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Wallingford

RUN-UP ON SHINGLE BEACHES

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This report describes work carried out by Hydraulics Research as part of its strategic research programme. The work was initiated by the Water Directorate of the Department of the Environment (Contract PECD 7/7/055). Responsibility for this work passed to the Ministry of Agriculture, Fisheries and Food at 1 April 1985 and finally reporting was carried out under MAFF Contract CSA 1034.

At the time of reporting this project, the MAFF nominated officer was Mr A Allison and the HR project officer was Dr S W Huntington.

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ABSTRACT

Physical model studies have been conducted investigating run-up on shingle beaches. This report summarises experiments carried out between 1982 and 1984. These experiments were carried out using regular waves, to facilitate interpretation of the results obtained. Further tests using random waves will be described in a future report.

CONTENTS

NOMENCLATURE

SYMBOL		DEFINITION
H	Wave height	
R	Run-up	The vertical distance between maximum level reached by the wave and still water level
T	Wave period	
L	Wave length	
H_o	Deep water wave heights	
L_o	Deep water wave length	
α	Beach slope angle	The angle that the surface of the beach makes with the horizontal

1 INTRODUCTION

A considerable amount of work, both theoretical and experimental, has been carried out in the past to investigate wave run-up on smooth impermeable beaches and on sandy beaches but very little, if any, on shingle beaches.

The ultimate aim of this research project is to produce criteria to assist engineers in the design of shingle beaches for shore protection. It was considered that the existing run-up wave height relationships derived from impermeable or sandy beaches would produce an over-design when applied to shingle beaches and that worthwhile economies were possible by the application of a more appropriate relationship.

It was originally proposed to carry out measurements in the field to investigate the relationships between the various parameters affecting run-up but in view of the time and cost involved in obtaining comprehensive field observations it was decided to carry out laboratory experiments first and then to follow up with limited field tests to check the validity of the laboratory results.

2 THE EXPERIMENTAL FACILITY

The study was carried out in a wave flume measuring 23.2m long by 0.75m wide and 1m deep (Fig 1). At one end of the flume a deep sump accomodates the wave paddle which is pivoted at the floor of the sump. The speed and linkages to a motor are infinitely variable; permitting the generation of regular waves from 0.5 seconds to 10 seconds period and up to 0.3m in height. At the opposite end of the flume a test section of beach was constructed (Fig 1, Plate 2). Details of the construction can be found in chapter 4.

The operational still water depth in the flume was 0.56m. This water depth remained unaltered for all stages of the study described in this report.

The instrumentation used to measure the wave heights and run-up are described separately in the following chapter.

3 INSTRUMENTATION

Wave heights outside the breaker zone were measured using a twin wire wave probe. This was mounted onto a carriage that ran along a length of channel section. The carriage was electrically winched at a constant speed of 0.064m/s along part of the longitudinal axis of the flume. This method prevented the measurement of reflection effects and provided an easily repeatable sampling sequence (Fig 1).

Waves running up the beach were measured using a run-up probe. This is essentially a long twin wire wave probe and for this particular application was 2m long. The 2 conductors are laid parallel to each other and are either bonded to or just above a PVC insulation strip. The upper face of this assembly is installed flush with the surface of the beach (Plate 1). The conductors can be made of silver steel wire or alternatively from foil. In this case, the foil type was preferred, permitting measurements to be made at the beach face. A constant voltage is applied to the electrodes. The resistance of the electrical path varies depending upon the length immersed by the waves. The resulting variation in current is measured and displayed in a convenient form. During the study care was taken to prevent stones and water globules influencing the run-up readings.

Signals from both channels were recorded on an ultra-violet recorder giving a continuous trace of water level variation with time. Calibration of the two instruments was carried out by recording the signals from the instruments on the ultra-violet trace, for a range of known still water levels. These water-levels were measured using a point gauge calibrated to 0.1mm.

4 THE MODEL TESTS

4.1 The Model Beach

In each of the main conditions the core of the model beach was made up of coarse river gravel. Pea shingle was used for the surface layer of the model beach. This was approximately 0.2m deep and was separated from the underlayer by a perforded metal sheet (Fig 1). During the unconstrained tests using large waves, movement of this surface layer of pea shingle produced complex beach forms. This gave rise to problems in both interpreting and predicting the final beach slope. To overcome this problem the beach was constrained using gabions. These gabions encased the upper 0.1m of pea shingle (Fig 2, Plate 2). The results of a sieve analysis carried out on the beach material are shown in Figure 3 and indicate a mean grain size of 8.2mm.

4.2 Small wave tests (unconstrained beach)

Measurements were made of wave height and run-up for a wide range of wave periods and wave heights using regular waves on beach slopes of 1:6, 1:8, and 1:10. The wave heights and run-up in the flume experiments were observed and the mean of three observations was used.

The deep-water wave length was calculated from:

$$L_o = gT^2/2$$

and the deep-water wave height was found from tables in Ref 2.

For smooth impermeable slopes the run-up of periodic (regular) waves is given by:

$$R = (H_o L_o)^{0.5} \tan \alpha$$

where R is the vertical height above still water in the absence of waves and H_o and L_o are respectively the deep-water wave height and deep-water wave length. This equation was proposed by Hunt (ref 3) and appears to be based on the results of Saville (Ref 4). There is some ambiguity of definition in this equation. It is known (Ref 5) that waves breaking on a slope cause a local rise in mean sea level inside the breaker zone. This means that the run-up from mean water level measured in the presence of waves will be less than the run-up from the still water level measured in the absence of waves. It has been assumed that the above equation refers to run-up from still water level in the absence of waves and throughout this report this assumption holds.

In order to check the validity of this equation for smooth, impermeable beaches when applied to shingle beaches, run-up against $H_o L_o$ was plotted on log.log paper for the three beach slopes tested in the flume (Figs 4 - 6).

Although scatter is evident it appears reasonable to draw a straight line through the plotted points. To this end the equation of the best-fit straight line was computed using a least squares linear regression analysis. The equations resulting from this computation were:

Beach slope	Equation of best-fit line	Correlation coefficient
1/6	$R = 0.0390(H_o\ L_o)^{0.56}$	0.951
1/8	$R = 0.0279(H_o\ L_o)^{0.55}$	0.916
1/10.45	$R = 0.0211(H_o\ L_o)^{0.54}$	0.868

A correlation coefficient of 1.0 indicates a perfect alignment of the points. The high values of the coefficients shown above indicate that it is justifiable to assume a power law relationship between run-up (R) and H_oL_o for the flume experiments.

If the tangent of the beach slope is taken out of the constant multiplier in each of the three equations, the following relationships result.

Beach slope	Equation
1/6	$R = 0.234 \tan \alpha\ (H_o\ L_o)^{0.56}$
1/8	$R = 0.223 \tan \alpha\ (H_o\ L_o)^{0.55}$
1/10.45	$R = 0.220 \tan \alpha\ (H_o\ L_o)^{0.54}$

The resulting constant multipliers are sufficiently similar to justify the use of one equation for all three beach slopes. In order to be dimensionally correct the value of H_oL_o should be raised to the power 0.5. The actual powers obtained were fairly close to this value, varying from 0.54 to 0.56, it was therefore considered reasonable to substitute a power of 0.5. A further graph is presented in Figure 7 which shows R plotted against Tan $(H_oL_o)^{0.5}$ for all the results obtained. The best fit straight line through these results, using a least squares linear regression analysis, gave a slope of the line 0.393 and an intercept of -0.255 resulting in the relationship:-

$$R = 0.393 \tan \alpha (H_o L_o)^{0.5} - 0.255$$

The correlation coefficient was again good at 0.945 giving a high degree of confidence in the relationship.

It is evident that there is a discrepancy here, for as H_o tends to zero R should also tend to zero. The graph should therefore pass through the origin. The equation of the least squares linear regression line passing through the origin was:

$$R = 0.349 \tan \alpha (H_o L_o)^{0.5}$$

The first stage of laboratory experiments designed to explore the applicability to shingle beaches of Hunt's run-up equation for smooth, impermeable beaches, viz,

$$R = \tan \alpha (H_o L_o)^{0.5}$$

indicated the run-up on shingle beaches to be about one-third of that on smooth, impermeable beaches: the following relationship was obtained:

$$R = 0.349 \tan \alpha (H_o L_o)^{0.5}$$

4.3 Large wave tests (unconstrained beach)

During the tests using small waves no movement of the test material occurred. Later tests were undertaken with waves of greater amplitude. The first of these tests allowed the beach to move and thus establish a stable profile.

The beach was again composed of pea shingle (size grading Fig 3) and was initially moulded to a flat slope of 1:6.

Waves were generated and, when the beach reached a stable profile with no net movement of material, the wave height, period and beach profile were measured. It was not possible to use the run-up gauge employed for the earlier tests because of the complex beach profile. Instead, the run-up was assumed to be equal to the vertical distance between the beach crest level and the still water level. The beach slope was defined as the slope of the line joining the beach crest to the point at which the still water level met the beach profile. Fig 8 illustrates these definitions for a typical beach profile. The assumption that the beach crest level defined to the run-up was based on a visual examination of the tests which demonstrated the existence of only a very thin layer of water running over the top of the beach crest once a stable profile had been achieved. The definition of beach slope is somewhat arbitrary but has the advantage of being easily defined and measured.

Tests were carried out using wave heights ranging from 59.2mm to 179.5mm (trough to crest) and for wave periods ranging from 0.86 seconds to 2.40 seconds. There were a total of 22 tests in all. The deep water wave height and wave length were calculated as described above, the results of which were presented with run-up plotted against $\tan \alpha \, (H_o L_o)^{0.5}$ (Fig 9). The use of a least squares linear regression analysis gave a best fit straight line resulting in the relationship:

$$R = 0.327 \tan \alpha \, (H_o L_o)^{0.5} + 1.280$$

The correlation coefficient obtained from the above least squares linear regression analysis was 0.872 and gave a high degree of confidence in the relationship.

Unlike the above expression the equation of the line should pass through the origin. The reason for this is that when H_o is zero, R will also be zero. The equation of the line passing through the origin from the least square linear regression is:-

$$R = 0.358 \tan \alpha \, (H_o L_o)^{0.5}$$

This expression compares favourably with the previous equation derived from the small waves tests.

In order for this relationship to be of use for engineers in the design of shingle beaches for shore protection, design values for the unknowns (tan , H_o and L_o) are required. The deep water wave height and wave length can be calculated, given wind speed duration and fetch, but the resulting beach slope is less easy to predict. The relationship between tan and function of the wave steepness (H_o/L_o) has been investigated the particular material tested in the flume. A plot of tan against H_o/L_o is shown in Figure 10. The best fit straight line using linear regression analysis has been computed giving the relationship:-

$$\tan \alpha = 0.565 - 1.471 \, H_o/L_o$$

with a correlation coefficient of 0.613. Although this is not as good as the coefficients in earlier parts of the report, it nevertheless indicates a 99% certainty that there is a correlation (Ref 6). It should be stressed that the above relationship is based on a limited number of experiments, on one specific material. It therefore cannot be applied universally, as obviously the beach slope depends not only on the wave characteristics, but also on the material with which the beach is composed. Additionally the beach slopes resulting from various beach compositions will have been influenced by model scale effects. This is an area where there is a need for further research.

4.4 Large Wave Tests constrained by gabions)

The unconstrained beach tests produced a complex profile at the end of testing. The calculated beach slope used in the analysis was difficult to define in advance and this leads to difficulties in predictions.

It was thus decided to restrain the beach to a predetermined slope by means of wire baskets or gabions. This method would also provide valuable information for beaches restrained in this way. Gabions have often been used for coast protection work but with little available information concerning wave run-up.

Pea shingle was again used as test material and the size grading was as before (Fig 3). Coarse river gravel formed the core of the beach with perforated metal sheet separating the two layers as before (Fig 2). The pea shingle was nominally 0.2m thick. The upper 0.1m was encased in wire baskets formed from 9mm square wire mesh.

A central rectangular recess was formed in the top of the baskets to allow the run-up gauge to be mounted flush with the gabions. These gabions formed the whole of the upper part of the beach and extended to a depth of 0.28m, where wave action had little effect on the loose pea shingle (Fig 3, Plate 2).

Two beach slopes were tested, namely a 1:9 and a 1:6 slope. For each of these test beaches, wave periods ranging from 0.86 seconds to 2.4 seconds were used. Wave heights ranged from 53mm to 156mm to form a total of 45 wave conditions.

Run-up against H_oL_o was plotted on a log.log scale paper for the two beach slopes tested in the flume (Figs 11, 12). Scatter was again evident, so the equations of the best fit straight lines for the two beach slopes was computed using a least squares linear regression analysis. The equations resulting from this computation are:-

Beach slope	Equation of best-fit line	Correlation coefficient
1:9	$R = 0.036\ (H_o\ L_o)^{0.57}$	0.953
1:6	$R = 0.036\ (H_o\ L_o)^{0.60}$	0.940

The high values of the correlation coefficients shown above indicate that the logarithmic relationship assumed between run-up (R) and H_oL_o for the flume experiments is justifiable.

If the tangent of the beach slope was taken out of the factors for both the equations the following relationships result.

Beach slope	Equation of best-fit line
1:9	$R = 0.328 \tan\alpha\ (H_o\ L_o)^{0.57}$
1:6	$R = 0.219 \tan\alpha\ (H_o\ L_o)^{0.60}$

The factors describing the slopes of the two equations are sufficiently similar to justify the use of one equation for both beach slopes. For the resulting equation to be dimensionally correct the value of H_oL_o should be raised to the power 0.5. The powers obtained from the regression analysis for the two beach slopes were marginally higher than had been evaluated for previous work. These powers were 0.57 and 0.60 for the 1:9 and 1:6 slopes respectively. A graph is presented in Figure 13 of run-up plotted against tan α $(H_oL_o)^{0.5}$ for all the results obtained for the two slopes. The best fit straight line through these combined results using a least squares linear regression analysis gives a slope of 0.550 and an intercept of +0.083. The resulting equation given below:-

$$R = 0.550 \ (H_oL_o)^{0.5} + 0.083$$

has a correlation coefficient of 0.905 giving a high degree of confidence in this relationship. Since R is zero when H_o is zero the equation should not have an intercept value. The equation of the least squares linear regression line passing through the origin is:-

$$R = 0.557 \ (H_oL_o)^{0.5}$$

The above result indicates that the run-up on the beach restrained by gabions is nearly 1.6 times than for the unconstrained beaches (run-up factors 0.557 constrained and a 0.358 unconstrained). This increase in run-up with the groyne experiments can be explained in two ways.

The restrained beach does not permit the same level of energy dissipation due to the beach material being unable to move. Settlement of beach material and trapped air within the model gabion may have restricted permeation of the waves within the beach material so preventing the same level of energy dissipation. The model beach in this instance behaved more like the smooth slope than an effective dissipator of wave energy. The latter effect may largely be due to the model and in turn may not reflect the run-up for the prototype beach. More model work using larger material within the gabions would have to be done to substantiate this.

5 RESULTS AND CONCLUSIONS

The laboratory tests described in this report have shown that run-up on shingle beaches is very much smaller for given wave conditions, than for a smooth impermeable sea wall at the same slope. For all the tests carried out, it was possible to express the results using a modification to Hunt's formulae for smooth sea walls.

It has also been shown that run-up on a constrained shingle beach is greater than on a beach which is free to move, but still substantially less than on a smooth slope. This indicates the possibility of using a gabion system to create a shingle beach, as an apron to a sea wall for example, with better hydraulic performance than a concrete slope. Such an apron would need to be covered, however, in sufficient free shingle to prevent exposure and abrasion except in very severe storms.

The tests described in this report have been at a modest scale with regular waves. Further, the problems associated with adjustments of the beach profile under wave action have only briefly been mentioned. A subsequent report, dealing with these topics, will be presented in the near future.

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F I G U R E S

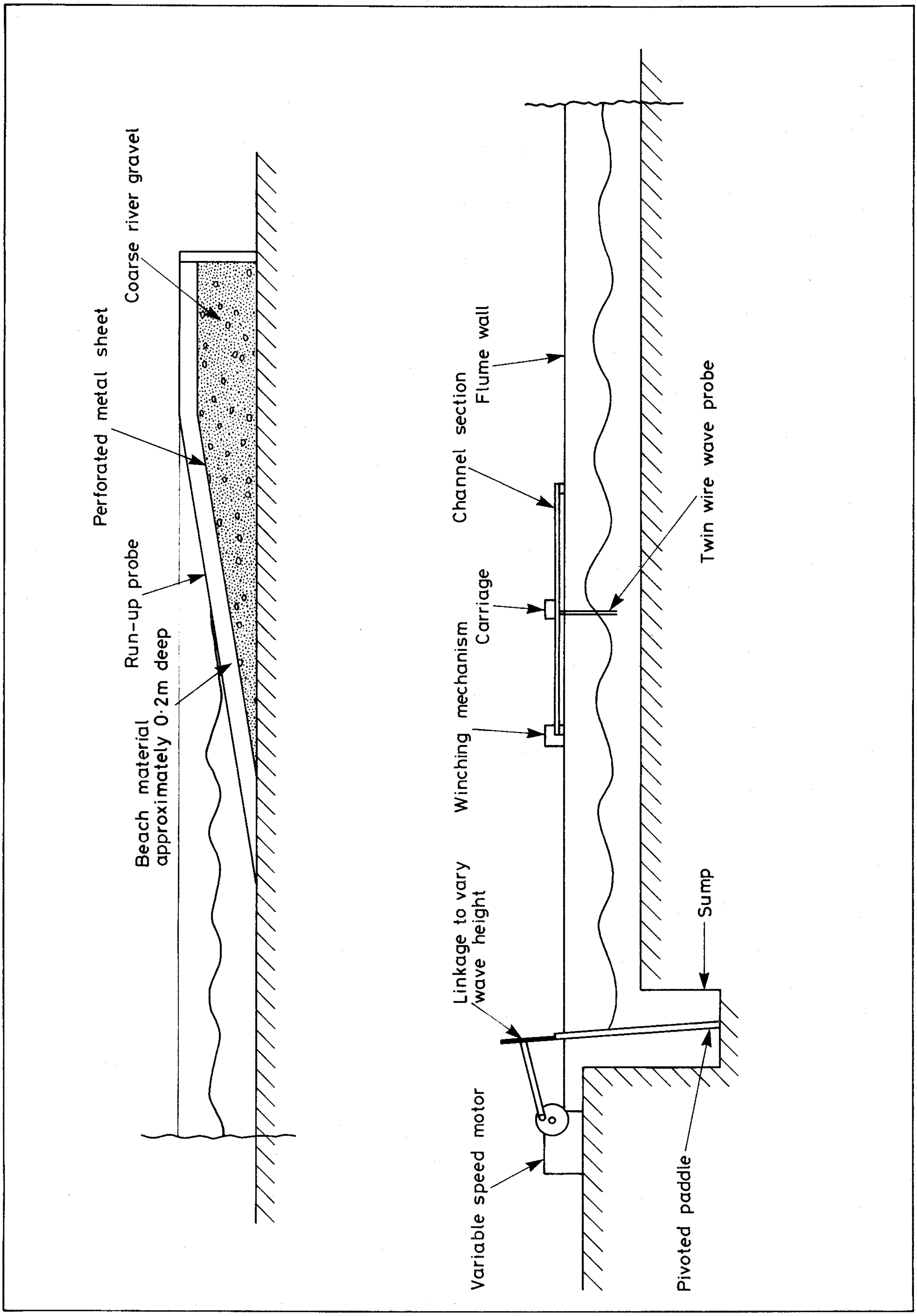

Fig 1 The experimental facility

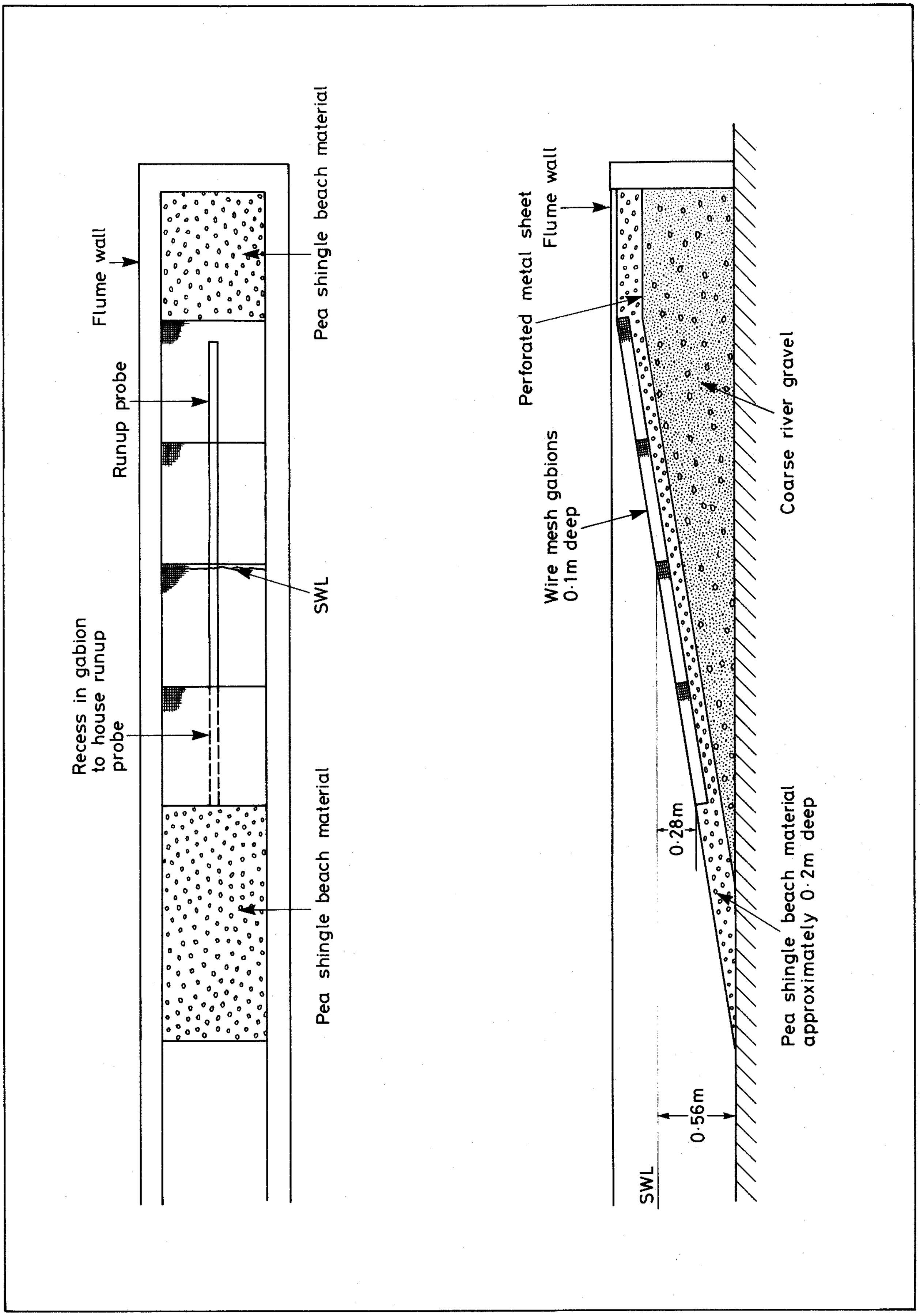

Fig 2 The constrained beach

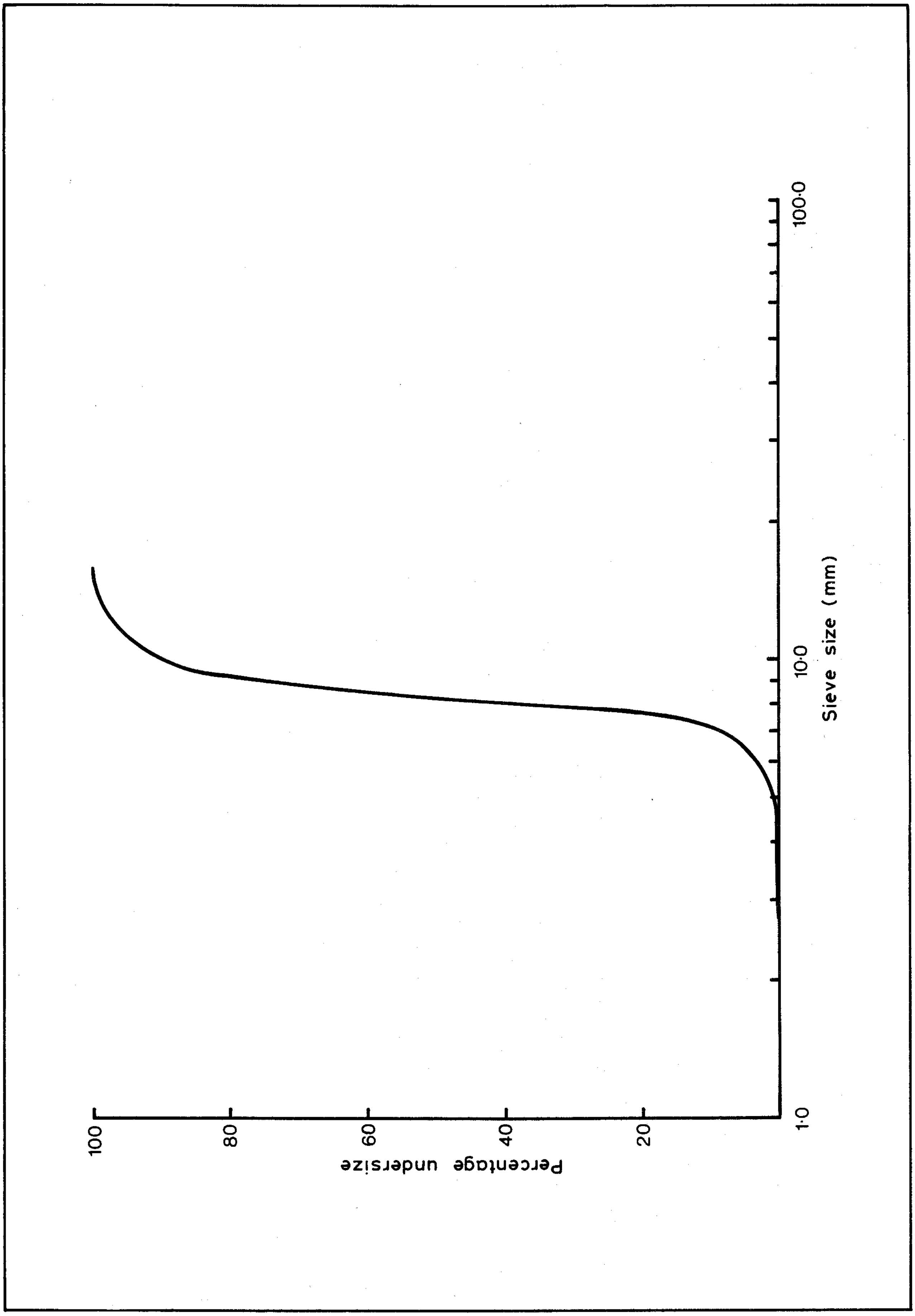

Fig 3   Sieve analysis of beach material

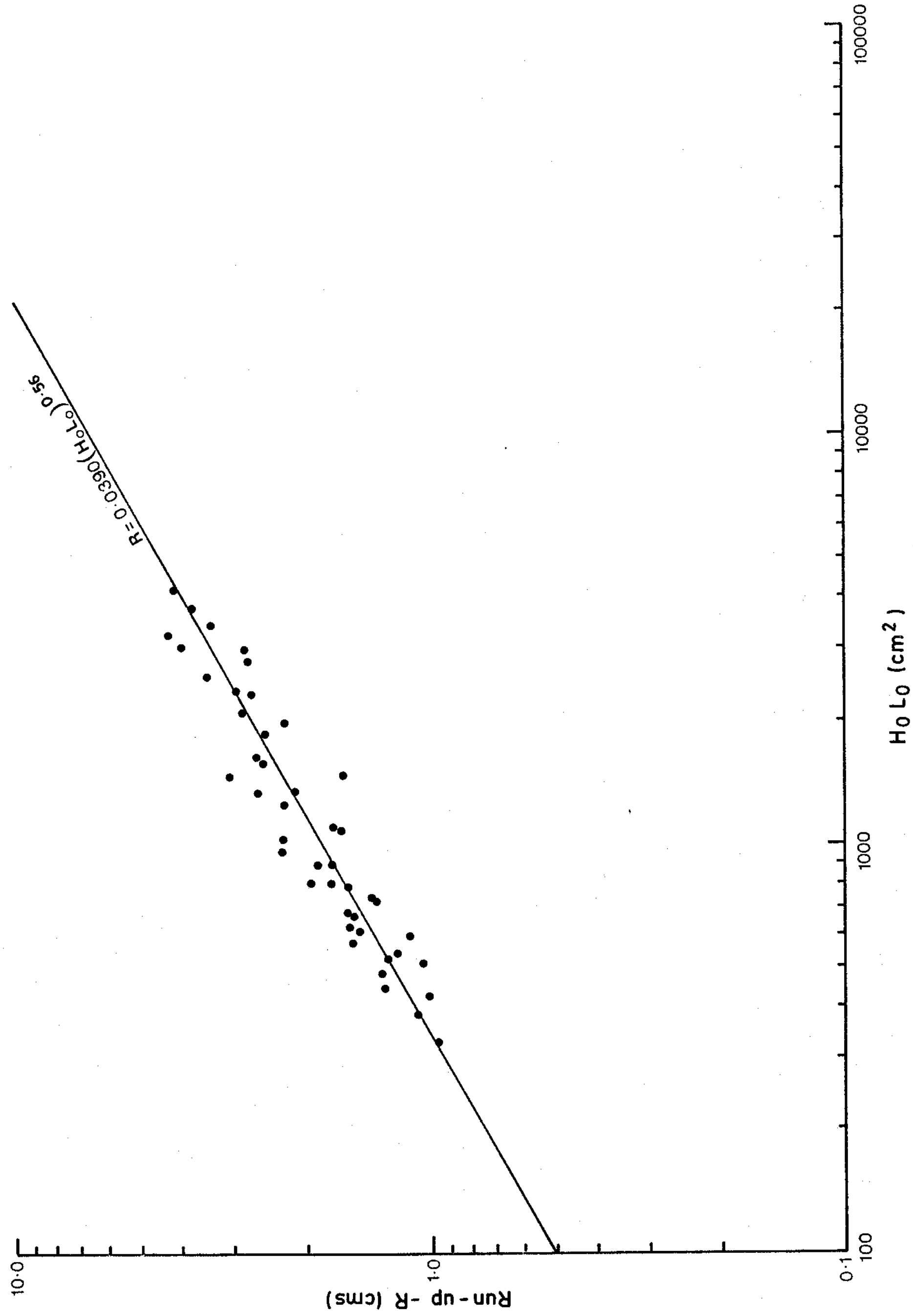

Fig 4 Run-up $\sim$ H_0L_0 for a beach slope of 1:6 for unconstrained tests using small waves

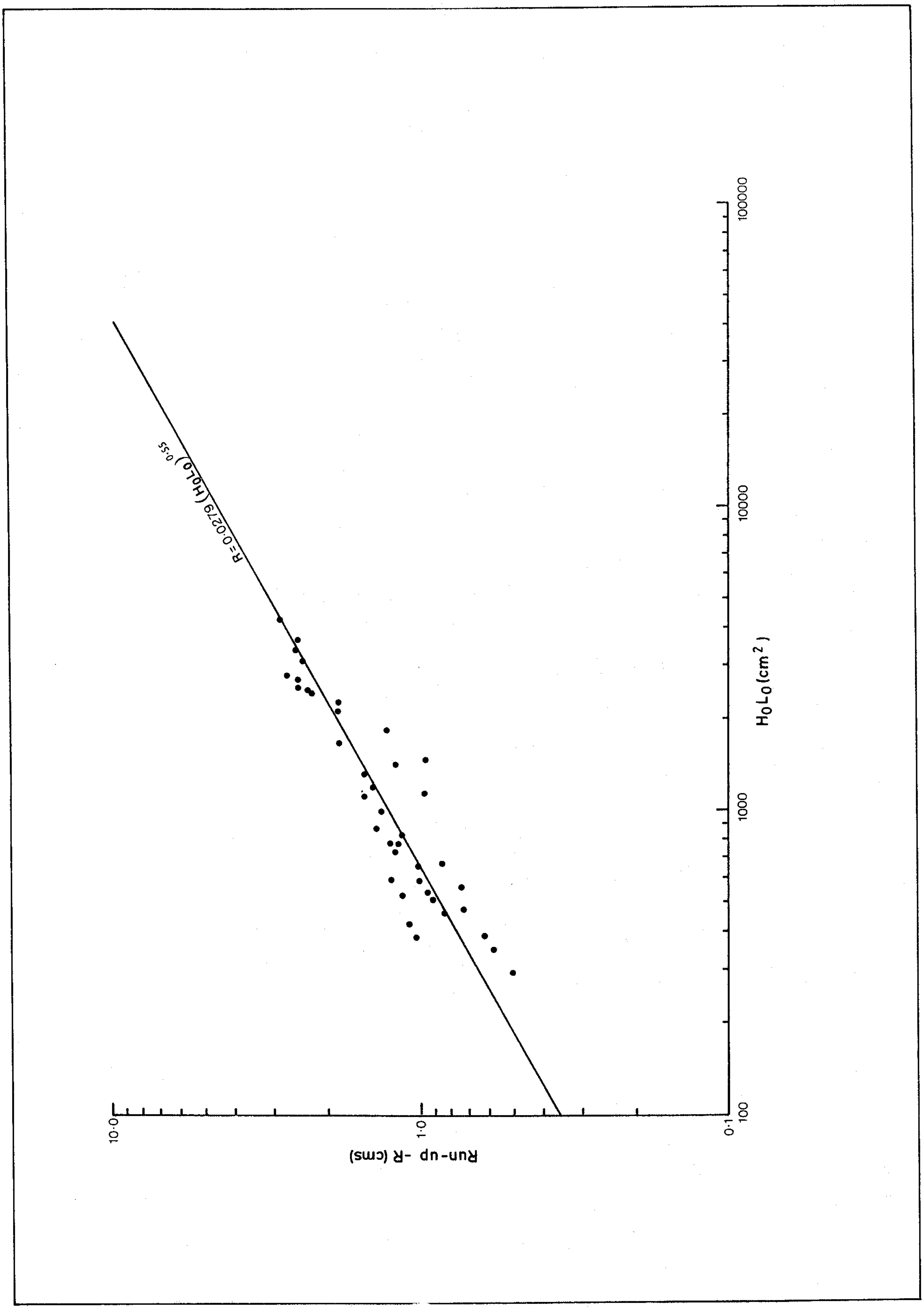

Fig 5 Run-up $\sim H_0L_0$ for a beach slope of 1:8 for unconstrained tests using small waves

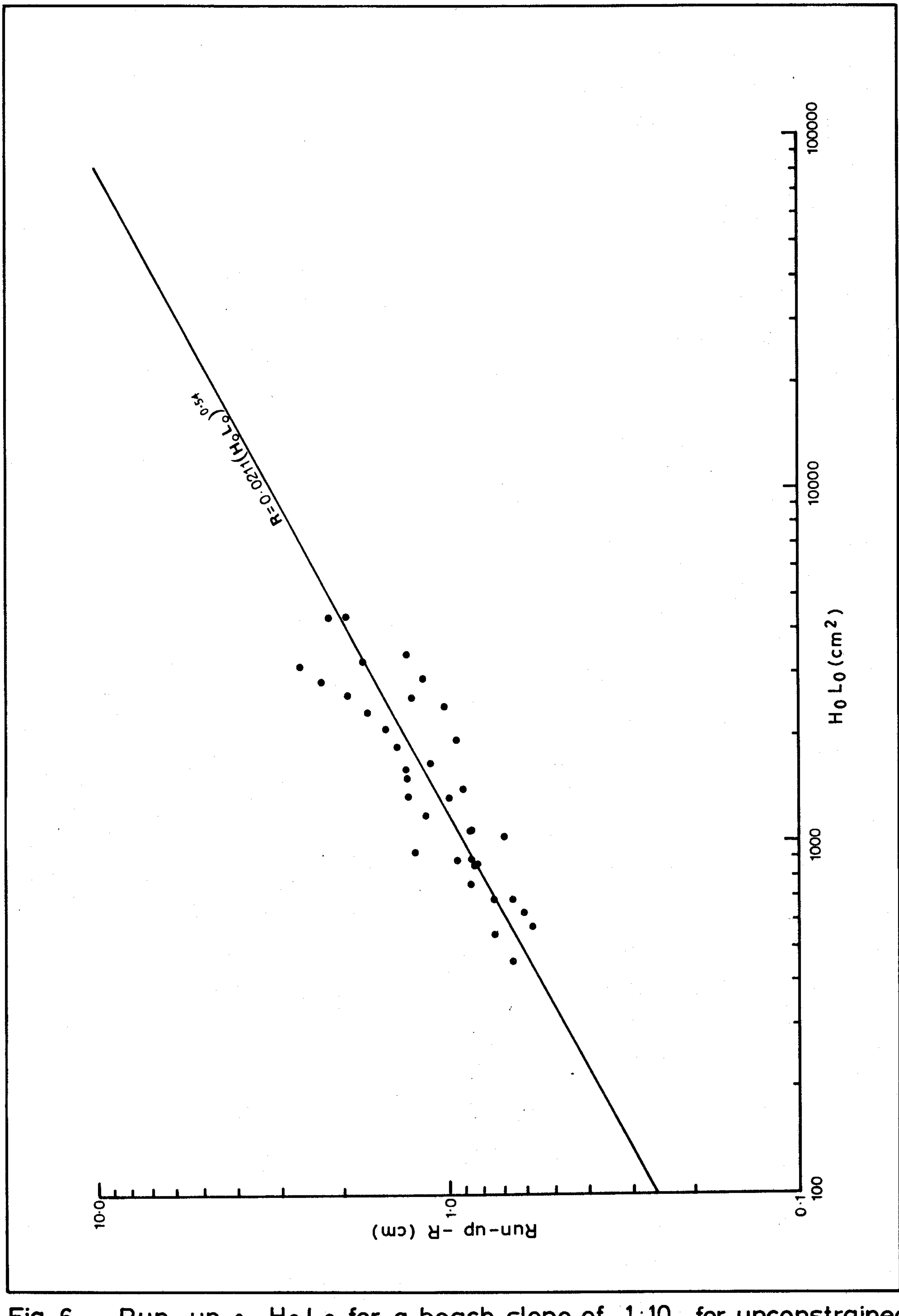

Fig 6 Run-up $\sim H_0 L_0$ for a beach slope of 1:10 for unconstrained tests using small waves

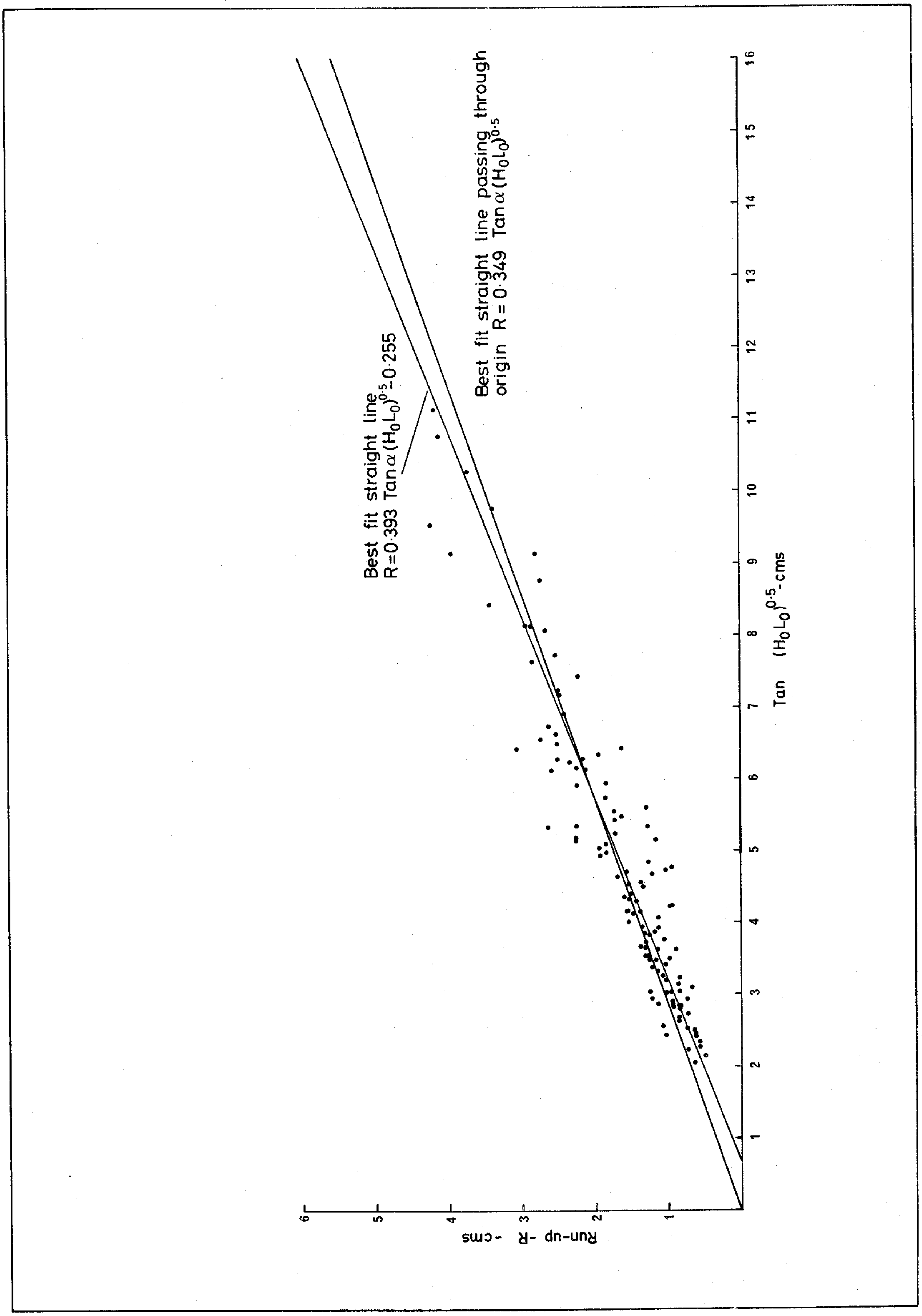

Fig 7 Run-up ∼ Tan α $(H_0L_0)^{0.5}$ for all results of unconstrained tests using small waves

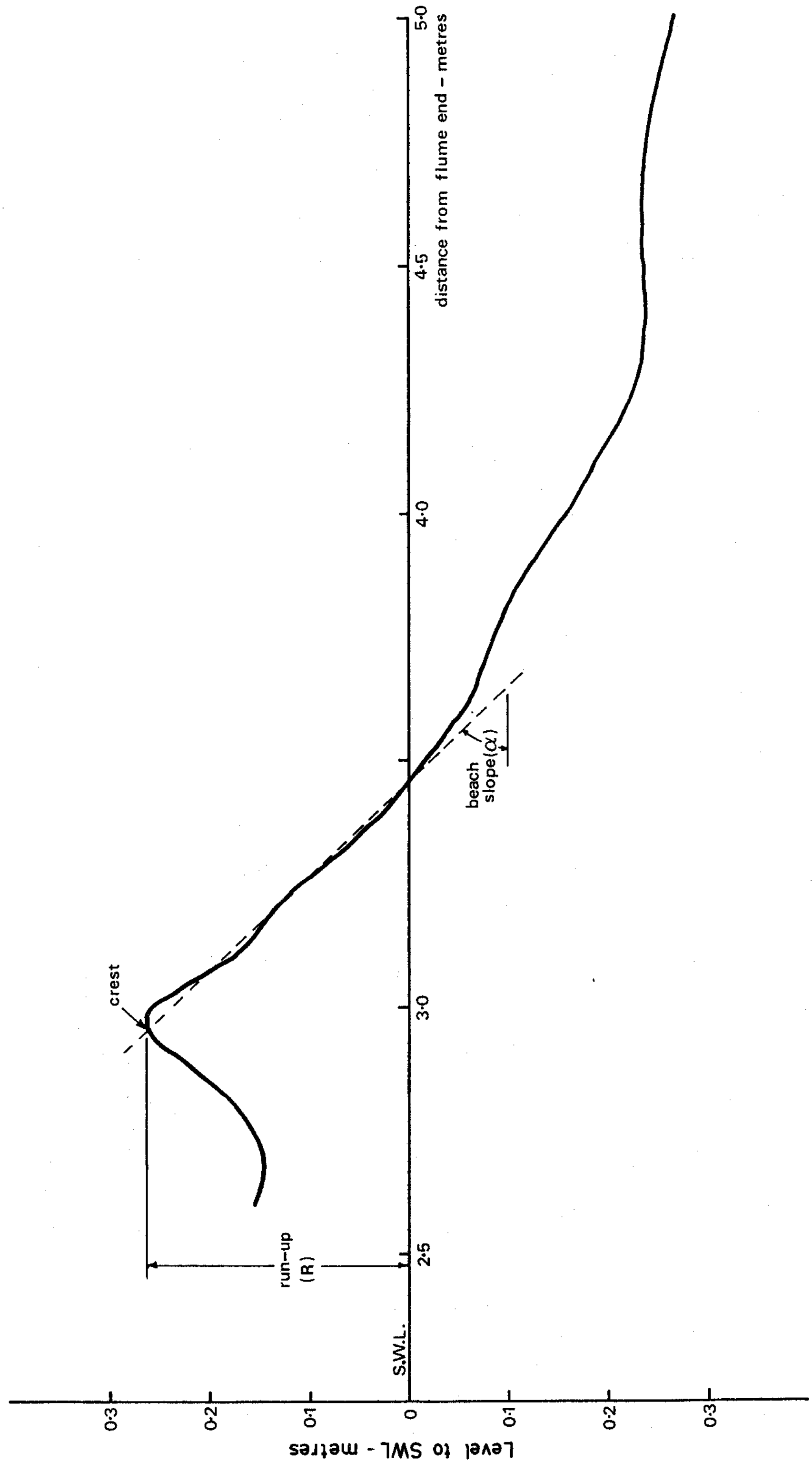

Fig 8 A typical post test beach profile for unconstrained tests using large waves

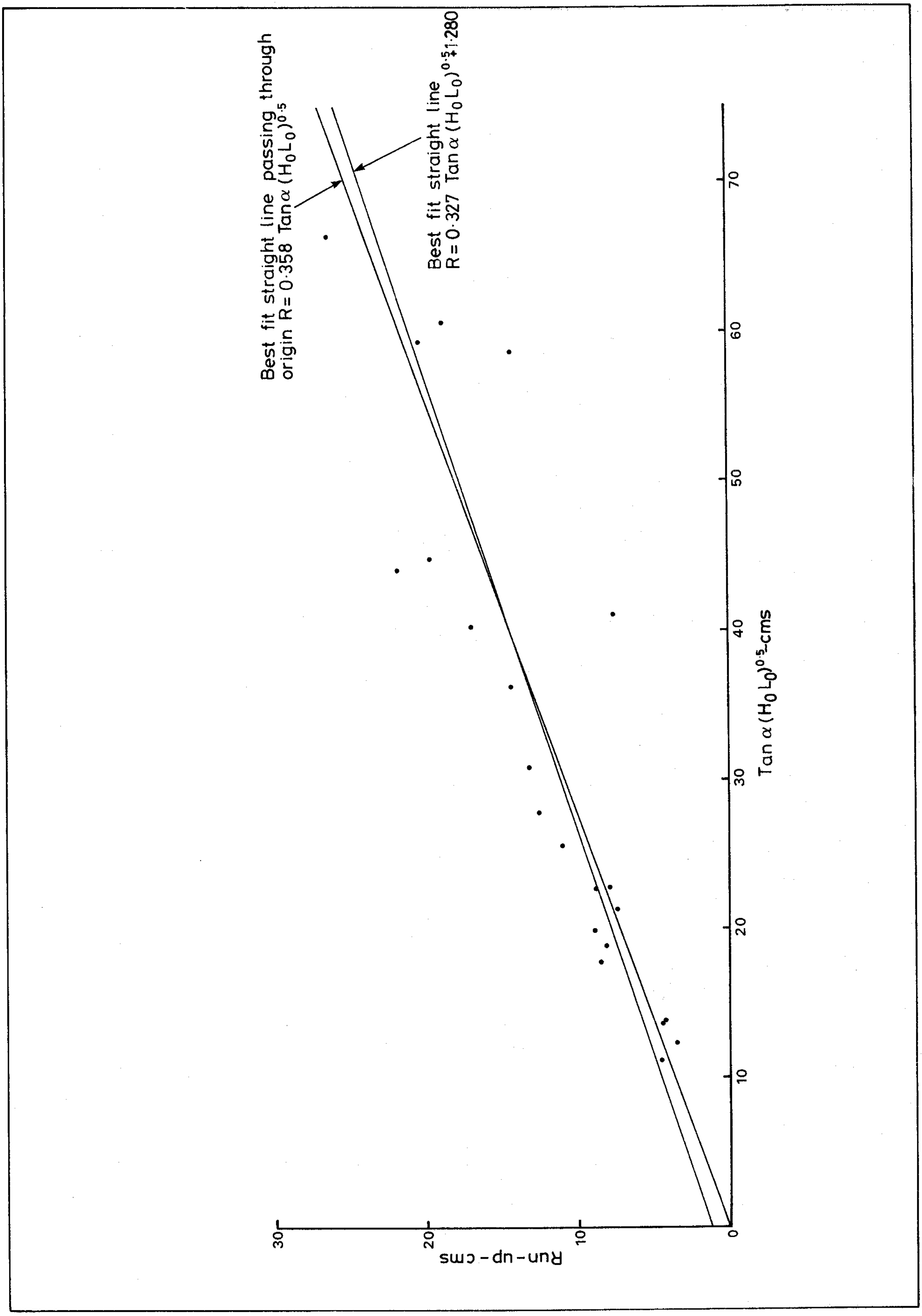

Fig 9 Run-up $\sim$ Tan α $(H_0L_0)^{0\cdot5}$ for an initial beach slope of 1:6 for unconstrained tests using large waves

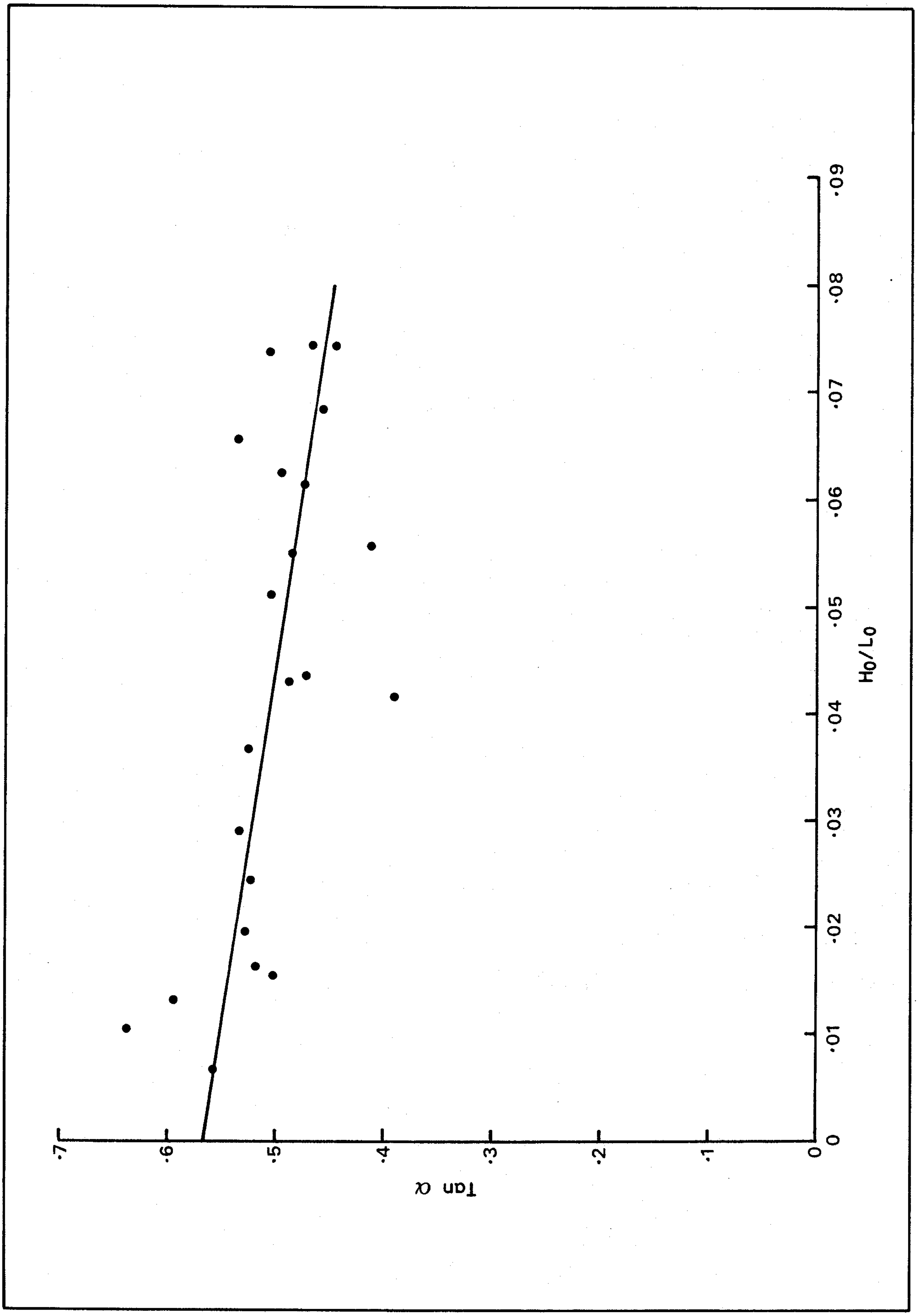

Fig 10 Relationship between beach slope and wave steepness for 8·2 mm pea shingle

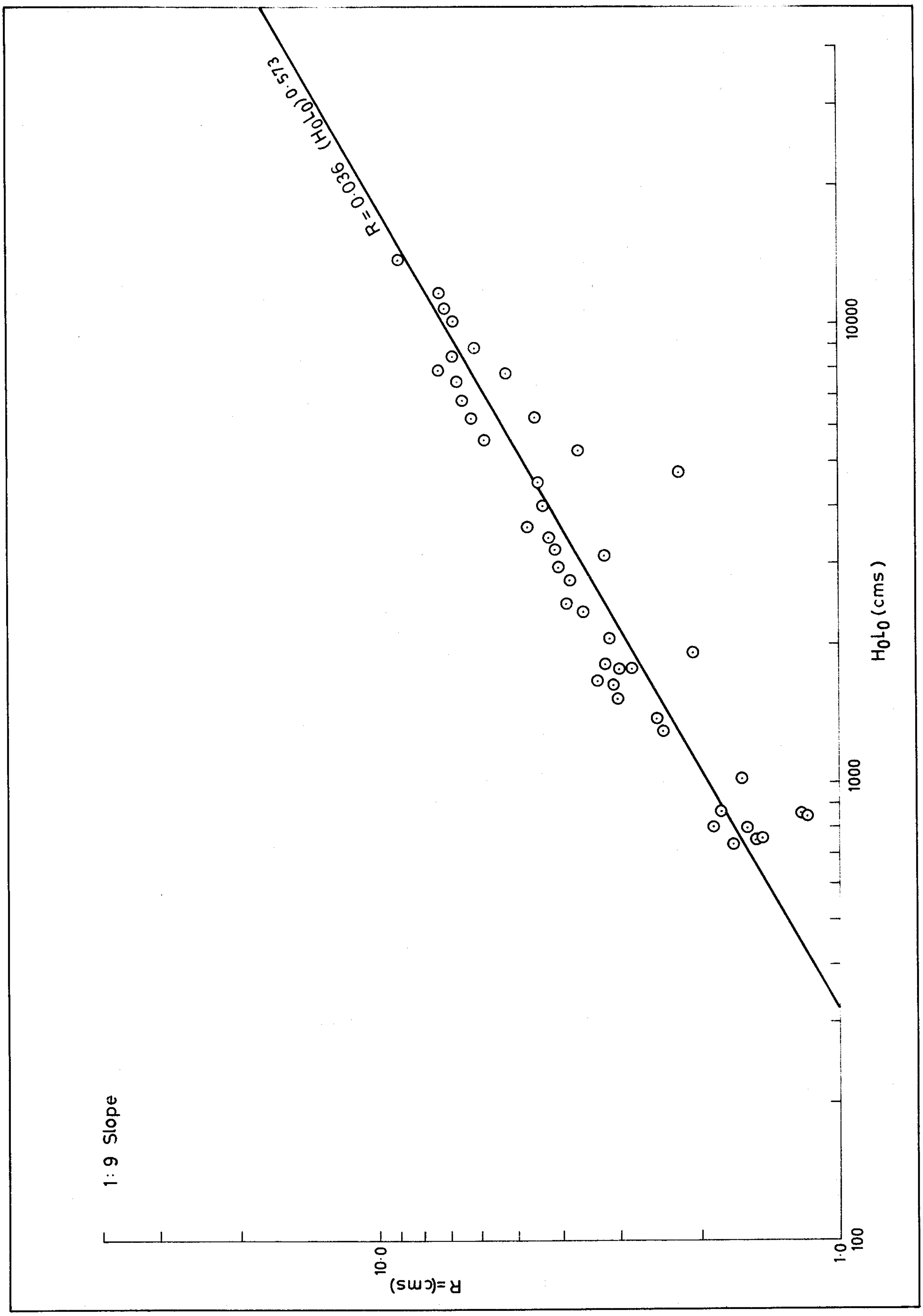

Fig 11 Run-up $\sim H_0 L_0$ for a beach slope of 1:9 for constrained tests

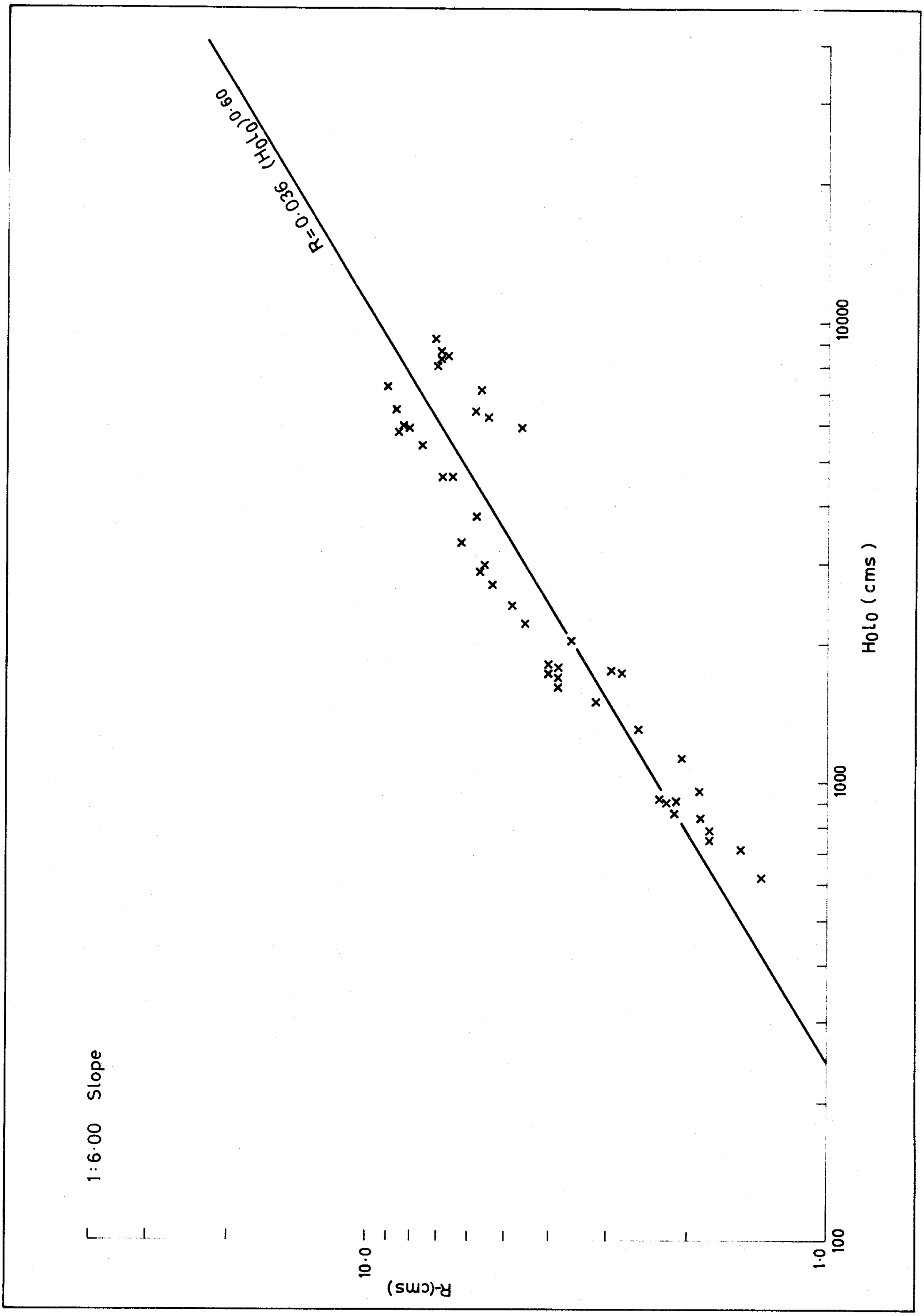

Fig 12 Run-up ~ H_0L_0 for a beach slope of 1:6 for constrained test

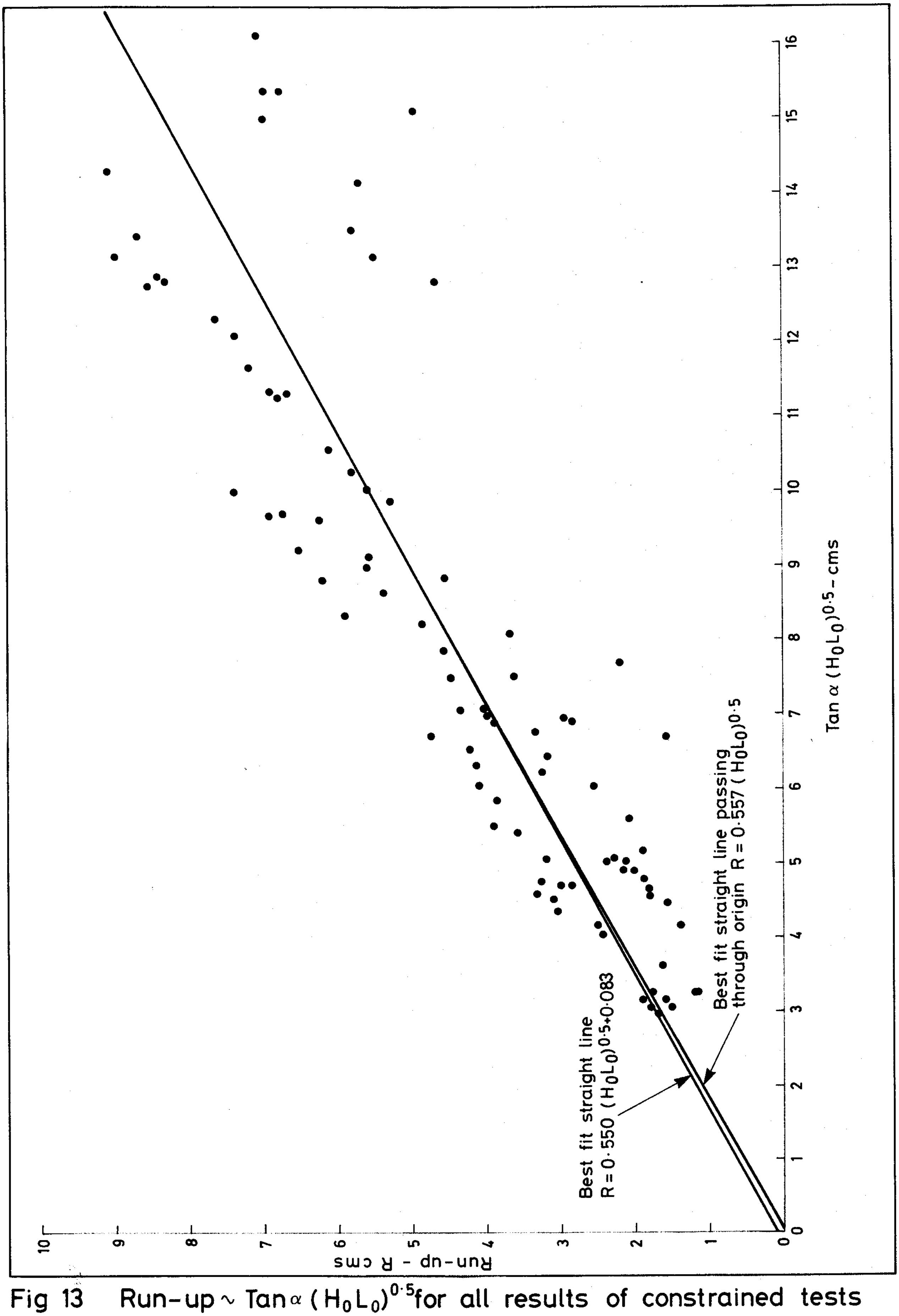

Fig 13 Run-up ∼ Tan α $(H_0L_0)^{0.5}$ for all results of constrained tests

P L A T E S

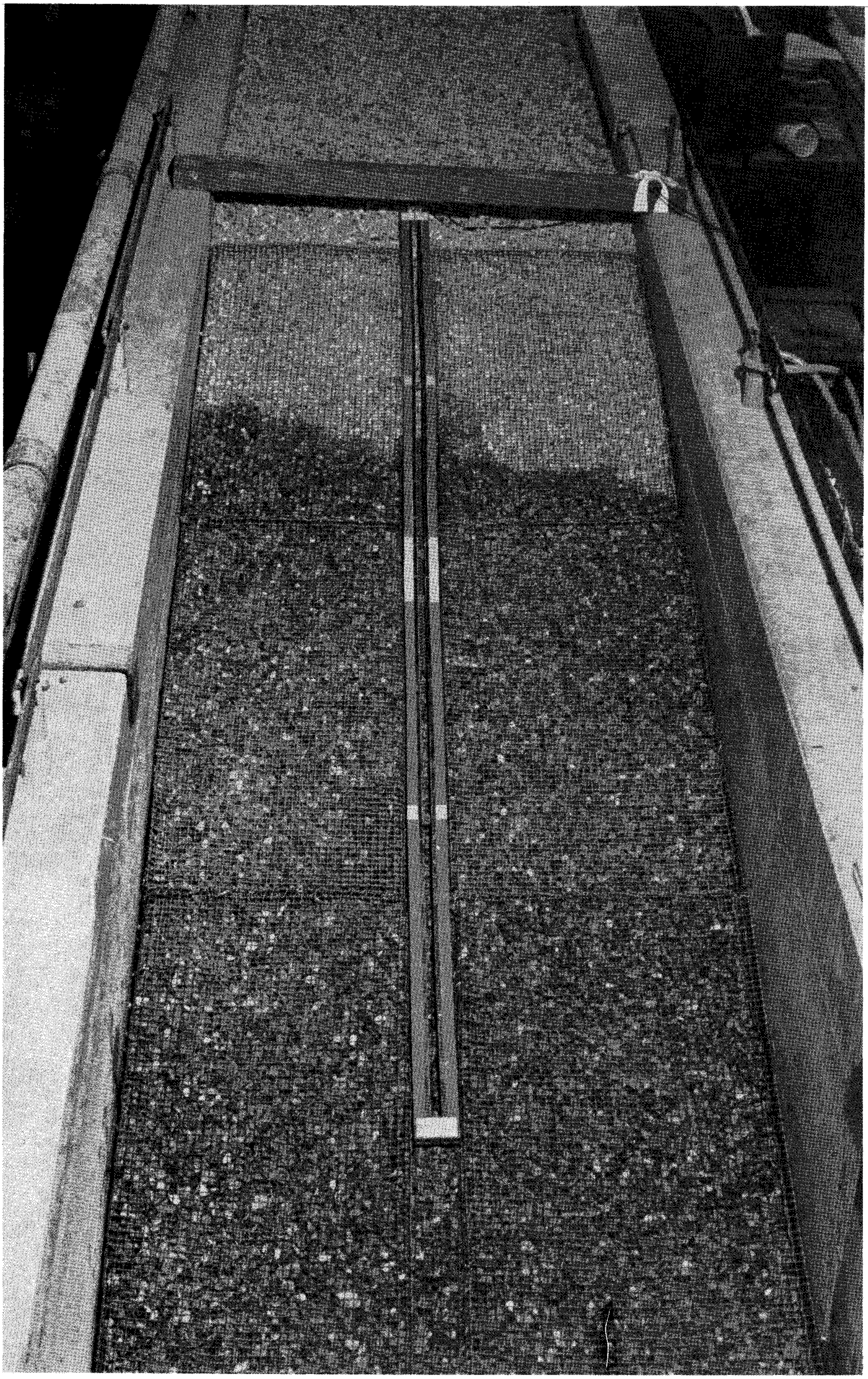

Plate 1 The runup probe

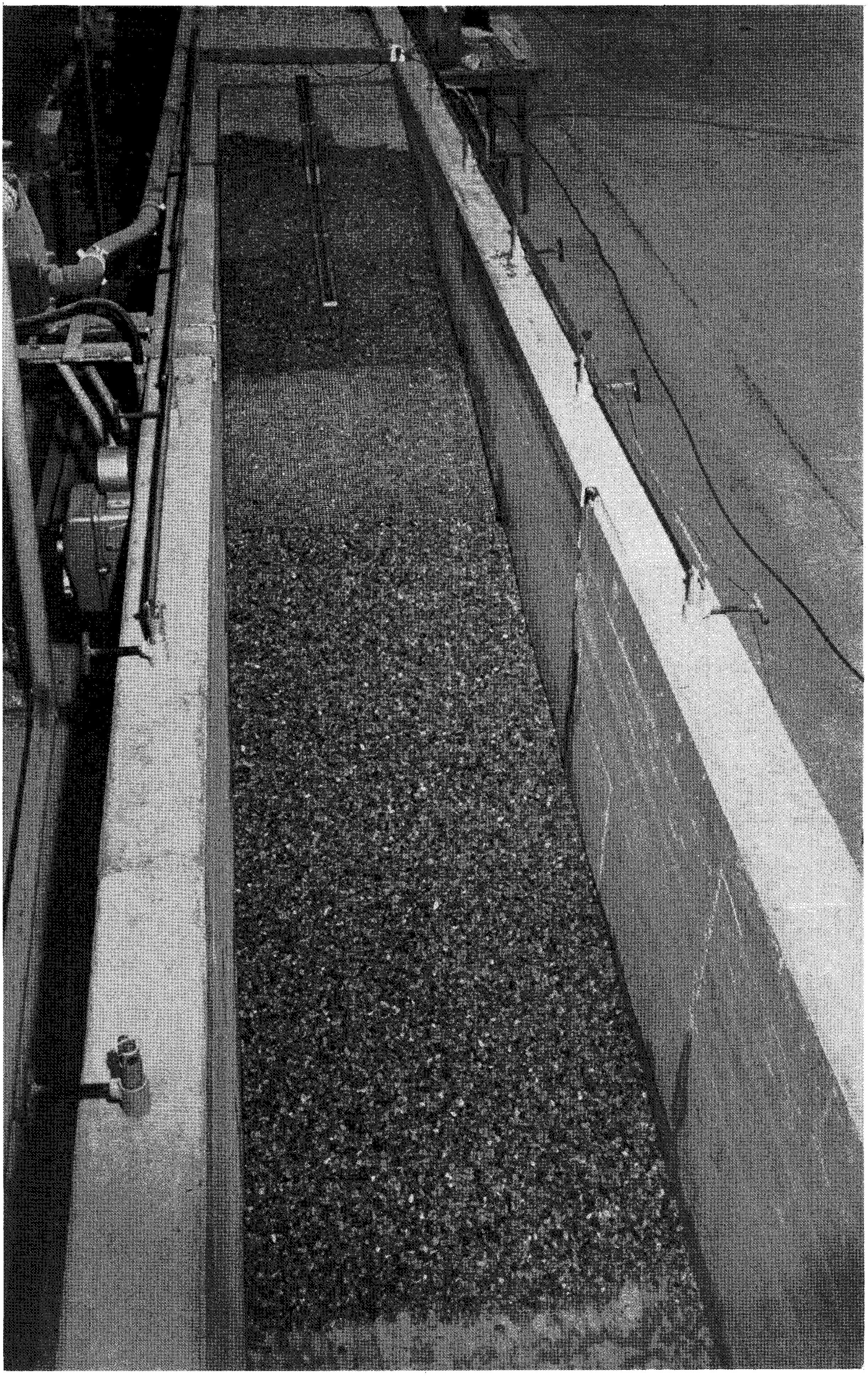

Plate 2 The constrained 1:9 beach