allows the 'equilibrium' slope of shingle sized sediments to be predicted as a function of median sediment size, sediment grading and incident wave climate. The results have been shown to be in general agreement with field data.

(x) Over time a replenished beach will evolve a grading more closely allied to that of a natural beach. In doing so it will automatically adopt a steeper 'equilibrium' slope. Thus, provided losses in the longshore direction are minimised, an appropriately designed beach replenishment cross-section will provide a consistent standard of defence even if it employs dissimilar sediments.

8.2 Recommendations for future research

This study has highlighted two areas which need to be explored more fully. These are, firstly, the requirement for further validation of the 'equilibrium' slope concept and model against good quality field data. Some data is presently being collected under a related MAFF research commission (FD0702) which is examining the performance of replenished beaches. This should be augmented by measurements of the short term (ie. single storm) response of replenished beaches. Efforts should be made to ensure that when complete this entire data set is used for a full evaluation of the ideas and model put forward in this report.

Secondly, the validation that has been undertaken has shown that the existing equilibrium slope model is not applicable to sand-sized sediments. This would not necessarily be of concern if other methods and models were capable of providing realistic results for sand beaches. However, to an extent they are all handicapped by their inability to take account of wave conditions within the calculation of the recharge volumes. There is therefore a strong case to be made for extending the present 'equilibrium' slope concept to sand-sized sediments and sand beach replenishments.

9 Acknowledgements

This report describes work carried out by the Coastal Group at HR Wallingford. The study was designed and supervised by Dr K A Powell who also carried out the majority of the analysis and reporting. Mr R E Sadler and Mr J C Borges undertook the experimental work, while Mr P B Sayers wrote the literature review section of the report.

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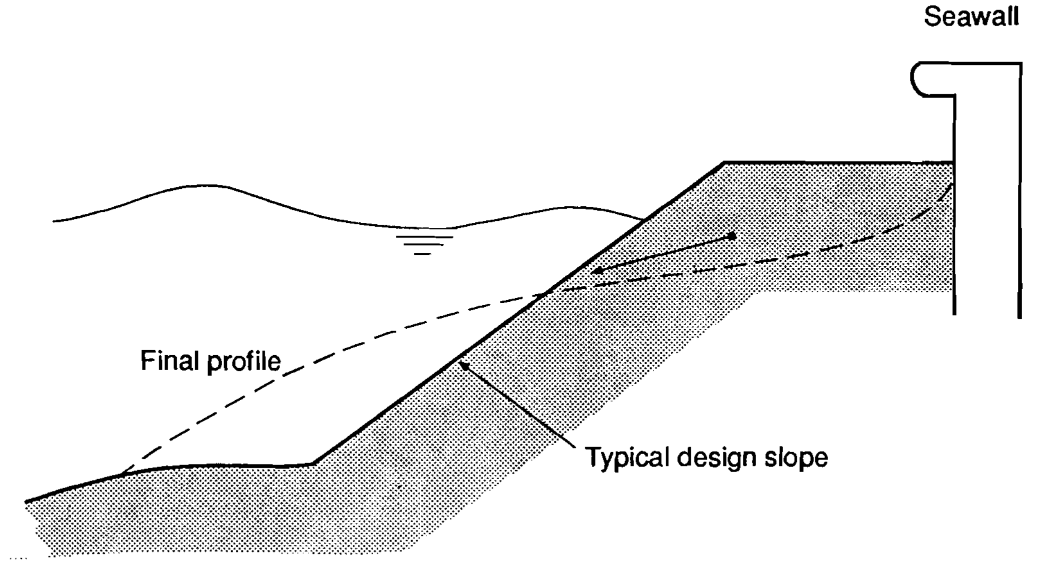
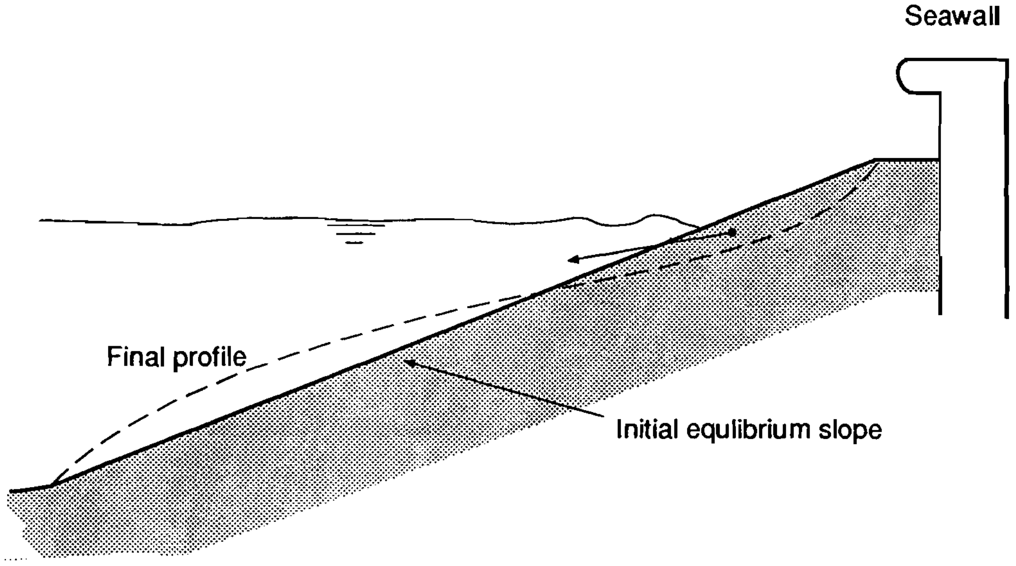
Figures



(a) Response of widely graded material laid at typical (1:7 - 1:8) slope



(b) Response of widely graded material laid at equlibrium slope

KP/1.1/7-93/OW

Figure 1.1 Postulated response of widely graded beaches to storm action

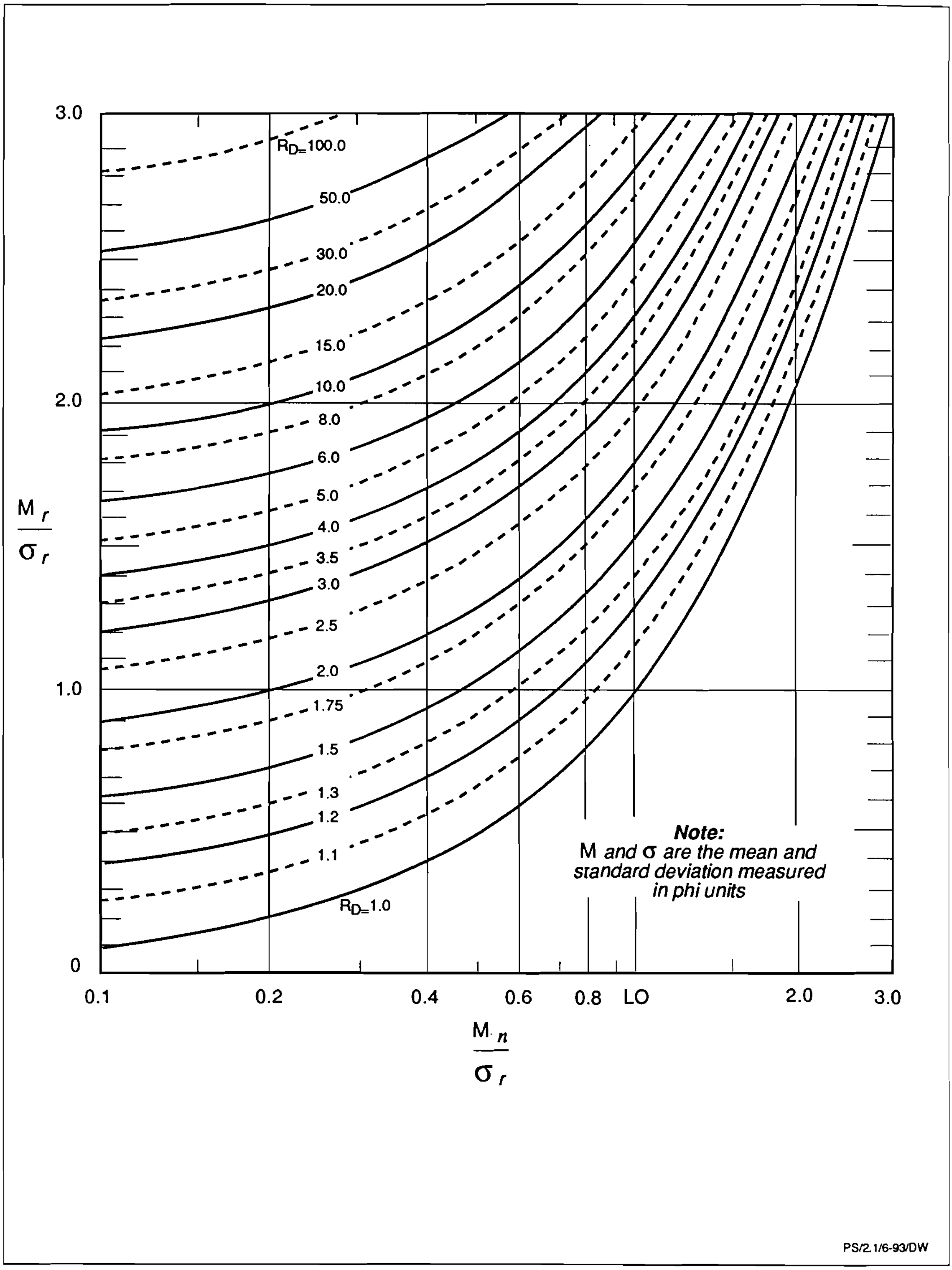


Figure 2.1 Isolines of the Dean fill factor (R_D) (Dean, 1974)

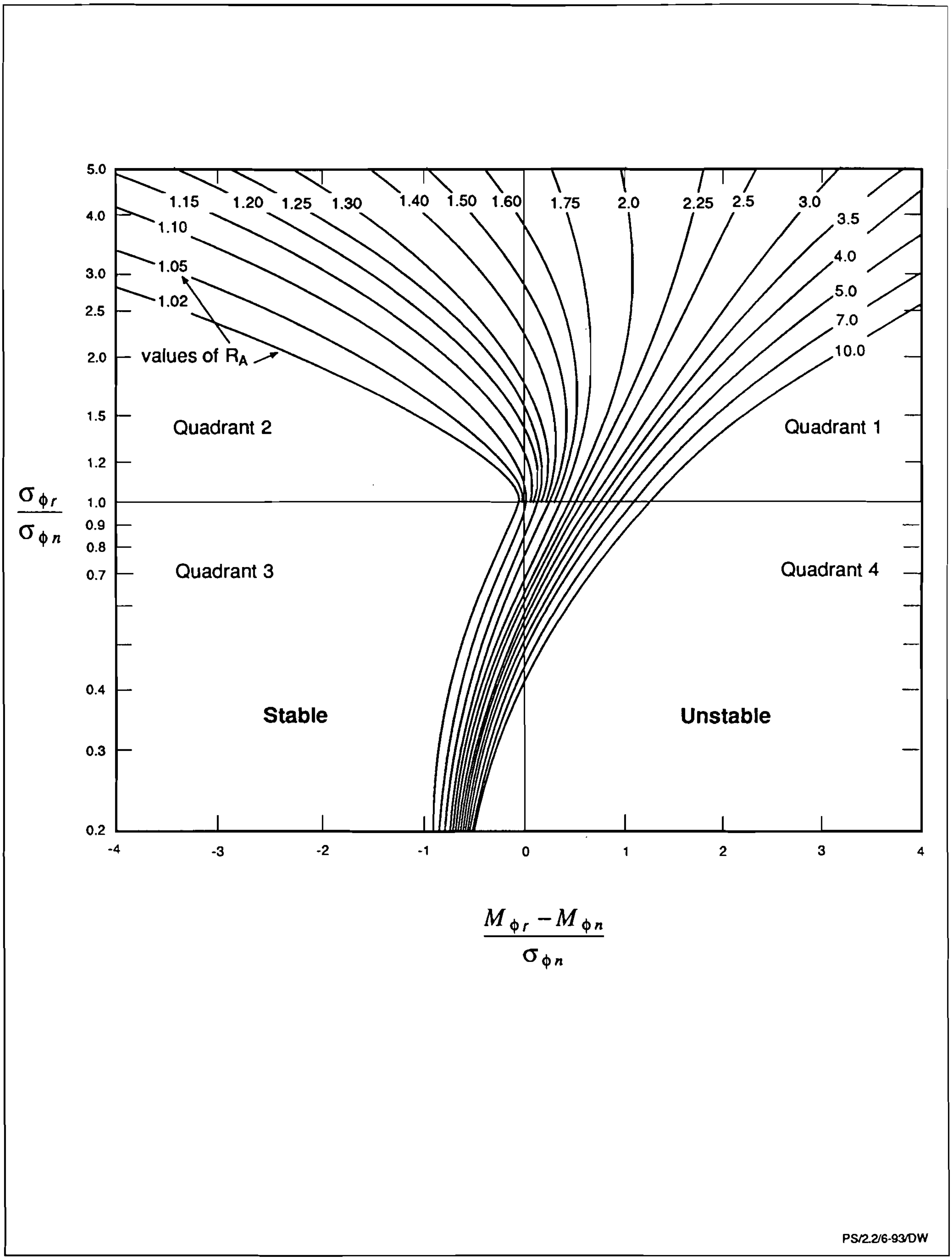
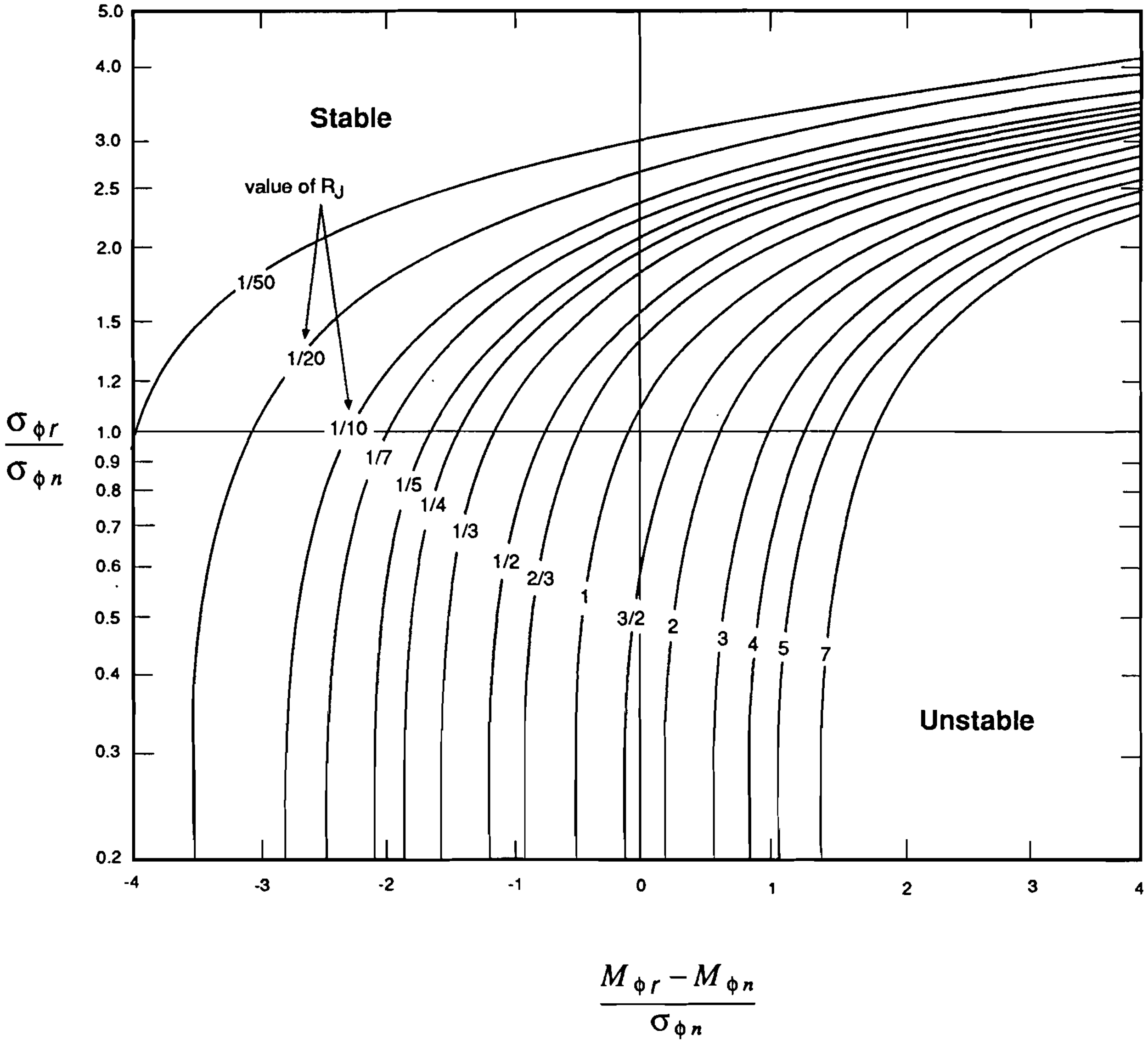
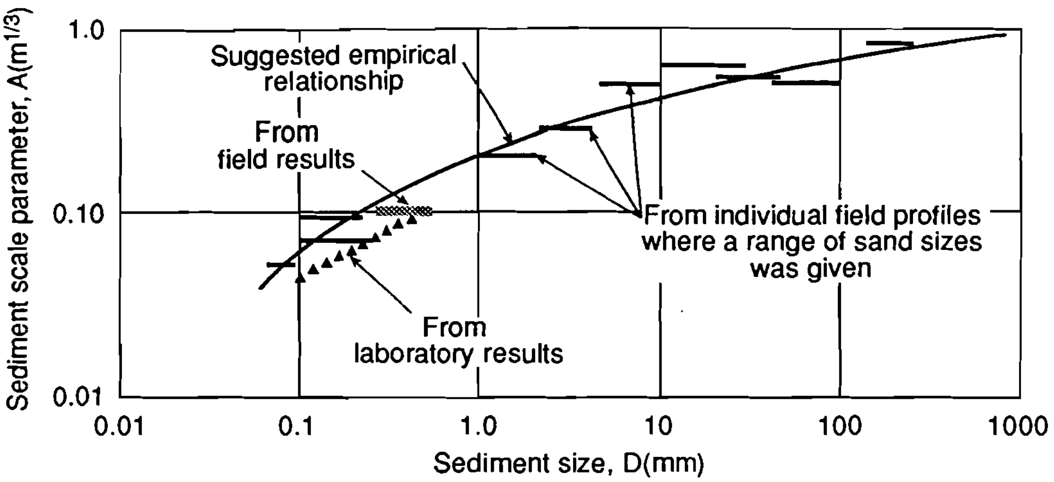


Figure 2.2 Isolines of adjusted overfill factor, R_A (James 1975)

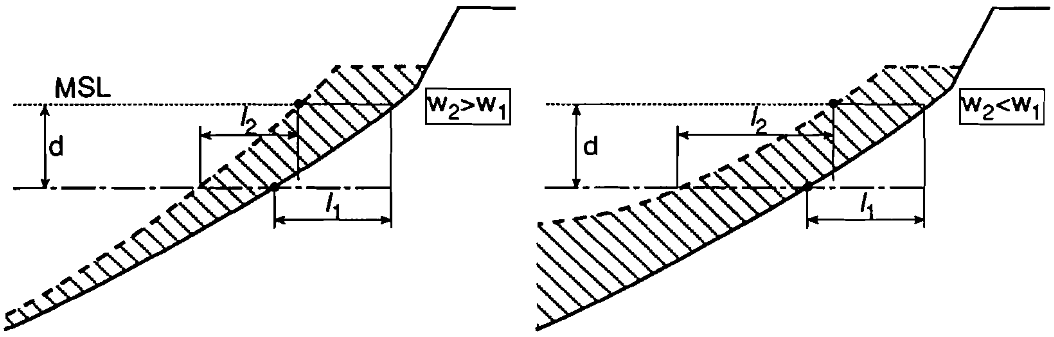


PS/2.3/6-93/DW

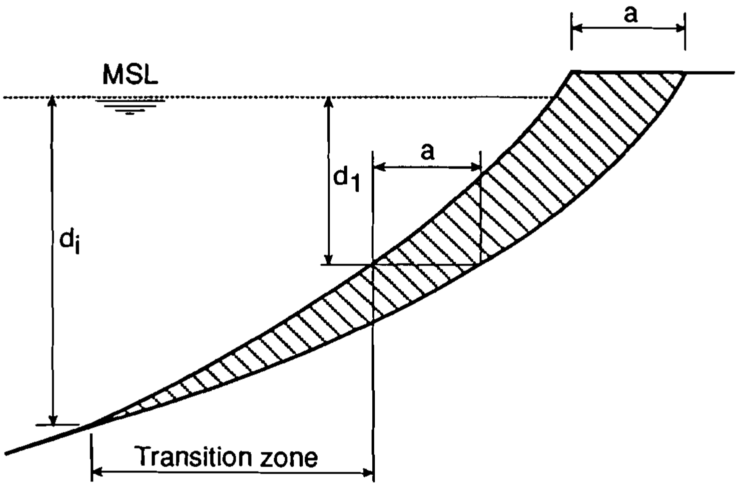
Figure 2.3 Isolines of the renourishment factor, R_J (James 1975)



(a) Values of A in $h = Ay^m$ versus sediment size (Dean, 1973)



(b) Effect of grain size on profile steepness (Vellinga, 1982)



(c) Profile of beach fill: recharge sand equal to native sand

PS/2.4/6-93/DW

Figure 2.4 Beach replenishment design considerations (Pilarczyk, van Overeem and Bakker, 1986)

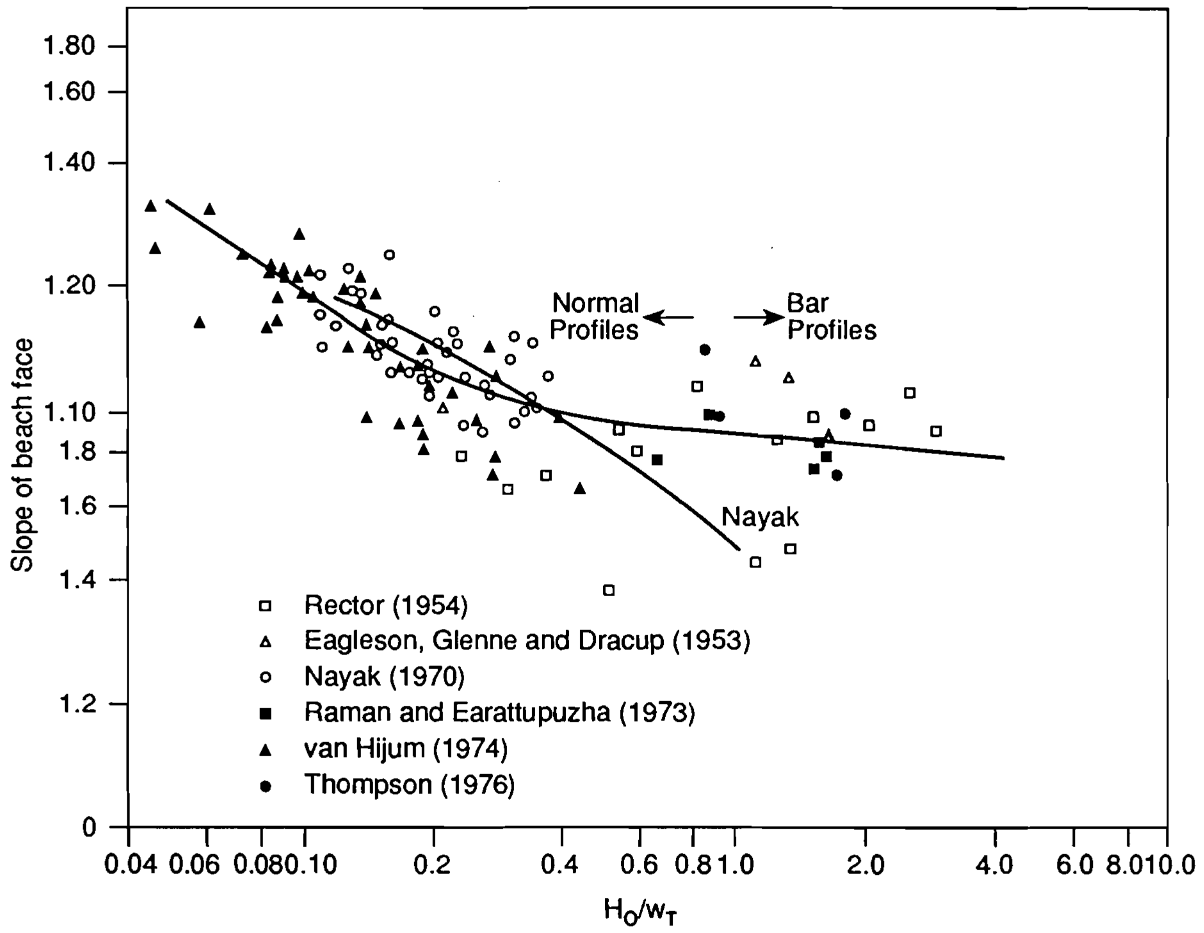


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Figure 2.5 Beach slope versus fall velocity parameter (Dalrymple and Thompson, 1976)

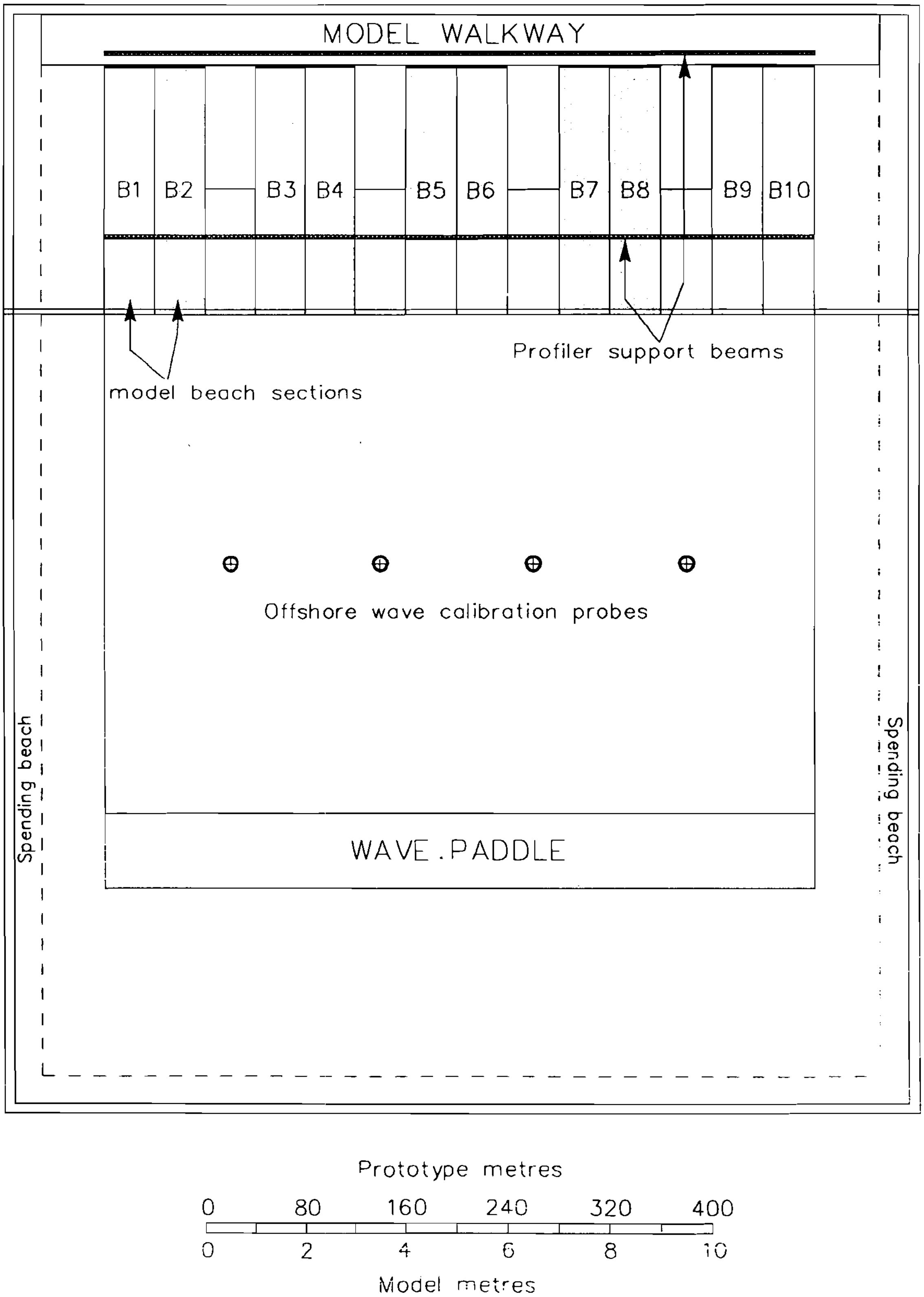


Figure 3.1 Layout of physical model test facility

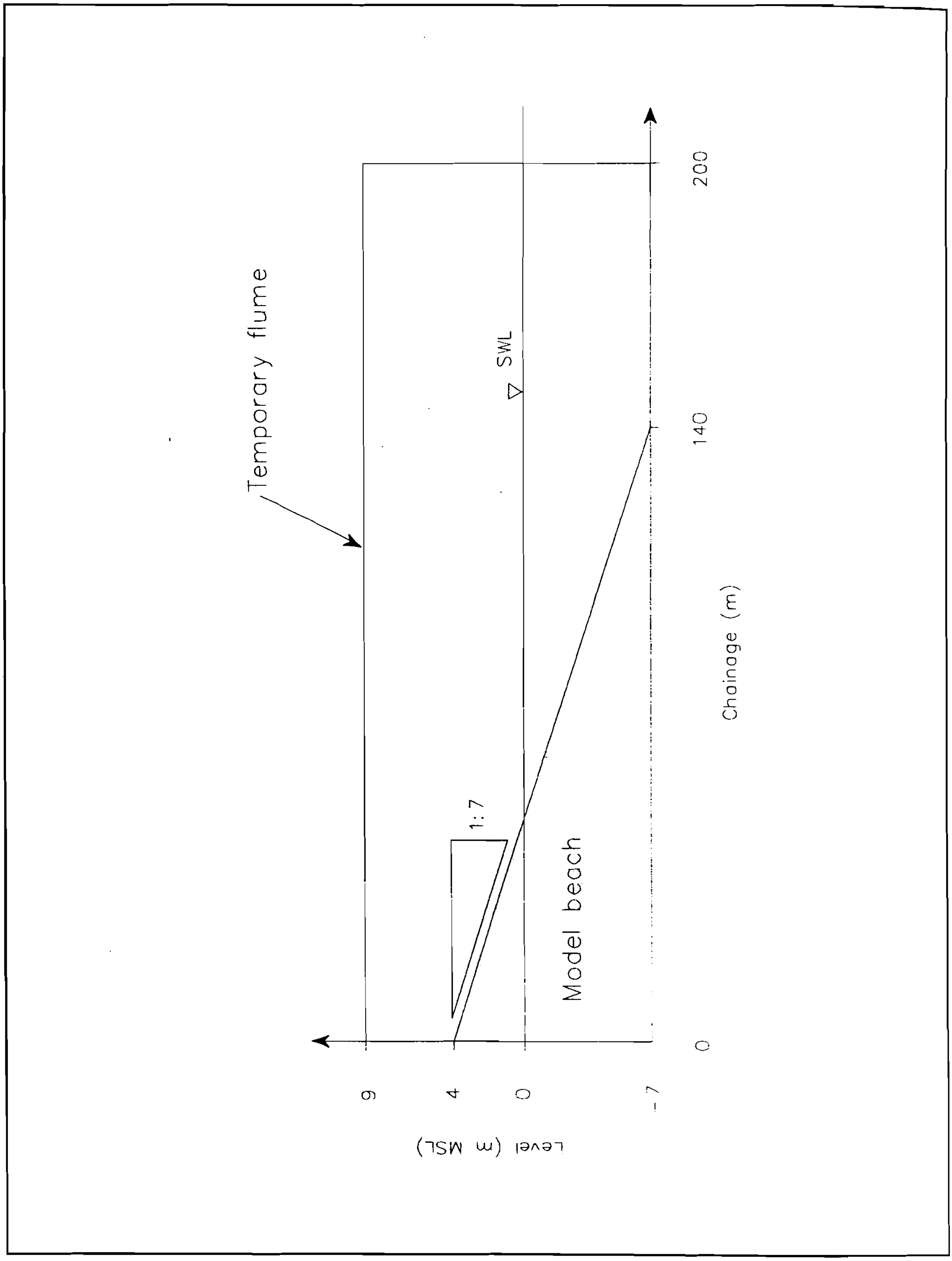
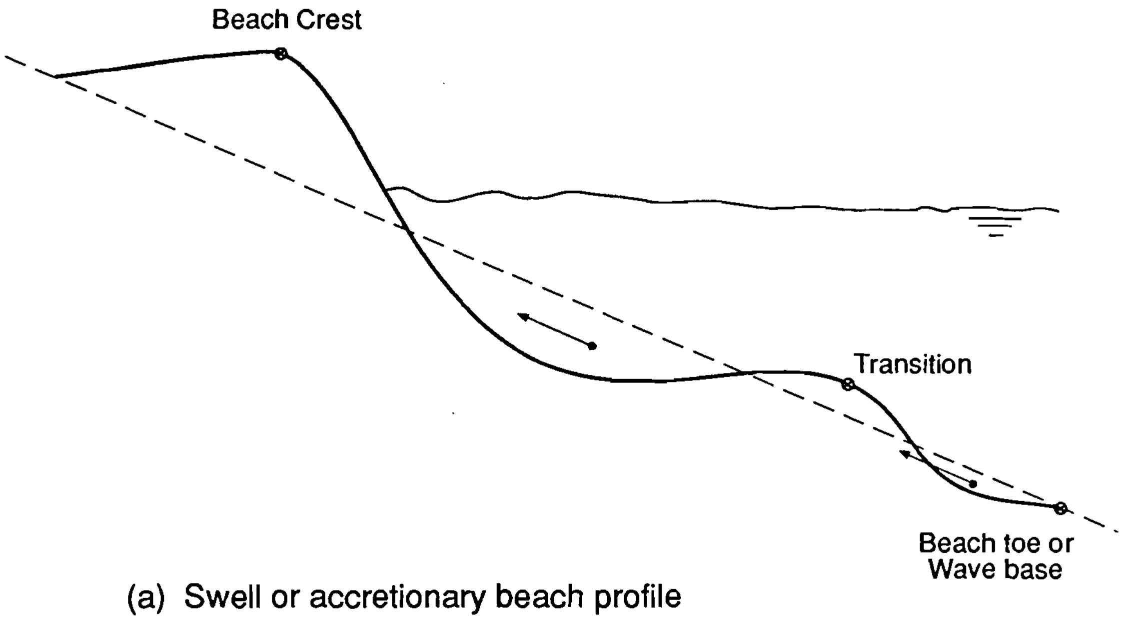
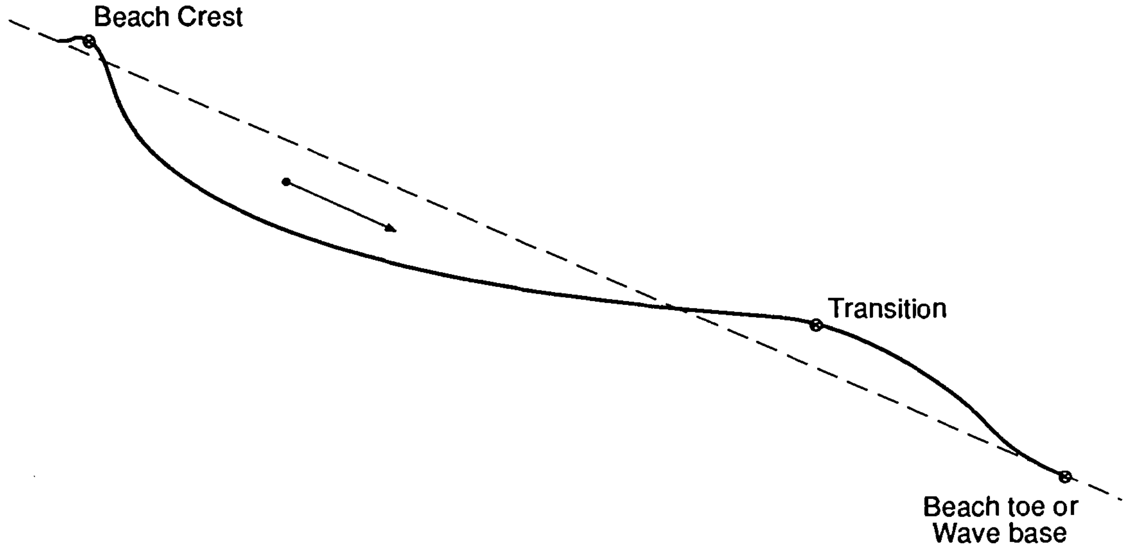


Figure 3.2 Cross-section through typical model beach



(a) Swell or accretionary beach profile



(b) Storm or erosion beach profile

KP/4.1/7-93/DW

Figure 4.1 Beach profile definitions

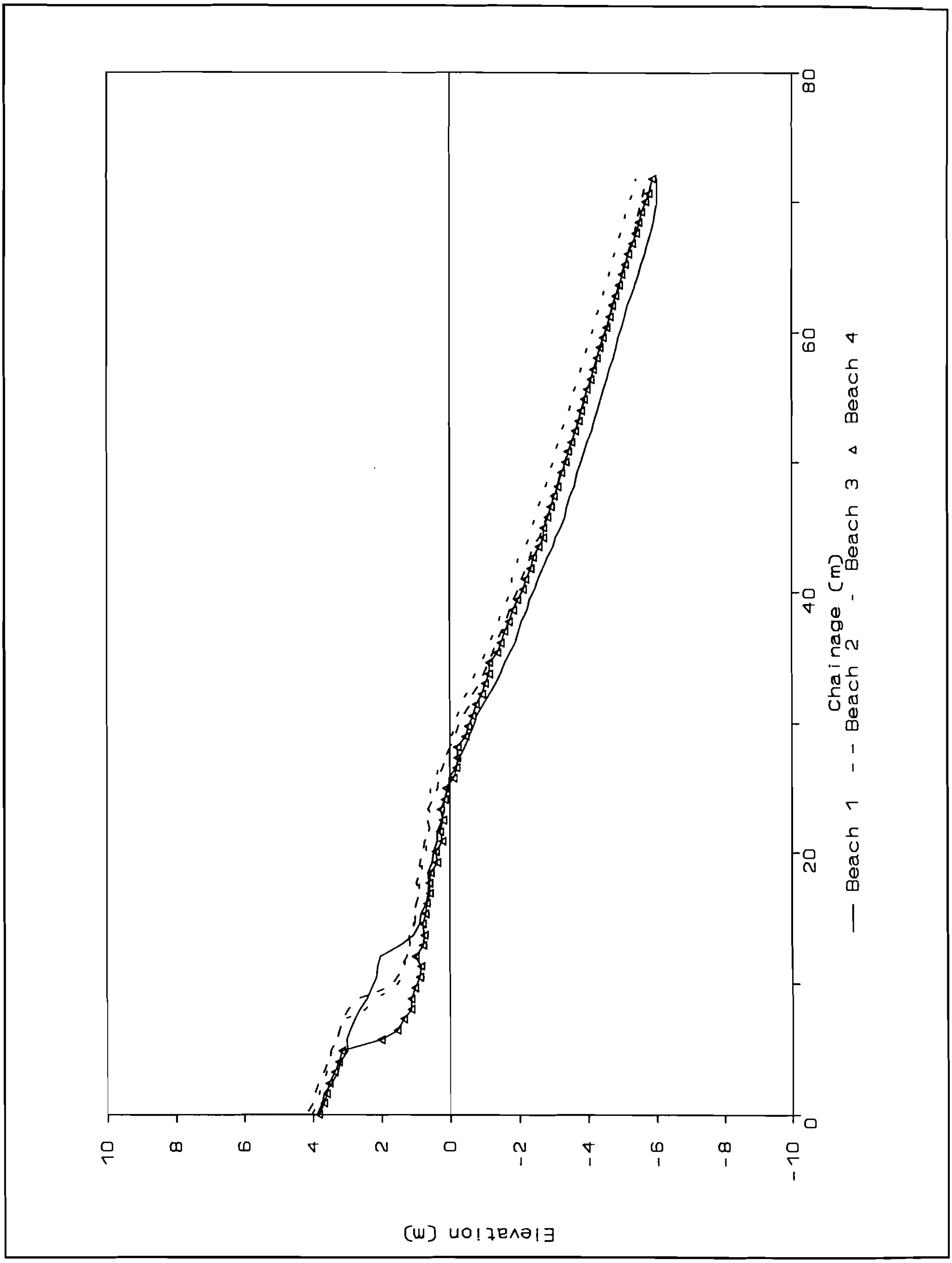


Figure 4.2 Beach profile development as a function of sediment grading: mean sea steepness 0.06

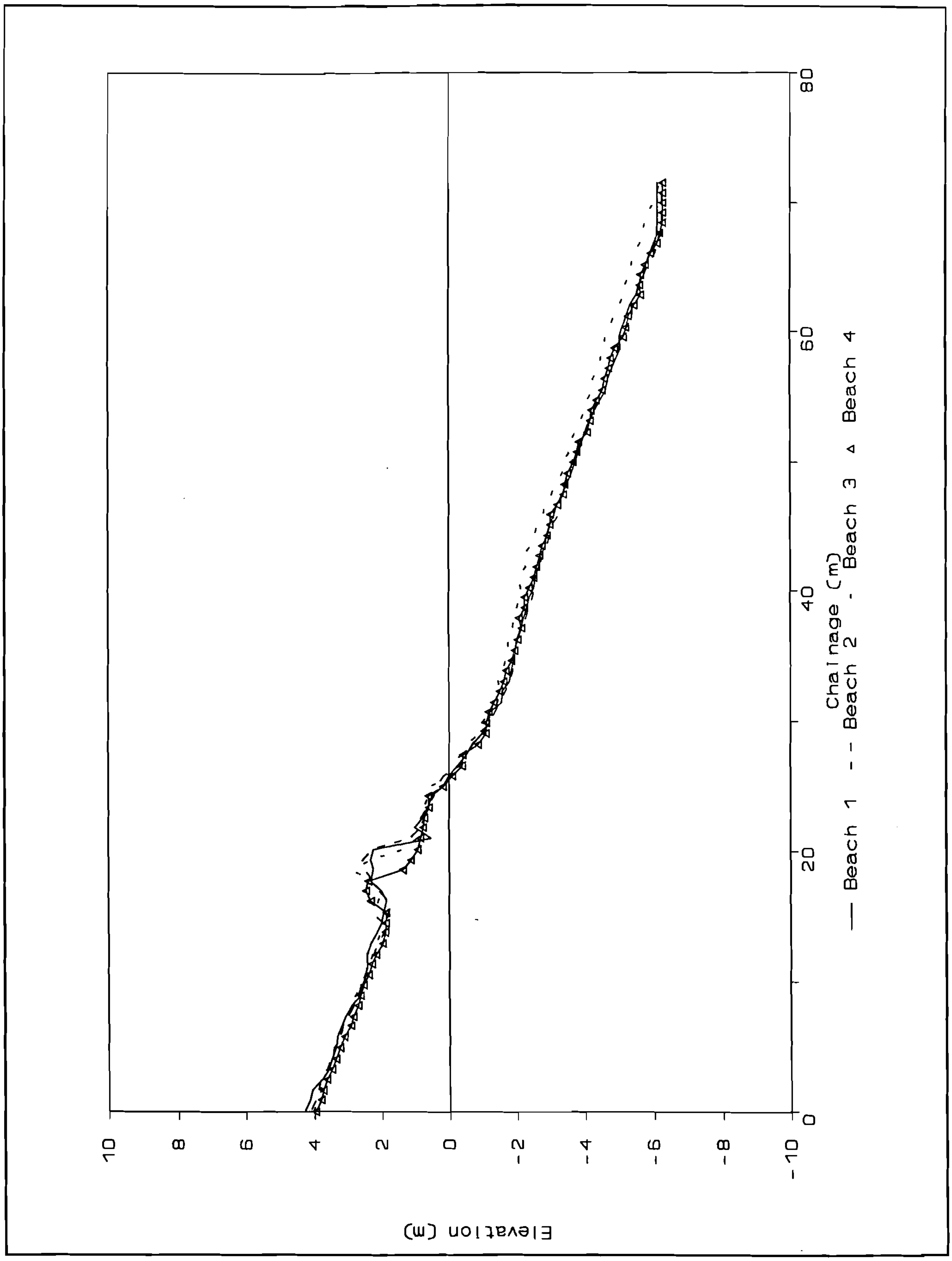


Figure 4.3 Beach profile development as a function of sediment grading: mean sea steepness 0.01

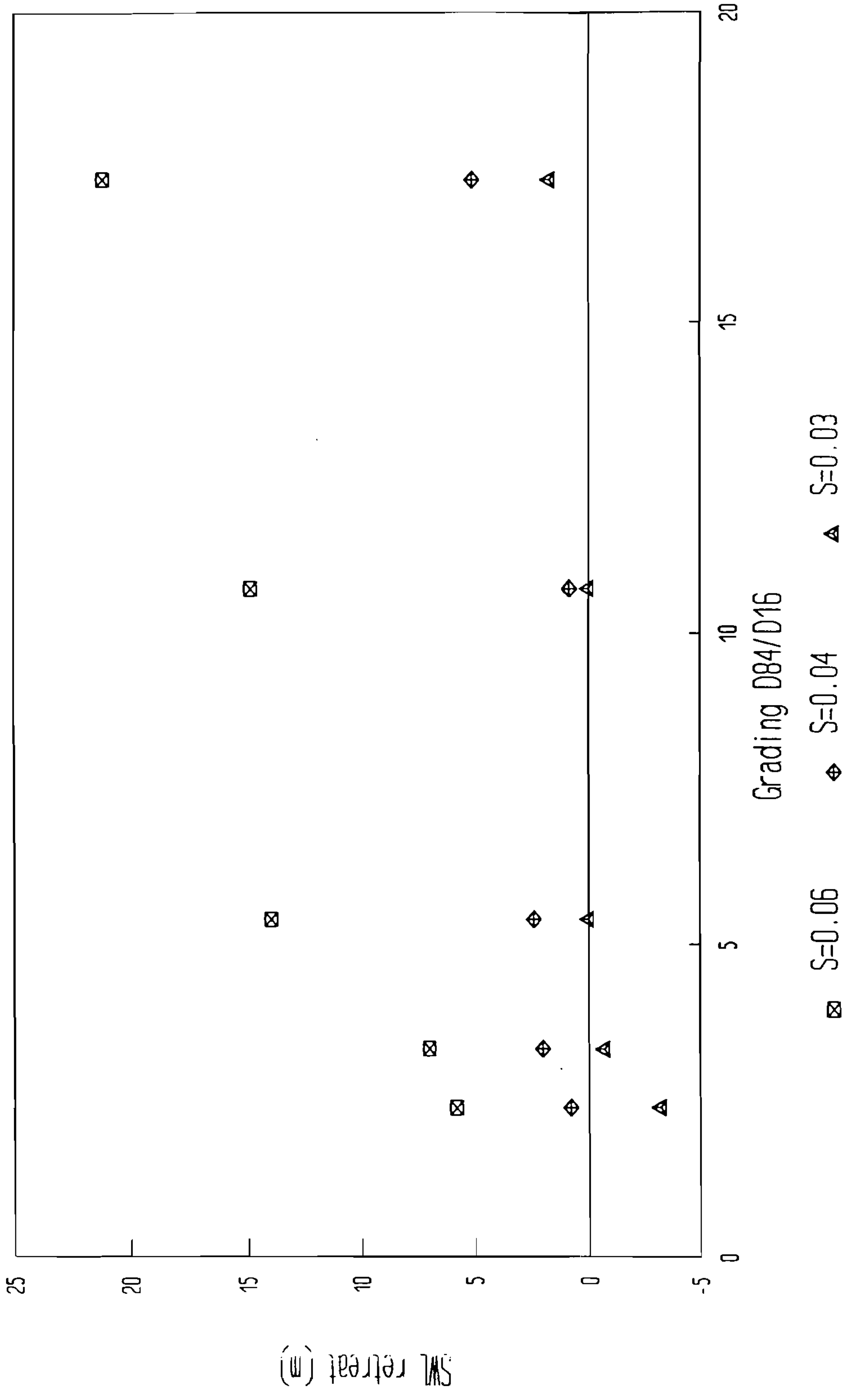


Figure 4.4 Still water level retreat as a function of sediment grading

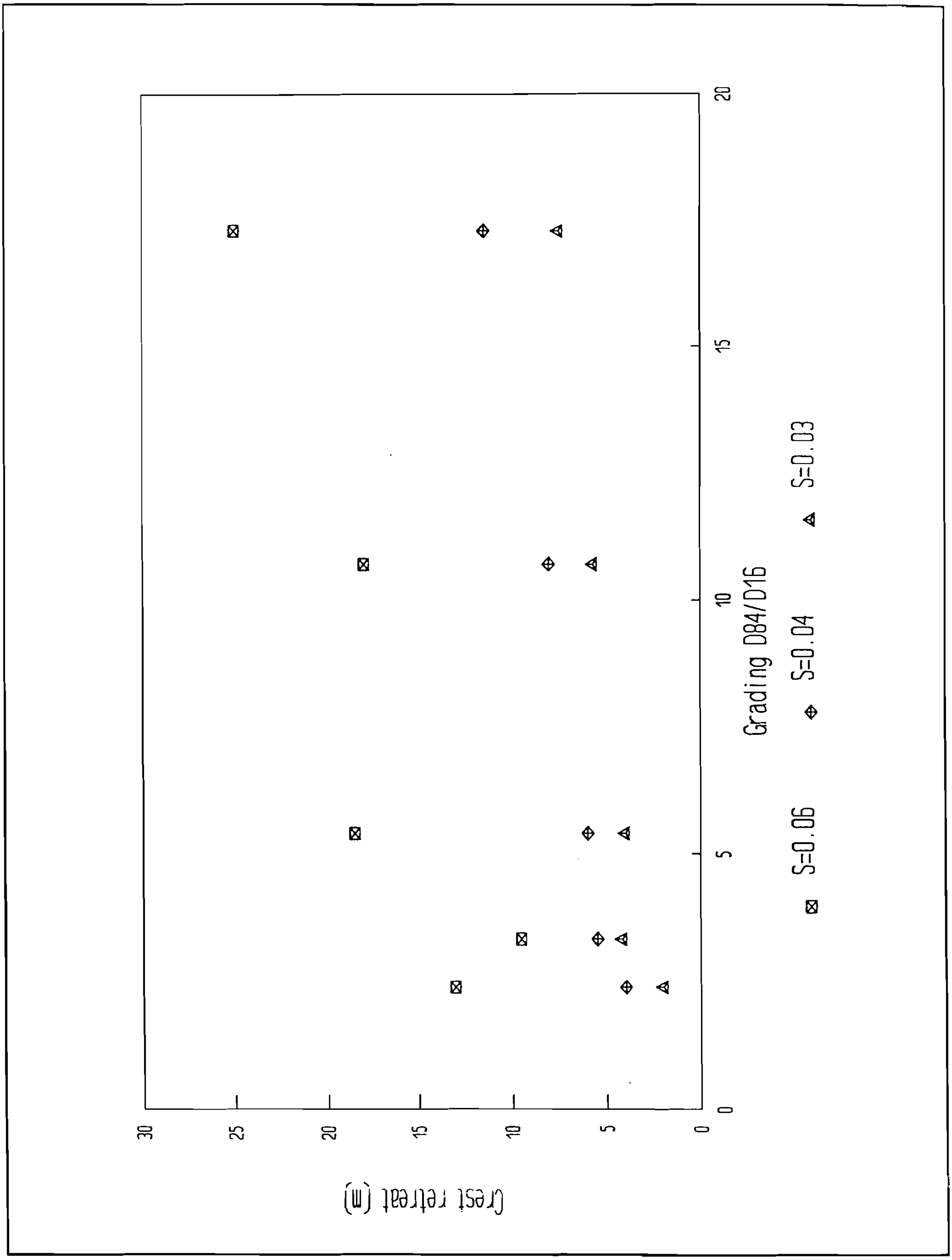


Figure 4.5 Beach crest retreat as a function of sediment grading

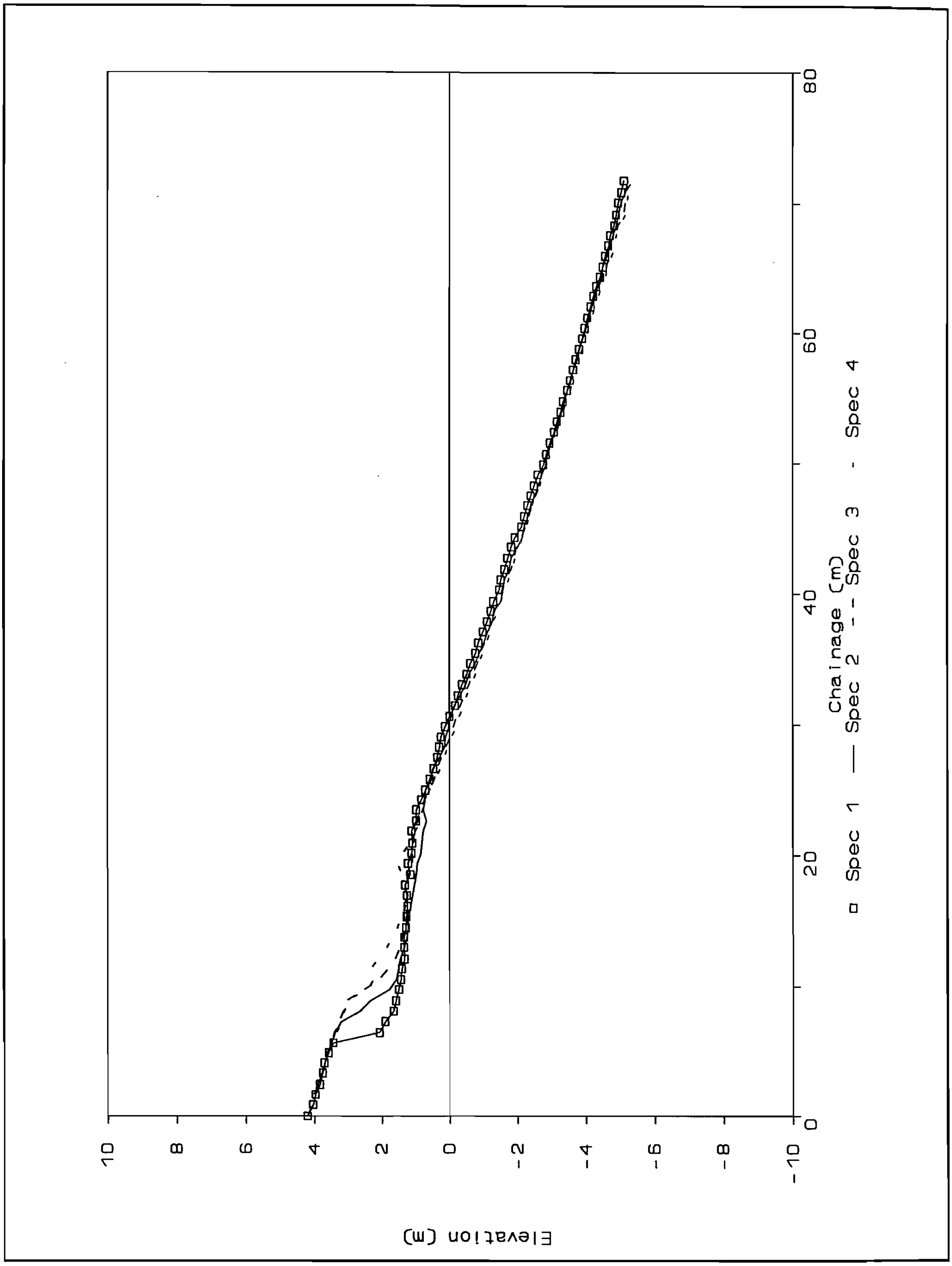


Figure 4.6 Influence of wave height and period on beach profile development: mean sea steepness H_s/L_m=0.06

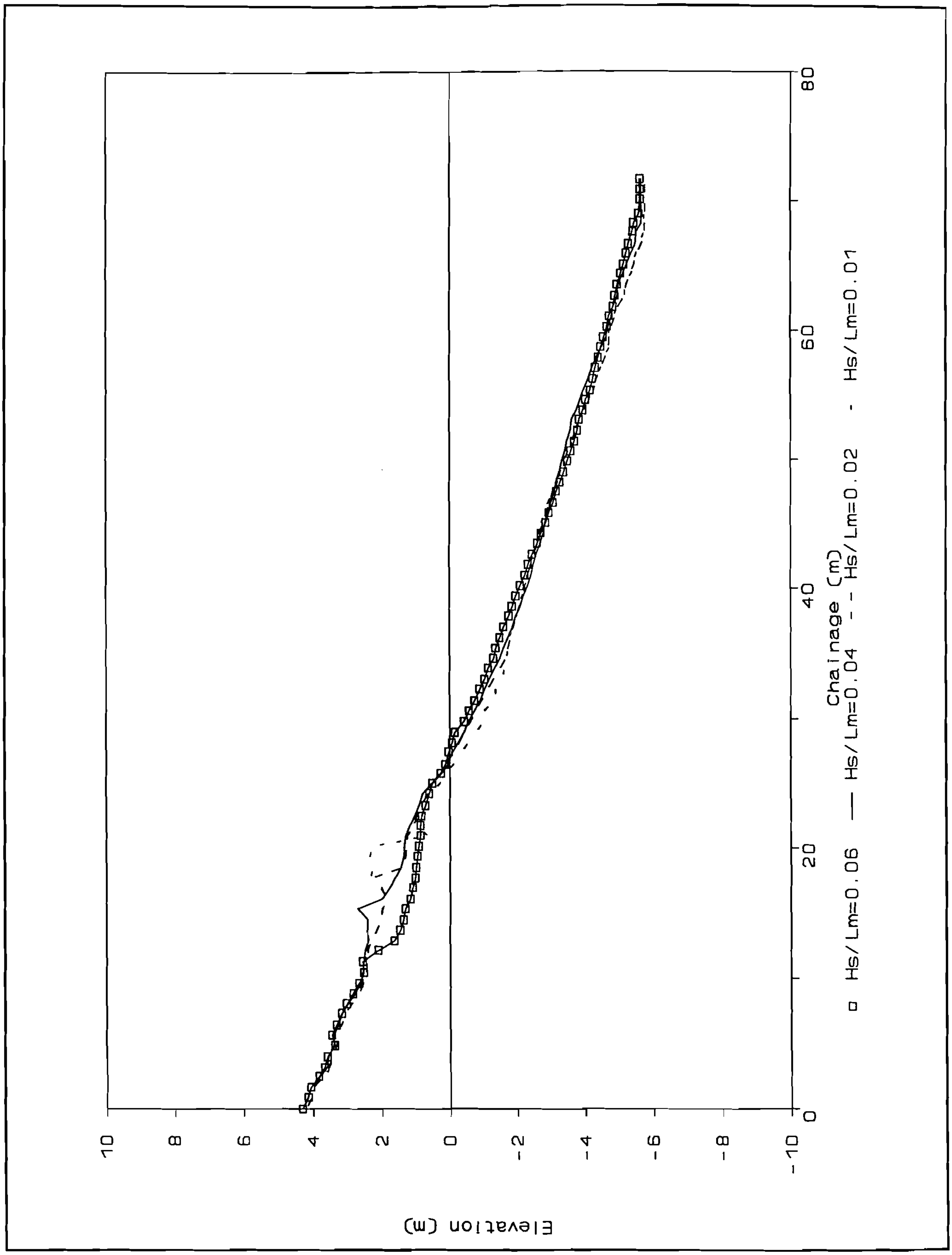


Figure 4.7 Influence of mean sea steepness on beach profile development: narrow sediment grading

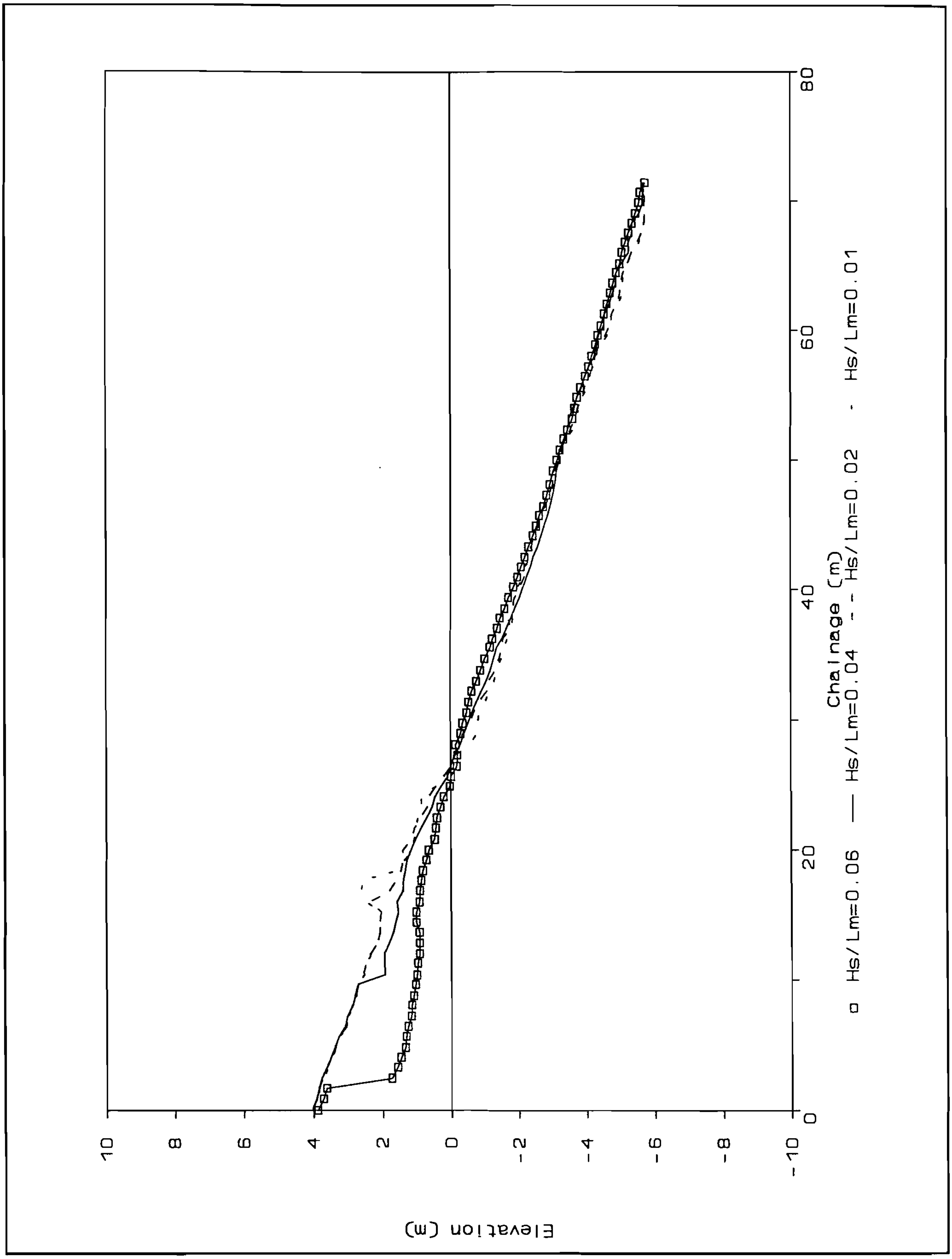


Figure 4.8 Influence of mean sea steepness on beach profile development: wide sediment grading

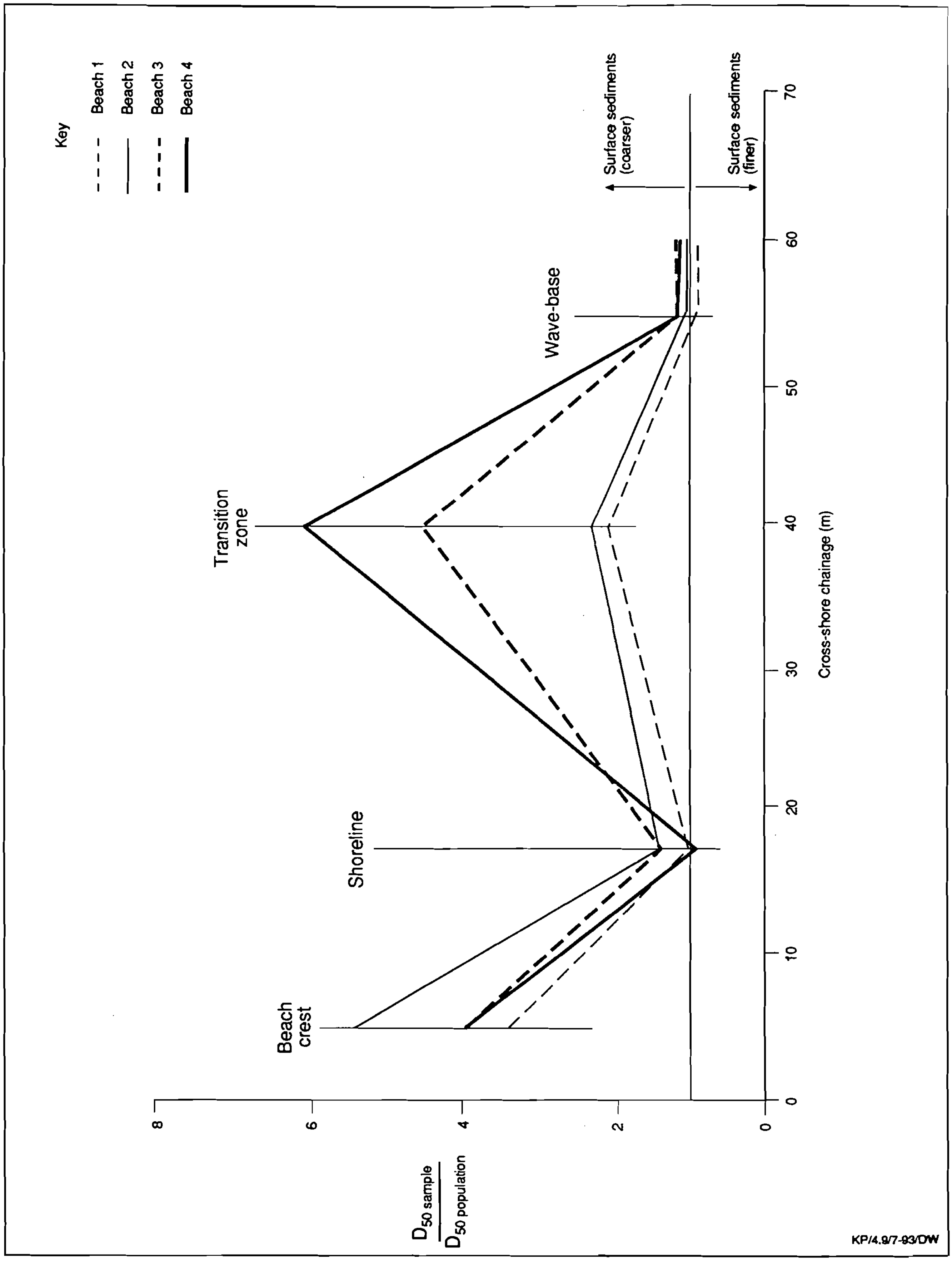


Figure 4.9 Cross-shore sediment size distributions : Wave spectrum 1

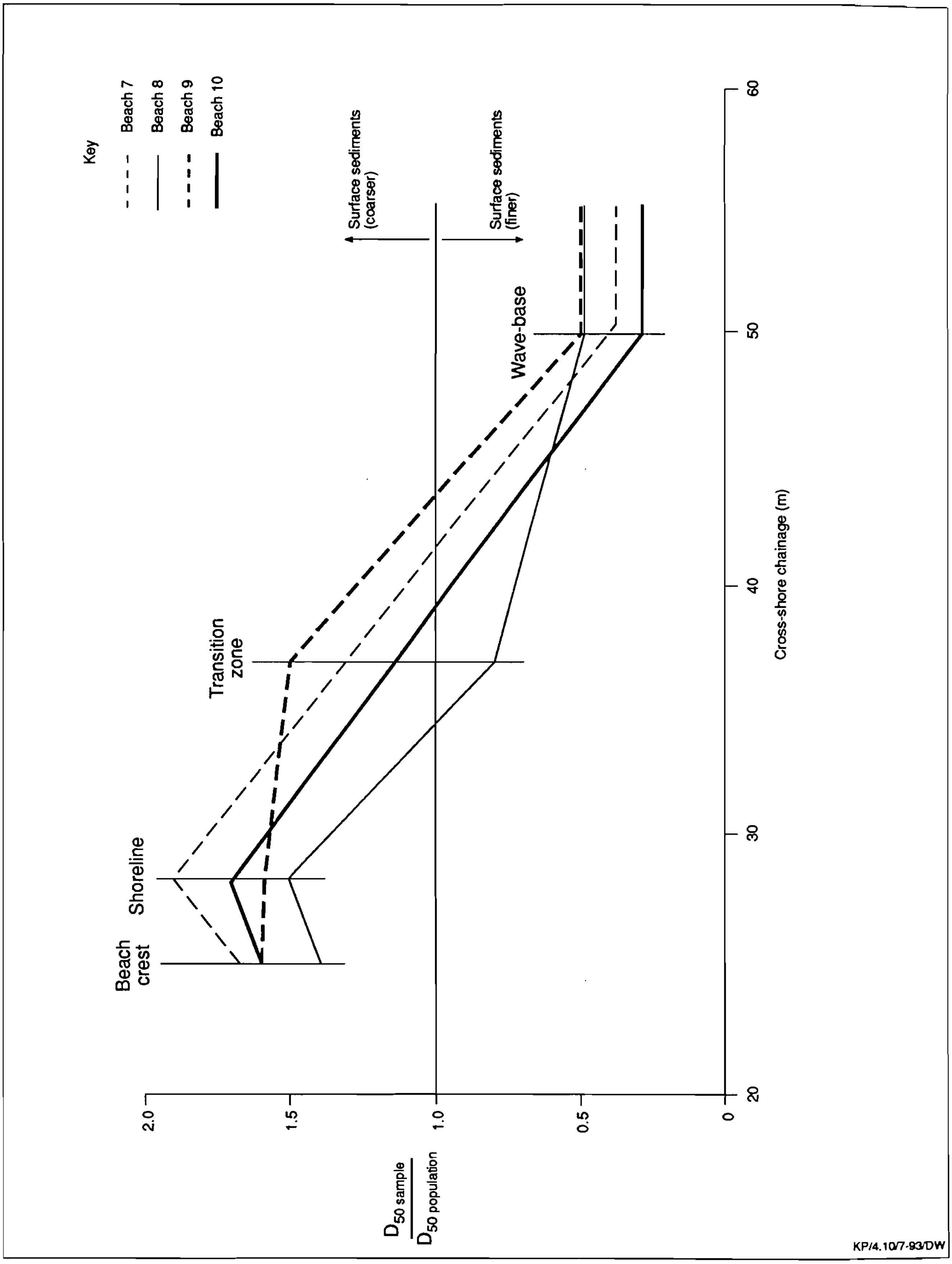


Figure 4.10 Cross-shore sediment size distributions : Wave spectrum 1

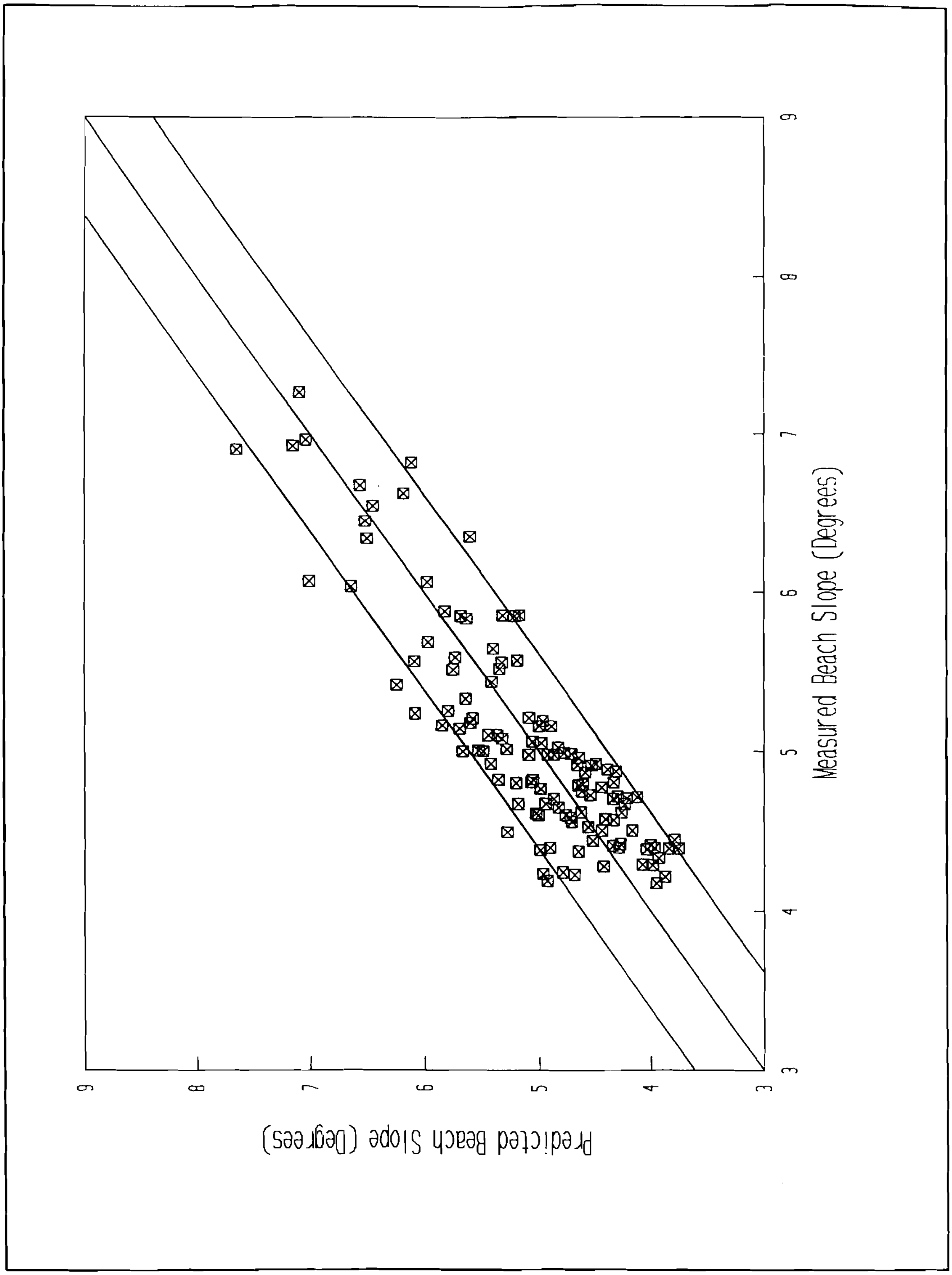


Figure 5.1 Comparison of predicted and measured beach slopes

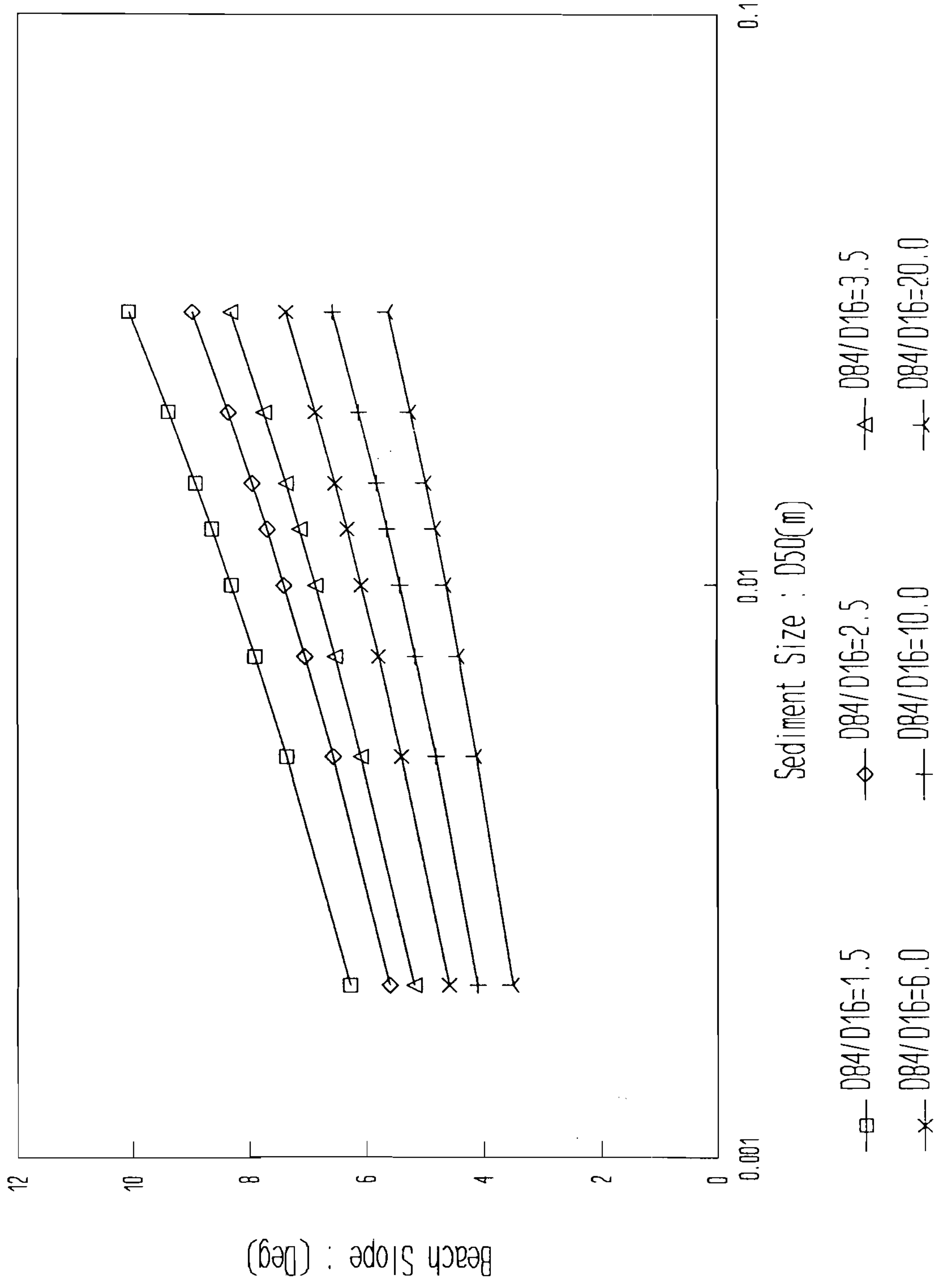


Figure 5.2 Beach slope dependency: mean sea steepness, Hs/Lm=0.005. Significant wave height, Hs=2.0m

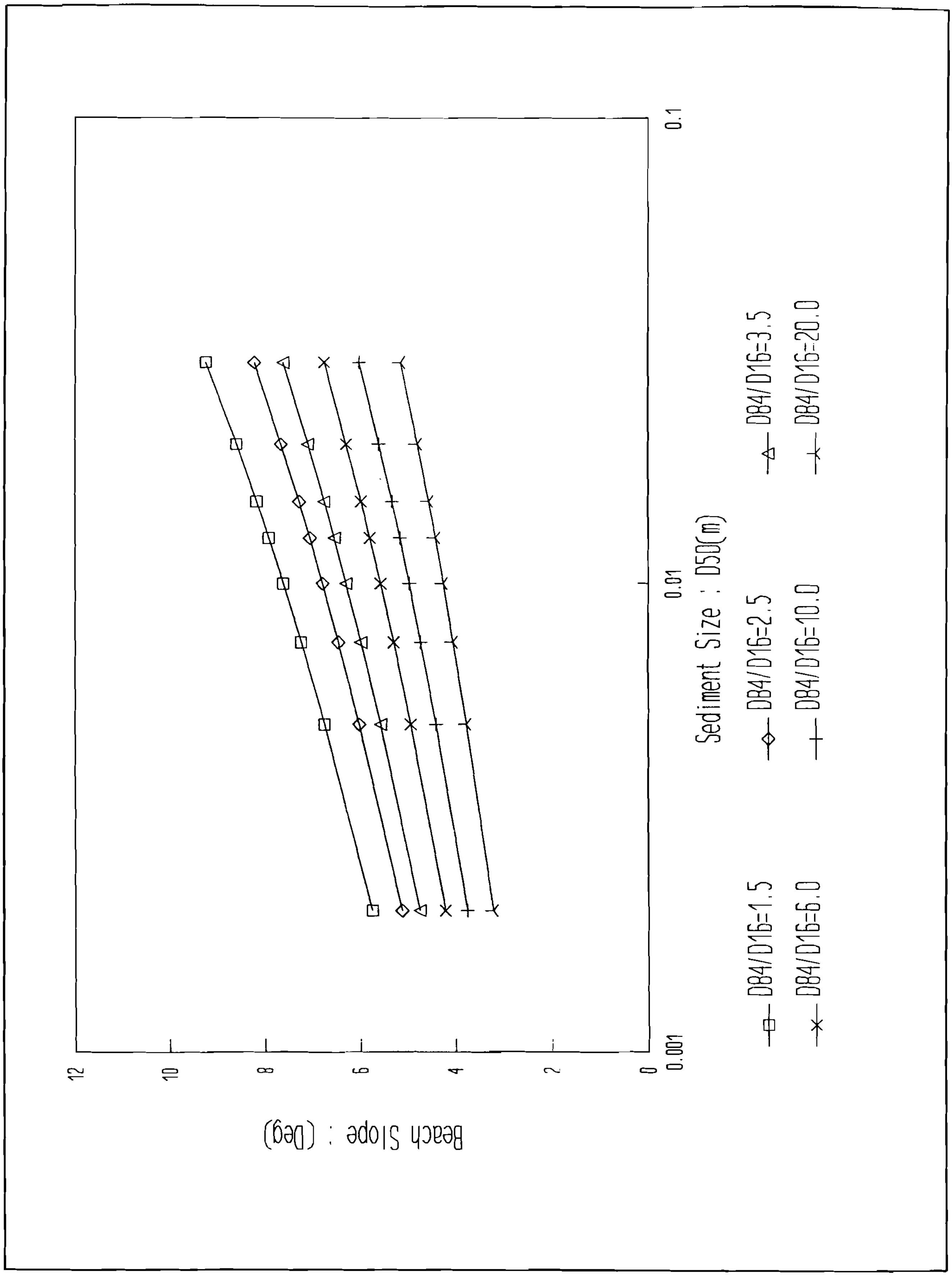


Figure 5.3 Beach slope dependency: mean sea steepness, Hs/Lm=0.01. Significant wave height, Hs=2.0m

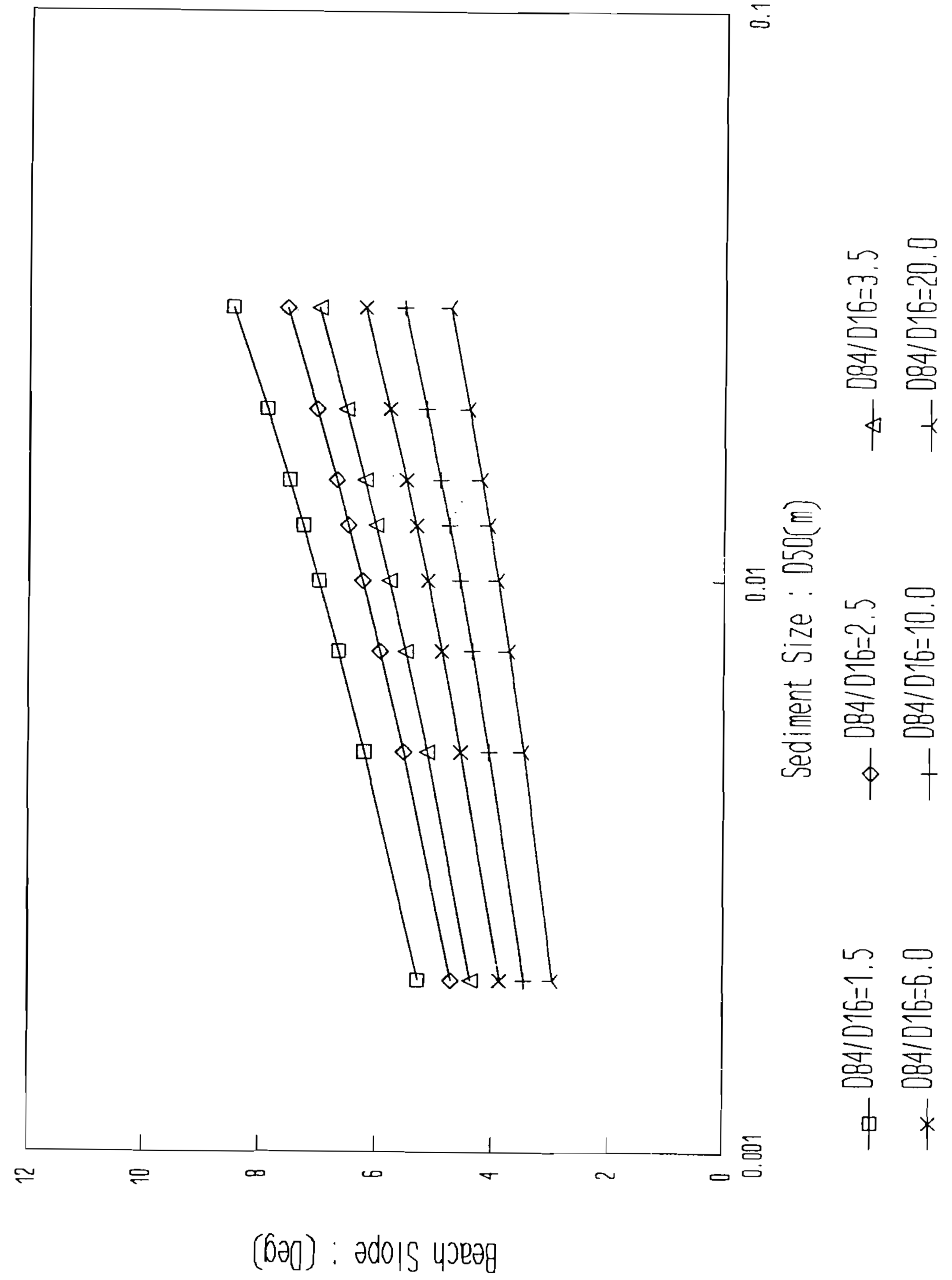


Figure 5.4 Beach slope dependency: mean sea steepness, Hs/Lm=0.02. Significant wave height, Hs=2.0m

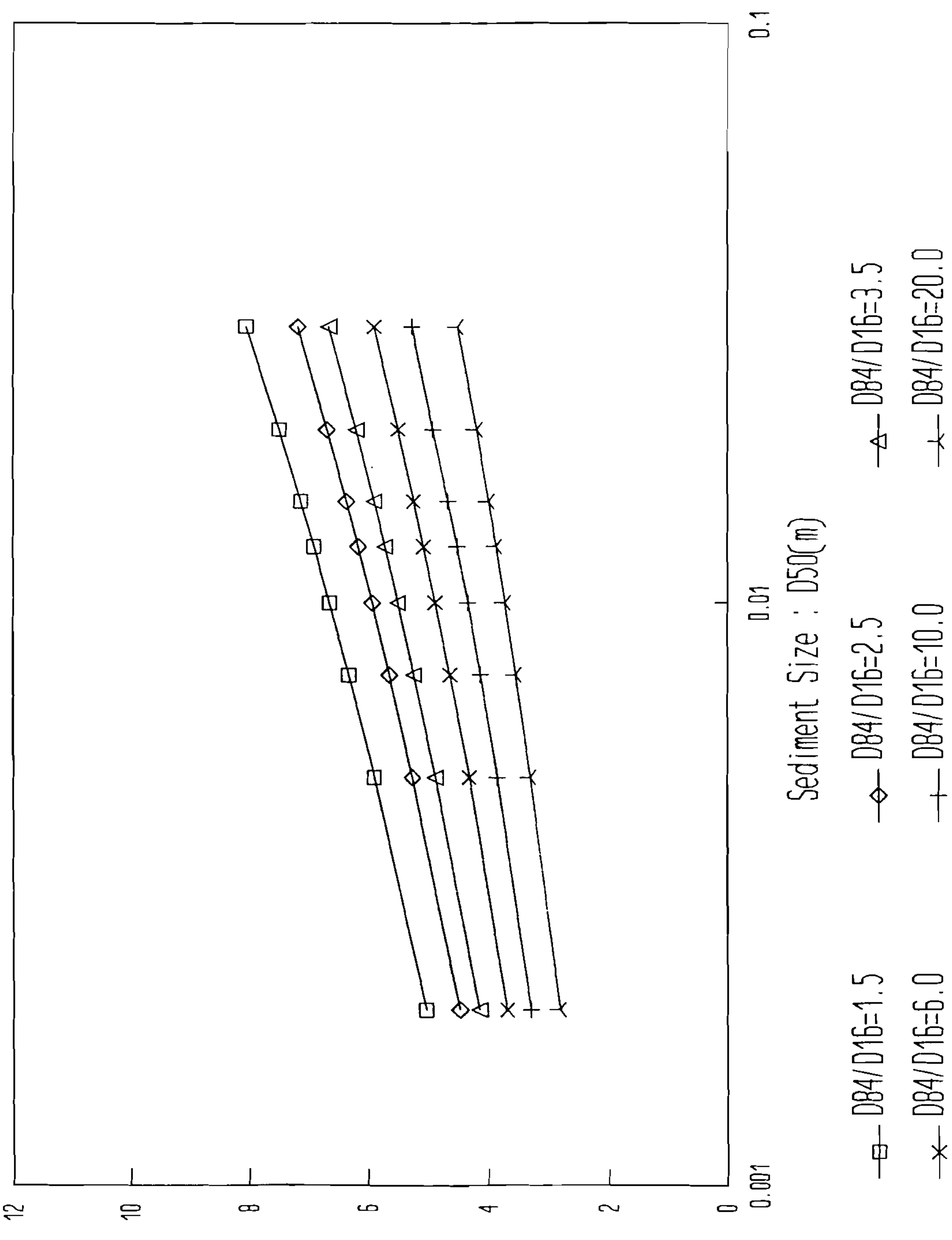


Figure 5.5 Beach slope dependency: mean sea steepness, Hs/Lm=0.03. Significant wave height, Hs=2.0m

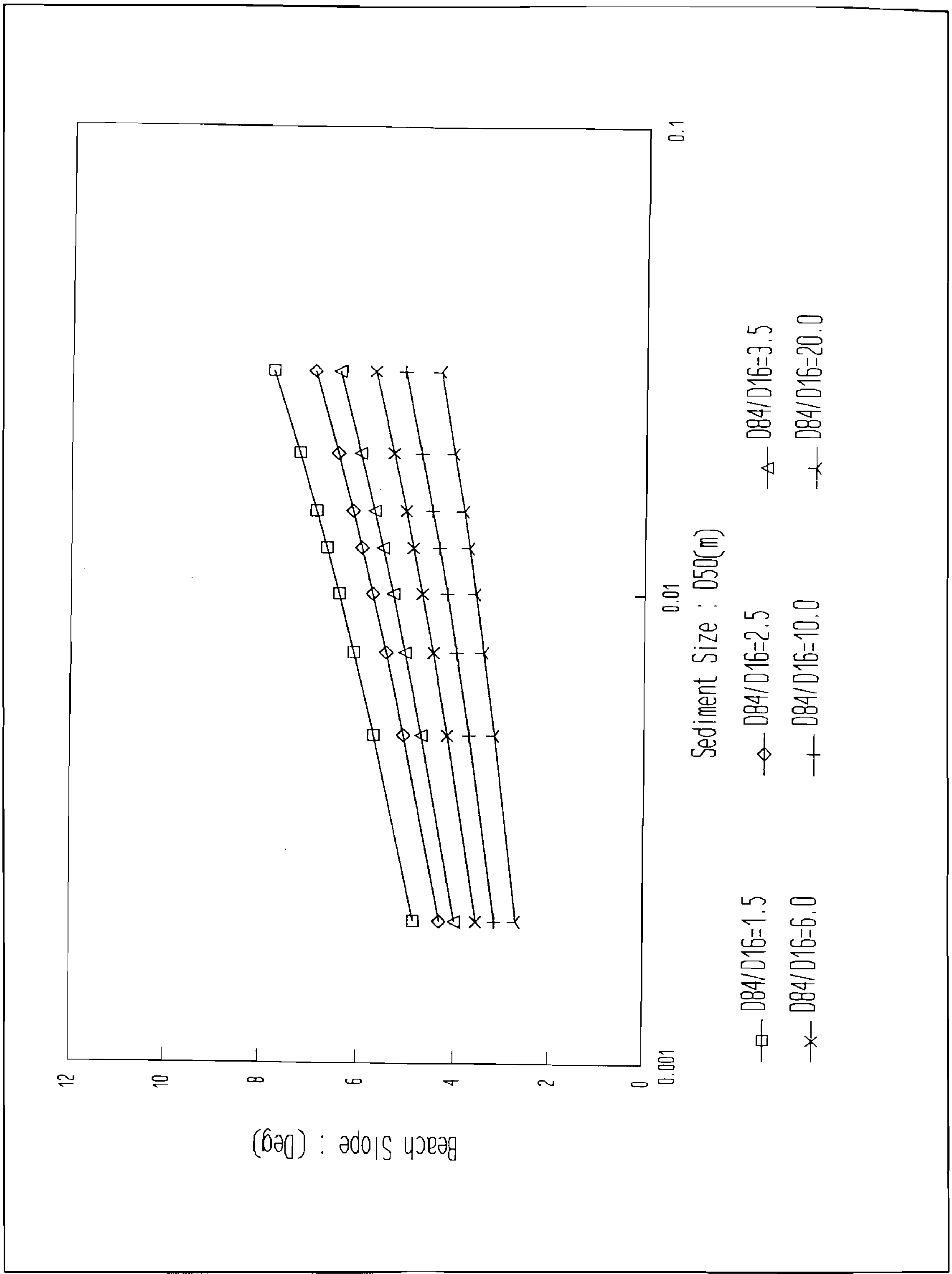


Figure 5.6 Beach slope dependency: mean sea steepness, Hs/Lm=0.04. Significant wave height, Hs=2.0m

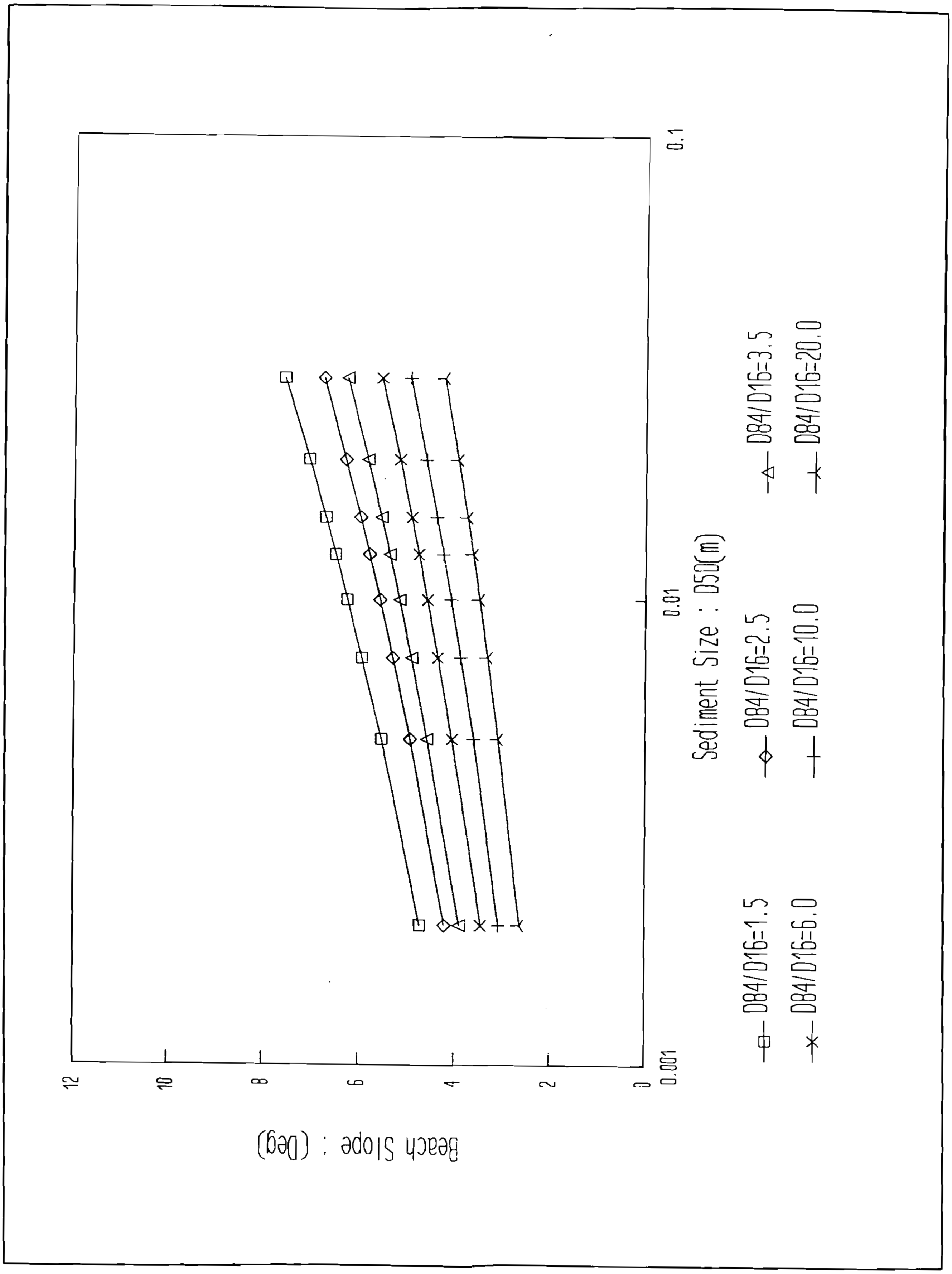


Figure 5.7 Beach slope dependency: mean sea steepness, Hs/Lm=0.05. Significant wave height, Hs=2.0m

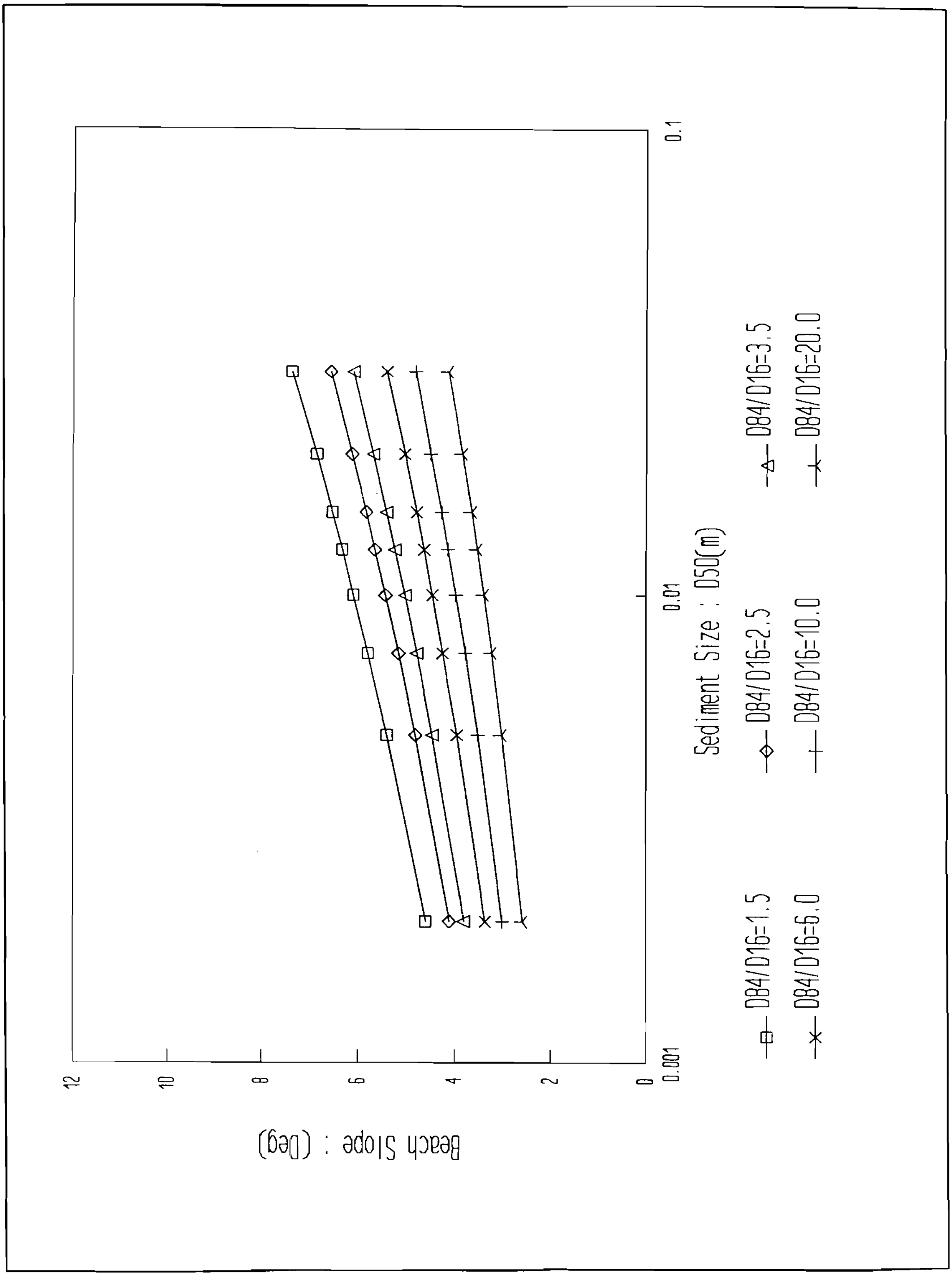


Figure 5.8 Beach slope dependency: mean sea steepness, Hs/Lm=0.06. Significant wave height, Hs=2.0m

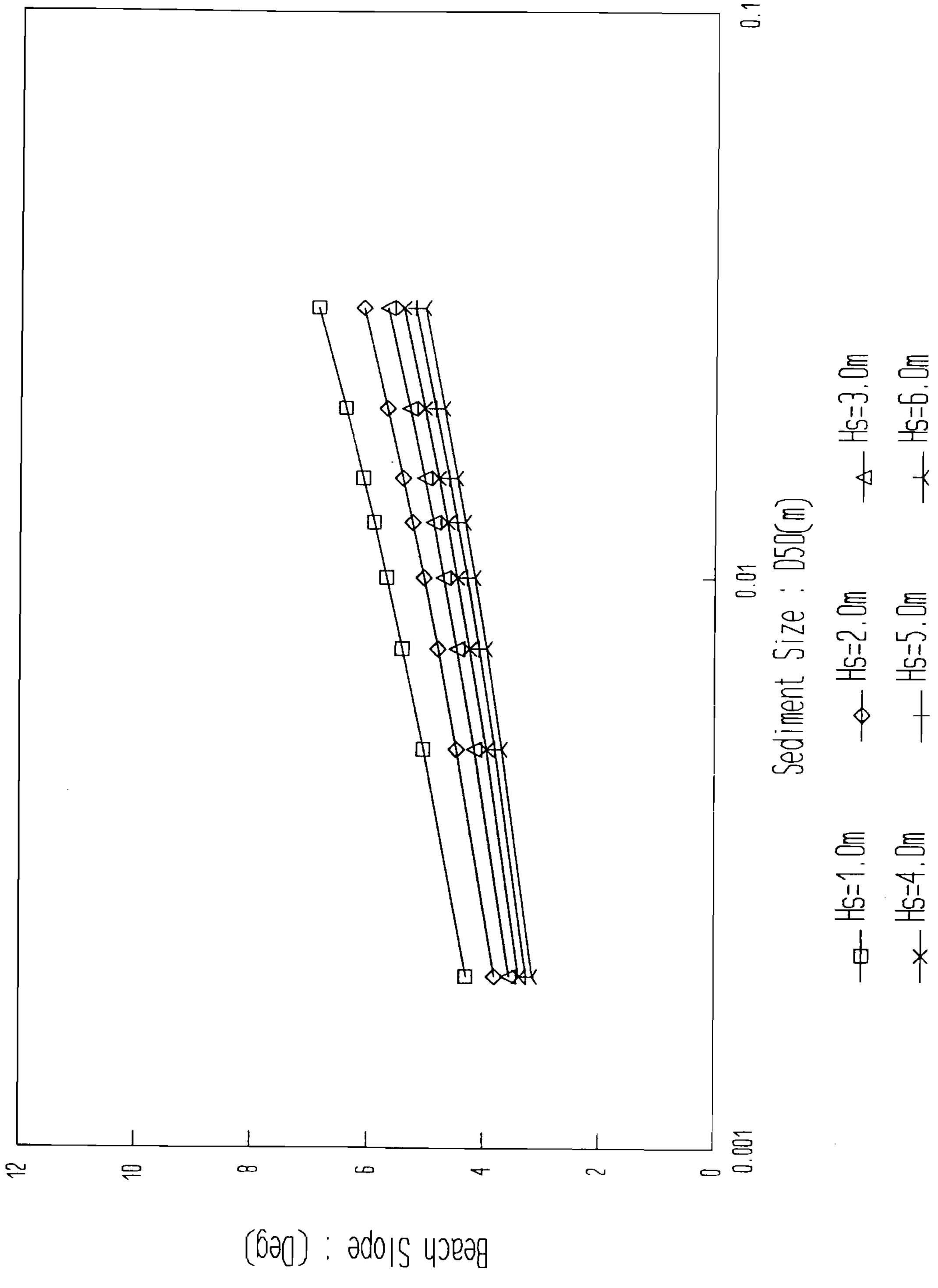


Figure 5.9 Influence of wave height on beach slope trend

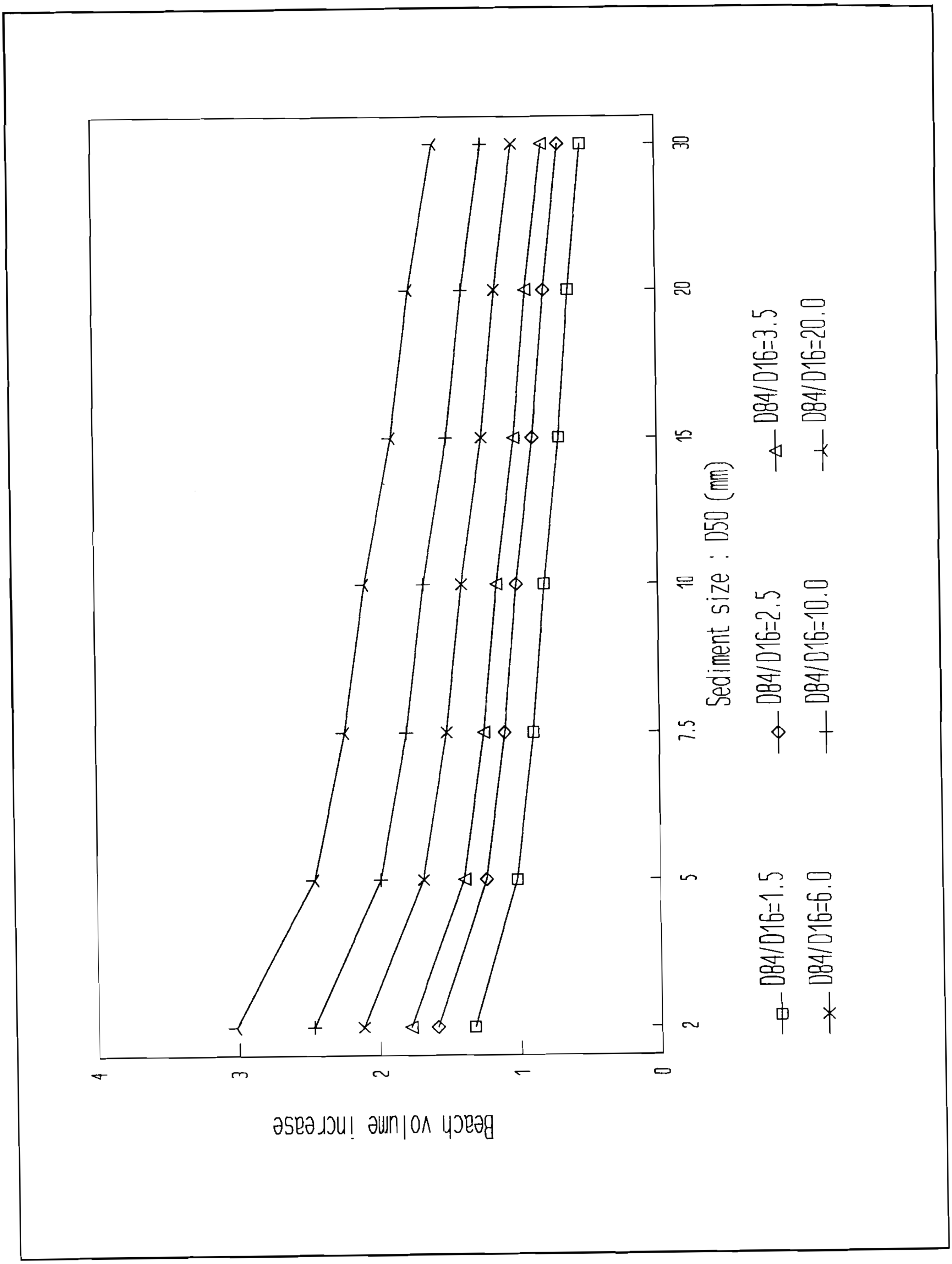


Figure 6.1 Replenishment volume requirements as a function of sediment size and grading

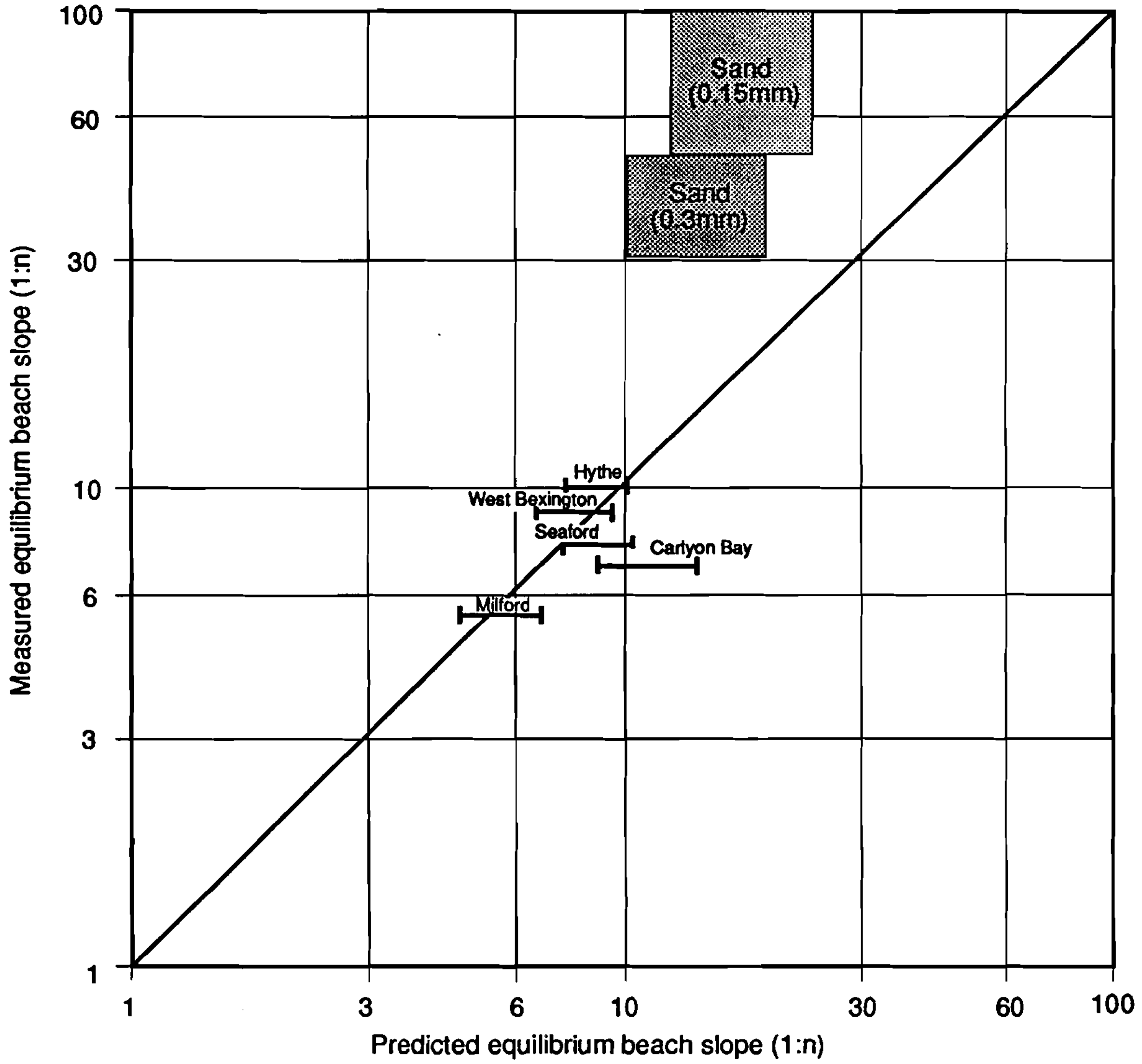


Figure 7.1 Comparison of predicted and field data

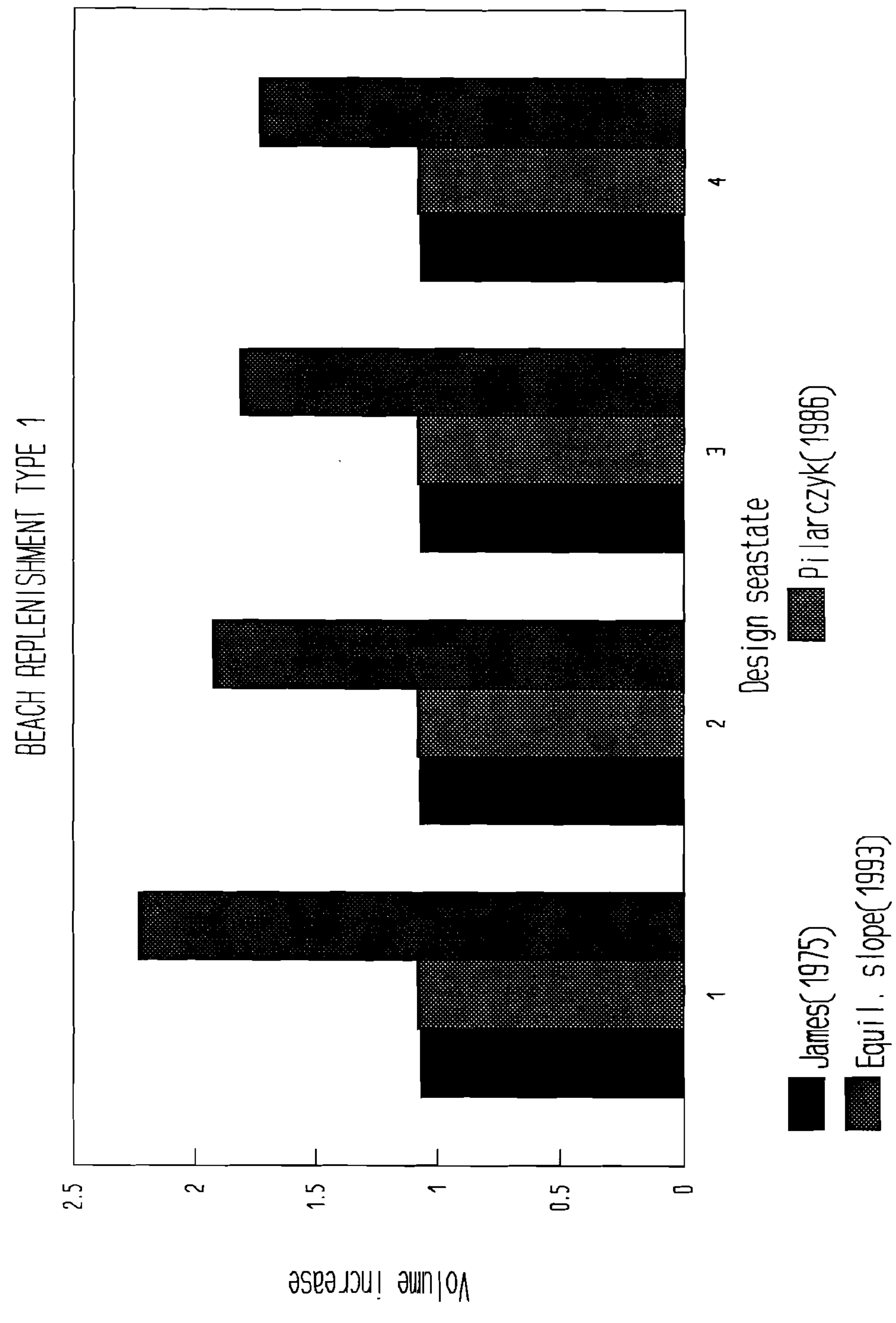


Figure 7.2 Comparison of design of methodologies
- Type 1 replenishment

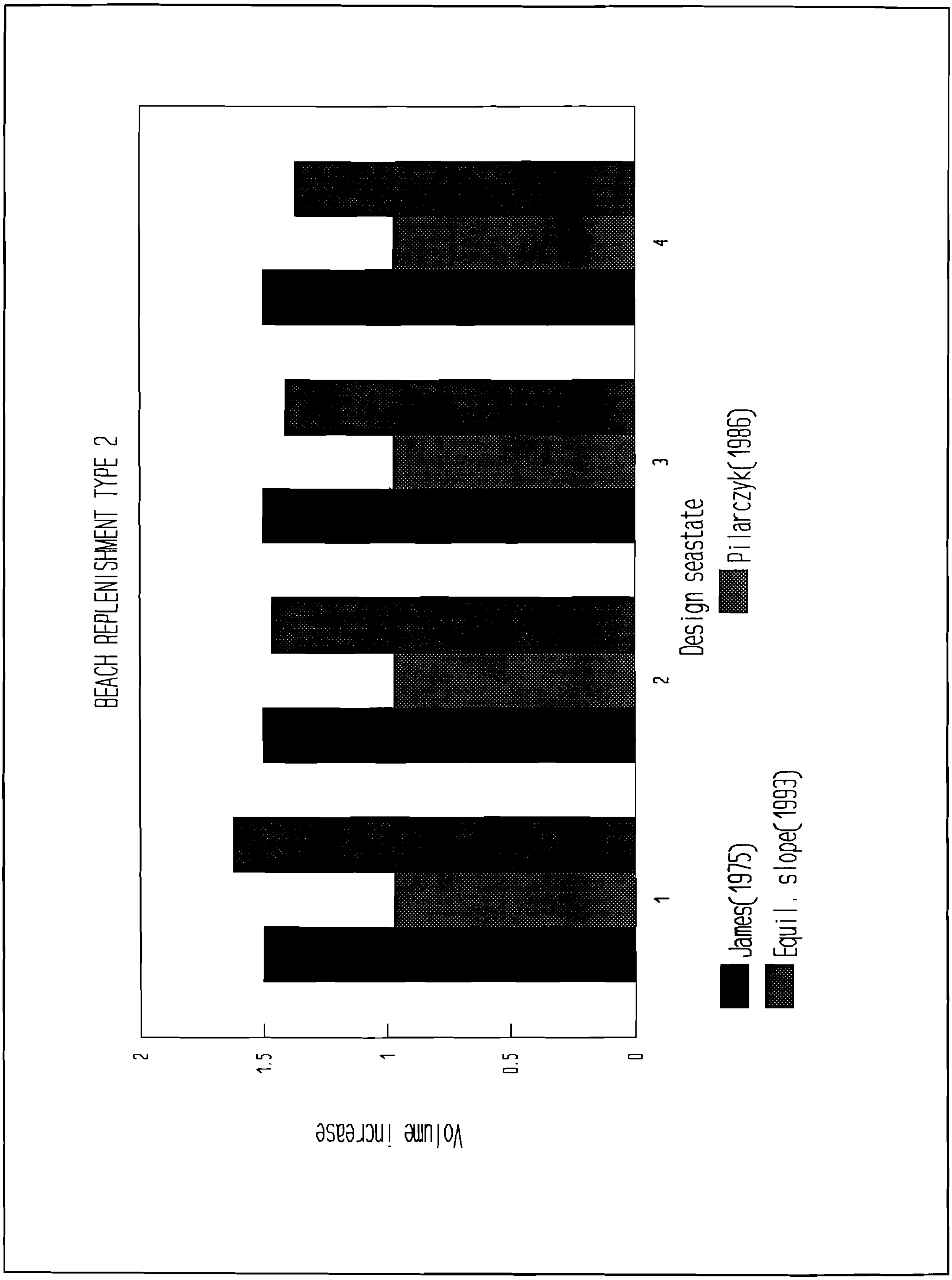


Figure 7.3 Comparison of design of methodologies
- Type 2 replenishment

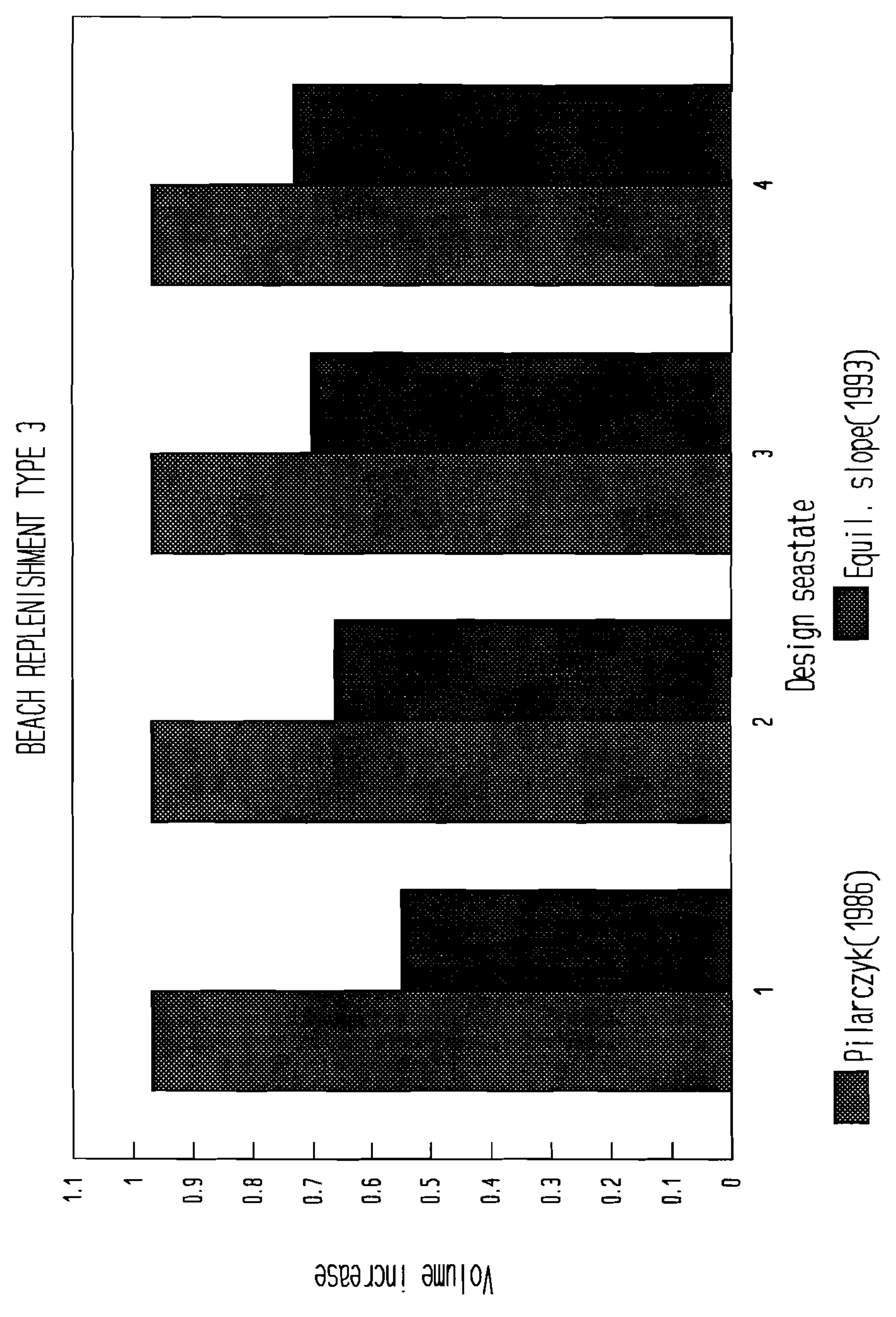


Figure 7.4 Comparison of design of methodologies - Type 3 replenishment

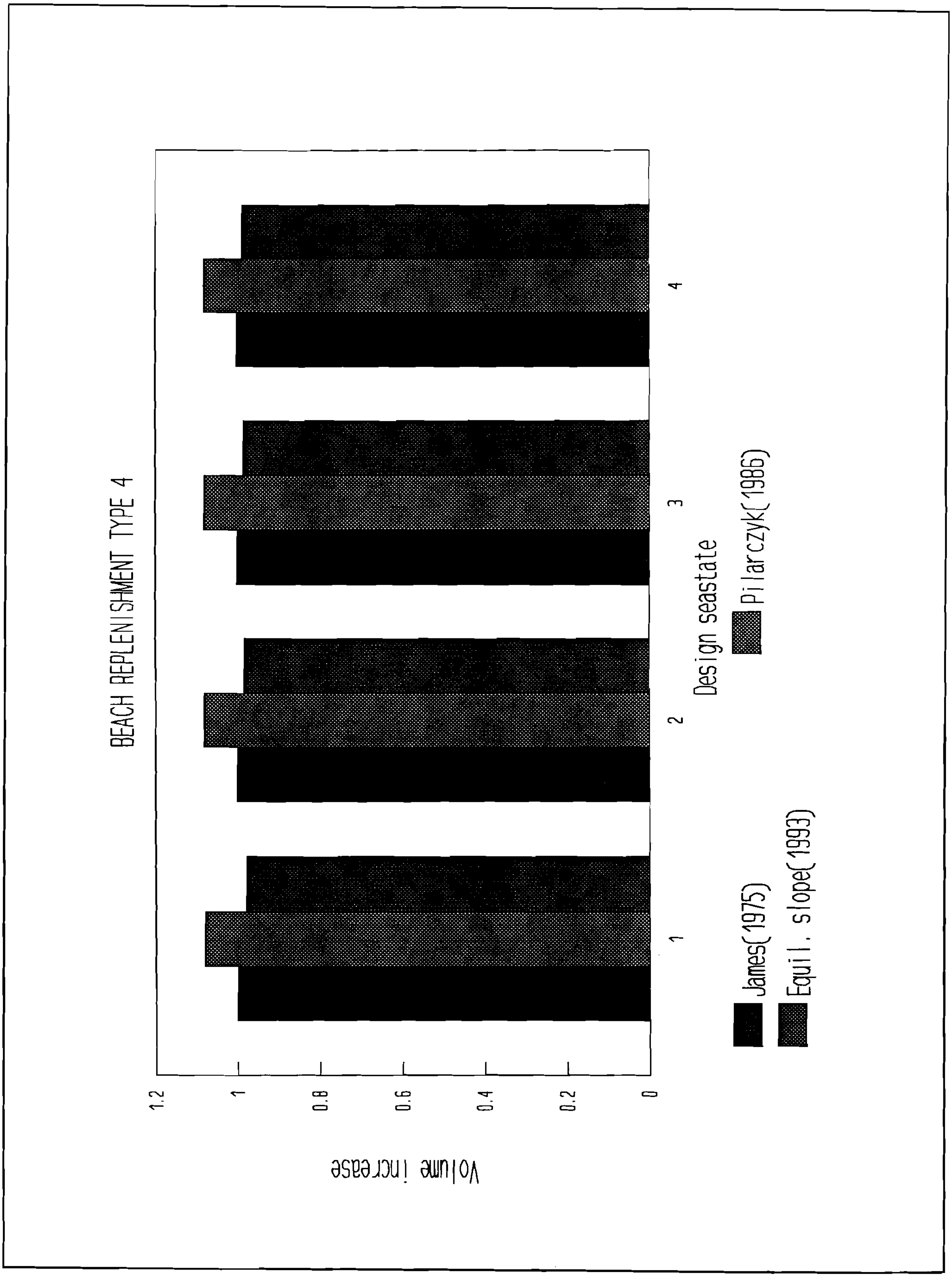


Figure 7.5 Comparison of design of methodologies
- Type 4 replenishment

Plate

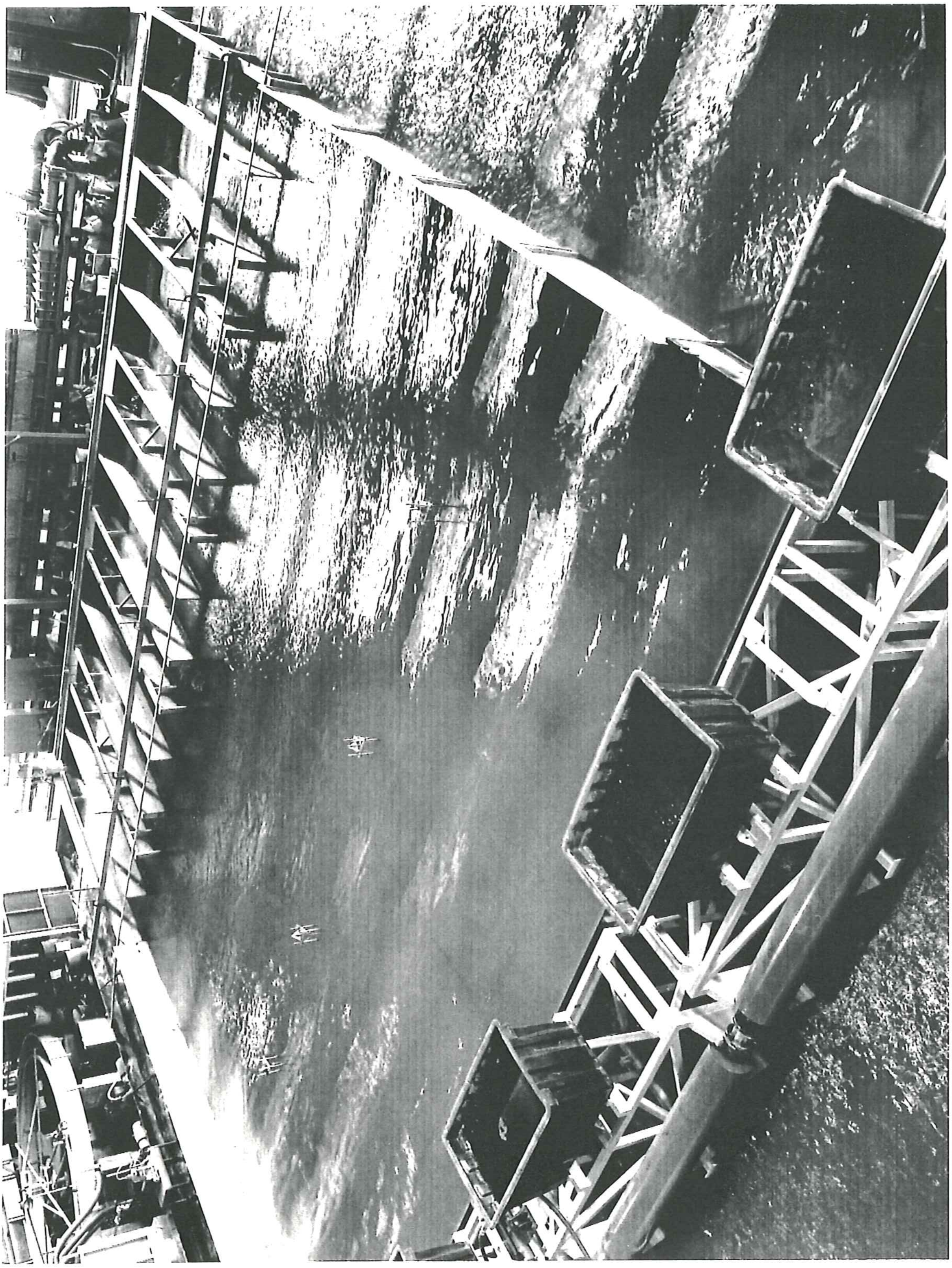


Plate 1 View of wave basin

Appendix

Appendix 1 Summary of profile parameters

Beach no	Spectrum	Crest		Transition		Toe	
		pc (m)	hc (m)	pt (m)	ht (m)	pb	hb
1	1	18.50	1.40	35.70	-1.39	52.90	-4.26
1	2	19.90	1.21	39.70	-1.87	52.90	-4.02
1	3	19.90	1.36	33.10	-0.87	52.90	-4.18
1	4	21.20	1.38	33.10	-0.74	52.90	-4.18
1	5	21.20	1.46	33.10	-0.65	45.00	-2.89
1	6	25.20	1.69	33.10	-0.55	47.50	-3.46
1	7	22.50	1.54	33.10	-0.83	47.50	-3.34
1	8	23.70	1.13	31.80	-0.89	47.50	-3.36
1	9	23.70	1.09	34.40	-0.58	45.00	-2.84
1	10	23.70	1.11	34.40	-0.63	45.00	-2.71
1	11	29.10	1.12	34.40	-0.52	45.00	-3.22
1	12	33.00	1.21	36.90	-1.18	n/r	n/r
2	1	9.30	2.77	33.10	-1.07	59.40	-4.80
2	2	13.20	2.26	38.20	-1.55	59.40	-4.82
2	3	15.80	1.82	33.10	-0.99	59.40	-4.89
2	4	15.80	1.78	33.10	-0.84	39.70	-1.85
2	5	14.50	2.04	38.20	-1.08	48.80	-2.74
2	6	22.50	1.68	33.10	-0.54	48.80	-3.43
2	7	22.50	1.44	35.70	-0.84	48.80	-3.48
2	8	23.70	0.84	36.90	-0.98	48.80	-3.31
2	9	23.70	1.05	36.90	-1.07	48.80	-3.38
2	10	23.70	1.10	39.70	-1.12	50.00	-3.38
2	11	29.10	1.38	39.70	-1.11	63.30	-5.84
2	12	31.80	1.78	48.80	-4.14	n/r	n/r
3	1	9.30	2.93	35.70	-0.88	59.40	-4.13
3	2	12.00	2.56	41.00	-1.52	59.40	-4.37
3	3	14.50	2.22	35.70	-1.05	41.00	-1.81
3	4	14.50	2.22	31.80	-0.26	41.00	-1.76

Beach no	Spectrum	Crest		Transition		Toe	
		pc (m)	hc (m)	pt (m)	ht (m)	pb	hb
3	5	14.50	2.17	36.90	-0.85	47.50	-2.58
3	6	19.90	1.80	33.10	-0.10	47.50	-2.81
3	7	19.90	1.70	36.90	-0.87	47.50	-2.92
3	8	0.04	1.21	36.90	-0.98	47.50	-2.84
3	9	22.50	1.35	33.10	-0.32	47.50	-2.79
3	10	22.50	1.33	36.90	-0.51	56.50	-4.20
3	11	26.40	1.33	n/r	n/r	62.00	-4.84
3	12	30.40	2.15	n/r	n/r	67.50	-5.30
4	1	2.80	3.52	29.10	-0.98	60.80	-4.67
4	2	8.10	2.84	36.90	-2.00	58.10	-4.58
4	3	9.30	2.59	31.80	-1.16	50.20	-3.63
4	4	9.30	2.63	30.40	-0.74	50.20	-3.71
4	5	9.30	2.55	42.30	-1.84	50.20	-3.37
4	6	15.80	1.78	31.80	-0.59	47.50	-3.31
4	7	15.80	1.80	31.80	-0.75	47.50	-3.34
4	8	19.90	1.21	31.80	-0.77	47.50	-3.34
4	9	19.90	1.23	31.80	-0.74	47.50	-3.31
4	10	21.20	1.16	31.80	-0.35	47.50	-3.33
4	11	26.40	1.31	39.70	-1.69	47.50	3.15
4	12	27.90	1.66	39.70	-1.22	58.10	-5.22
5	1	12.00	2.35	36.90	-0.92	54.10	-3.76
5	2	13.20	2.13	41.00	-1.92	62.10	-5.13
5	3	17.20	1.69	34.40	-0.80	50.20	-3.45
5	4	17.20	1.62	34.40	-0.65	50.20	-3.55
5	5	17.20	1.64	36.90	-1.00	48.80	-3.07
5	6	15.80	2.30	27.90	0.42	50.20	-3.57
5	7	18.50	1.91	34.40	-0.53	50.20	-3.53
5	8	23.70	0.97	35.70	-0.90	50.20	-3.60
5	9	23.70	1.26	31.80	-0.31	50.20	-3.63

Beach no	Spectrum	Crest		Transition		Toe	
		pc (m)	hc (m)	pt (m)	ht (m)	pb	hb
5	10	23.70	1.31	34.40	-0.35	50.20	-3.57
5	11	26.40	1.20	41.00	-1.26	50.20	-3.60
5	12	30.40	2.03	n/r	n/r	66.00	-5.62
6	1	12.00	2.61	35.70	-0.98	58.10	-4.38
6	2	14.50	2.38	42.30	-1.76	58.10	-4.38
6	3	17.20	1.96	35.70	-1.00	52.90	-3.65
6	4	17.20	1.92	30.40	-0.36	52.90	-3.74
6	5	15.80	1.26	31.80	-0.02	48.80	-2.96
6	6	21.20	1.45	35.70	-0.41	50.20	-3.44
6	7	21.20	1.54	36.90	-0.82	50.20	-3.35
6	8	22.50	1.36	33.10	-0.45	50.20	-3.41
6	9	22.50	1.38	34.40	-0.39	50.20	-3.33
6	10	23.70	1.18	38.20	-0.74	51.50	-3.57
6	11	27.90	1.47	38.20	-0.77	51.50	-3.79
6	12	30.40	2.01	n/r	n/r	6.00	-4.84
7	1	8.10	3.16	50.20	-2.61	60.80	-4.56
7	2	12.00	2.43	39.70	-1.97	54.10	-3.41
7	3	13.20	2.29	38.20	-1.83	54.10	-3.63
7	4	13.20	2.29	35.70	-1.42	54.10	-3.62
7	5	13.20	2.32	35.70	-1.10	48.80	-2.70
7	6	17.20	1.75	31.80	-0.80	51.50	-3.24
7	7	17.20	1.73	33.10	-0.34	51.50	-3.43
7	8	21.20	1.30	39.70	-1.42	51.50	-3.47
7	9	22.50	1.21	38.20	-1.18	51.50	-3.49
7	10	22.50	1.07	39.70	-1.36	51.50	-3.51
7	11	27.90	1.48	38.20	-0.89	56.00	4.37
7	12	27.90	1.71	n/r	n/r	64.00	-5.30
8	1	2.80	3.60	52.90	-3.07	60.80	-4.47

Beach no	Spectrum	Crest		Transition		Toe	
		pc (m)	hc (m)	pt (m)	ht (m)	pb	hb
8	2	8.10	2.94	52.90	-3.17	60.80	-4.42
8	3	9.30	2.70	36.90	-1.51	50.20	-3.22
8	4	9.30	2.65	36.90	-1.51	50.20	-3.25
8	5	9.30	2.75	38.20	-1.28	42.30	-1.80
8	6	15.80	1.77	39.70	-1.69	50.20	-3.32
8	7	17.20	1.77	38.20	-1.60	50.20	-3.48
8	8	21.20	1.30	39.70	-1.52	50.20	-3.49
8	9	22.50	1.28	34.40	-0.69	59.40	-5.04
8	10	22.50	1.40	38.20	-1.35	54.10	-4.37
8	11	27.90	1.45	n/r	n/r	51.00	3.59
8	12	29.50	2.03	n/r	n/r	63.00	-5.77
9	1	8.10	3.04	51.50	-2.79	58.10	-4.19
9	2	12.00	2.50	47.50	-2.60	58.10	-4.17
9	3	13.20	2.17	42.30	-1.95	50.20	-3.11
9	4	13.20	2.18	39.70	-1.53	50.20	-3.20
9	5	13.20	2.23	38.20	-1.39	46.20	-2.30
9	6	17.20	1.85	38.20	-1.27	62.10	-5.06
9	7	18.50	1.56	39.70	-1.64	52.90	-3.79
9	8	21.20	0.97	41.00	-1.51	52.90	-3.81
9	9	22.50	1.06	38.20	-1.05	52.90	-3.88
9	10	21.20	1.12	39.70	-1.19	51.50	-3.67
9	11	26.40	1.02	39.70	-1.29	51.50	-3.86
9	12	29.10	2.46	n/r	n/r	70.00	5.46
10	1	13.20	2.08	45.00	-2.54	51.50	-3.87
10	2	15.80	1.80	46.20	-2.73	50.20	-3.54
10	3	15.80	1.85	43.40	-2.26	52.90	-4.12
10	4	15.80	1.84	34.40	-1.20	45.00	-2.87
10	5	15.80	1.86	31.80	-0.61	39.70	-1.93
10	6	19.90	1.31	39.70	-1.71	50.20	-3.65

Beach no	Spectrum	Crest		Transition		Toe	
		pc (m)	hc (m)	pt (m)	ht (m)	pb	hb
10	7	19.90	1.32	38.20	-1.67	50.20	-3.67
10	8	21.20	1.09	38.20	-1.24	50.20	-3.72
10	9	23.70	0.79	35.70	-1.09	50.20	-3.57
10	10	22.50	0.93	38.20	-1.05	46.00	-2.81
10	11	26.40	0.93	38.20	-1.08	48.50	-3.12
10	12	29.10	1.22	n/r	n/r	53.00	-4.06