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WAVE REFLECTIONS IN HARBOURS:

Reflection performance of rock armoured slopes in random waves

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ABSTRACT

Wave reflections from sea walls or breakwaters often cause difficulties in the navigation and/or mooring of ships, and may also cause or exacerbate toe scour or local sea bed erosion. Such erosion is a common cause of the failure of many coastal structures. Rock armoured rubble structures often provide efficient energy dissipation reducing both wave run-up levels and wave reflections.

This report presents the results of a short series of hydraulic model tests on rock-armoured rubble sea wall sections under random waves. Wave reflections were measured for 9 wave conditions on structures of simple slopes of 1:1.5, 1:2.0, and 1:2-5; and 3 berm widths with upper and lower slopes of 1:1.5.

The results of these measurements are presented as values of the reflection coefficient, C_r. The results may be used to predict, and/or compare, the performance of rock armoured sea walls and breakwaters. The values of the coefficient derived can be used to give boundary conditions for mathematical models of wave action in harbours.

The work reported here has been conducted by Hydraulics Research for the British Overseas Development Administration. For further information on this work, please contact Mr N W H Allsop, manager of the Coastal Structures Section within the Maritime Engineering Department, Hydraulics Research.

NOTATION

A, B	Empirical coefficients
a, b	"
B	Structure width, in direction normal to face
C_1, C_2, C_i	Empirical or shape coefficients
C_r	Coefficient of reflection, defined in Equation 1
$C_r(f)$	Reflection coefficient function, defined in Equation 2
D	Particle size or typical dimension
D_n	Nominal particle diameter
E_i	Incident wave energy
E_r	Reflected wave energy
E_t	Transmitted wave energy
E_d	Energy absorbed or dissipated
F_c	Crest freeboard, level of crest less static water level
f	Wave frequency
f_p	Frequency of peak period = $1/T_p$
g	Gravitational acceleration
H	Wave height, from trough to crest
H_b	Wave height at onset of breaking
H_o	Offshore wave height, unaffected by shallow water processes
H_s	Significant wave height, average of highest one-third of wave heights
H_i	Incident wave height
H_{max}	Maximum wave height in a record
h	Water depth
Ir	Iribarren or surf similarity number, defined in Equation 3
Ir'	Modified Iribarren number, = $\tan\alpha/s_p^{1/2}$
k	Wave number, $2\pi/L$
L	Wave length, in the direction of propagation
L_{ps}, L_{ms}	Wave length at structure toe, of peak and mean wave periods respectively
L_o	Deep water or offshore wave length, $gT^2/2\pi$
L_s	Wave length, at the structure toe
M	Armour unit mass
M_{50}	Median armour unit mass
N	Number of waves in a storm, record or test
n	Porosity, usually taken as n_v

n_v	Volumetric porosity, volume of voids expressed as proportion of total volume
R	Run-up level, relative to static water level
$\bar{R}$	Mean run-up level
R_s	Run-up level of significant wave
$R_{u2\%}$	Run-up level exceeded by only 2% of run-up crests
R^*	Dimensionless freeboard
$R_{d2\%}$	Run-down level, below which only 2% pass
r	Roughness value, usually relative to smooth slopes
S_i	Incident spectral energy density
S_r	Reflected spectral energy density
s	Wave steepness, H/L_o
s_m	Wave steepness for mean period, $2\pi\ H_s/g\ T_m{}^2$
s_p	Wave steepness for peak period, $2\pi H_s/g\ T_p{}^2$
T	Wave period
T_m	Mean wave period
T_p	Spectral peak period, inverse of peak frequency
u, v	Flow velocities, often orthogonal components of velocity
W	Armour unit weight
W_{50}	Median armour unit weight
α	Structure front slope angle
β	Angle of wave attack
γ	Weight density
γ_w	Weight density of (sea) water
γ_r, γ_c	Weight density of rock (or concrete)
ρ	Mass density, usually of fresh water
ρ_w	Mass density of sea water
ρ_r, ρ_c	Mass density of rock (or concrete)
Δ	Relative buoyant density, eg $(\frac{\rho_r}{\rho_w} - 1)$

CONTENTS

1 INTRODUCTION

1.1 Purpose and organisation of the project

In 1985 the Engineering Adviser's conference of the Overseas Development Adminstration (ODA) identified a general problem of wave disturbance in a number of small harbours in the Caribbean. Changed patterns and degrees of wave activity were thought to be due to wave reflections from new structures, and/or the effects of wave refraction from dredged areas. A proposal was therefore submitted to ODA by Hydraulics Research (HR) to study the design and performance of wave absorbing structures, and to advise on the most effective and appropriate methods to reduce existing problems, and avoid further such problems. A review of literature and practice was conducted, and has been previously reported (Ref 1). In addition a paper has summarised some of the more important results of the review (Ref 2).

Following a site visit to review harbour performance (Ref 3), it was agreed that HR would run mathematical models of wave disturbance of three harbours in the Caribbean: Port Castries, St Lucia; St George's harbour, Grenada; and St John's, Antigua (Refs 4,5). The use of these models required the derivation of appropriate reflection coefficients for the harbour boundaries. It had previously been noted in the review (Ref 1), that data on the reflection performance of rock armoured slopes was sparse and uncertain. It was therefore agreed that a short series of random wave flume studies would be conducted to provide data both of general application, and to be used in the numerical modelling of wave disturbance. This report describes the model tests and summarises the test results.

1.2 Wave reflections at coastal structures

The importance of wave reflections from coastal and harbour structures has historically been given relatively little weight in the design of harbours or of coastal protection schemes, despite the problems that may arise from the cumulative local increase in wave energy. Typically, increased wave action due to reflections may lead to:

a) danger in navigating vessels through steep seas arising from the interaction of incident and reflected wave trains, this often occurs at harbour entrances;

b) increased berth down-time within the harbour arising from unacceptable vessel motions during loading or unloading;
c) damage to vessels, moorings, or fenders, arising from increased mooring forces;
d) increased wave velocities, and hence shear stresses, at the structure toe, leading to potentially greater local scour or sea bed erosion;
e) changes to wave-induced currents, and hence to the sediment movement regime.

All coastal structures reflect back some proportion of the wave energy incident upon them. The reflection performance of such structures is often described by a reflection coefficient, C_r. This is defined in terms of the incident and reflected wave heights, H_i and H_r respectively, or the total incident and reflected wave energies, E_i and E_r:

$$C_r = H_r/H_i = (E_r/E_i)^{1/2} \tag{1}$$

When considering random waves, values of C_r may be defined using the significant wave height as representative of the energy in the sea state. On occasions it is more useful to consider a reflection coefficient function $C_r(f)$, defined in terms of the incident and reflected spectral densities, $S_i(f)$ and $S_r(f)$, at each value of frequency, f:

$$C_r(f) = (S_r(f)/S_i(f))^{1/2} \tag{2}$$

Values of C_r, and less commonly $C_r(f)$, have been measured for a few example structures, but for many structure types little or no data has been published. A recent study reviewed that data available on a wide range of coastal structures (Ref 1). It was noted that very little data was available on reflections from rock armoured rubble structures. Only one report gave data on the reflection performance under random waves (Ref 15), and that was restricted to a single wave condition. A short series of hydraulic model tests were therefore initiated to derive values of C_r for typical rock armoured structures under a range of wave conditions. The results of this study are intended to complement those reported for a much wider range of structures in Reference 1.

1.3 Outline, and use, of this report

The main part of the report covers the design and preparations of the model tests, Chapter 2; the description of the test results, Chapter 3; and a discussion on the use and applicability of the data, Chapter 4. The main test parameter and results are listed in the tables.

Further general information on the test facility and model procedures is given in Appendices 1 and 2.

The study has been concerned with the wave reflection performance of rock armoured rubble mound structures, and has not studied aspects of armour response. It is noted however that the principal problem in the design of such structures is the determination of an appropriate armour size. This is discussed in Appendix 3.

This report is not intended to be used as a design guide or manual. It is anticipated that the reader intending to use the results here will be familiar with the design of coastal structures, and will consider the data in this report within the framework provided by Reference 1.

2 DESIGN OF MODEL TESTS

2.1 General

Hydraulic model testing can provide an inexpensive, rapid, and reliable method to quantify many of the wave/structure response functions for sea walls and breakwaters. Such model tests can examine the response of a structure to a wide range of water level and wave conditions, and can quantify the influence of many of the structure geometry variables. The principal hydraulic response functions studied in such models are wave run-up levels; overtopping discharges; wave reflections; and armour movement. The results of such studies are widely used in the design of sea walls, breakwaters, and related shoreline structures. Some examples of such recent studies on the design and performance of coastal structures have been discussed in References 7-12. A comprehensive review of the literature covering the design, construction and performance of sea walls is given by Reference 13. Previous studies identifying the reflection performance of coastal structures were reported earlier in this project in Reference 1.

The test facility selected for these studies was HR's wind wave flume. This is now a conventional wave flume equipped with a hydraulically-driven random wave paddle, and is described further in Appendix 1.

For this study a simplified approach bathymetry was used, shown in Figure 1. The wave paddle was operated in a water depth of 0.610m. Use of this depth at the test sections however would have required relatively large sections, and a consequent reduction in the number of tests possible with the resources available. An elevated sea bed was therefore built, reducing the test water depth to h_s = 0.380m. A range of test sections were used in the study and these are described further in section 2.2 below.

Before any test sections were constructed, the wave conditions to be used were measured immediately seaward of the position of the test section. For these calibration tests an absorbing beach at the end of the flume prevented the incident wave conditions from being corrupted by reflections. The wave conditions used for testing are discussed in section 2.3 below and summarised in Table 2.

During testing both incident and reflected wave spectra were measured with an array of 3 wave probes, positioned over the horizontal approach bed section. These allowed the derivation of the reflection coefficient function, $C_r(f)$, at around 16 values of f. Measurement and analysis procedures are discussed further in section 2.4, and in Appendix 2.

2.2 Test sections

A total of 19 cross-sections were used in these tests to explore the effects of:

a) front face slope angle, α;
b) smooth or armoured facing;
c) armour layer thickness, t_a;
d) armour unit size, M_{50}, D_{n50};
e) berm length, B.

The primary geometric parameter affecting the hydraulic performance of a rubble coastal structures is the front face slope angle, α. The practical range for α is relatively narrow, limited by economic and construction considerations. The steepest slope angle will be set by the natural angle of repose of the rubble, and the stable slope for armour, under the influence of wave loading. A limit of 1:1.33, $\cot\alpha$ = 4/3 is commonly accepted, although the steepest slope generally adopted is probably 1:1.5, $\cot\alpha$ = 3/2. Shallower slope angles may often be used to reduce the armour size required, and/or to improve the hydraulic performance. Naturally such shallow slopes will require more fill material. The choice of a shallow slope angle involves a balance between cost and performance. These considerations generally seem to limit the range of slopes used between 1:1.5 to 1:2.5. Three slope angles adopted for these studies were $\cot\alpha$ = 1.5, 2.0 and 2.5.

The simplest structure type from the hydraulic viewpoint has a plane impermeable smooth front face. Three smooth-faced test sections, A/1-3, were used to give control sections at $\cot\alpha$ = 1.5, 2.0, and 2.5 respectively. All subsequent test sections used a rubble core of 0-0.01m crushed rock , with an underlayer of D = 0.02-0.03m, and then armour layer, or layers.

Test sections B/1-3, C/1-3, D/1-3, and E/1-3 were used to study the effects of variations of slope angle α; armour size M_{50}, D_{n50}; and number of armour layers, t_a/D_{n50}. For test sections B/1-3 and C/1-3 the armour was laid in conventional 2 layer construction. Sections B/1-3 used armour of mass 0.206-0.411kg, M_{50} = 0.326kg. The nominal median diameter for this armour size was D_{n50} = 0.0494m. Sections C/1-3 used a larger armour size, M = 0.411-0.685kg, M_{50} = 0.485kg, D_{n50} = 0.0563m.

Conventionally armour rock is laid in a 2 layer thickness, and it is for this construction that empirical design methods have been developed. In some instances, including locations in the Caribbean and the UK, a single layer of armour has been laid. Whilst this form of armouring would not be recommended from stability considerations, it was recognised that its historic use meant that an assessment of its reflection performance was needed. Sections D and E used the standard and larger armour used in sections B and C respectively, but laid to a single layer only.

In previous studies of sea walls and breakwaters it has been noted that a step or berm placed at, or close to, the design water level will often yield a considerable improvement in hydraulic performance. Previous work by HR on overtopping (Ref 6) demonstrated that a greater improvement in overtopping performance may often be achieved by placing material to form a berm, rather than using the same volume to increase the section crest height. The position of the berm is again governed by cost; practical construction considerations; and hydraulic efficiency. For these studies three berm lengths were tested, all placed so that the upper surface of the armour on the berm was at the static water level. The larger rock, M_{50} = 0.485kg, was used in 2 layer construction for all bermed test sections. For sections F/1-3 berm widths of B = 0.20, 0.40 and 0.80 metres respectively were used, and the upper and lower slope angle was kept constant at 1:1.5. Section G/1 used upper and lower slope angles of 1:2.5 and a berm length B = 0.40m.

The main features of the model test sections are illustrated in Figures 1 and 2, and are summarised in Table 1.

This study was not intended to consider aspects of armour stability. It may however be helpful to the reader to summarise briefly the main sources of data available for the determination of armour unit size required, and this is done in Appendix 3.

2.3 Test conditions

Previous work (Refs 1 and 2) has identified a number of empirical prediction methods for C_r using dimensionless parameters such as the Iribarren number:

$$Ir = \tan\alpha / s_m^{1/2} \tag{3}$$

where the wave steepness for the mean wave period may be defined:

$$s_m = 2\pi \, H_s / g \, T_m^2 \tag{4}$$

At and within a coastal harbour wave conditions will vary significantly. At any outer breakwater the incident waves will be relatively large and steep, with values for s_m often around 0.04-0.05 or greater. Under more common conditions wave heights will be less, and wave periods may often be greater, leading to markedly less steep wave conditions. Similarly, at structures within a harbour wave heights are reduced, whilst the mean or peak periods are less affected. Sea steepness here may then be around 0.004 or lower.

For these studies a set of 9 sea states were used with mean steepness, s_m, from 0.0043 to 0.052. Relative local wave lengths, L_{ms}/h_s, varied from 6.2 to 14.8. In each instance standard JONSWAP wave spectra were generated at the wave paddle.

The test conditions used in this study are summarised in Table 2.

2.4 Test procedures and measurements

The purpose of these tests was to quantify the reflection performance of a range of structure configurations under a variety of wave conditions. No other measurements were made. For each test a relatively short sequence of random waves was generated, typically around 250 waves long. The incident and reflected wave spectra were measured for a sample length exactly matching the sequence length generated. An array of 3 wave probes in a constant water depth seaward of the test sections ensured that a wide range of wave frequencies were covered. The probe output was scaled and analysed on a PDP 11 mini-computer. Incident and reflected wave spectra, and values for the reflection coefficient function, $C_r(f)$, were calculated over the frequency band from $0.5\ f_p$ to $2.0\ f_p$. For each test condition a single value of C_r has been used in all further analysis. The results of the study are summarised in Tables 3 and 4.

3 ANALYSIS OF TEST RESULTS

3.1 Simple slopes

The test results for simple, or plain, slopes are summarised in Table 3. Values of C_r are derived for each test section, slope angle, and test wave condition. Following the previous work (Ref 1), the main dimensionless parameter used to describe wave behaviour on a plain slope is the Iribarren number, Ir, defined in equation 3. The results for sections A-E are shown in Figures 3-12 as C_r against Ir.

Previous studies have explored the use of a number of simple empirical equations. That used earlier in this project may be given in terms of C_r and Ir, and empirical coefficients a and b:

$$C_r = \frac{a\ Ir^2}{Ir^2 + b} \qquad (5)$$

In previous work this equation has been fitted by regression to give values for a and b, (Refs 1-2). This exercise has been repeated in this study. The regression curves are shown in Figures 3-7 and the coefficients derived may be summarised:

Test section	a	b
Smooth, A/1-3	1.02	5.57
Armoured, B/1-3	0.52	5.97
Armoured, C/1-3	0.52	6.82
Armoured, D/1-3	0.53	5.14
Armoured, E/1-3	0.56	5.69

Inspection of Figures 3-7 suggests that this method gives a tolerable description of the data for smooth slopes (Fig 3), but the regression curve does not fit the data well for armoured slopes, particularly for $Ir > 5$.

It was noted that the test conditions used gave greater emphasis to results for $Ir < 5$, resulting in more test results in that range. A revised regression was therefore attempted in which each of the data values in the range above $Ir = 5$ was progressively weighted more heavily. This had the effect of placing more emphasis on this end of the data. The revised regression curves, still based on equation 5, are shown in Figures 8-12. The coefficients derived in this modified regression may be summarised:

Test section	a	b
A/1-3	0.96	4.80
B/1-3	0.64	8.85
C/1-3	0.64	9.64
D/1-3	0.64	7.22
E/1-3	0.67	7.87

A comparison of these curves with those derived from the un-weighted regression, shows better agreement. It is clear however that neither method is fully successful. It is probable that an alternative general empirical equation would give a better fit, particularly for the armoured slope. It may be noted however that this approach still involves considerable simplifications. For example, it may be seen from numerical models of wave reflection and transmission that the wave/structure interaction is controlled by many more parameters than the Iribarren number. Time and resources did not however allow a further analysis of the test results here. The use of the test results is discussed further in Chapter 4.

3.2 Bermed slopes

In this study four series of tests were run on bermed slopes, F/1-3 and G/1. Test sections F/1-3 differed only in the berm length, B, which varied from 0.2 to 0.8m. Sections F/2 and G/1 had the same berm length, B = 0.4m, but the slope angle for both upper and lower slopes was changed from 1:1.5 for F/2 to 1:2.5 for G/1. In considering bermed slopes it is more difficult to establish a simple and reliable dimensionless parameter comparable with the Iribarren number. A composite slope angle cannot be defined unambiguously to be used in Ir. In analysing the reflection performance it is more useful to establish other dimensionless parameters than to use the berm length in model units. A number of dimensionless parameters have therefore been developed using the berm length, B; the local water depth, h_s; the wave length at the structure, L_{ms}; and the wave length offshore, L_{mo}.

The wave parameter used previously in the description of the reflection performance of simple slopes was the wave steepness, s_m, using the wavelength of the mean period, T_m, in deep water. It might be argued that it would be more appropriate to calculate wave steepness using the wave length in the water depth at the structure, L_{ms}, rather than the offshore wavelength, L_{mo}. In discussing wave breaking, Southgate (Ref 14) has noted that paradoxically the parameter using L_{mo} often gives a better classification of wave breaking than that using L_{ms}. It is possible that this effect might similarly influence the reflection behaviour. In this analysis both values have therefore been used. The reflection coefficient C_r is plotted against H_s/L_{ms} in Figure 13, and against H_s/L_{mo} in Figure 14. In neither instance does a clear view of the reflection response emerge.

The picture is clearer when C_r is plotted against the relative berm length B/L. The local wave length, L_{ms}, is used in Figure 15, and the offshore, L_{mo}, in Figure 16. In each instance three curves are shown, each for the different values of B/h_s. It may be noted that a careful examination suggests a residuary effect of H_s giving the spread of C_r in each set. This effect is not strong, and it would seem appropriate to use either the mean curve, or an upper bound, in predictions. For the range of conditions tested in this study there is no clear reason to prefer L_{ms} or L_{mo}. It should be noted that the main advantage of Figures 15 and 16 is that the introduction of the berm length separates the sets of data. The different berm lengths do not of themselves yield greatly different reflection results, although all offer lower reflections than the equivalent simple slopes.

The final series of tests were intended to explore the effect of a shallow slope angle (1:2.5) in combination with a berm, B = 0.4m, comparing the performance of section G/1 with F/2. The reflection coefficient, C_r, is plotted against B/L_{ms} in Figure 17. The change of slope angle leads to a reduction in C_r, and this would appear to be greater than that resulting from extending the berm length B. No further combinations of berm length and slope angle were tested in this study. It may be reasonable to assume that structures with berms shorter than that tested would show a greater influence of α on C_r, whilst larger values of B would reduce the effect of α.

4 APPLICATION AND USE OF RESULTS

The results presented in this report are intended to assist a coastal engineer who requires:

a) to identify the comparative effects on wave reflections of changes to an existing, or proposed structure, or of alternative structures;
b) to calculate values of the reflection coefficient for use in the definitions of boundary conditions in models of harbour wave disturbance, or in the estimation of toe erosion or beach scour.

For those instances where the absolute level of C_r is less important than the change in C_r for changes in the structure, or for alternative configuration, it will be sufficient to compare prediction curves or equations. When comparing values of C_r derived here with those derived in other studies, the user is cautioned to compare the definitions and methods, as usage varies widely. For example, many methods are based only on the results of tests with regular waves. Their applicability to real sea conditions will often not be well established.

Where a value must be determined for C_r for use in later calculations, the engineer must decide on the level of any safety factors to be applied, and/or the sensitivity testing needed. Much of the data used here and in previous work embodies considerable scatter. A further concern will be the application of tests at small scale to the prototype situation. These experiments were not intended to cover a particular site, or sites, so no scale factor has been used. Nor were the tests sufficiently comprehensive to yield a general design method of wide application. The analysis presented in Chapter 3 was therefore intentionally simple. The results of the tests are summarised fully in Tables 1-4, allowing the user to compare the data with other prediction methods if required. The analysis in Chapter 3 has produced prediction curves that can be used directly, and this may be particularly useful in estimating boundary conditions for numerical models of wave disturbance (Refs 4,5). To the results of the tests on simple slopes have been fitted equations of the form developed by Seelig (see discussion in Reference 1). Values of coefficients a and b in equation 5 have been derived using simple regression. A modified regression, in which larger values or Ir are weighted more heavily, has generated an alternative set of coefficients. It should however be noted that neither approach is fully successful in describing the full data set. A further weakness is the lack of any quantitative assessment of the scatter. The simplest way of overcoming this is to estimate an upper bound to the results given in the figures.

For simple slopes the results are presented in terms of dimensionless parameters C_r and Ir. In use it is expected that typical values of Ir will be calculated for design and service wave conditions. Values of C_r can then be estimated using the relevant graph, or equation 5 with appropriate values of coefficients a and b. If the structure considered is in relatively shallow water these methods may overestimate C_r. A reduction factor as used by Seelig is discussed in Reference 1, although its use has not been validated here.

For bermed structures Figures 13 and 14, 15 and 16, or 17 may be used directly. In each instance it may be useful to estimate the value of C_r for the equivalent simple slope. Then for the same sea states values of wave steepness, H_s/L_{ms} or H_s/L_{mo}, can be used to estimate C_r from Figures 13 or 14 respectively. Alternatively C_r can be estimated for given berm lengths and wave lengths from Figures 15-17.

5 RECOMMENDATIONS

In the studies reported here coefficients of wave reflections have been derived for a wide range of structure configurations and sea states. To establish reflection coefficients for the range of conditions tested, it will generally be sufficient to use the values measured, interpolate between test results, or use the prediction equations where derived.

For structure configurations, or wave conditions, lying outside of the ranges tested, recourse should be made where possible to the methods and data discussed in the earlier review (Ref 1).

Where reflection characteristics are required for configurations not tested here, nor covered in sufficient detail in the published literature, it is recommended that hydraulic model tests be carried out to establish the reflection performance.

6 ACKNOWLEDGEMENTS

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TABLES.

TABLE 1 Test Sections

Section	Tests	Slope angle, cotα	Armour unit mass M_{50}(kg)	Armour layer thickness	Berm length B(m)
A/1	/1-9	1.500	Smooth	-	-
A/2	/1-9	2.000	Smooth	-	-
A/3	/1-9	2.500	Smooth	-	-
B/1	/1-9	1.500	0.326	2	-
B/2	/1-9	2.000	0.326	2	-
B/3	/1-9	2.500	0.326	2	-
C/1	/1-9	1.500	0.485	2	-
C/2	/1-9	2.000	0.485	2	-
C/3	/1-9	2.500	0.485	2	-
D/1	/1-9	1.500	0.326	1	-
D/2	/1-9	2.000	0.326	1	-
D/3	/1-9	2.500	0.326	1	-
E/1	/1-9	1.500	0.485	1	-
E/2	/1-9	2.000	0.485	1	-
E/3	/1-9	2.500	0.485	1	-
F/1	/1-9	1.500	0.485	2	0.200
F/2	/1-9	1.500	0.485	2	0.400
F/3	/1-9	1.500	0.485	2	0.800
G/1	/1-9	2.500	0.485	2	0.400

Notes
a) All tests were conducted with a water depth at the toe of the structure, h_s = 0.380m
b) For sections F/1-3 and G/1 the berm level was set at static water level.

TABLE 2 Summary of Test Conditions

Test part	Significant wave height H_s(m)	Mean wave period T_m(s)	Peak wave period T_p(s)	Mean sea steepness s_m	Relative wave length at structure L_{ms}/h_s
/1	0.060	1.400	1.610	0.020	6.190
/2	0.120	1.400	1.610	0.039	6.190
/3	0.160	1.400	1.610	0.052	6.190
/4	0.060	1.700	1.950	0.013	7.876
/5	0.120	1.700	1.950	0.027	7.876
/6	0.160	1.700	1.950	0.035	7.876
/7	0.060	2.200	2.530	0.008	10.589
/8	0.120	2.200	2.530	0.016	10.589
/9	0.060	3.000	3.450	0.004	14.810

TABLE 3 Summary of test results, simple slopes

Section	Test part	Wave height H_s(m)	Wave period T_m(s)	Sea steepness s_m	Iribarren number Ir	Reflection coefficient C_r	Mean offshore wave length L_{mo}	Mean local wave length L_{ms}
A/1	/1	0.060	1.400	0.020	4.761	0.83	3.059	2.352
	/2	0.120	1.400	0.039	3.367	0.75	3.059	2.352
	/3	0.160	1.400	0.052	2.916	0.71	3.059	2.352
	/4	0.060	1.700	0.013	5.781	0.84	4.511	2.993
	/5	0.120	1.700	0.027	4.088	0.82	4.511	2.993
	/6	0.160	1.700	0.035	3.540	0.77	4.511	2.993
	/7	0.060	2.200	0.008	7.482	0.83	7.554	4.024
	/8	0.120	2.200	0.016	5.290	0.79	7.554	4.024
	/9	0.060	3.000	0.004	10.202	0.88	14.047	5.628
A/2	/1	0.060	1.400	0.020	3.571	0.72	3.059	2.352
	/2	0.120	1.400	0.039	2.525	0.55	3.059	2.352
	/3	0.160	1.400	0.052	2.187	0.51	3.059	2.352
	/4	0.060	1.700	0.013	4.336	0.80	4.511	2.993
	/5	0.120	1.700	0.027	3.066	0.69	4.511	2.993
	/6	0.160	1.700	0.035	2.655	0.62	4.511	2.993
	/7	0.060	2.200	0.008	5.611	0.81	7.554	4.024
	/8	0.120	2.200	0.016	3.968	0.75	7.554	4.024
	/9	0.060	3.000	0.004	7.652	0.86	14.047	5.628
A/3	/1	0.060	1.400	0.020	2.857	0.60	3.059	2.352
	/2	0.120	1.400	0.039	2.020	0.39	3.059	2.352
	/3	0.160	1.400	0.052	1.749	0.34	3.059	2.352
	/4	0.060	1.700	0.013	3.469	0.72	4.511	2.993
	/5	0.120	1.700	0.027	2.453	0.53	4.511	2.993
	/6	0.160	1.700	0.035	2.124	0.47	4.511	2.993
	/7	0.060	2.200	0.008	4.489	0.78	7.554	4.024
	/8	0.120	2.200	0.016	3.174	0.65	7.554	4.024
	/9	0.060	3.000	0.004	6.121	0.82	14.047	5.628
B/1	/1	0.060	1.400	0.020	4.761	0.33	3.059	2.352
	/2	0.120	1.400	0.039	3.367	0.34	3.059	2.352
	/3	0.160	1.400	0.052	2.916	0.35	3.059	2.352
	/4	0.060	1.700	0.013	5.781	0.45	4.511	2.993
	/5	0.120	1.700	0.027	4.088	0.45	4.511	2.993
	/6	0.160	1.700	0.035	3.540	0.45	4.511	2.993
	/7	0.060	2.200	0.008	7.482	0.57	7.554	4.024
	/8	0.120	2.200	0.016	5.290	0.55	7.554	4.024
	/9	0.060	3.000	0.004	10.202	0.68	14.047	5.628
B/2	/1	0.060	1.400	0.020	3.571	0.24	3.059	2.352
	/2	0.120	1.400	0.039	2.525	0.23	3.059	2.352
	/3	0.160	1.400	0.052	2.187	0.27	3.059	2.352
	/4	0.060	1.700	0.013	4.336	0.33	4.511	2.993
	/5	0.120	1.700	0.027	3.066	0.34	4.511	2.993
	/6	0.160	1.700	0.035	2.655	0.35	4.511	2.993
	/7	0.060	2.200	0.008	5.611	0.48	7.554	4.024
	/8	0.120	2.200	0.016	3.968	0.46	7.554	4.024
	/9	0.060	3.000	0.004	7.652	0.62	14.047	5.628

B/3	/1	0.060	1.400	0.020	2.857	0.20	3.059	2.352
	/2	0.120	1.400	0.039	2.020	0.18	3.059	2.352
	/3	0.160	1.400	0.052	1.749	0.20	3.059	2.352
	/4	0.060	1.700	0.013	3.469	0.25	4.511	2.993
	/5	0.120	1.700	0.027	2.453	0.25	4.511	2.993
	/6	0.160	1.700	0.035	2.124	0.27	4.511	2.993
	/7	0.060	2.200	0.008	4.489	0.37	7.554	4.024
	/8	0.120	2.200	0.016	3.174	0.37	7.554	4.024
	/9	0.060	3.000	0.004	6.121	0.54	14.047	5.628
C/1	/1	0.060	1.400	0.020	4.761	0.34	3.059	2.352
	/2	0.120	1.400	0.039	3.367	0.33	3.059	2.352
	/3	0.160	1.400	0.052	2.916	0.34	3.059	2.352
	/4	0.060	1.700	0.013	5.781	0.44	4.511	2.993
	/5	0.120	1.700	0.027	4.088	0.44	4.511	2.993
	/6	0.160	1.700	0.035	3.540	0.44	4.511	2.993
	/7	0.060	2.200	0.008	7.482	0.55	7.554	4.024
	/8	0.120	2.200	0.016	5.290	0.54	7.554	4.024
	/9	0.060	3.000	0.004	10.202	0.68	14.047	5.628
C/2	/1	0.060	1.400	0.020	3.571	0.21	3.059	2.352
	/2	0.120	1.400	0.039	2.525	0.21	3.059	2.352
	/3	0.160	1.400	0.052	2.187	0.23	3.059	2.352
	/4	0.060	1.700	0.013	4.336	0.31	4.511	2.993
	/5	0.120	1.700	0.027	3.066	0.31	4.511	2.993
	/6	0.160	1.700	0.035	2.655	0.33	4.511	2.993
	/7	0.060	2.200	0.008	5.611	0.45	7.554	4.024
	/8	0.120	2.200	0.016	3.968	0.43	7.554	4.024
	/9	0.060	3.000	0.004	7.652	0.60	14.047	5.628
C/3	/1	0.060	1.400	0.020	2.857	0.18	3.059	2.352
	/2	0.120	1.400	0.039	2.020	0.17	3.059	2.352
	/3	0.160	1.400	0.052	1.749	0.18	3.059	2.352
	/4	0.060	1.700	0.013	3.469	0.24	4.511	2.993
	/5	0.120	1.700	0.027	2.453	0.23	4.511	2.993
	/6	0.160	1.700	0.035	2.124	0.25	4.511	2.993
	/7	0.060	2.200	0.008	4.489	0.36	7.554	4.024
	/8	0.120	2.200	0.016	3.174	0.35	7.554	4.024
	/9	0.060	3.000	0.004	6.121	0.53	14.047	5.628
D/1	/1	0.060	1.400	0.020	4.761	–	3.059	2.352
	/2	0.120	1.400	0.039	3.367	–	3.059	2.352
	/3	0.160	1.400	0.052	2.916	–	3.059	2.352
	/4	0.060	1.700	0.013	5.781	–	4.511	2.993
	/5	0.120	1.700	0.027	4.088	–	4.511	2.993
	/6	0.160	1.700	0.035	3.540	–	4.511	2.993
	/7	0.060	2.200	0.008	7.482	–	7.554	4.024
	/8	0.120	2.200	0.016	5.290	–	7.554	4.024
	/9	0.060	3.000	0.004	10.202	–	14.047	5.628

D/2	/1	0.060	1.400	0.020	3.571	0.28	3.059	2.352
	/2	0.120	1.400	0.039	2.525	0.26	3.059	2.352
	/3	0.160	1.400	0.052	2.187	0.29	3.059	2.352
	/4	0.060	1.700	0.013	4.336	0.38	4.511	2.993
	/5	0.120	1.700	0.027	3.066	0.37	4.511	2.993
	/6	0.160	1.700	0.035	2.655	0.38	4.511	2.993
	/7	0.060	2.200	0.008	5.611	0.53	7.554	4.024
	/8	0.120	2.200	0.016	3.968	0.50	7.554	4.024
	/9	0.060	3.000	0.004	7.652	0.66	14.047	5.628
D/3	/1	0.060	1.400	0.020	2.857	0.25	3.059	2.352
	/2	0.120	1.400	0.039	2.020	0.21	3.059	2.352
	/3	0.160	1.400	0.052	1.749	0.21	3.059	2.352
	/4	0.060	1.700	0.013	3.469	0.29	4.511	2.993
	/5	0.120	1.700	0.027	2.453	0.28	4.511	2.993
	/6	0.160	1.700	0.035	2.124	0.29	4.511	2.993
	/7	0.060	2.200	0.008	4.489	0.42	7.554	4.024
	/8	0.120	2.200	0.016	3.174	0.41	7.554	4.024
	/9	0.060	3.000	0.004	6.121	0.58	14.047	5.628
E/1	/1	0.060	1.400	0.020	4.761	0.36	3.059	2.352
	/2	0.120	1.400	0.039	3.367	0.36	3.059	2.352
	/3	0.160	1.400	0.052	2.916	–	3.059	2.352
	/4	0.060	1.700	0.013	5.781	0.49	4.511	2.993
	/5	0.120	1.700	0.027	4.088	0.48	4.511	2.993
	/6	0.160	1.700	0.035	3.540	–	4.511	2.993
	/7	0.060	2.200	0.008	7.482	0.60	7.554	4.024
	/8	0.120	2.200	0.016	5.290	0.58	7.554	4.024
	/9	0.060	3.000	0.004	10.202	0.70	14.047	5.628
E/2	/1	0.060	1.400	0.020	3.571	0.27	3.059	2.352
	/2	0.120	1.400	0.039	2.525	0.25	3.059	2.352
	/3	0.160	1.400	0.052	2.187	0.28	3.059	2.352
	/4	0.060	1.700	0.013	4.336	0.37	4.511	2.993
	/5	0.120	1.700	0.027	3.066	0.37	4.511	2.993
	/6	0.160	1.700	0.035	2.655	0.37	4.511	2.993
	/7	0.060	2.200	0.008	5.611	0.52	7.554	4.024
	/8	0.120	2.200	0.016	3.968	0.50	7.554	4.024
	/9	0.060	3.000	0.004	7.652	0.65	14.047	5.628
E/3	/1	0.060	1.400	0.020	2.857	0.25	3.059	2.352
	/2	0.120	1.400	0.039	2.020	0.21	3.059	2.352
	/3	0.160	1.400	0.052	1.749	0.22	3.059	2.352
	/4	0.060	1.700	0.013	3.469	0.30	4.511	2.993
	/5	0.120	1.700	0.027	2.453	0.28	4.511	2.993
	/6	0.160	1.700	0.035	2.124	0.29	4.511	2.993
	/7	0.060	2.200	0.008	4.489	0.44	7.554	4.024
	/8	0.120	2.200	0.016	3.174	0.41	7.554	4.024
	/9	0.060	3.000	0.004	6.121	0.59	14.047	5.628

TABLE 4 Summary of Test Results, Bermed slopes

Section	Test part	Wave height H_s(m)	Wave period T_m(s)	Mean local wave length L_{ms}(m)	B/h_s	B/L_{ms}	Mean offshore wave length L_{mo}(m)	B/L_{mo}	Reflection Coefficient Cr
F/1	/1	0.060	1.400	2.352	0.526	0.085	3.059	0.065	0.16
	/2	0.120	1.400	2.352	0.526	0.085	3.059	0.065	0.18
	/3	0.160	1.400	2.352	0.526	0.085	3.059	0.065	0.21
	/4	0.060	1.700	2.993	0.526	0.067	4.511	0.064	0.21
	/5	0.120	1.700	2.993	0.526	0.067	4.511	0.044	0.26
	/6	0.160	1.700	2.993	0.526	0.067	4.511	0.044	0.29
	/7	0.060	2.200	4.024	0.526	0.050	7.554	0.046	0.34
	/8	0.120	2.200	4.024	0.526	0.050	7.554	0.026	0.37
	/9	0.060	3.000	5.628	0.526	0.036	14.047	0.024	0.52
F/2	/1	0.060	1.400	2.352	1.053	0.170	3.059	0.131	0.21
	/2	0.120	1.400	2.352	1.053	0.170	3.059	0.131	0.25
	/3	0.160	1.400	2.352	1.053	0.170	3.059	0.131	0.25
	/4	0.060	1.700	2.993	1.053	0.134	4.511	0.139	0.26
	/5	0.120	1.700	2.993	1.053	0.134	4.511	0.089	0.28
	/6	0.160	1.700	2.993	1.053	0.134	4.511	0.089	0.29
	/7	0.060	2.200	4.024	1.053	0.099	7.554	0.083	0.33
	/8	0.120	2.200	4.024	1.053	0.099	7.554	0.053	0.32
	/9	0.060	3.000	5.628	1.053	0.071	14.047	0.058	0.45
F/3	/1	0.060	1.400	2.352	2.105	0.340	3.059	0.262	0.12
	/2	0.120	1.400	2.352	2.105	0.340	3.059	0.262	0.18
	/3	0.160	1.400	2.352	2.105	0.340	3.059	0.262	0.22
	/4	0.060	1.700	2.993	2.105	0.267	4.511	0.267	0.19
	/5	0.120	1.700	2.993	2.105	0.267	4.511	0.177	0.27
	/6	0.160	1.700	2.993	2.105	0.267	4.511	0.177	0.28
	/7	0.060	2.200	4.024	2.105	0.199	7.554	0.176	0.31
	/8	0.120	2.200	4.024	2.105	0.199	7.554	0.106	0.33
	/9	0.060	3.000	5.628	2.105	0.142	14.047	0.107	0.39
G/1	/1	0.060	1.400	2.352	1.053	0.170	3.059	0.051	0.13
	/2	0.120	1.400	2.352	1.053	0.170	3.059	0.131	0.16
	/3	0.160	1.400	2.352	1.053	0.170	3.059	0.131	0.18
	/4	0.060	1.700	2.993	1.053	0.134	4.511	0.139	0.15
	/5	0.120	1.700	2.993	1.053	0.134	4.511	0.089	0.18
	/6	0.160	1.700	2.993	1.053	0.134	4.511	0.089	0.19
	/7	0.060	2.200	4.024	1.053	0.099	7.554	0.083	0.20
	/8	0.120	2.200	4.024	1.053	0.099	7.554	0.053	0.22
	/9	0.720	3.000	5.628	1.053	0.071	14.047	0.058	0.30

FIGURES.

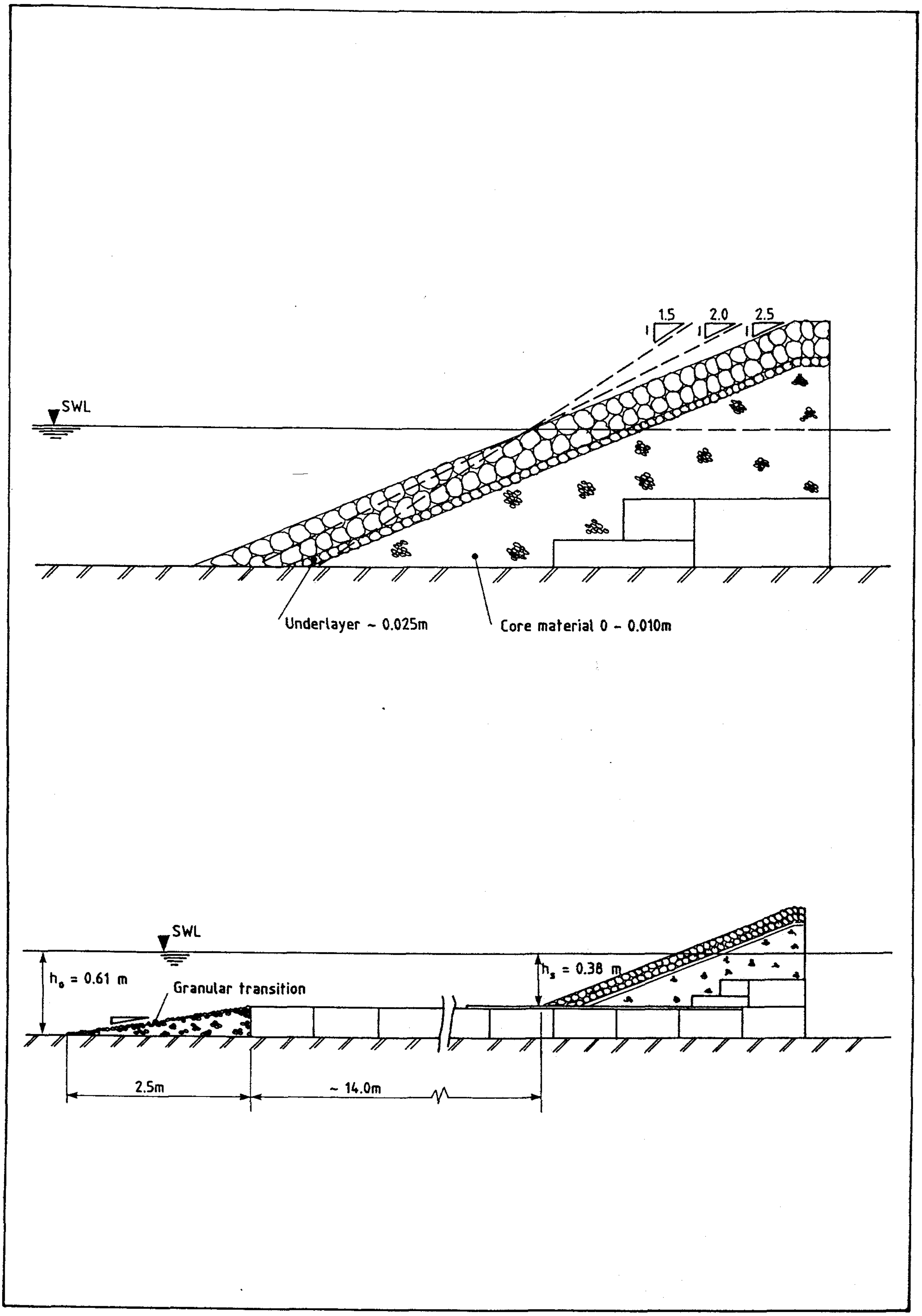

Fig 1 Test section layout and simple slopes

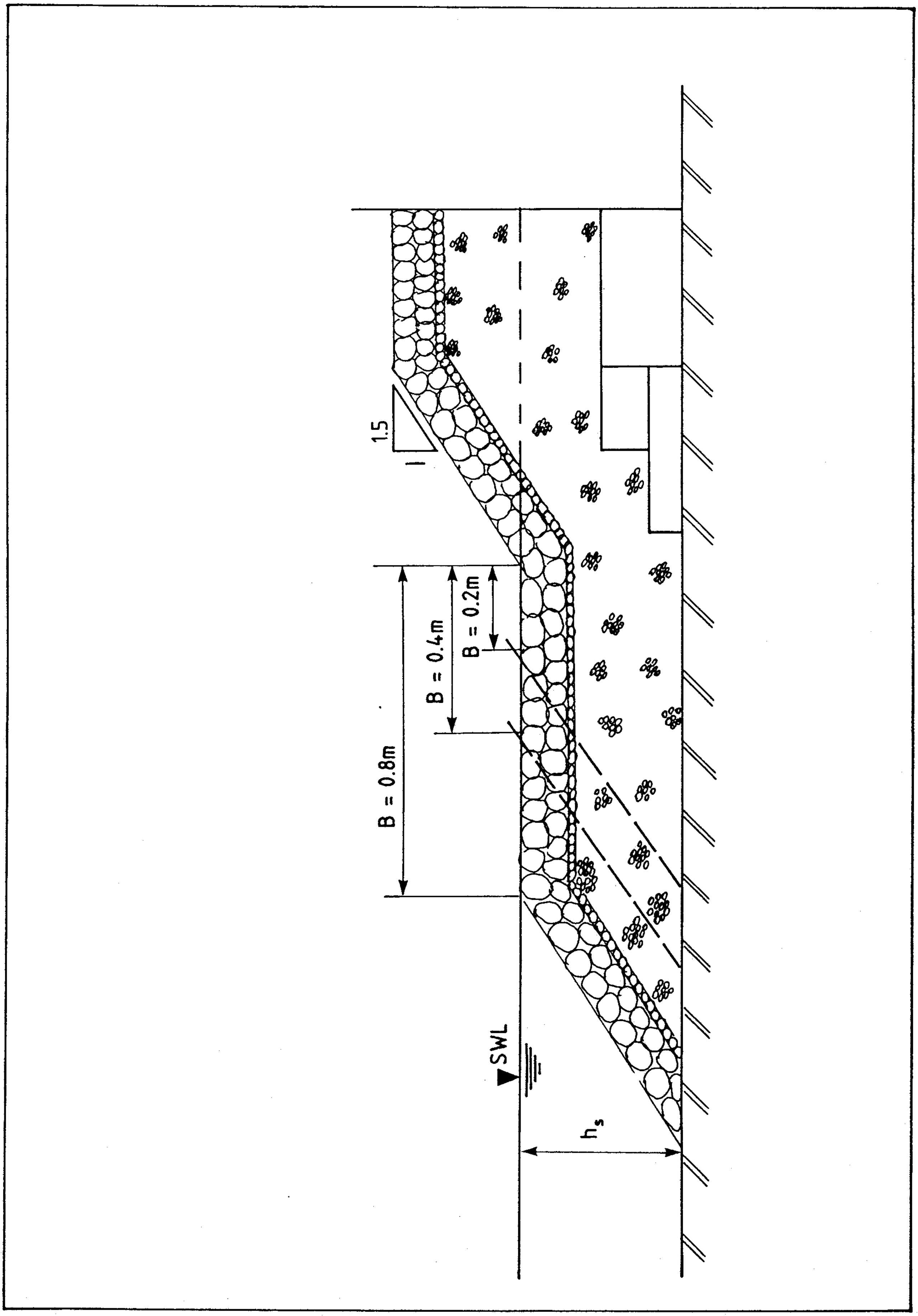

Fig 2 Bermed test section

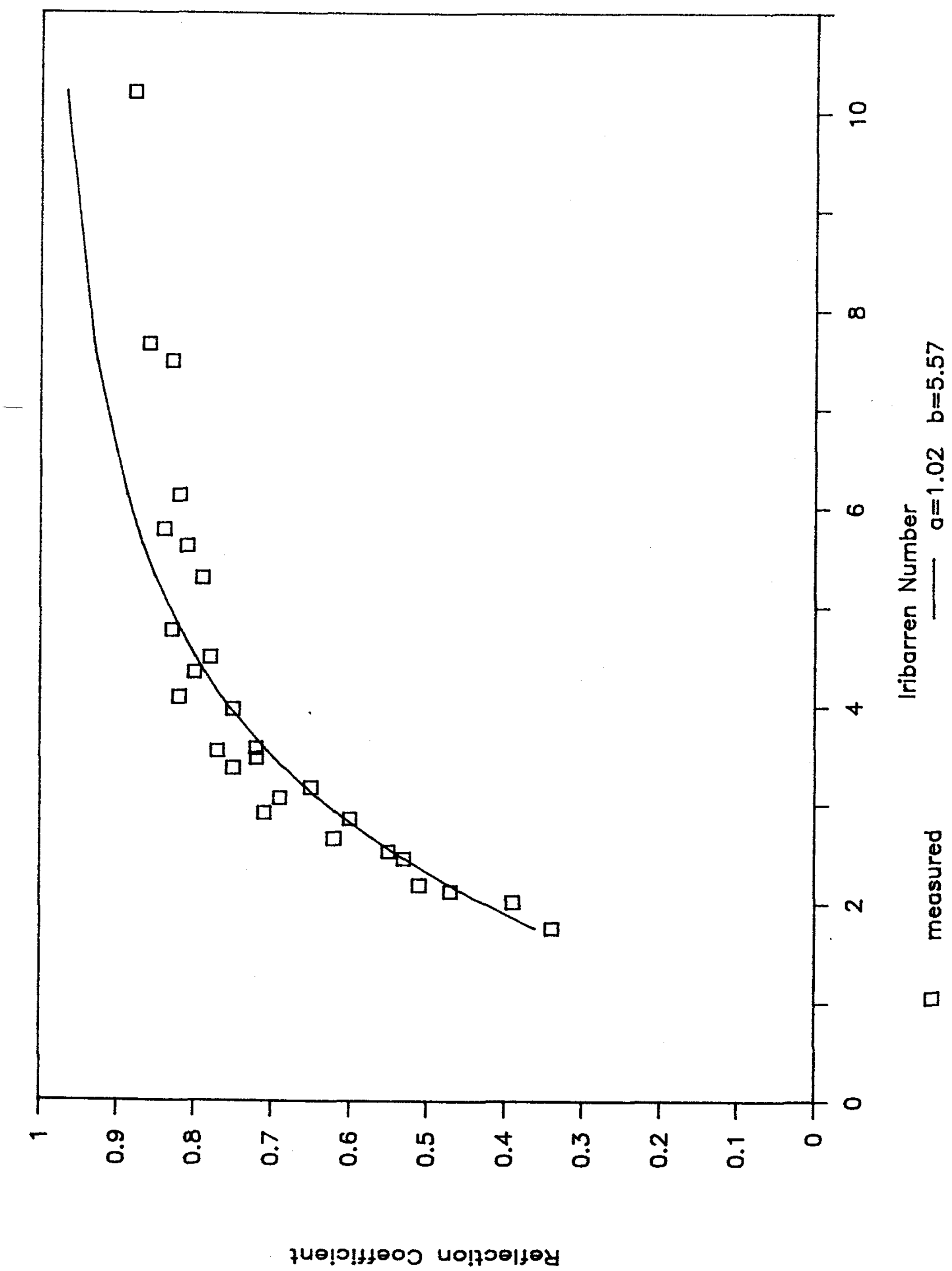

Fig 3 Smooth slopes, sections A/1–3

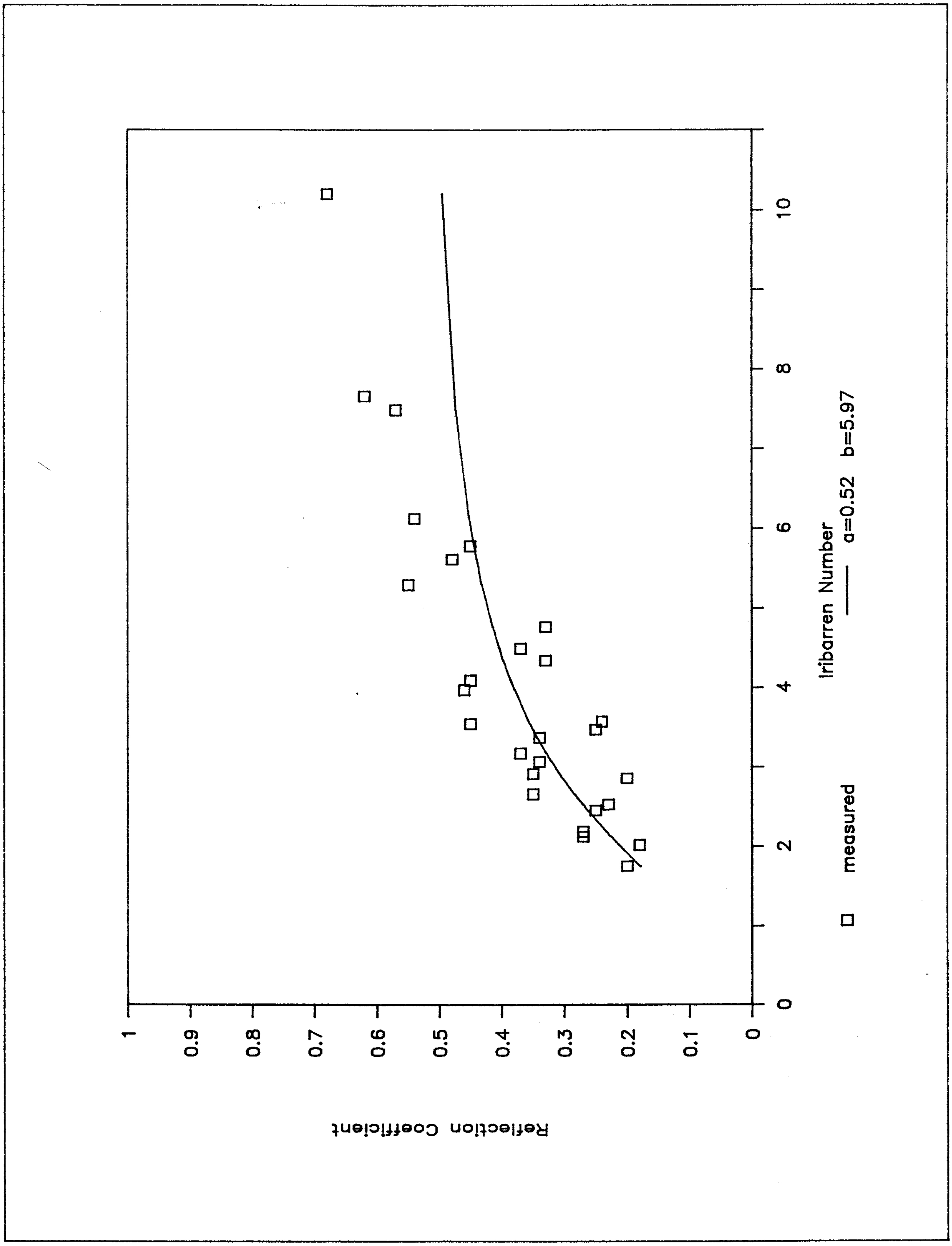

Fig 4 Armoured slopes, sections B/1—3

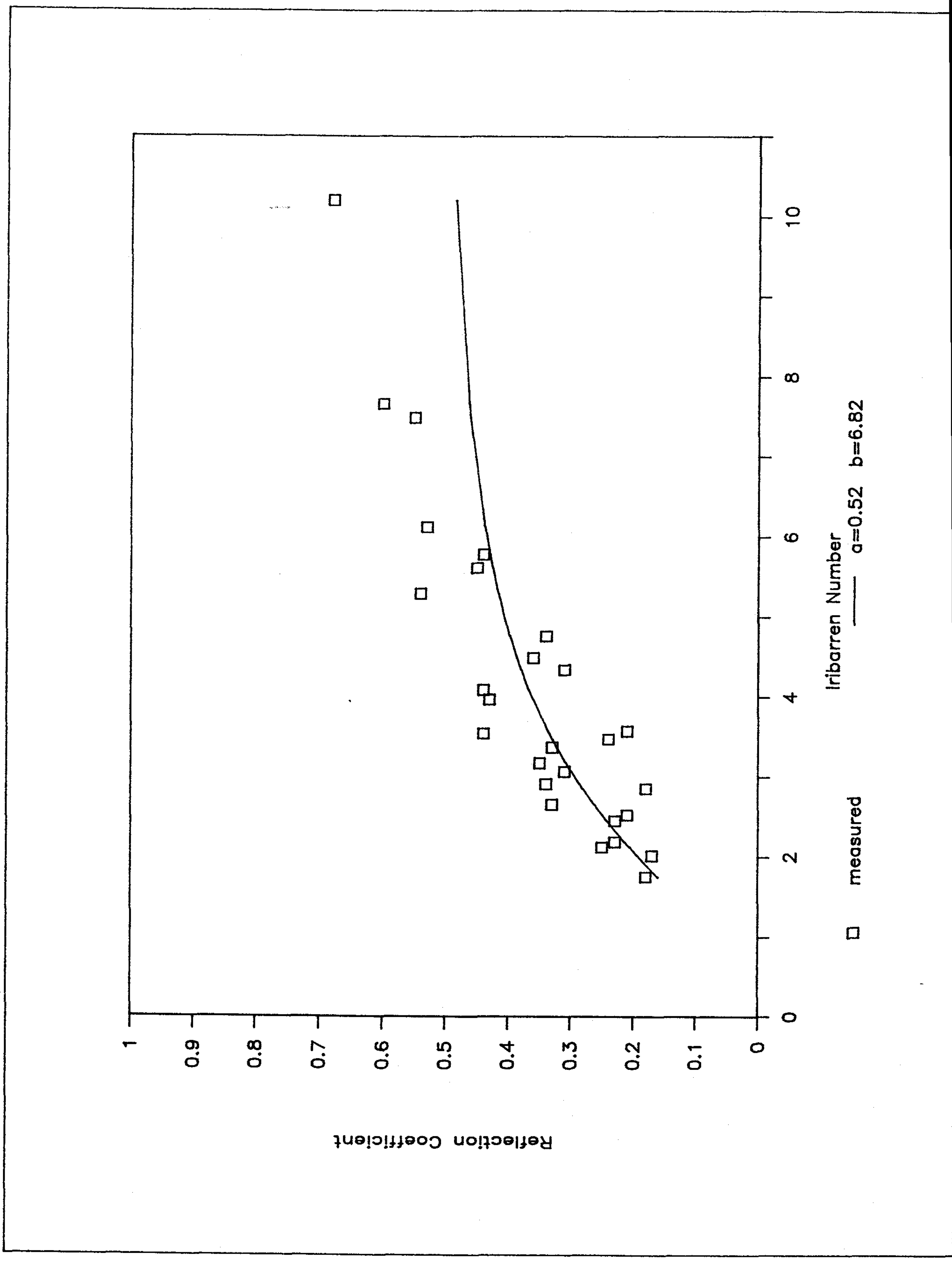

Fig 5 Armoured slopes, sections C/1–3

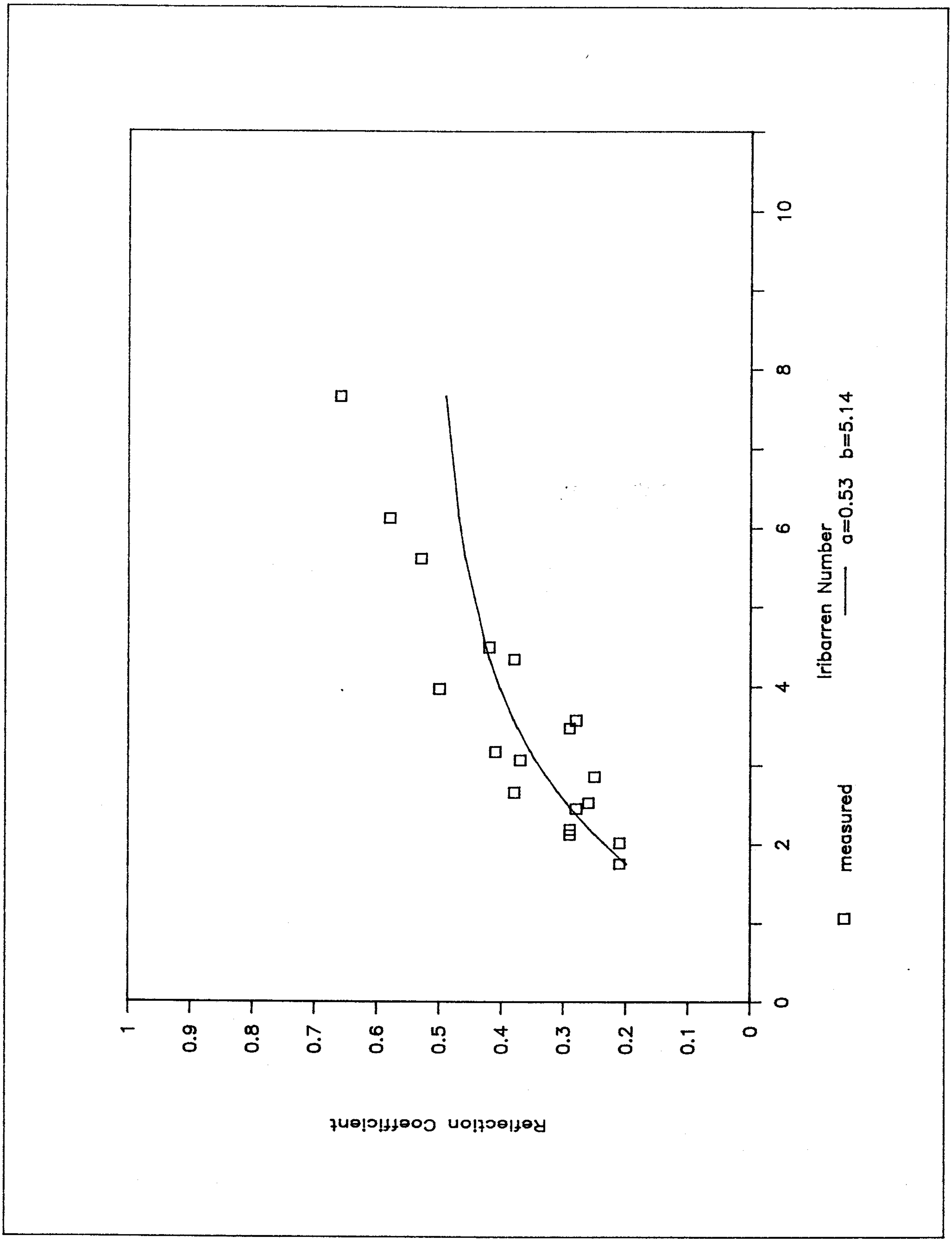

Fig 6 Armoured slopes, sections D/1–3

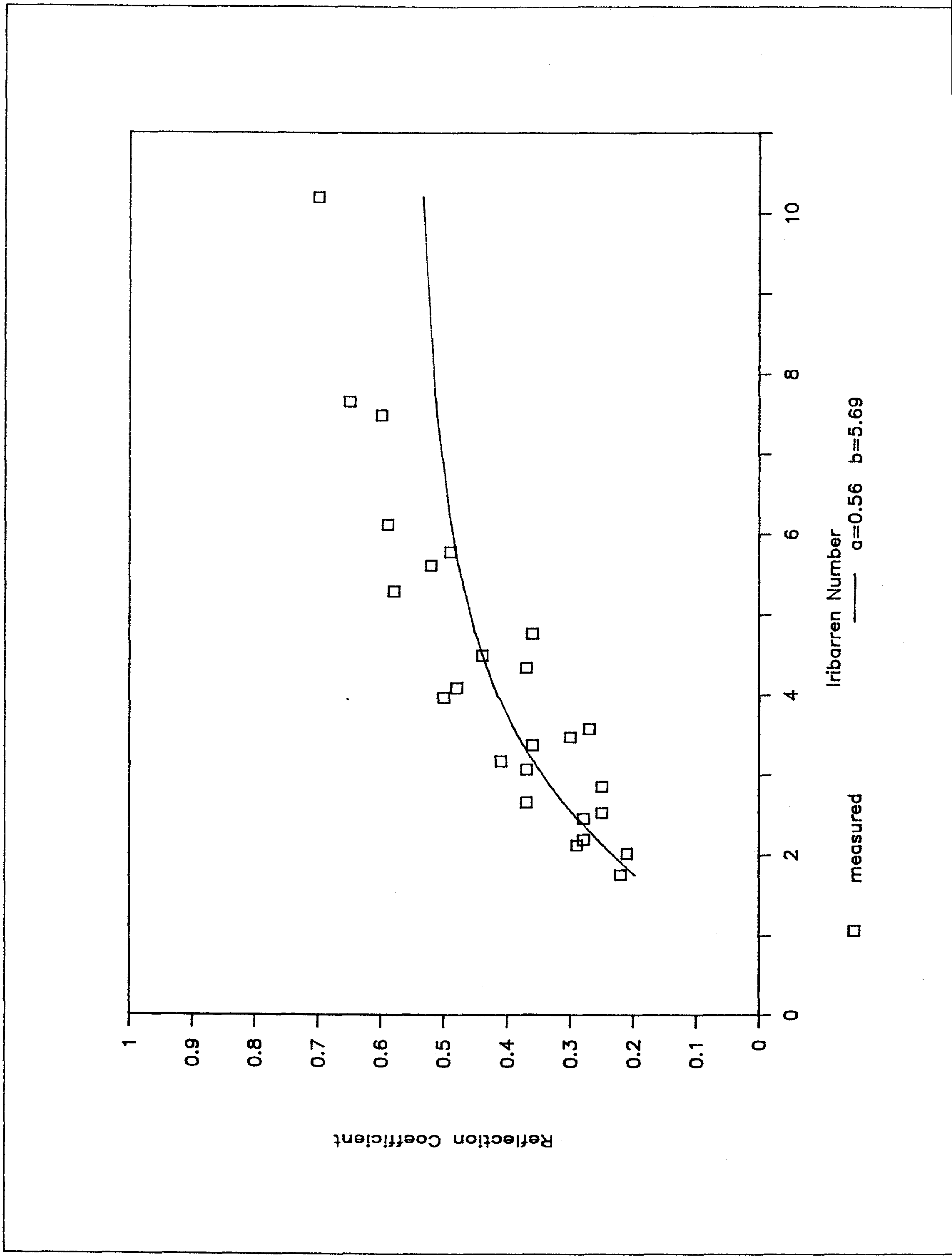

Fig 7 Armoured slopes, sections E/1–3

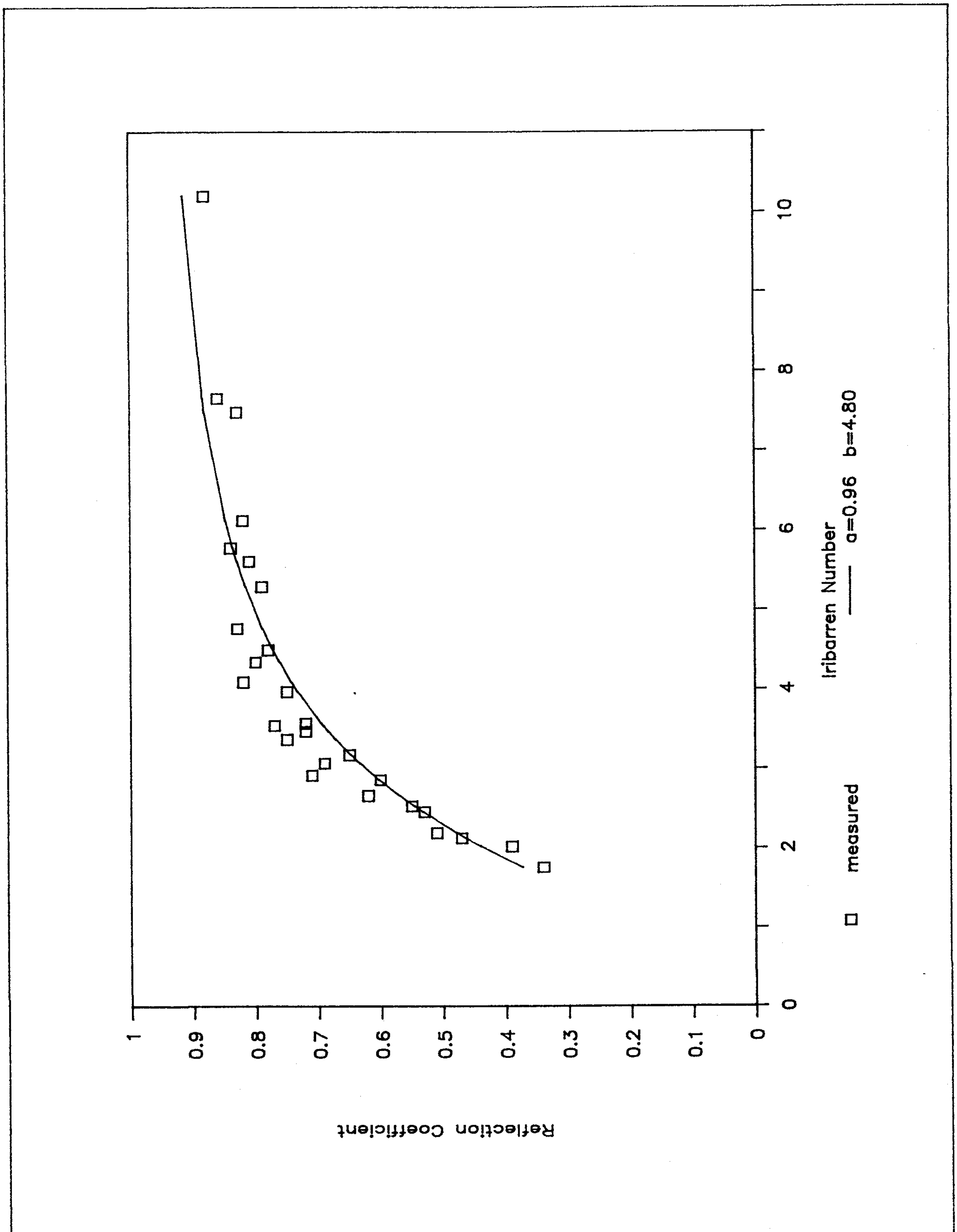

Fig 8 Smooth slopes, sections A/1–3
modified regression

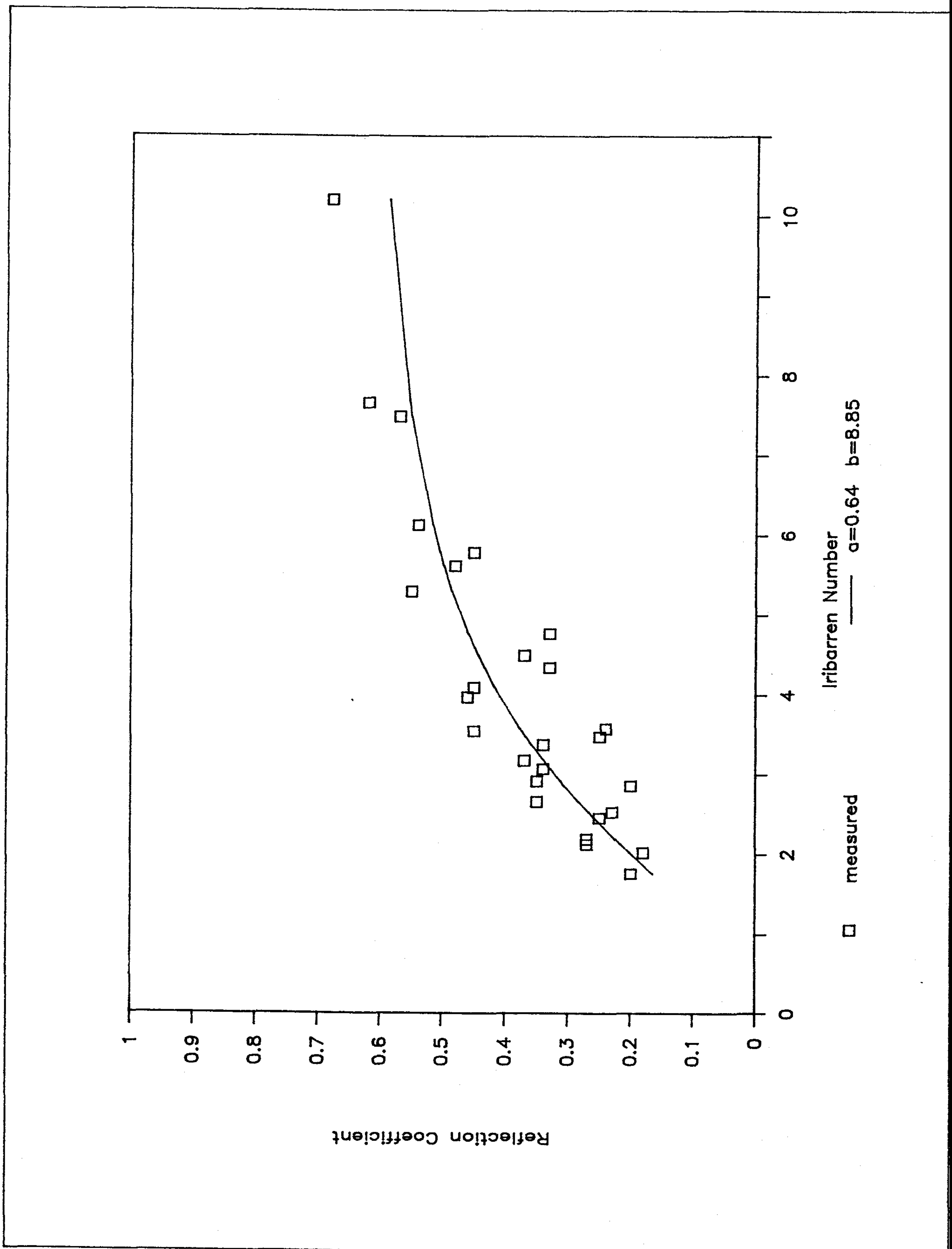

Fig 9 Armoured slopes, sections B/1–3
modified regression

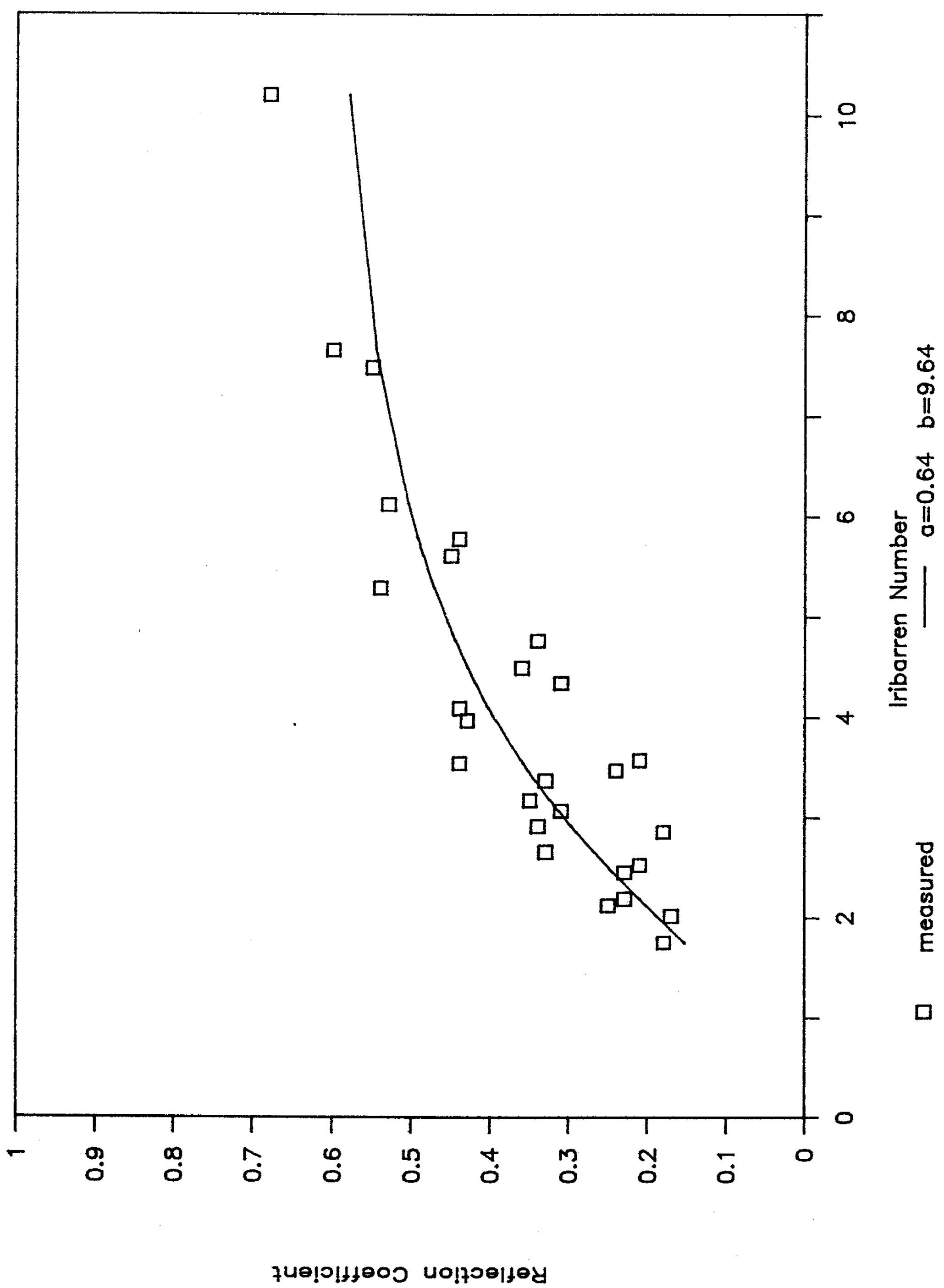

Fig 10 Armoured slopes, sections C/1–3
modified regression

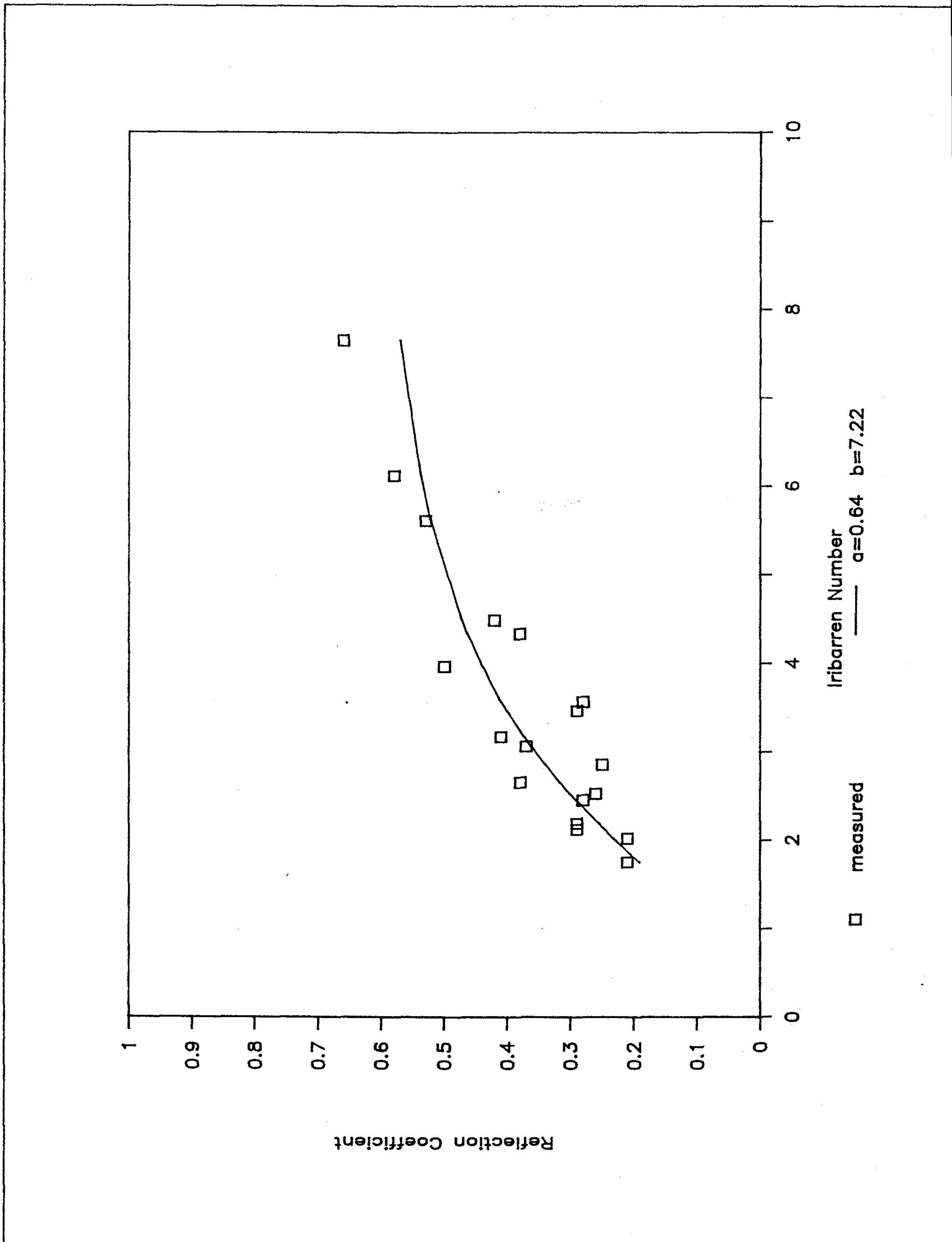

Fig 11 Armoured slopes, sections D/1–3 modified regression

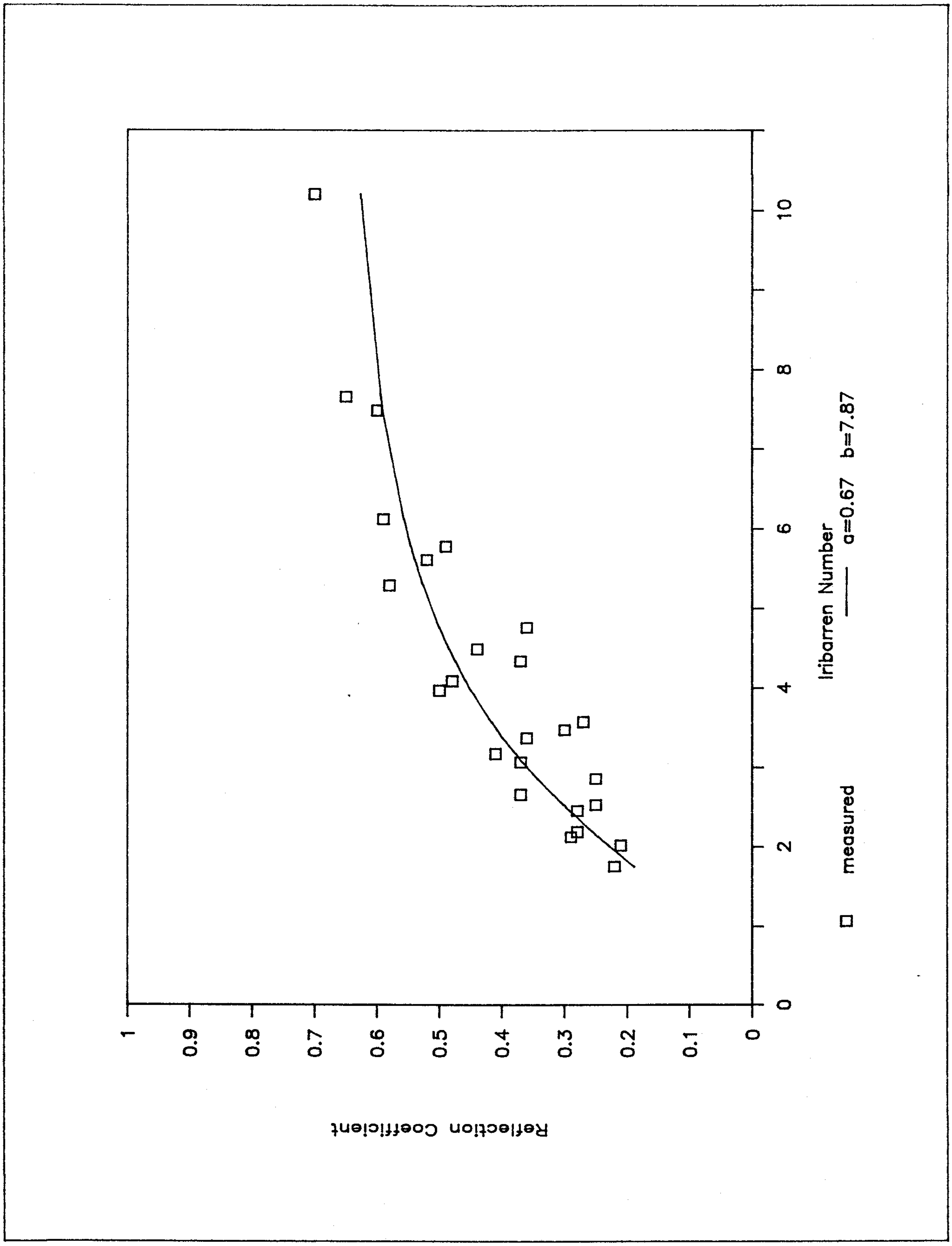

Fig 12 Armoured slopes, sections E/1–3
modified regression

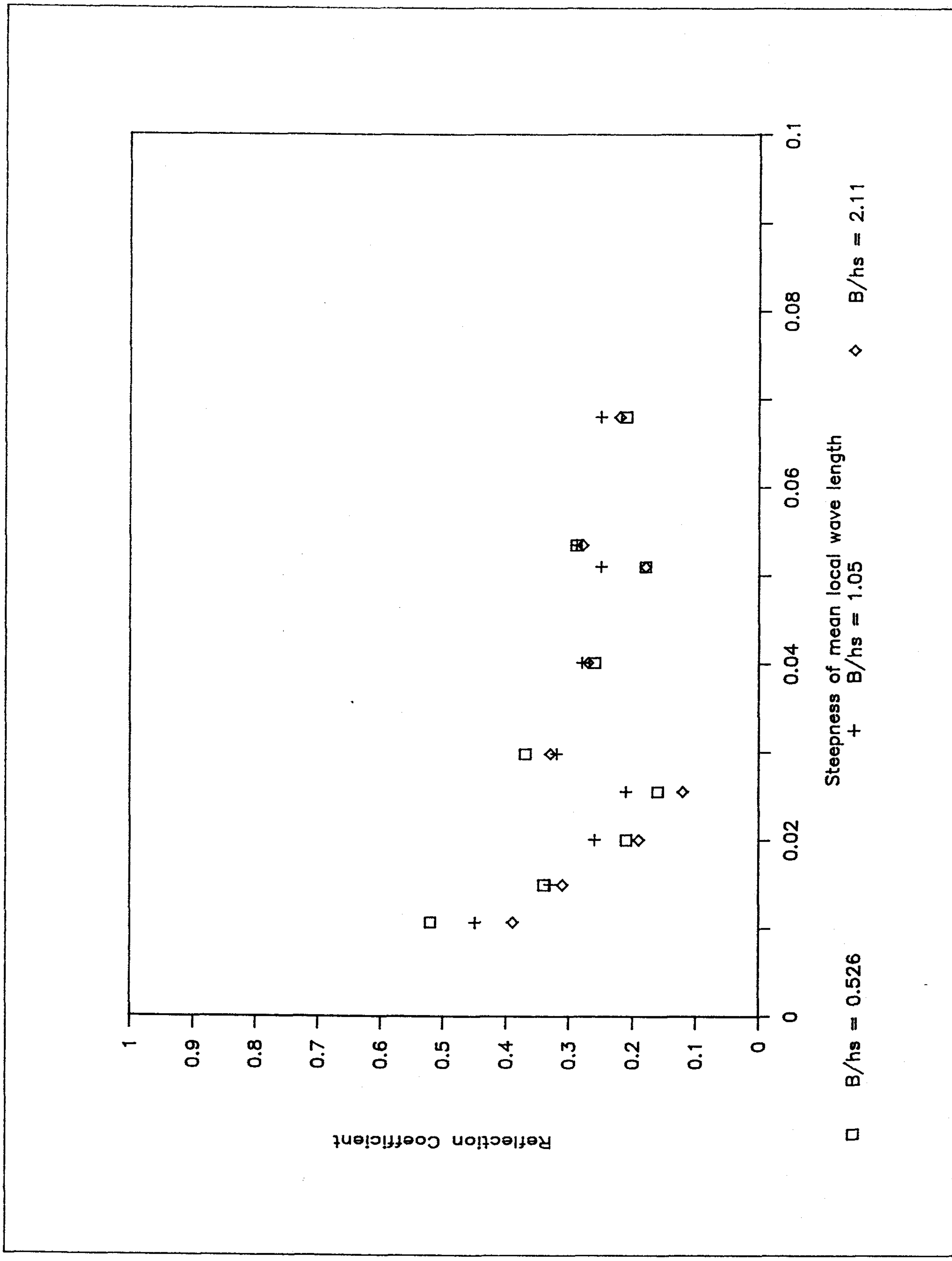

Fig 13 Bermed slopes, sections F/1–3, effect of steepness of mean local wave length

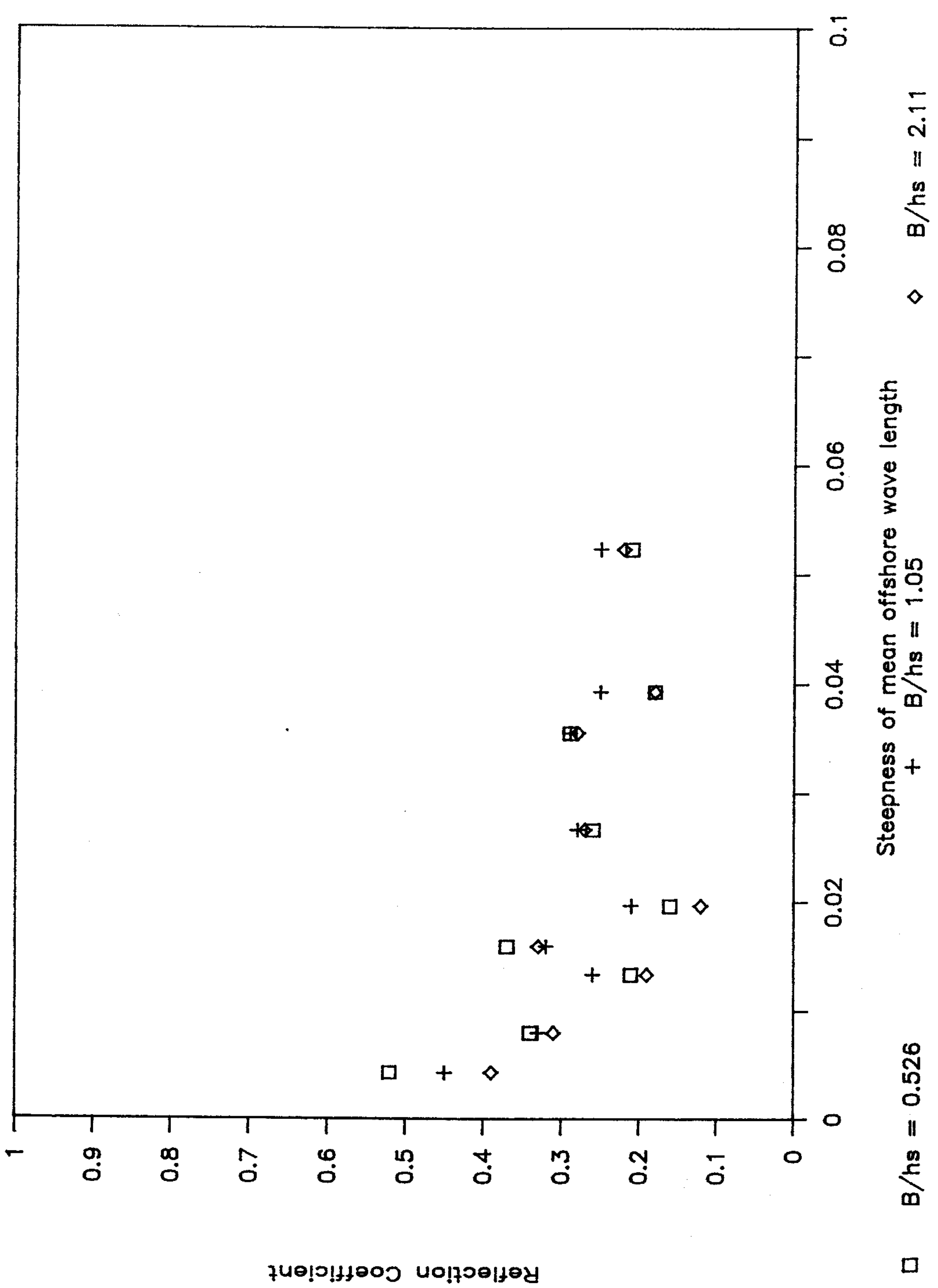

Fig 14 Bermed slopes, sections F/1–3, effect of steepness of mean offshore wave length

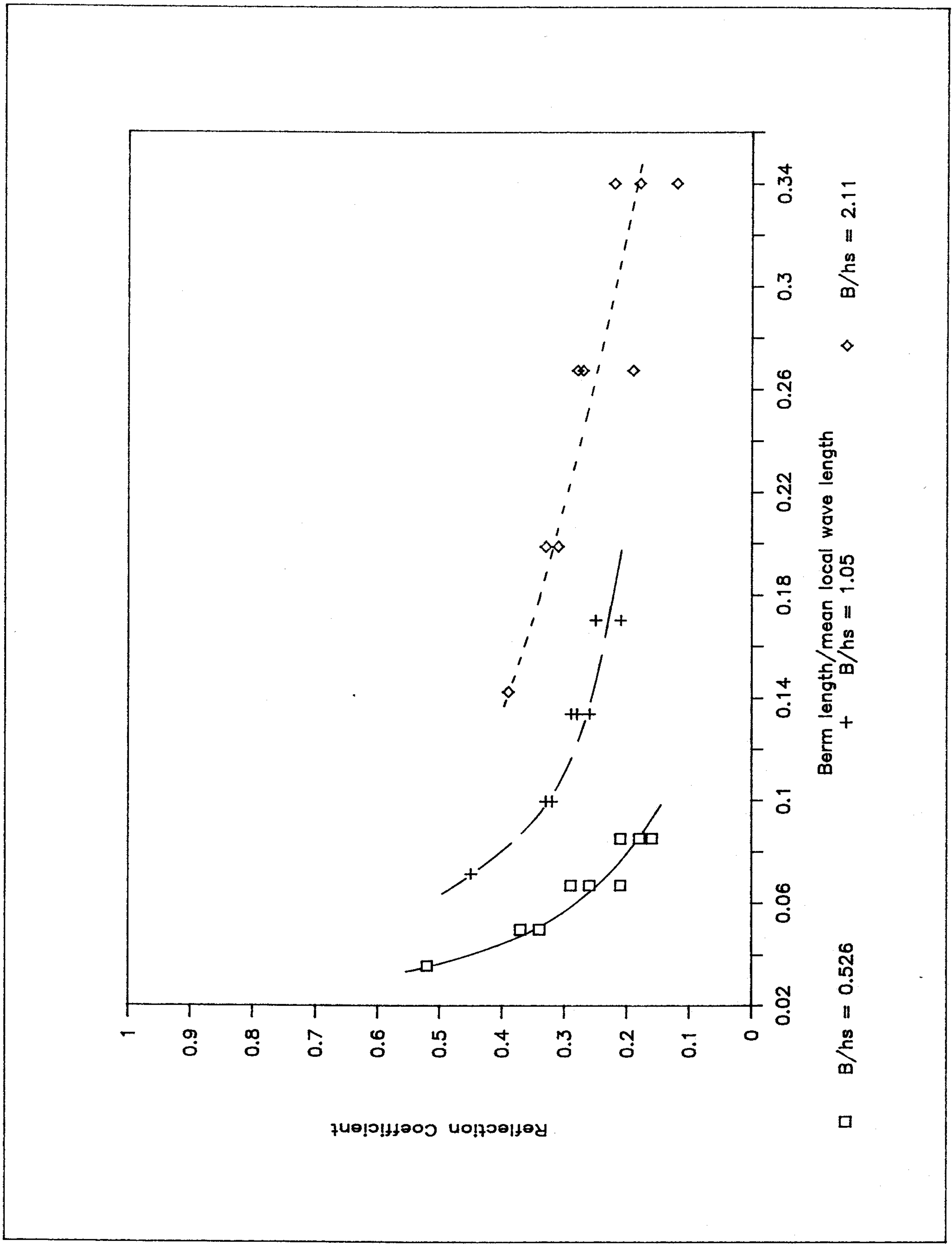

Fig 15 Bermed slopes, sections F/1–3, effect of relative berm length to local wave length

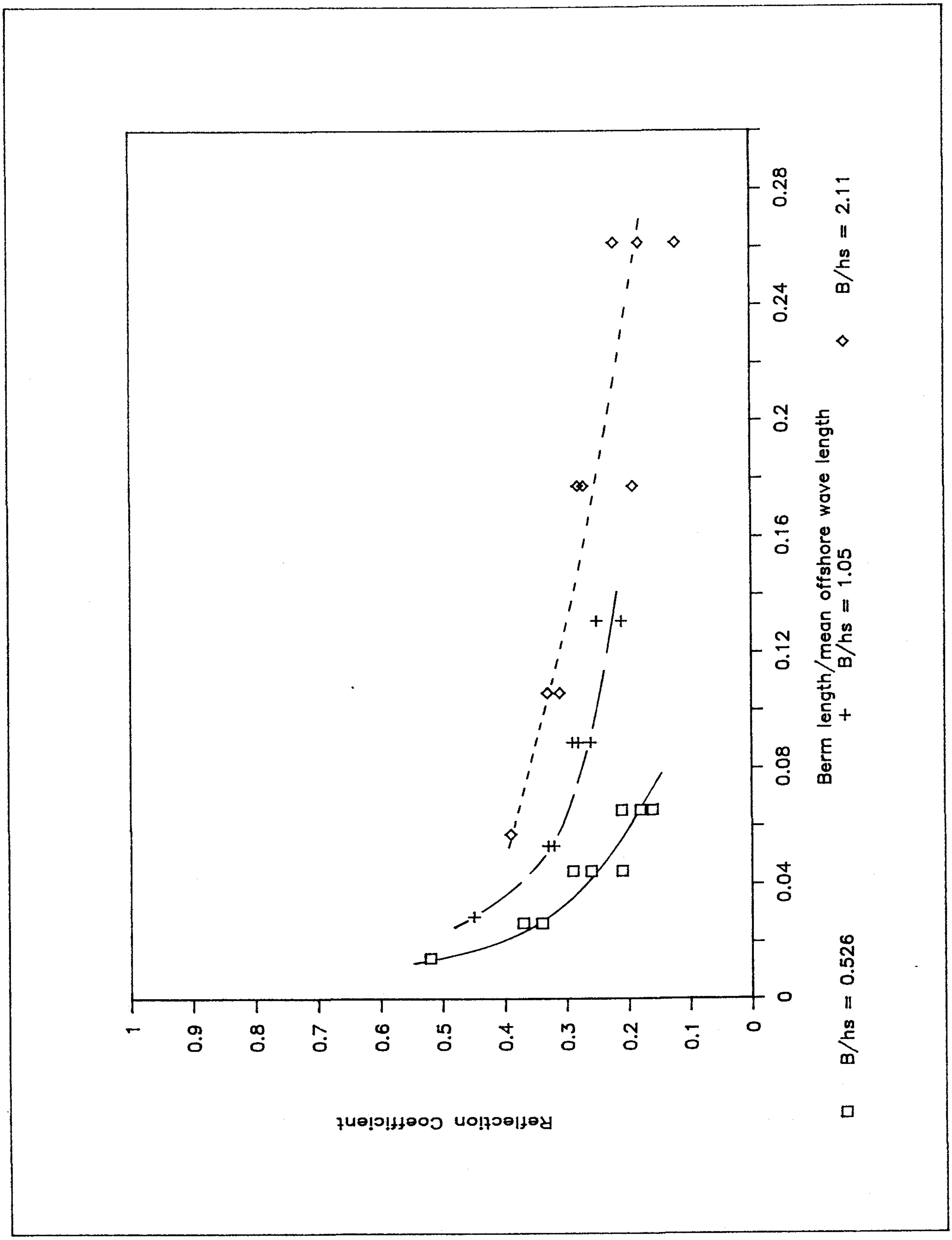

Fig 16 Bermed slopes, sections F/1–3, effect of relative berm length/offshore wave length

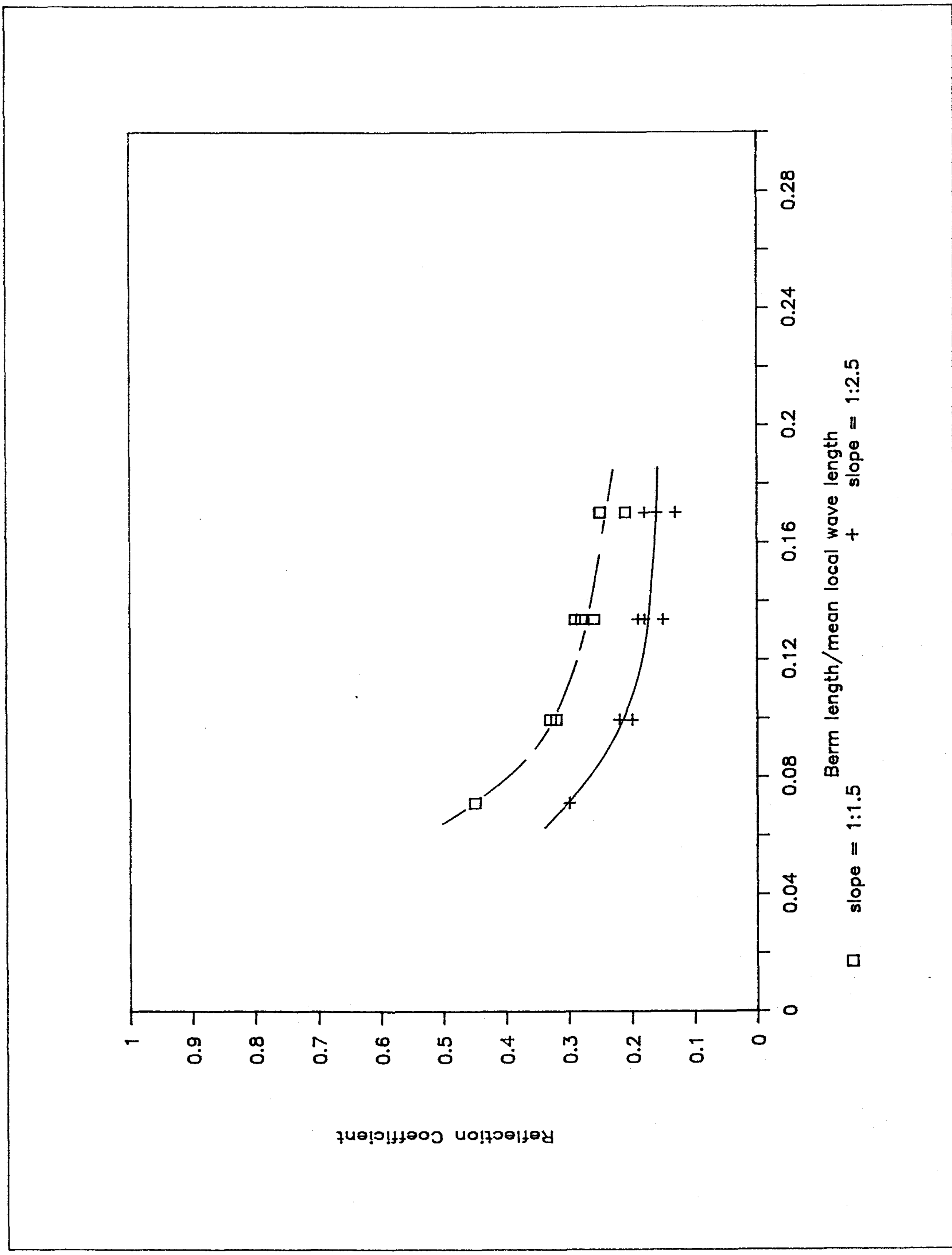

Fig 17 Bermed slopes, sections F/2 and G/1, effect of slope angle

APPENDICES.

APPENDIX 1

Model Test Facility

The model tests for this study were carried out in a wave flume or channel, 50m long by 1.22m wide by 1.1m deep and having a nominal working depth of 0.61m. The wave generator is a wedge type random wave paddle powered by a double acting electro-hydraulic ram controlled by a micro-computer. This system was developed at HR from an older wave spectrum synthesizer. This synthesizer is capable of producing any required deep water ocean wave spectrum that can be described by 16 spectral ordinates. The micro computer wave spectrum synthesizer produces a random wave spectrum by digitally filtering a white noise signal via a shift register. Varying lengths of wave sequence can be produced on this shift register which is used in conjunction with a clock pulse generator (Ref 1). This allows a repeatable pseudo-random sequence of outputs to be generated creating sequences of waves with repeat times varying from a few minutes to several years, depending on the test requirements.

For this study short sequences of waves were used, and a spectral analysis was used where data recording takes place over one complete sequence, thus eliminating any statistical uncertainty in the results. The water level at each twin wire wave probe (Ref 2) is recorded by the mini-computer at every clock pulse of the synthesizer, typically every 0.1-0.2 seconds. A maximum of 4096 data points can be collected from up to 16 probes at one time using this program, although in this study only 3 probes were used. The analogue output of the wave probe, representing a displacement relative to a static water level, is first converted to a digital form, and then to an elevation in prototype metres. This program then uses a Fast Fourier Transform (Ref 3) to transfer the time domain data into the frequency domain and divide the energy between individual frequency components.

This wave flume is divided along its length into two channels by a vertical splitter wall which increases in porosity as it approaches the generator end of the flume. This porous divide wall helps prevent the generation of cross waves as well as dissipating any energy reflected back from the structure being tested. The smaller of these two channels (0.47m wide) is of constant depth and ends in a shingle spending beach of 1:5 gradient. This channel is used to measure the "deep water" wave conditions produced by the generator. The wider channel (0.75m wide) contained the model under test.

Appendix 1 References

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2. Twin wire wave probe modules. Tech Memo 3/1974, Hydraulic Research Station, October 1974.

3. Thompson DM & Gilbert G. "The fast Fourier transform with applications to spectral and cross spectral analysis." Report IT 100, Hydraulics Research Station, Wallingford, December 1972.

APPENDIX 2

Measurement and analysis of wave reflections under random waves

Definition of reflection coefficient

For irregular waves measurement and analysis of wave reflection is best interpreted in terms of sine waves. A certain proportion of the energy of a sine wave incident on a structure will be reflected as a sine wave of the same period but of a lower height. The coefficient of reflection, C_r, may be defined as the reflected wave height divided by the incident wave height.

If irregular waves are regarded as the sum of sine waves of different frequencies, then the reflection coefficients can be calculated for each frequency considered in the incident wave spectrum. The reflection coefficient, function $C_r(f)$ may then be defined for any frequency band width in terms of the reflected and incident energy densities in that band width, $S_r(f)$ and $S_i(f)$ respectively:

$$C_r(f) = (S_r(f)/S_i(f))^{½}$$

In contrast to the above method it is also possible to determine a representative single value of the reflection coefficient, C_r, for a given sea state characterised by H_s and T_m. In this method it is necessary to evaluate the area under the energy density - frequency curves for both reflected and incident energy densities. The computations are performed over a selected band width. Reference to this band width will be made later. By adopting this method it is possible to plot C_r against any appropriate parameter such as the Iribarren number Ir.

Measuring techniques and analysis of data

At Hydraulics Research measurements of wave reflections are usually made using three wave probes. The incident and reflected wave spectra cannot be measured directly but are calculated by a computer programme developed by Gilbert & Thompson (Ref 1), based on the method Kajima (Ref 2).

This method of analysis calculates values of $C_r(f)$ over a wide range of frequencies, but the method is only valid over a restricted band related to the probe spacing. When using two wave probes for measurements, a single probe spacings is used for each test, only allowing a single range of wave frequencies to be covered. However, the use of three wave probes separated by distance, Δx_1, Δx_2 and Δx_3

($\Delta x_3 = \Delta x_1 + \Delta x_2$) permits a much wider range of frequencies to be covered.

The presentation of the results follows directly from the method of analysis in which $C_r(f)$, is calculated either at each of a number of frequency bands, or as a single value of C_r over the entire frequency range selected.

It must be appreciated that the analysis technique assumes that energy is not shifted from one frequency band to any other. However, in some situations, an incident long period wave may well give rise to a number of smaller and much shorter waves. If these short waves reflect, the analysis may calculate a greater coefficient of reflection for the high frequency short waves than is due to the incident waves of the frequency. For example where waves break at or on the test slope, low frequency waves may reflect partially as high frequency waves. In these circumstances some measurements may suggest low values of C_r at the lower frequencies and high values at the high frequencies. This shift of energy from low frequencies will only occur when long waves are of sufficient steepness to break, and not when long waves of relatively low steepness are present.

When the coefficient of reflection, C_r, is calculated as a representative value for a given sea state it is possible to plot C_r against any suitable parameter characterising the wave climate or the structure or both. For sloping structures it may be useful to compare C_r against the Iribarren number. The plots obtained by adopting these dimensionless parameters are assumed to be valid for both model and prototype neglecting the influence of scale effects. Once the data is presented in this form it is possible to develop empirical relationships between structural and wave parameters and the reflection performance.

Appendix 2 References

1. Gilbert G & Thompson D M. "Reflections in random waves, the frequency response function method". HRS Report IT 173, Hydraulics Research, March 1978.

2. Kajima R. "Estimation of an incident wave spectrum under the influence of reflection". Coastal Eng. in Japan, Vol 12, 1969.

APPENDIX 3

Design methods to determine armour size

1. Introduction

In the design of rock armoured rubble structures, the size of the armour required to resist the design wave condition without significant armour displacement constitutes the most important parameter to be determined. Many methods for the prediction of rock size have been proposed, and have been discussed previously in other reports and reviews (Refs 1-4). Those discussed in more detail here may be summarised:

a) the Hudson formulae as used in the Shore Protection Manual (Ref 5);
b) CIRIA 61 based on studies by Thompson & Shuttler (Refs 6,7);
c) van der Meer's equations (Refs,4,9,10).

2. Hudson's formula

On the basis of an extensive series of tests with regular waves and permeable mounds, Hudson developed a simple expression for the minimum armour weight required for a given wave height. This formula may be expressed in terms of the median armour mass, M_{50}, and rock density, ρ_r:

$$M_{50} = \frac{\rho_r \, H^3}{K_D \, \Delta^3 \cot\alpha} \qquad (1)$$

where α is the structure slope angle
Δ is the relative buoyant density defined in terms of the rock density, ρ_r, and (sea) water density, ρ_w, $\Delta = (\rho_r/\rho_w)-1$.

and K_D is a stability coefficient taking account of the other variables. For wide graded rock armour, or rip-rap, values of a similar coefficient K_{RR} are substituted for K_D. Values of K_D and K_{RR} were derived from the results of hydraulic model tests with permeable cross-sections subject to no overtopping. A range of wave heights and periods were studied. In each case the value of K_D derived corresponded to the wave condition giving the worst stability condition. Some re-shaping or re-arrangement of the armour was expected, and values of K_D suggested for design correspond to a "no damage" condition where up to 5% of the armour units may be displaced.

In the 1973 edition of the Shore Protection Manual the values given for K_D for rough, angular stone in 2 layers on a breakwater trunk were:

a) $K_D = 3.5$ for breaking (plunging) waves;
b) $K_D = 4.0$ for non-breaking (surging) waves.

No tests with random waves had been conducted, but it was suggested that "the design wave ... is usually the significant wave". Designers therefore generally used equation 1 with $H_s = H$.

By 1984 the advice given was more cautious. The SPM now recommends "the design wave height ... should usually be the average of the highest 10 percent of all waves", $H_{1/10} = H$. Furthermore the values of K_D were revised. For the case considered above the value of K_D for breaking waves was revised downward from 3.5 to 2.0. The effect of these two changes is equivalent to an increase in the unit stone mass required by a factor of about 3.5!

The main advantages of the Hudson equation are its simplicity, and the wide range of armour units and configurations for which values of K_D have been derived. The Hudson equation also has many limitations, most of which are described in the SPM. Briefly they include:-

a) potential scale effects due to the small scales at which most of the tests were conducted;
b) the use of regular waves only;
c) no account taken in the equation of wave period, or storm duration;
d) no description of the damage level;
e) the use of non-overtopped and permeable core structures only.

Some of these limitations have been addressed by later studies, and are discussed further below. Before turning to other methods, however, it is convenient to consider another way of looking at equation 1.

It is noted in the SPM, and elsewhere, that the use of $K_D \cot\alpha$ does not always best describe the effect of the slope angle. In some circumstances it may not always be easy to assign a single value to α. It may therefore be convenient to define a single stability number to substitute for $K_D \cot\alpha$. Further, it may, often be more helpful to work in terms of a linear armour unit size, such as a typical or nominal diameter. The Hudson equation can be re-arranged to:

$$\frac{H_s}{\Delta D_{n50}} = N_s = K_D(\cot \alpha)^{1/3} \qquad (2)$$

where the nominal median stone diameter, D_{n50}, is defined in terms of M_{50} and ρ_r:

$$D_{n50} = (M_{50}/\rho_r)^{1/3} \qquad (3)$$

Values of N_s or $H_s/\Delta D_{n50}$ have been determined from model tests for a wide range of conditions, and some are discussed further below.

3. CIRIA 61

In the early 1970 a series of random wave model studies were conducted at HR by Thompson & Shuttler (Ref 6). Wide graded armour, riprap, placed on an impermeable foundation, was tested for various durations under random waves. Armour displacement was measured by profiling over the structure face. The results of the studies were used to derive a design method published by CIRIA (Ref 7).

The use of CIRIA 61 to predict riprap size for a design duration of wave action is relatively simple, being based almost entirely upon one table. This table gives values of the parameter Hs/D_{R50} (where D_{R50} is a median rock diameter defined as 1.22 D_{n50}) for various acceptable damage criteria and slopes.

The damage criteria employed may be summarised as:

Criterion A - No erosion of riprap for a given significant wave height.

Criterion B - Intermediate damage, an absolute measure, equivalent to the erosion of one D_{R50} sized stone per D_{R50} width of slope.

Criterion C - Intermediate damage, a relative measure equivalent to the erosion of 15% of the mean number of stones that would be eroded at failure.

Criterion D - Failure, taken as occurring when the filter layer is first exposed.

The test results allow the effect on armour displacement to be established for changes in wave height, H_s; structure slope angle, α; storm duration, T_R, or number of waves N.

The limitations of the CIRIA 61 method are that it is only applicable to structures with relatively impermeable cores, and it does not explicitly take account of wave period, nor whether the incident wave are breaking or non-breaking.

Due to different structure core permeabilities for which they were evolved, the methods of CIRIA 61 and Hudson cannot strictly be compared. If they were however, it would be Criterion C that would most closely correspond to Hudson's zero-damage. Indeed it might be expected that most structures designed using CIRIA 61 would be designed to one of the intermediate damage levels (B or C). It is worth noting that the use of criterion A as the damage standard can typically result in rock weights of up to 8 times greater than those demanded by the intermediate standards.

4. Van der Meer's equations

Van der Meer and co-workers (Refs 4,9 and 10) have conducted a very wide series of model tests including and extending Thompson & Shuttler's results. The new tests included structures with a wide range of core/underlayer permeabilities, and a wider range of wave conditions. Two formulae are derived for plunging and surging wave conditions respectively. These formulae may be written for plunging waves:

$$\frac{H_s}{\Delta D_{n50}} = 6.2\ P^{0.18}\left(\frac{S}{\sqrt{N}}\right)^{0.2} Ir^{-0.5} \tag{4}$$

and for surging waves:

$$\frac{H_s}{\Delta D_{n50}} = 1.0\ P^{-0.13}\left(\frac{S}{\sqrt{N}}\right)^{0.2} (\cot\alpha)^{0.5}\ Ir^{P} \tag{5}$$

The transition from plunging to surging waves can be calculated using a critical value of Ir:

$$Ir_c = (6.2\ P^{0.31}\ (\tan\alpha)^{0.5})^{1/(P+0.5)} \tag{6}$$

The parameters not previously defined are:

P	a notional permeability factor
S	a design damage number = A_e/D_{n50}
A_e	erosion area from profile
N	the number of waves
α	the structure slope
Ir	the Iribarren number = $\tan\alpha/s_m^{½}$
s_m	wave steepness for mean period = $2\pi\ H_s/g\ T_m^2$
T_m	mean wave period.

The recommended values of the design damage number, S, are given below, as a number of D_{n50} sized stones extracted from a D_{n50} wide strip of the structure, for each of the damage criteria. The three criteria employed are initial damage, intermediate damage, and failure. Failure is assumed when the filter layer is first exposed. It is worth noting that it is CIRIA criterion C that is equivalent to van der Meer's initial damage and Criterion D that corresponds to failure.

Table 1 **Values of design damage number, S.**

Slope	Damage Criterion		
	Initial	Intermediate	Failure
1:1.5	2	-	8
1:2	2	5	8
1:3	2	8	12
1:4	3	8	17
1:6	3	8	17

The damage criterion chosen at the design stage will determine the maintenance requirements for the structure over its lifetime. In general it may be expected that the majority of structures will be designed to Hudson's zero damage/CIRIA Criterion C/ van der Meer's initial damage. There may, however, be some structures whose purpose and/or ease of maintenance dictates a more, or indeed less, severe design damage criterion. In all such cases the choice of damage criterion must ultimately lie with the designer.

In van der Meer's tests a range of core/underlayer configurations were used, each with an armour layer thickness, $t_a = 2.2D_{n50}$. To each of these a value of the permeability factor, P, was assigned. Values of P given by van der Meer vary from 0.1 for armour on an underlayer over an impermeable embankment, to 0.6 for a homogeneous mound of armour size material. Intermediate values of 0.4 and 0.5 are also described. It is not yet possible to determine values for P analytically, so one would expect a designer to explore the sensitivity of particular calculations to the assumptions made. It should be noted that values lower than P = 0.1 may be appropriate for certain configurations, this is discussed further later.

5. HR/QMC tests

Recent studies by Hydraulics Research and Queen Mary College (Refs 2,3) have sought to quantify the effect of particle shape on rock armour stability. These tests have also provided comparison data for van der Meer's equations. Of particular interest from these recent studies is a comparison of the effect of armour layer thickness, t_a. Van der Meer's tests were run with $t_a = 2.2D_{n50}$. Bradbury et al laid armour in two layers, but to a measured mean layer thickness of $t_a = 1.4D_{n50}$. [It should be noted in passing that a layer thickness calculated from profile measurements will include most of the 'hollows' in the armour surface as well as the 'peaks', hence giving a rather low layer thickness.] In considering the test results Latham et al (Ref 3) suggest that an appropriate value for P for the configuration tested might be 0.05, markedly lower than van der Meer's lower limit.

From the analysis of the influence of particle shape, Latham et al tentatively suggest further modifications to van der Meer's equations by replacing the coefficients 6.2 and 1.0 in equations 4 and 5 by C_{pl} and C_{su} respectively, where:

$$C_{pl} = 5.6 + 60\ P_R \qquad (7)$$

and $$C_{su} = 0.8 + 20\ P_R \qquad (8)$$

In each instance P_R is an asperity roughness factor derived from a Fourier shape analysis of a sample of the armour units (Ref 3).

Values for P_R have been derived from shape analysis at Queen Mary College of the rock used for the model tests:

a) equant $P_R = 0.0117$
b) tabular $P_R = 0.0165$
c) fresh $P_R = 0.0138$
d) semi-round $P_R = 0.0087$
e) very round $P_R = 0.0046$

These test results and analysis are very recent, and have not yet been validated by further testing, or by independent data.

6. Discussion

The main advantages of the Hudson equation are its extreme simplicity, and the wide availability of values of K_D. In a time of programmable calculators, personal computers, and spreadsheet programs, a very simple formula has however no significant advantage.

Conversely the limitations of the Hudson equation are now seen to be significant. The use of it for random waves, storms of different durations, and structures with impermeable, or less permeable, core/underlayers is particularly ill-supported.

Van der Meer's studies have effectively included the data from Thompson & Shuttler's work, and have expanded the data set by further tests. The new equations allow the designer to explore the influence of important parameters such as mound permeability; storm duration; and acceptable damage levels. Coefficients in the equation are empirically derived, giving a central fit to the data. The reliability of these formulae is discussed by van der Meer & Pilarczyk (Ref 10) who showed that the coefficient 6.2 in equation 4 has a standard deviation of 0.4, equivalent to a coefficient of variation of 6.5%. The coefficient 1.0 in equation 5 has a standard deviation of 0.08 (8%). These values are significantly lower than that for the Hudson formula at 18%.

In use one would expected a designer to apply appropriate partial safety factors to the parameters calculated to account for the essential variability of rock armour response, and the uncertainties in the application of the formulae to the particular design case.

It should be noted that these equations have not yet been fully validated by independent laboratory tests, although they have included tests in different flumes at small and large scale. Complimentary tests in the UK suggest that permeability factors lower than van der Meer's lower limit of P = 0.1 may be justified for some structure configurations. Further modifications have been tentatively suggested to account of different particle shapes. The data set was very restricted, and the results should be used with caution.

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