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LOW-CREST BREAKWATERS, HYDRAULIC PERFORMANCE
AND STABILITY

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ABSTRACT

The research reported advances design methods for low crest and submerged breakwaters. A low-crest breakwater may be defined as a breakwater that is frequently overtopped by wave action, but that still performs a significant function in dissipating wave energy. Such breakwaters are considerably simpler and cheaper to construct than conventional harbour breakwaters, but do allow higher levels of wave activity in their lee. Low-crest breakwaters may be of considerable benefit in shoreline protection, and their use in this context is discussed separately by Brampton and Smallman⁽¹⁸⁾.

This report reviews techniques available to predict wave transmission and overtopping effects and armour displacement. Hydraulic model test results obtained both at Hydraulics Research and in other labs are presented graphically in terms of appropriate non-dimensional parameters. The degree of attenuation of wave effects may be estimated from these graphs for a wide range of structure crest levels and/or incident wave conditions. Test results for structures of approximately 40% porosity allow the estimation of the wave transmission and overtopping performance with sufficient accuracy for preliminary design purposes. The attenuation provided by breakwaters of other porosities is less certain, but a basis for estimating it is suggested. More accurate prediction will depend on an ability to define the permeability of the breakwater to oscillatory flow. At the moment this is limited to estimating the porosity of the prototype breakwater and testing an appropriate scale model.

Measurements of armour movement under random waves have also been analysed. The results have been presented graphically in terms of non-dimensional stability numbers. This provides the basis of a method for estimating armour sizes needed for breakwaters subject to frequent overtopping. A number of anomalies are discussed and it is concluded that all such designs should be finalised by appropriate physical model testing.

NOTATION

B	Breakwater crest width
C	Wave celerity
d	Nominal water depth
d_b	Depth at wave break point
d_s	Depth of water at structure
D	Representative stone diameter
E_d	Wave energy dissipated
E_i	Incident wave energy
E_r	Reflected wave energy
E_{to}	Wave energy transmitted by overtopping
E_{tt}	Wave energy transmitted through structure
g	Acceleration due to gravity
\bar{H}	Mean wave height
H_b	Breaking wave height
H_i	Incident wave height
H_r	Reflected wave height
H_s	Significant wave height
H_{to}	Wave height corresponding to E_{to}
H_{tt}	Wave height corresponding to E_{tt}
K_d	Coefficient of dissipation
K_r	Coefficient of reflection
K_t	Coefficient of total transmission
K_{to}	Coefficient of transmission by overtopping
K_{tt}	Coefficient of transmission through structure
L_p	Wave length corresponding to T_p
L_s	Wave length at structure toe
m	Sea bed slope ($= \tan \theta$)
N	Number of waves overtopping
N_a	Number of armour units in structure
P	Porosity
R	Notional run-up level
R_c	Breakwater freeboard
S	Sea steepness ($= H/L_o$)
T	Nominal wave period
T_p	Period of peak spectral energy
T_z	Zero crossing wave period
W_{50}	Median weight of armour stone
Δ	Number of armour stones displaced

ρ_r
 ρ

Density of rock
Density of water

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1 **LOW-CREST BREAKWATERS**

1.1 Introduction

The hydraulic performance of a breakwater with a low crest subjected to frequent wave overtopping may be treated as resulting from the combination of four different mechanisms:-

(a) Where the sea bed topography changes, and particularly in the form of a structure that pierces the water surface, some proportion of the incident wave energy will be reflected. In general the longer the incident wave length, the higher the proportion that may be reflected by a breakwater. Shorter or steeper waves may dissipate more of their energy at the structure reflecting a lower proportion.

(b) Where the crest of the structure is always immersed, some of the energy not reflected will then be dissipated in turbulent friction in the flow over the structure crest. Short period waves may pass over such a breakwater relatively unchanged, but longer period waves will tend to dissipate some energy.

(c) Under some combinations of crest freeboard and incident wave conditions, waves may shoal and break on the structure, or run up and overtop, dissipating further energy. In breaking, or overtopping, each wave will give rise to a number of smaller and shorter waves, in the lee of the structure, causing a shift in the wave period, or frequency, corresponding to the energy peak, as well as a reduction in the total energy. Again the degree and form of energy dissipation or change will depend strongly on the incident wave period, or wave length, as well as the wave height. Generally longer shallow waves will give rise to greater levels of overtopping, and hence wave transmission over the structure, than shorter steep waves. Steep waves may break on the breakwater dissipating much of their energy.

(d) Wave energy may also be transmitted by wave-induced oscillatory flow through the voids in a porous rubble, or other, breakwater. In rubble structures such flow occurs principally in the outer and upper layers, especially if a layered construction is used. Wave transmission may usually be regarded as negligible for short period waves, but may be of importance for porous structures subjected to long waves.

The transmitted wave may therefore be viewed as the result of the combined effects of each of these four mechanisms. As, however, their interaction is complex and ill-defined most researchers have chosen to concentrate on only one, or two, mechanisms at a time. The technical literature available may therefore be considered under the following general headings:-

- (a) Reflection at a structure
- (b) Transformations over submerged structures
- (c) Transmission by overtopping
- (d) Transmission through a porous structure

As indicated above, the relative effect of each of these mechanisms will differ with both wave conditions and structure geometry. In considering each mechanism, researchers have therefore often considered different aspects of wave behaviour, usually with widely differing analytical or experimental techniques. Their results may therefore not be fully compatible. The rest of this report does however seek to identify the principal effects of each mechanism on the overall performance of low-crest breakwaters.

1.2 Object of study

The primary aim of this study is to provide design methods to allow the estimation of the hydraulic performance and stability characteristics of rock armoured breakwaters. Using these estimates, it is suggested that preliminary cross-sections may be produced for initial costing and feasibility purposes.

The data needed to provide this information has been obtained from recent site specific and fundamental research studies conducted at Hydraulics Research and from published work principally by the US Army Corps of Engineers. Most of the data considered in detail was derived from random wave model studies.

1.3 Outline of report

Before considering the data available in detail, a summary of previous work in this field is given in chapter 2. The main transmission characteristics of low-crest breakwaters are considered in chapter 3. Special attention is paid to the mechanisms of wave transmission as well as to the effects of wave period, breakwater porosity and breakwater crest width on that transmission. Dimensionless design graphs are given, from which the wave transmission coefficient may be estimated for various breakwater freeboards and incident wave conditions. Chapter 4 considers the stability of low crest breakwaters in terms of the long established Hudson's stability number and the relatively recent spectral stability number. Qualitative comparisons of the two parameters are made. In chapter 5 the accuracy and applicability of the results is assessed, particularly with regard to their use as design guidelines. Further discussion is offered concerning the recurrent anomalies observed within the data scatter. A tentative explanation of these anomalies is given. Chapter 6 draws together the conclusions arising from this report and makes recommendations for further work in this field.

2 PREVIOUS WORK

2.1 Definitions

In considering the reflection, dissipation and transmission of wave energy at a breakwater or similar coastal structure, a number of simple coefficients and terms may be useful. Each of these may most easily be described by considering a simple structure in water of constant depth, d , subjected to waves of incident energy, E_i , and having a wave height, H_i , generally proportional to $E_i^{1/2}$ (For the purposes of definition regular waves will be considered, however for random waves energy may be divided into frequency bands).

At the structure part of the incident energy is reflected, E_r , equivalent to a reflected wave height, H_r . The coefficient of reflection, K_r may then be defined:-

$$K_r = \frac{H_r}{H_i} = \left(\frac{E_r}{E_i} \right)^{1/2} \quad (2.1)$$

Some of the remaining energy may be transmitted by overtopping, E_{to} , equivalent to a wave height, H_{to} . Similarly energy may be transmitted through a porous structure, E_{tt} , equivalent to H_{tt} . Wave transmission coefficients may be defined for each of these two cases, using the same terminology as above:-

$$K_{to} = \frac{H_{to}}{H_i} = \left(\frac{E_{to}}{E_i} \right)^{1/2} \quad (2.2)$$

and

$$K_{tt} = \frac{H_{tt}}{H_i} = \left(\frac{E_{tt}}{E_i} \right)^{1/2} \quad (2.3)$$

That proportion of the energy incident on the structure that is neither reflected nor transmitted must necessarily be dissipated in the various processes at the structure. The energy dissipated, E_d , may be used to define a coefficient of energy dissipation, K_d :-

$$K_d = \left(\frac{E_d}{E_i} \right)^{1/2} \quad (2.4)$$

The energy balance may be written:-

$$E_i = E_d + E_r + E_{to} + E_{tt} \quad (2.5)$$

and the coefficient of dissipation:-

$$K_d = (1 - K_{to}^2 - K_{tt}^2 - K_r^2)^{1/2} \quad (2.6)$$

In some circumstances no distinction is drawn between energy transmitted through the structure and that transmitted by overtopping, a single coefficient may

then be defined:-

$$K_t = (K_{to}^2 + K_{tt}^2)^{\frac{1}{2}} \quad (2.7)$$

As mentioned earlier, in the process of wave reflection and transmission by overtopping, a shift of energy from low frequencies to high frequencies may occur as single large waves break and reform as a number of small short waves. Care must be taken in using the definitions above to ensure such a frequency shift is allowed for.

2.2 Wave reflections

The prediction of the level of reflected wave energy is addressed by various researchers using different approaches. Both analytical and experimental techniques are reported. In general, however, most methods available rely on model tests to determine values of the empirical coefficients used, and many of these tests are reported in the literature. The case of breakwaters that both transmit and reflect is relatively lightly covered.

Much of the recent work is summarised by Seelig⁽¹⁾, and, in a longer version, by Seelig & Ahrens⁽²⁾. Both present simple prediction methods for reflections from beaches, seawalls and breakwaters. The special case of both transmission and reflection is covered briefly citing the model work of Sollitt & Cross⁽³⁾ and the prediction method of Madsen & White⁽⁴⁾. Seelig^(1,5) presents a simple prediction equation for the reflection coefficient at a rubble breakwater in terms of the surf similarity parameter, or Iribarren number, I_r , and empirical coefficients α and β :-

$$K_r = \frac{\alpha I_r^2}{\beta + I_r^2} \quad (2.8)$$

where $I_r = \tan \theta / s^{\frac{1}{2}}$ and $s = H/L_0$

Using $\alpha = 0.6$ and $\beta = 6.6$ this is likely to give conservative results. It should be noted however that it is only directly applicable to regular waves, and moreover it does not account for any significant transmission of wave energy.

Seelig & Ahrens⁽²⁾ also discuss the influence of layers of armour over an impermeable core or embankment. They suggest that equation 2.8 should be used with $\beta = 5.5$ and α defined by:-

$$\alpha = \exp \left[-1.7 \left(\frac{D}{L_s} \right)^{0.5} \cot \theta - 0.5 \left(\frac{H_i}{H_b} \right)^{1.3} \right] \quad (2.9)$$

where L_s is the wavelength at the structure toe, D is the representative stone diameter $(W/\rho)^{1/3}$ and H_b is a representative breaking wave height. Values of a correction factor α' to be applied to α are tabulated for 2-4 layers of armour, and for ranges of relative

armour size, D/H_1 .

Other methods for the prediction of wave reflections are presented by Moraes⁽⁶⁾, Battjes⁽⁷⁾, Madsen & White⁽⁴⁾ and Madsen and others^(24,25,26).

Methods for the measurement and analysis of incident and reflected waves have been discussed by Madsen & White⁽⁴⁾, Thornton & Calhoun⁽⁸⁾, Kajima⁽¹²⁾, Gilbert & Thompson⁽¹³⁾, Gaillard et al⁽¹⁴⁾, and Goda & Suzuki⁽¹⁵⁾. Measurements of reflections from breakwaters in model studies are presented by Seelig & Ahrens⁽²⁾, Madsen & White⁽⁴⁾, Seelig⁽⁵⁾, CERC⁽⁹⁾, Kondo et al⁽¹⁰⁾, and Ijima et al⁽¹¹⁾. Apparently the sole analysis of wave reflections from a prototype structure is presented by Thornton & Calhoun⁽⁸⁾.

For convenience, wave reflections and transmission at wholly submerged structures are considered separately as elements of wave transformations in the following section.

2.3 Wave transformations

When waves encounter a change in sea bed level a number of transformations may occur to the waves. These may be simply summarised as:-

- a) refraction
- b) diffraction
- c) friction
- d) shoaling
- and e) breaking.

Of these, refraction and diffraction effects are not considered further in this report. The interested reader is referred to work by CERC⁽⁹⁾, Longuet-Higgins⁽¹⁶⁾, Brampton⁽¹⁷⁾ and Brampton & Smallman⁽¹⁸⁾.

The effects of significant changes in sea bed level on wave transformations, such as produced by a submerged breakwater, are considered by Brampton⁽¹⁹⁾, Miles⁽²⁰⁾ and in passing, by Lamb⁽²¹⁾. The effect of turbulent friction at the sea bed boundary layer has been discussed by Treloar & Abernethy⁽²³⁾ and by Hydraulics Research⁽²²⁾. Both laminar and rough turbulent friction laws are considered, and simplified calculation methods are presented.

The influence of sea bed slope and water depth on wave height and length, shoaling, has been discussed by very many authors, and is summarised by CERC⁽⁹⁾. Many authors have also addressed wave breaking, and this remains an active topic for research. For waves passing over a simple sea bed of slope m ($= \tan \theta$), the relationship between the wave height at breaking, H_b , and the depth at breaking, d_b , is given by⁽⁹⁾:-

$$\frac{H_b}{d_b} = b - a H_b / gT^2 \quad (2.10)$$

where a and b are functions of the beach slope, m:

$$a = 43.75 (1 - \exp(-19m))$$

$$b = 1.56 / (1 + \exp(-19.5m)).$$

2.4 Wave transmission by overtopping

Research into the performance of low-crest breakwaters has concentrated primarily on the level and form of wave transmission. As no fully satisfactory formulation for the hydrodynamic processes of wave overtopping has been produced, researchers have generally concentrated on physical model studies, from which they have derived empirical expressions for the wave transmission coefficient. These model tests have generally studied one or more of three principal structure types:-

- (a) wholly impermeable, solid;
- (b) rubble mound with impermeable layer, barrier or core;
- (c) permeable rubble mound.

The general form of such structures is trapezoidal, although some experimenters have used rectangular sections, principally to study transmission through porous structures.

In a single instance, reported by Thornton and Calhoun⁽⁸⁾, measurements were made of both wave reflection and transmission at a prototype structure, a rubble mound breakwater at Monterey, California.

The results of model studies of wave transmission, principally by overtopping, are presented by Seelig⁽⁵⁾, CERC⁽⁹⁾, Dattatri et al⁽²⁷⁾, Bade & Kaldenhoff⁽²⁸⁾, Ouellet & Eubanks⁽²⁹⁾ and Allsop⁽³⁰⁾. Seelig⁽⁵⁾ presents wave transmission coefficients for a wide variety of breakwater sections, principally in terms of a wave steepness parameter, H/gT^2 . A major conclusion of that study is that the transmission by overtopping may be estimated from the crest freeboard, R_c , and a notion run-up level, R , by:-

$$K_{to} = C \left(1 - \frac{R}{R_c}\right) \quad (2.11)$$

where

$$C = 0.51 - \frac{0.11 B}{R_c + d_s},$$

and B is the structure crest width and d_s is the water depth at the structure. For submerged breakwaters with approach sea bed slopes around 1:15 an extended expression is suggested:-

$$K_{t0} = C(1 - \frac{R_c}{R}) - (1 - 2C) \frac{R_c}{R} \quad (2.12)$$

For both equation 2.11 and 2.12, Seelig suggests that the value of the notional run-up level, R , may be calculated using normal prediction methods for run-up levels. A number of analytical and empirical methods are discussed. For stable rock breakwaters a single expression is suggested:

$$\frac{R}{H} = \frac{a \text{ Ir}}{1 + b \text{ Ir}} \quad (2.13)$$

where

$$\text{Ir} = \frac{\tan \theta}{s^{\frac{1}{2}}}, \quad s = \frac{H}{L_0}$$

and a and b are empirical coefficients having values $a = 0.692$ and $b = 0.504$. Recent work by Allsop et al^(31,32) has discussed the prediction of wave run-up levels under both regular and random waves, and a number of empirical expressions have been identified for various armour units.

CERC⁽⁹⁾ use Seelig's method for predicting K_{t0} . Graphs are presented allowing the prediction of the overtopping coefficient under random as well as regular waves. It is however noted that this method may overpredict the value of K_{t0} .

Dattatri et al⁽²⁷⁾ present results of regular wave tests on submerged breakwater sections of rectangular, triangular and trapezoidal section. The effects of the relative breakwater crest width, B/L , and relative freeboard, R_c/d_s , on the wave transmission are presented, but no general prediction method is derived. The authors conclude that the permeability of a submerged breakwater has relatively little effect on the wave transmission characteristics of such a structure. Bade & Kaldenhoff⁽²⁸⁾ studied the transmission of short sequences of irregular waves over a cube armoured trapezoidal breakwater of crest freeboard, R_c/d_s from -0.1 to $+0.1$. The authors present graphs of the transmission coefficient K_t against a dimensionless freeboard, $1 - R_c/H_s$, but concentrate primarily on the different transmission characteristics of the wave groups used. Ouellet & Eubanks⁽²⁹⁾ also used irregular waves against a trapezoidal rubble breakwater, armoured with dolos on the seaward face. A range of water levels were used, all below the structure crest level. Values of the transmission coefficient, K_t , are plotted against frequency. In general, however, the results of this study are presented in an unconventional manner, and it is not possible simply to generalise the results.

Allsop⁽³⁰⁾ considers a series of multi-layer rock armoured breakwaters of conventional form with crest

freeboards in the range, $R_c/d_s = 0.23$ to 0.56 , subjected to random wave attack. Measurements of the number of waves overtopping, N , and the transmission coefficient, K_t , are presented against values of dimensionless freeboard R_c/H_s and R^* , where:-

$$R^* = \frac{R_c}{H_s} \left(\frac{g}{2\pi} \right)^{\frac{1}{2}} \quad (2.14)$$

Allsop concludes that N may be described by a function of R_c/H_s , but that K_t is better described by R^* . Expressions suggested by regular wave tests are tried, but are found to underpredict N and K_t at low values of relative freeboard.

2.5 Wave transmission through porous structures

The transmission of wave energy through the voids of rubble mound breakwaters has been studied both experimentally and analytically. Some researchers have sought to derive mathematical models using linearized formulae for viscous drag, calibrating the empirical expressions against physical model results, and then re-running the mathematical model for prototype conditions. Mathematical models have been presented by Madsen and co-authors^(4,24,25,26), Kondo et al⁽¹⁰⁾, Iijima et al⁽¹¹⁾, Seelig⁽³³⁾ and Massel & Butowski⁽³⁴⁾. The results of physical model tests are presented by Madsen & White⁽⁴⁾, Seelig⁽³³⁾ and Kondo et al⁽¹⁰⁾. Measurements of transmission through a prototype breakwater are presented by Thornton & Calhoun⁽⁸⁾.

Madsen & White⁽⁴⁾ describe the derivation of, and Seelig⁽⁵⁾ documents and lists, a computer program to estimate wave transmission, and reflection, at a porous rubble breakwater. The method models transmission of long waves through and reflection from a porous structure in two stages. Energy dissipated and reflected at the seaward face is considered in the first stage. The second considers the energy dissipated in viscous drag in the flow through the voids of the structure. Simple empirical relationships tested against model test results are used to estimate energy losses. In their derivation, Madsen & White only consider regular waves, having height, H , and period T . Seelig⁽⁵⁾ however suggests that the method may be used to estimate the performance of structures subjected to random waves by setting $H=\bar{H}$ and $T=T_p$.

In later work, Madsen et al⁽²⁶⁾ attempt to analyse the flow at the seaward face of a trapezoidal breakwater in a less artificial manner in order to refine the estimation of wave reflections. They conclude however that neither the new method⁽²⁶⁾, nor the previous method⁽⁴⁾, estimate the level of reflections well, although both give relatively good estimates of

transmission.

Ijima et al⁽¹¹⁾ describe an analytical method for the estimation of wave reflection and transmission at various structures. Work with triangular and rectangular section porous breakwaters is presented very briefly. The prediction method is claimed to work well for rectangular sections, but less well for triangular, and therefore trapezoidal, structures. Massel & Butowski⁽³⁴⁾ use arbitrary wave spectra and rectangular porous breakwaters. Using a similar argument to that presented by Hydraulics Research⁽²²⁾, random waves are treated as the sum of small amplitude periodic waves, each of which may be treated linearly. The fluid damping in flow through the porous structure is represented by a linear term, approximating turbulent friction. Values for transmission and reflection are determined by integrating over the spectrum. The method is tested against the results measured at Monterey breakwater, California by Thornton & Calhoun⁽⁸⁾. Agreement with the total level of energy dissipation is reasonable, but the division between reflection and transmission is not well described.

Kondo et al⁽¹⁰⁾ present model test results demonstrating the influence of core permeability, and position within the structure, on the reflection and transmission performance. The results are again compared with those calculated by an analytical approach using a linearized friction loss method.

2.6 Rock armour stability

Low-crest breakwaters, designed to allow some overtopping, may have a stricter design criterion, with regard to the stability of the primary armour layers than breakwaters which do not overtop. A number of authors (Lording and Scott⁽³⁵⁾, Raichlen⁽³⁶⁾ and Lillevang⁽³⁷⁾) have noted that the armour on the back face of a low-crest breakwater is more likely to be displaced by heavy overtopping than the armour on the seaward face. Allsop⁽³⁰⁾ concludes that the total damage to a low-crest rock armoured breakwater attacked by waves of steepness $H_s/L_p > 0.03$ is dependent upon the stability number N_s , but not upon the freeboard. Furthermore damage to the back face of such a breakwater is best described by a dimensionless freeboard parameter, R^* .

Ahrens⁽³⁸⁾ relates low-crest breakwater damage to the stability number, N_s . He concludes that there is a wave period effect with damage increasing with increasing T_p . Furthermore he suggests that this wave period effect might be accounted for by the use of a modified stability number, N_s^* , which includes a measure of the wave length.

Little other stability testing of low-crest breakwaters in random waves has been documented.

Indeed, most designers seem to rely on stability formulae and coefficients derived from regular wave tests only.

3 WAVE TRANSMISSION

3.1 General

As discussed previously there are two basic modes of wave transmission for surface piercing, permeable low-crest breakwaters, that is, transmission through and transmission by overtopping. In the case of submerged permeable structures shorter waves will propagate above the breakwater while longer waves pass partly above the structure and partly through it. Thus there is a third mode of wave transmission, that is, transmission above a submerged structure. The relative proportions of any of these modes of transmission are dependant upon the relative freeboard, the breakwater permeability, the water depth and the wave period.

The coefficients of transmission by overtopping, K_{to} , and through the structure, K_{tt} , have been defined in Section 2.1. However, K_{to} and K_{tt} , as defined, have a basic failing in that they are determined solely by the ratio of transmitted to incident wave height. They cannot therefore account for any frequency shifts which may occur as waves are transmitted through or over the structure. To incorporate such information in a simple transmission coefficient may well require the coefficient to be defined in terms of frequency, as are the incident and transmitted wave spectra. This may however be over-complex given the present state of understanding of the hydrodynamics.

3.2 Wave transmission characteristics of low-crest breakwaters

The data used in this study was obtained from a number of different sources and consequently a wide range of breakwater constructions have been investigated. The cross-sections of these breakwaters are shown in Figures 3.1 to 3.3. All are of the low-crest type and both surface piercing and submerged structures are included.

The transmission performance data from reference 5 and references 30 and 39 is presented in Figures 3.4 and 3.5 respectively, in terms of K_t and the dimensionless parameter R_c/H_s ; R_c is the breakwater freeboard and may be either positive (surface piercing) or negative (submerged). As might be expected the overall trend is one of decreasing wave

transmission with increasing freeboard. For values of $R_c/H_s < 1.0$, Figure 3.5 suggests slightly higher levels of wave transmission than those given by Figure 3.4. However this discrepancy may be ascribed to the different permeabilities of the breakwaters tested. Where the breakwaters are more permeable, as in the case of Figure 3.5, the increased transmission through the structure will result in an increase in the overall value of K_t . It might also be expected that this effect would account for much of the data scatter apparent in Figure 3.4.

The Shore Protection Manual⁽⁹⁾ contains a curve for the prediction of wave transmission by overtopping only. This curve, when re-worked and plotted in Figures 3.4 and 3.5, provides qualitative agreement with the general trend. It also serves to emphasise that for $0.0 < R_c/H_s < 1.5$, overtopping is the major mode of wave transmission. The upper limit to this range will however vary from breakwater to breakwater depending on the attenuation to wave run-up afforded by the primary armour layers. For further details of run-up performance the interested reader is referred to work by Allsop, Hawkes, Jackson and Franco⁽³²⁾.

For surface piercing structures with $R_c/H_s > 1.5$, the qualitative agreement between Figures 3.4 and 3.5 ceases, with Seelig's⁽⁵⁾ data showing an increase in K_t while Allsop's⁽³⁰⁾ data suggests a small, but constant, level of wave transmission, presumably through the structure. The problem is that for relatively high breakwaters transmission is no longer dominated by wave overtopping but by energy transmission through the structure, which is a function of, amongst other things, wave steepness. It is this changing role in the dependence of K_t on H_s which causes the paradoxical trend in Seelig's data. Thus, Figure 3.4 does not imply that for a fixed incident wave height the transmitted height will increase if the freeboard is increased, but rather that with a fixed freeboard the transmission coefficient will increase if the incident wave period is increased (or if the wave height is reduced and the waves become less steep). A similar trend should be expected in Figure 3.5 but, owing to the lower permeability of the breakwater modelled by Allsop, its onset may be considerably delayed.

Owen⁽⁴⁰⁾ suggests that the overtopping performance of seawalls can be expressed in terms of a dimensionless discharge, Q^* , and a dimensionless freeboard, R^* , where

$$R^* = \frac{R_c}{T_z (g H_s)^{1/2}} \quad (3.1)$$

and T_z is the zero crossing wave period.

T_z is however a less desirable measure of periodicity than T_p , the period of maximum spectral energy, due to it being difficult to assess from wave spectra, and because it may vary with the frequency shifts occurring as a result of wave transformations at the breakwater. T_p , on the other hand, is relatively unaffected by wave transformations. Furthermore its use allows comparisons to be drawn with published American results⁽⁵⁾.

The dimensionless freeboard has therefore been re-defined as:-

$$R^{*'} = \frac{R_c}{T_p (gH_s)^{\frac{1}{2}}} \quad (3.2)$$

The physical significance of which is perhaps best appreciated if equation 3.2 is re-written in the form:-

$$R^{*'} = \frac{R_c}{H_s} \left(\frac{S}{2\pi} \right)^{\frac{1}{2}} \quad (3.3)$$

where S is the wave steepness corresponding to T_p .

Equation 3.3 implies that for waves of constant steepness, $R^{*'}$ is simply related to the ratio of structure freeboard to significant wave height.

Figures 3.6 and 3.7 present the transmission performance data from Seelig⁽⁵⁾ and HR⁽³⁹⁾, in terms of K_t and $R^{*'}$. Both figures exhibit a trend very similar to that given by previous plots of K_t against R_c/H_s . However the upward trend of K_t for large values of R_c/H_s is nullified in Figures 3.6 and 3.7 by the inclusion of the wave steepness in the parameter, $R^{*'}$. Furthermore, it appears that the use of $R^{*'}$ as the freeboard parameter may result in a slight reduction in the scatter of data, perhaps due to the inclusion of wave period effects within the parameter.

3.3 Effect of wave period

It may be assumed that the influence of wave period will differ for both submerged and surface piercing breakwaters. For permeable surface piercing structures K_t will increase with increasing wave period due to the longer waves propagating more freely through and over the structure. For submerged breakwaters, short period waves should pass almost unhindered over the structure (depending on the

freeboard), while longer period waves, which propagate deeper in the water, will be partially attenuated. However, no matter how permeable the breakwater is, the efficiency of wave transmission through it will never be 100%.

The changing influence of wave period on the transmission characteristics of low crested breakwaters is illustrated in Figures 3.8 and 3.9. Both figures are purely qualitative and no reliance should be placed on the absolute values they suggest. However the reversal of the wave period effect is clear, occurring at $R_c/d_s \approx 0.1$ for data from reference 39 and $R_c/d_s \approx -0.35$ for data from reference 5. The relative depth of submergence, R_c/d_s , at which this reversal occurs will probably depend upon the relative permeability of the breakwater to the wave periods being considered.

One further point that arises from Figures 3.8 and 3.9 is the apparent discontinuity of the general trend at $R_c/d_s = -0.05$, ie 5% submergence. Had such an effect occurred in just one of the figures it might well have been discounted as an anomaly, but for the same effect to occur at exactly the same point in both figures seems to be more than sheer coincidence. Indeed, retrospective analysis of Figures 3.4 to 3.7 reveals a similar trend within the scatter. The explanation for this phenomenon possibly lies with the mechanisms responsible for wave transmission, or more precisely with the interaction of the mechanism of wave transmission above a submerged structure and the mechanism of transmission over a surface piercing structure. It may be postulated that transmission by overtopping commences not at the still water level, but at the level at which the breakwater crest is first exposed. This level is determined by the incident wave conditions. Thus, the discontinuity in the overall trend may result from the superimposition, between the still water level and some critical level determined by the wave conditions, of the rapidly declining coefficient of transmission (wave transformation effect) above the structure and the comparatively large coefficient of transmission by overtopping.

This argument is lent credence by the theoretical considerations of Lamb⁽²¹⁾ concerning the reflection and transmission characteristics of shallow water waves ($L \gg d$) propagating over a submerged step. From these considerations Lamb derived reflection and transmission coefficients in terms of the wave celerity, C , where,

$$K_r = \frac{C_2 - C_1}{C_2 + C_1} \quad (3.4)$$

$$\text{and } K_t = \frac{2C_2}{C_2 + C_1} \quad (3.5)$$

For the shallow water approximation:-

$$C_{1,2} = (g d_{1,2})^{\frac{1}{2}} \quad (3.6)$$

The subscripts 1 and 2 refer to the wave celerities and water depths above and beyond the step respectively. Combining equations 3.5 and 3.6 yields an expression for the transmission coefficient in terms of the water depth only:-

$$K_t = \frac{2d_2^{\frac{1}{2}}}{d_2^{\frac{1}{2}} + d_1^{\frac{1}{2}}} \quad (3.7)$$

This expression has been plotted in Figures 3.8 and 3.9. Equation 3.7 effectively represents the transmission of wave energy above a submerged breakwater. The resulting curve agrees remarkably well with the general trend but, more importantly, it also passes almost directly through the point of discontinuity. As such Lamb's curve would appear to confirm the hypothesis previously suggested.

Further research into this phenomenon is however still required in order to;

- (a) Confirm the phenomenon
- (b) Substantiate the explanation given above
- (c) Assess the effect of this small 'window' in the general transmission trend on the optimum cost/benefit design of submerged breakwaters.

3.4 Effect of breakwater porosity

The porosity of a permeable breakwater may be defined as the ratio of the volume of voids within the breakwater to the total breakwater volume. As such, the porosity, together with the incident wave conditions, will determine the level of wave transmission through the structure. This scenario may however be complicated, particularly for low permeability structures, by the additional effects of void shape and the tortuosity of the flow path through the breakwater. There is however virtually no data available for these latter two effects. Similarly, there is very little published data concerning the effects of breakwater porosity on the transmission of wave energy.

Seelig⁽⁵⁾ presents a plot of wave transmission through a rubble mound breakwater as a function of wave steepness for different breakwater porosities. This graph is reproduced in Figure 3.10 and is based on transmission coefficients predicted by the computer program of Madsen and White⁽⁴⁾, for regular waves. The most important features of the graph are that:

1. The predicted transmission coefficient increases with decreasing wave steepness, and
2. The change in the predicted value of K_{tt} for a given change in porosity is greatest for waves of small steepness, i.e. long waves.

These two effects are likely to be true for both submerged and surface piercing structures.

Dattatri, Raman and Shankar⁽²⁷⁾ studied the effect of breakwater porosity on the transmission of wave energy past a submerged breakwater. Unfortunately the only data readily available relates to rectangular structures under regular waves. Nevertheless this data for breakwaters of porosity 0%, 41% and 42%, can be presented in terms of K_t and R_c/H - Figure 3.11.

The general trend agrees favourably with that of Figures 3.4 and 3.5. Dattatri et al's data also suggest however that there may be a non-linear increase in K_t with porosity, such that K_t is far more sensitive to porosity over a range of, say, $0.4 < P < 0.5$ than it is over a range of $0 < P < 0.2$. This sensitivity would appear to be heightened as R_c/H decreases.

The results presented by HR⁽³⁹⁾ (Figure 3.5) also relate to a breakwater of porosity 0.4. However, transmission coefficients for this breakwater are considerably higher than those given by Dattatri et al for breakwaters of comparable porosity. This is probably almost entirely due to the differing ranges of breakwater crest widths, B , relative to wave length, L , used in the two studies. Dattatri et al used $0.08 < B/L < 0.32$ while HR used $0.03 < B/L < 0.07$. Naturally, as the value of B/L increases the transmission through the structure will decrease, hence the lower values of K_t for Dattatri et al's results. The different breakwater shapes and wave conditions used in the two studies may also in part contribute towards the different values obtained for K_t .

For surface piercing structures, porosity may also act to restrict transmission by overtopping through the attenuation of wave run-up. The greater the porosity and roughness of the breakwater armour layer, the greater the attenuation of this run-up, and hence the

reduction of wave transmission by overtopping.

Due to the paucity of detailed information, regarding the effect of structure porosity on the transmission performance of low crest breakwaters, it is almost inevitable that further research will be required if realistic design guidelines are to be produced. However, it appears that a short, but well planned, series of model tests may provide sufficient information to allow the confident prediction of this transmission performance.

3.5 Effect of breakwater crest width

Results obtained in the previous section have indicated that the transmission through a porous breakwater may be reduced by increasing the crest width. It is logical to suppose that the transmission by overtopping may similarly be reduced by an increase in crest width. The problem arises in determining the size of any subsequent reduction in K_t for a given increase in crest width. Moreover, such an increase may not be cost effective.

At present there is little or no data available on the quantitative effect of increasing the breakwater crest width for surface piercing structures. However some, limited, data is available from recent studies at Hydraulics Research⁽³⁹⁾ for submerged structures. This data is presented, in terms of K_t and the dimensionless freeboard R^* , in Figure 3.12. Surprisingly the results indicate that increasing the breakwater crest width by 55% (equivalent to a change in crest width from 4.5 m to 7.0 m prototype) yields at best only a 10% reduction in wave transmission. This reduction comes about by reduced transmission through the structure and increased friction losses across the crest.

Dattatri, Raman and Shakar⁽²⁷⁾ also present data for crest width effects on wave transmission passed submerged structures. These results are plotted in terms of K_t and R_c/d in Figure 3.13. Unfortunately there is insufficient information available to allow a direct comparison with the results from HR⁽³⁹⁾. However Dattatri et al's results do appear to confirm the previous findings, with a 400% increase in crest width resulting in a reduction in K_t by a factor of only 0.2. This reduction factor appears to be fairly constant over the range of structure heights considered. Dattatri et al also suggest that the crest width influences the transformation of wave energy by prompting wave breaking above the structure. They conclude that any increase in the crest width over the minimum necessary to trigger breaking, is

unlikely to have any significant influence on the transmission characteristics. However, it should be emphasised that Dattatri et al considered a totally impractical range of relative crest widths, B/L , ($0.08 < B/L < 0.32$) from the point of view of prototype constructions.

4 BREAKWATER STABILITY

4.1 General

This report has so far been concerned mainly with the wave transmission characteristics of low crest breakwaters. It is however, worth remembering that these breakwaters will only continue to function as required whilst they are relatively undamaged by wave action. If the breakwater is unstable under the design wave conditions its performance in respect of, amongst other things, wave transmission, will be impaired. Consequently higher levels of wave activity may occur in the lee of the breakwater than allowed for. The stability of the breakwater crest is therefore of particular importance.

The stability data used in this study was taken principally from Allsop⁽³⁰⁾ and Ahrens^(38,41). Although the stability of a rubble mound breakwater is usually described by the "zero damage" wave height or sea state, it is clear that in random waves some small armour movement is possible at comparatively low sea states. It is, therefore, more useful to determine the damage behaviour of the structure over a range of wave heights, or sea states. In this study damage has been defined as the number of units extracted from their original position, Δ , expressed as a percentage of the total number of armour units, N_a .

4.2 Hudson stability number

Hudson and Davidson⁽⁴²⁾ concluded from the results of tests in regular waves with no overtopping that the stability of rubble mound breakwaters is a function of a dimensionless stability number, N_s ,

$$N_s = \frac{H_s}{\left(\frac{W_{50}}{\rho_r}\right)^{1/3} \left(\frac{\rho_r}{\rho_w} - 1\right)} \quad (4.1)$$

where W_{50} is the median armour stone weight
 ρ_r is the density of armour stone
 ρ_w is the density of water.

In effect N_s is a dimensionless wave height and as such does not contain a wave period or sea steepness term. Furthermore its derivation in tests that

allowed no overtopping suggest that it may overestimate the median armour weight required for front face stability on low-crest breakwaters.

Back face damage is considered by Allsop⁽³⁰⁾ to be dependent more upon the overtopping discharge and hence upon R^* , than on any stability number. However a comparison of the data for back and front face damage, as presented by Allsop, reveals that for long waves (Figure 4.1) damage to both faces is equally well described by N_s . Conversely, for short waves (Figure 4.2), N_s is a poor descriptor of the damage to both faces. Figures 4.1 and 4.2 also show very similar damage levels for both faces as a function of N_s . This suggests that one particular design value for N_s may be applicable to both the front and back face of a breakwater.

Figures 4.3 and 4.4 present damage, Δ/N_a , in terms of N_s for various relative freeboards R_c/d . The overall trend is that of an increasing number of stones being extracted as the wave attack becomes more severe. Although the stability number, N_s , would seem to offer a reasonable explanation of the damage to the structures, closer inspection of the data in each of the figures suggest that there is a wave period effect; damage increasing with increasing values of T_p , all other factors being equal.

An exponential regression analysis may be performed to fit curves of the following form to the data:-

$$\frac{\Delta}{N_a} = A \exp(B \cdot N_s) \quad (4.2)$$

where A and B are empirically derived coefficients. The results of such an analysis are summarised below:-

DATA	R_c/d	Coefficients		Correlation r^2
		A	B	
Ref (38)	0.0	0.23	0.86	0.90
Ref (41)	0.2	0.32	0.89	0.88
Ref (38)	0.4	0.21	1.19	0.62
Ref (30), Long Wave	0.29			
Ref (30), Long Wave	0.39	0.028	2.25	0.74
Ref (30), Long Wave	0.57			
Ref (30), Short Wave	0.23			
Ref (30), Short Wave	0.38	0.008	2.31	0.56
Ref (30), Short Wave	0.57			

Comparison of the resulting curves for the data of Ahrens^(38,41) and Allsop⁽³⁰⁾ is not strictly valid owing to the different breakwater constructions tested by the two authors. Ahrens used homogeneous surface

piercing rubble breakwaters constructed from a rock size sufficiently small that the structure would collapse under wave action, eventually forming a stable submerged mound. Allsop, on the other hand, used a multilayered surface piercing rubble mound breakwater that was designed so as to minimise damage. The use of the damage parameter Δ/Na , as defined, implies that damage for Ahrens homogeneous breakwaters is a volumetric measure whilst that for Allsop's breakwaters is effectively a surface area measure. Thus Allsop's damage level will, by definition, be much lower. Indeed the damage levels measured by Allsop are so small that the extrapolated trends may not be significant.

In conclusion the trend of increasing damage with increasing stability number, N_s , is reasonably described by curves of the form $\Delta/Na = A \exp(BN_s)$, however it is difficult to envisage such curves being universally applicable to all types of breakwater construction. Furthermore, use of the Hudson stability number tends to introduce a wave period effect, which increases the separation of the data.

4.3 Spectral stability number

In an attempt to account for the wave period effect apparent in plots of the Hudson stability number, Ahrens⁽³⁸⁾ suggested a modified stability parameter the spectral stability number, N_s^* , where,

$$N_s^* = \frac{(H_s^2 L_p)^{1/3}}{\left(\frac{W_{50}}{\rho_r}\right)^{1/3} \left(\frac{\rho_r}{\rho_w} - 1\right)} \quad (4.3)$$

and L_p is the wavelength corresponding to T_p .

A similarly modified parameter had previously been mooted by Gravesen et al⁽⁴³⁾, based on model studies of breakwaters which, similar to Hudson's⁽⁴²⁾, did not overtop. Again, therefore, this new stability number, N_s^* , may overestimate the weight of armour stones required for front face stability, on breakwaters that are designed to overtop.

Figures 4.5 and 4.6 present plots of damage, Δ/Na , against N_s^* for the various sets of data. The trends are very similar to those observed with the Hudson stability number but there is now no apparent dependence on T_p for Ahrens data. Allsop's data, however, still exhibits a wave period effect; damage occurring more rapidly under wave spectra with a greater T_p . It is interesting to note that Allsop⁽³⁰⁾ distinguishes between front and back face damage, though total combined damage has been plotted

in this study. Moreover Allsop's results indicate that back face damage is generally greater than front face damage, for the longer wave periods, while the trend is reversed for the shorter wave periods. This implies that the apparent wave period effect is in reality a result of the increased overtopping, which occurs under the longer waves. This leads to significantly increased back face damage and hence to disproportionately higher levels of total damage. It seems unlikely therefore that a stability number, specifically derived for the case of no overtopping, could be used to adequately account for the stability of low crest breakwaters over a realistic range of wave periods. By virtue of their design, Ahrens breakwaters were subjected to only a very short duration of overtopping before they retreated below the water level. It is not surprising therefore that the wave period effect is not apparent in Ahrens results when plotted in terms of N_s^* .

An exponential regression analysis to fit curves of the form given by equation 4.2 has been carried out. The results are summarised below:-

DATA	R_c/d	Coefficients		Correlation
		A	B	r^2
Ref (38)	0.0	0.152	0.31	0.75
Ref (41)	0.2	0.168	0.33	0.88
Ref (38)	0.4	0.048	0.05	0.77
Ref (30), Long Wave	0.29	0.0007	1.66	0.98
Ref (30), Long Wave	0.39	0.0018	1.58	0.96
Ref (30), Long Wave	0.57	0.0009	1.92	0.95
Ref (30), Short Wave	0.23			
Ref (30), Short Wave	0.38	0.0059	1.07	0.57
Ref (30), Short Wave	0.57			

On average there is little difference between the correlation coefficients for Hudson's stability number and the spectral stability number.

It is interesting to note however that the trends for R_c/d dependency in Figures 4.5 and 4.6 are qualitatively similar for both Allsop's⁽³⁰⁾ long wave regime and Ahrens data. A comparison of the range of wave steepnesses used in both studies reveals that,

1. for Allsop's long wave regime, $H_s/L_p < 0.03$
2. for Allsop's short wave regime, $H_s/L_p > 0.03$
3. for Ahrens data, $0.0012 < H_s/L_p < 0.036$

In other words, Ahrens wave conditions bear closest resemblance to Allsop's long wave regime. This suggests that the apparent dependence on relative freeboard, is characteristic of long waves ($H_s/L_p < 0.03$). The trend of increasing damage with increasing

values of R_c/d may therefore be partly due to the effect of the water depth, d , on the shoaling of the longer waves.

5 DESIGN GUIDELINES

5.1 Estimation of wave transmission

It is clear from the preceding discussions that the wave transmission characteristics of low-crest breakwaters are primarily determined by:-

- (a) the breakwater freeboard,
- (b) the incident wave conditions, and
- (c) the breakwater porosity.

The above factors may be adequately represented by a graph of K_t against dimensionless freeboard (R_c/H_s or R^*), for various values of breakwater porosity. In its ultimate form such a graph would comprise a series of curves each relating to a particular value of porosity. Unfortunately, there is at present only sufficient random wave data available to enable one curve, that for structures with a porosity of 40% ($P = 0.4$), to be plotted. This curve is presented in Figures 5.1 and 5.2 together with the envelope of wave transmission coefficients covering the range of realistic breakwater porosities (say $0 < P < 0.5$). The upper bound of this envelope represents relatively permeable structures and the lower bound relatively impermeable structures.

Given the 40% porosity curve and the boundary conditions it may be possible to estimate from Figures 5.1 and 5.2 the overall transmission coefficient, K_t , for any particular breakwater porosity. However it should be recognised that for a particular value of the dimensionless freeboard, K_t may not increase linearly with increasing porosity. Thus, the accuracy of any estimate must be uncertain, until such time as sufficient data is available to allow more porosity curves to be plotted. For surface piercing structures it is recommended that Figure 5.2 be used to obtain the transmission coefficients owing to the reduced range of K_t over that of Figure 5.1.

It is clear that the determination of the porosity of a breakwater is of particular importance if accurate values of K_t are to be obtained. This may lead to problems particularly if the breakwater is of multilayered construction, with an armour layer, filter layers and core. In such a case, if K_{tt} is predominant, then the required porosity ought to be that of the least permeable material, probably the core. If transmission by overtopping, K_{to} , dominates then the porosity chosen may be that of the primary

armour layers. Ultimately the choice of porosity used must lie with the designer.

5.2 Estimation of stability

The present study has demonstrated that the stability of the primary armour layers of low-crest breakwaters cannot, as yet, be adequately represented by a single simple design graph. However it is clear that if the structure has been designed to provide a certain level of wave attenuation, then the level of damage which that structure can sustain whilst still providing the required degree of wave attenuation will be very limited. This is particularly true if the damage should occur at the crest.

For structures similar to those of Allsop⁽³⁰⁾ and Ahrens⁽³⁸⁾, Figures 4.3 and 4.6 may be used to obtain a rough estimate of N_s or N_s^* for the permissible level of damage selected. For other structures N_s or N_s^* may be estimated by approximating that structure to those used by Allsop and Ahrens. It should however be noted that both N_s and N_s^* may overestimate the weight of armour required for front face stability. By the same token, they may underestimate the weight of crest and back slope armour required to resist overtopping forces.

Clearly there are still considerable uncertainties involved in selecting a suitable armour weight to ensure stability of a low crest breakwater under the design wave conditions. At present these uncertainties may only be satisfactorily resolved by physical model testing of the stability aspects of the breakwater design. It is therefore strongly recommended that all designs be finalised by physical model testing.

6 CONCLUSIONS AND RECOMMENDATIONS

The aim of this study has been the production of a design methodology for estimating the wave transmission coefficients of low crest breakwaters. This methodology is graphically illustrated in Figures 5.1 and 5.2. Further research is however required to improve the accuracy and applicability of these design graphs. In particular it is recommended that research be conducted into the influence of breakwater porosity on the wave transmission coefficient.

On the basis of this study it may be concluded that the wave transmission characteristics of low crest breakwaters are primarily dependent upon the incident wave conditions, the breakwater freeboard and the breakwater porosity. Other factors have, however, been considered. These factors include the breakwater

crest width and the effect of wave period. For submerged structures, increasing the crest width, above that needed for stability, has little effect on the wave transmission. It is therefore unlikely to be a cost effective method of improving the wave attenuation capability of such a breakwater. For surface piercing structures an increase in crest width may have a significant effect on the level of wave transmission.

The influence of wave period has proved difficult to quantify, with several anomalies apparent within the general trend. These anomalies suggest that there may be an optimum freeboard for a submerged breakwater at which maximum wave attenuation is achieved for minimum cost. However, further research is required to substantiate this hypothesis.

The stability of low crest breakwaters has been considered in relation to both the Hudson's stability number and the spectral stability number. It has been concluded that, although the spectral stability number may adequately account for wave period, neither of these two parameters is likely to form a satisfactory basis for a comprehensive design method. Further research into the mechanisms responsible for front and back face damage is required, and until these mechanisms are fully understood low-crest breakwater designs can only be finalised by physical model tests.

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Figures

Fig 3.1 Breakwater cross-sections - Reference 5

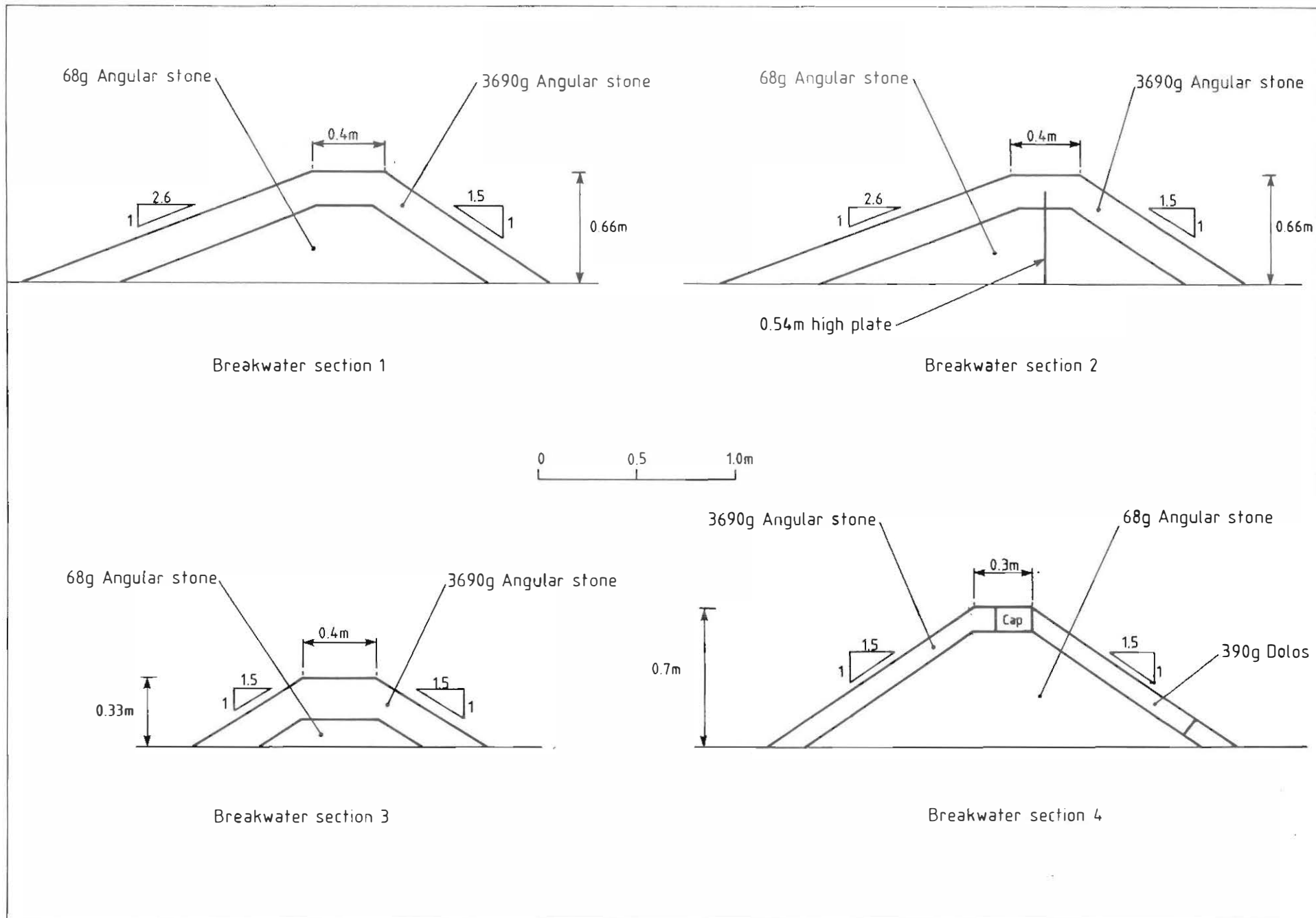
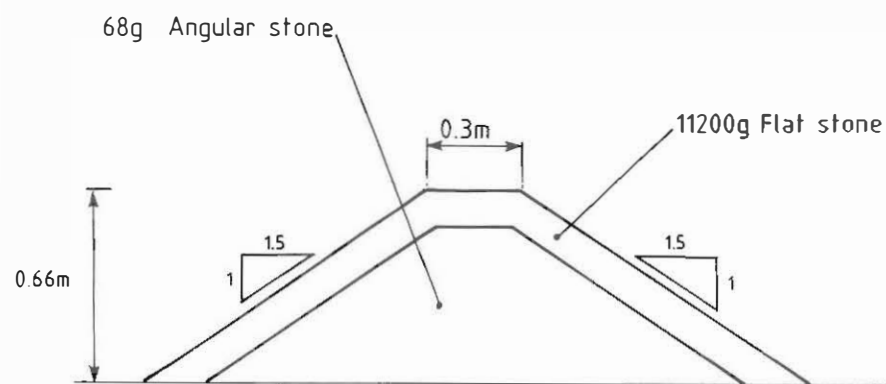
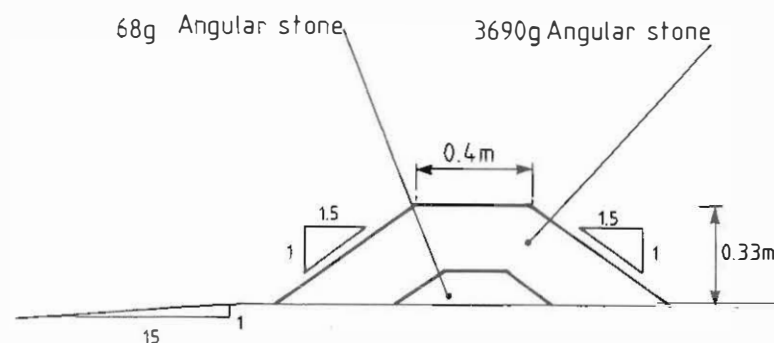


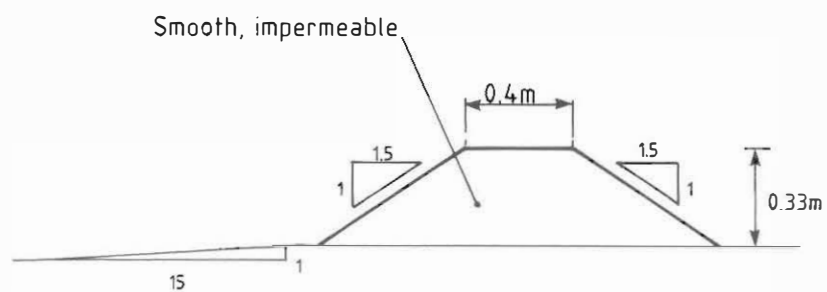
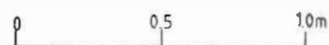
Fig 3.2 Breakwater cross-sections - Reference 5



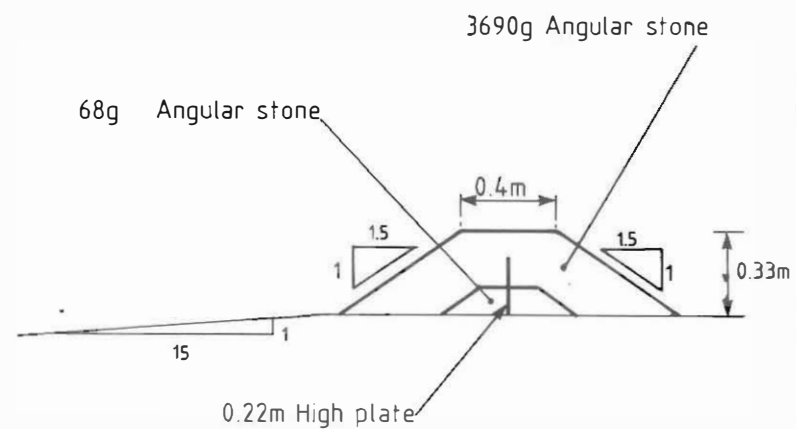
Breakwater section 5



Breakwater section 6 & 8

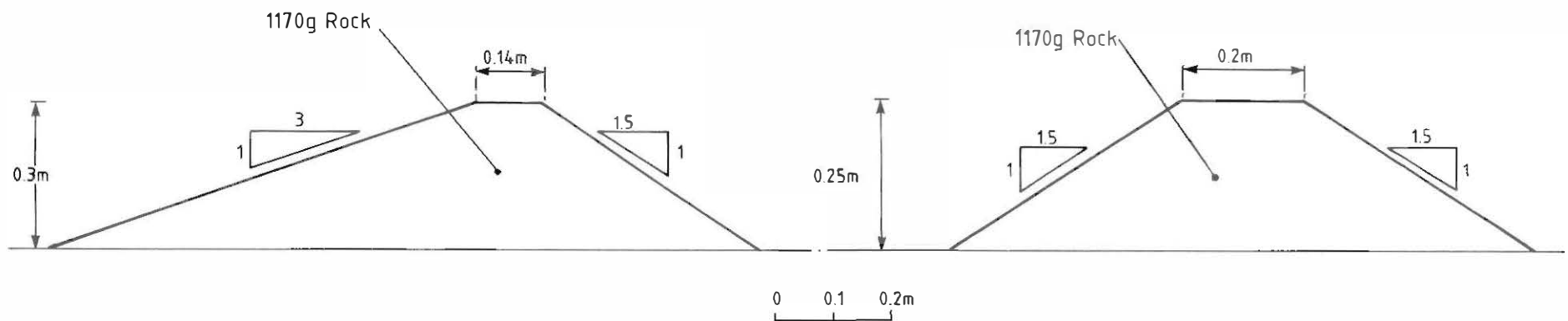


Breakwater section 7



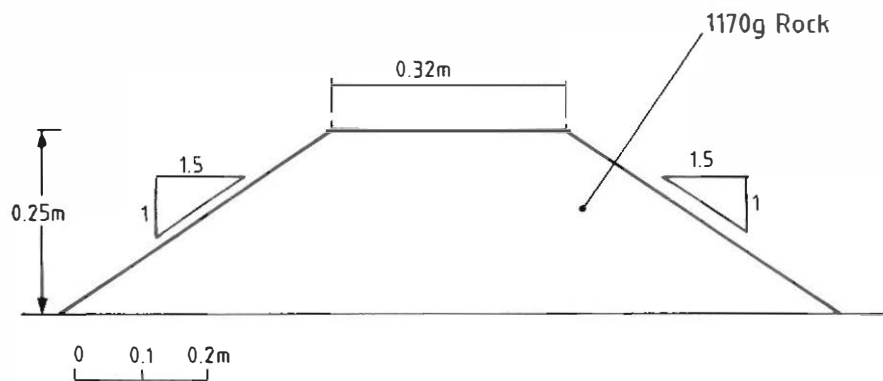
Breakwater section 9

Fig 3.3 Breakwater cross-sections - References 30 & 39

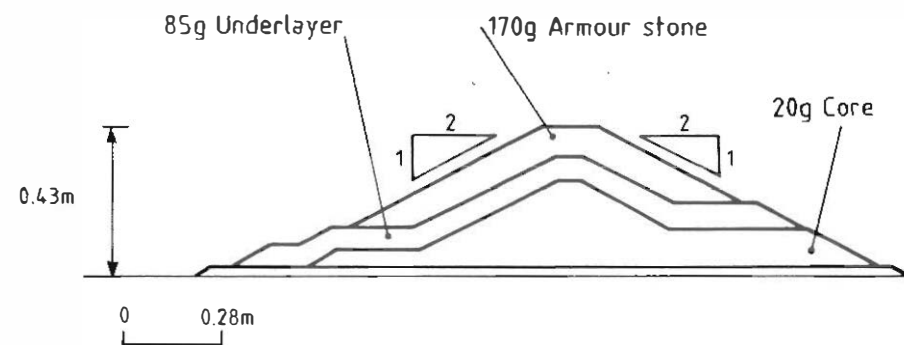


Breakwater section 1 (Ref 39)

Breakwater section 2 (Ref 39)



Breakwater section 3 (Ref 39)



Breakwater section (Ref 30)

Fig 3.4 Wave transmission data - Reference 5

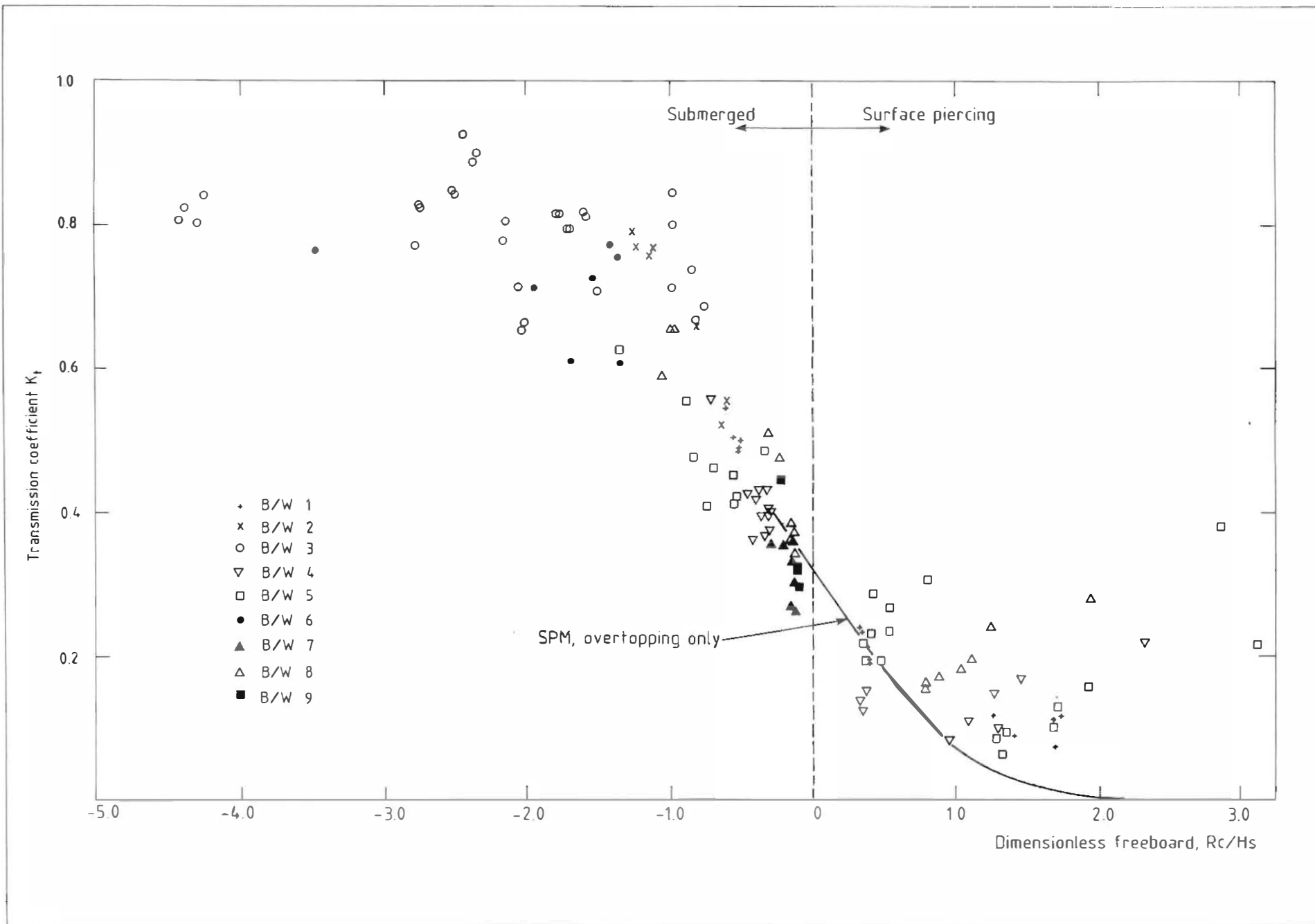
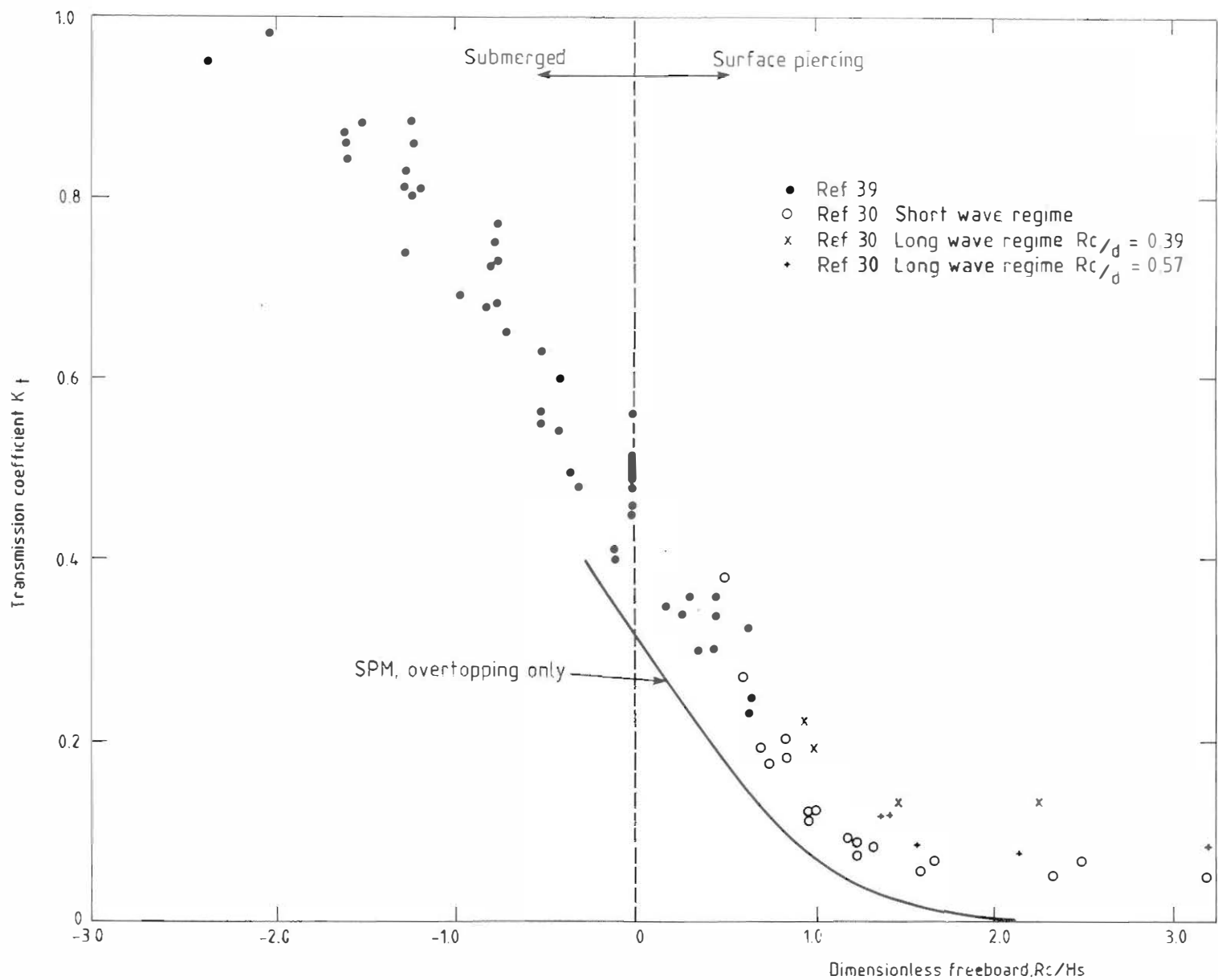


Fig 3.5 Wave transmission data - Reference 30 and 39



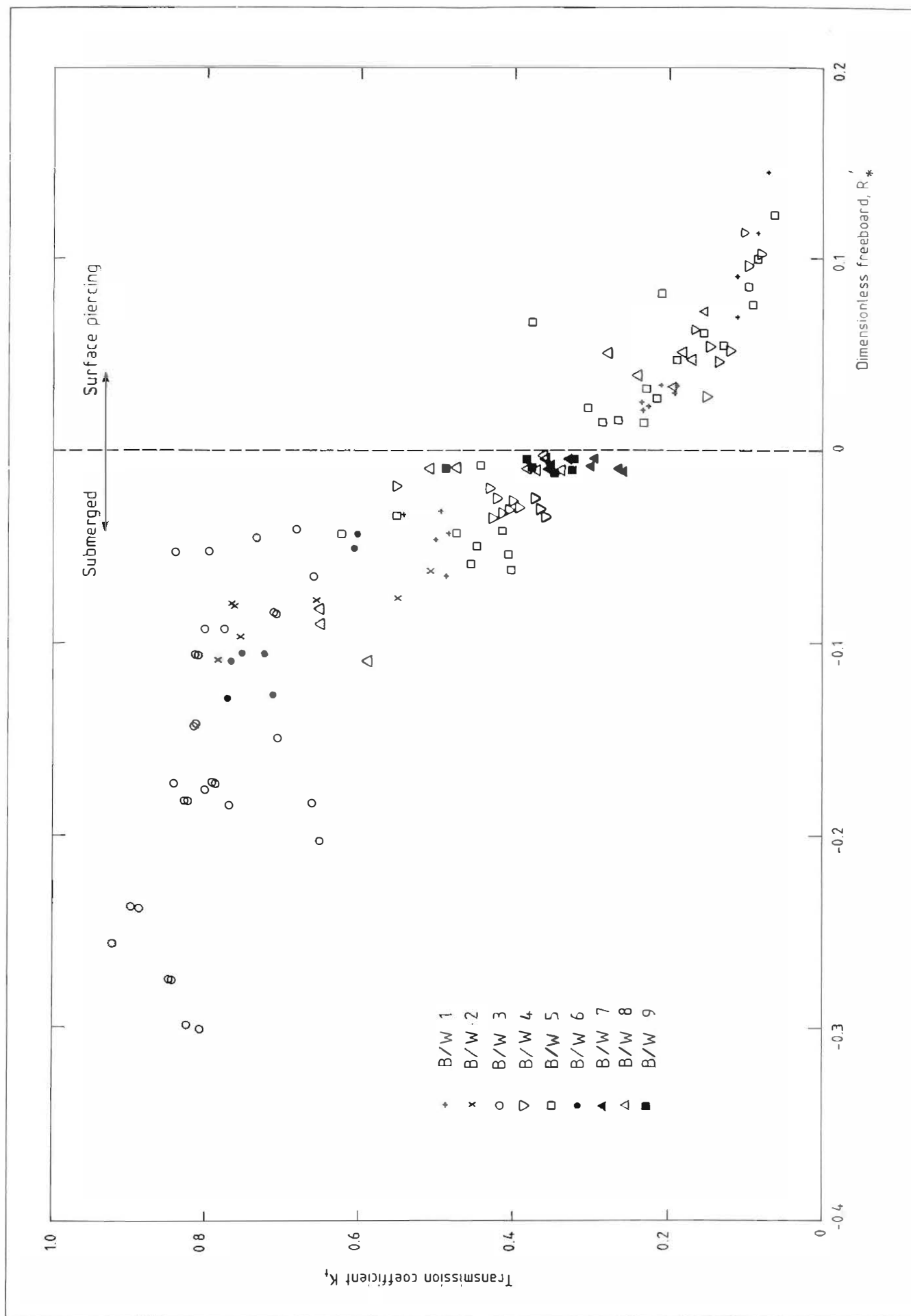
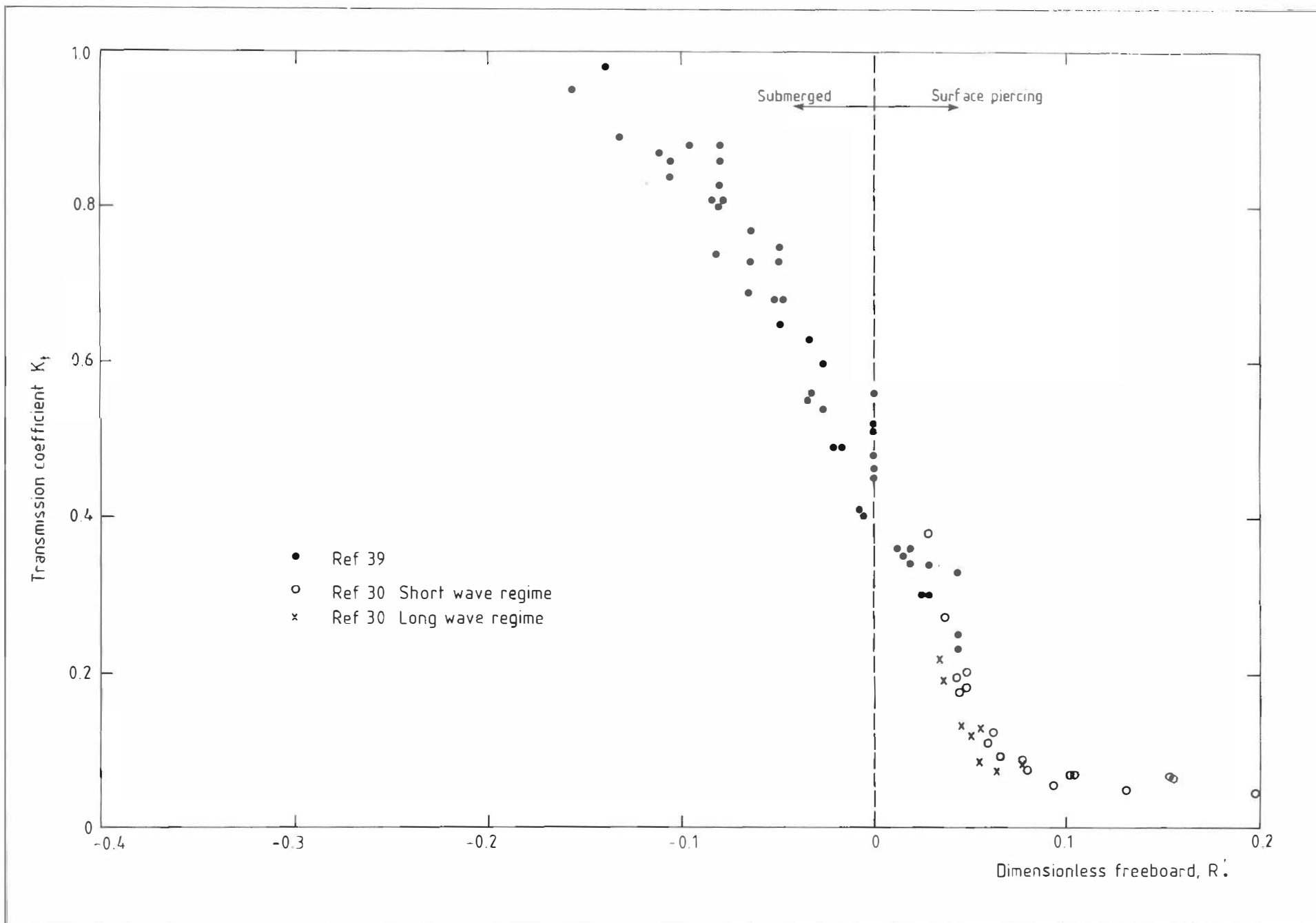


Fig 3.6 Wave transmission data - Reference 5

Fig 3.7 Wave transmission data - Reference 30 & 39



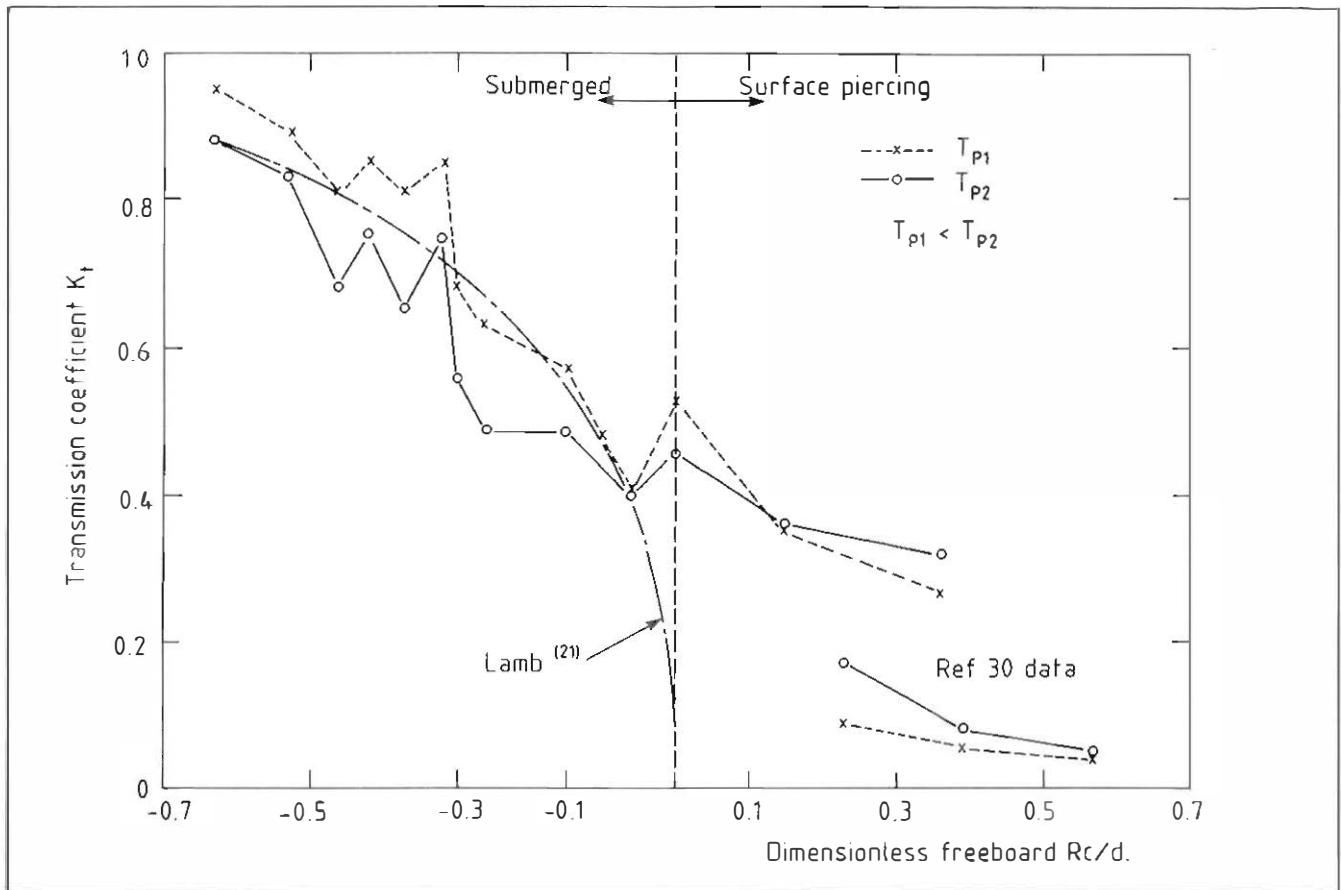


Fig 3.8 Variation of wave transmission with wave period - Reference 30 & 39

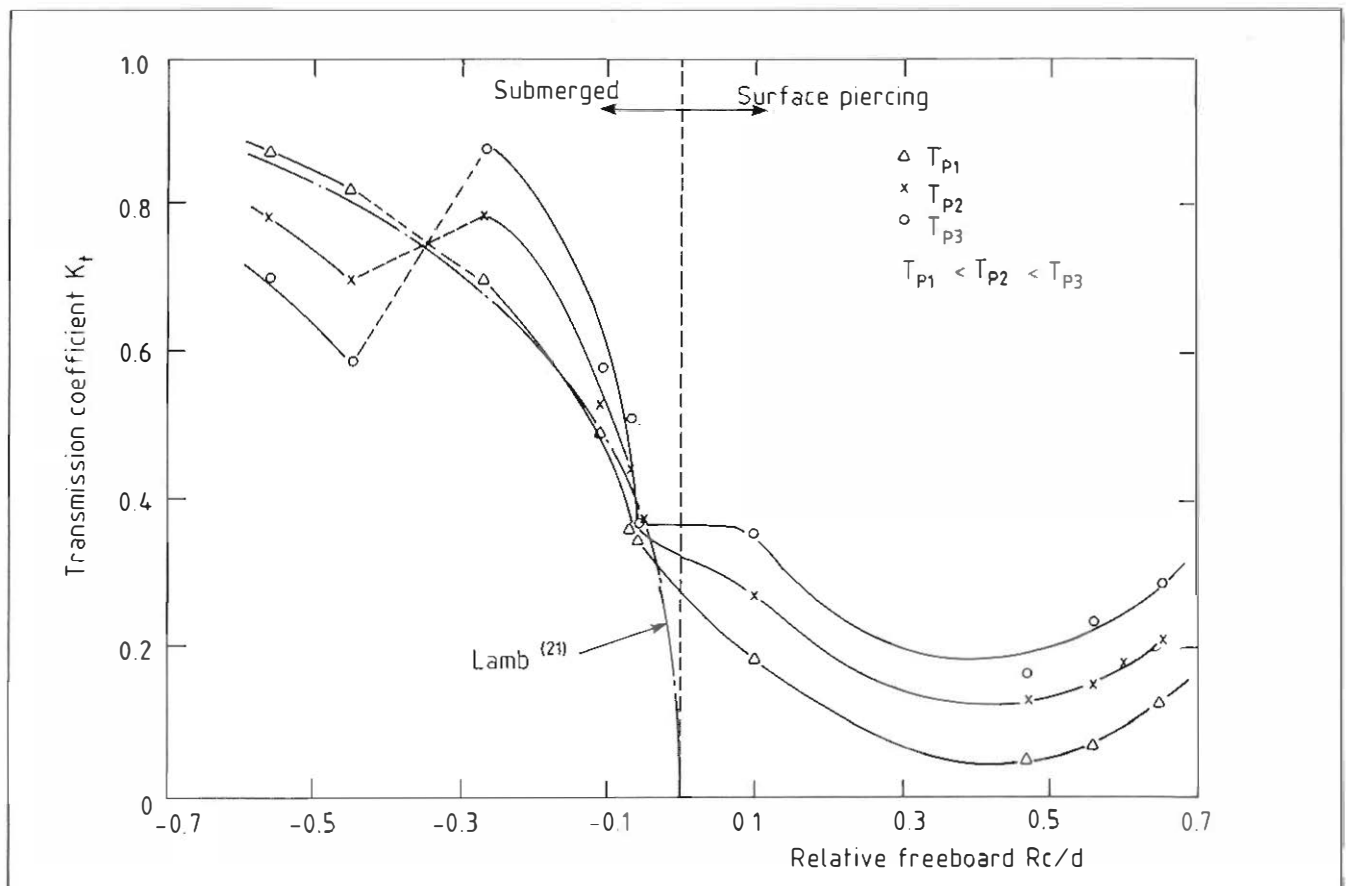


Fig 3.9 Variation of wave transmission with wave period - Reference 5

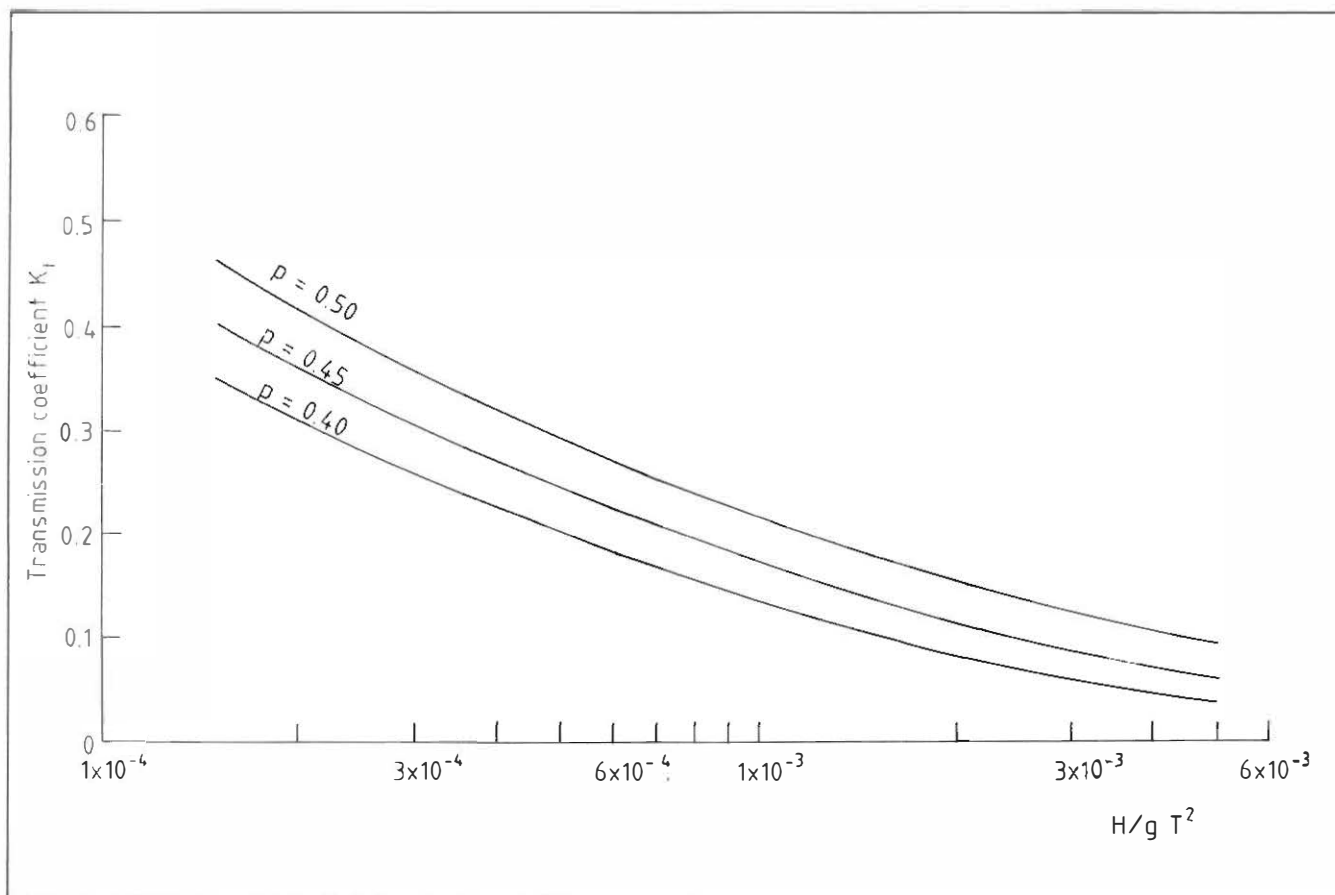


Fig 3.10 Effect of porosity on the wave transmission coefficient (after Seelig⁽⁵⁾)

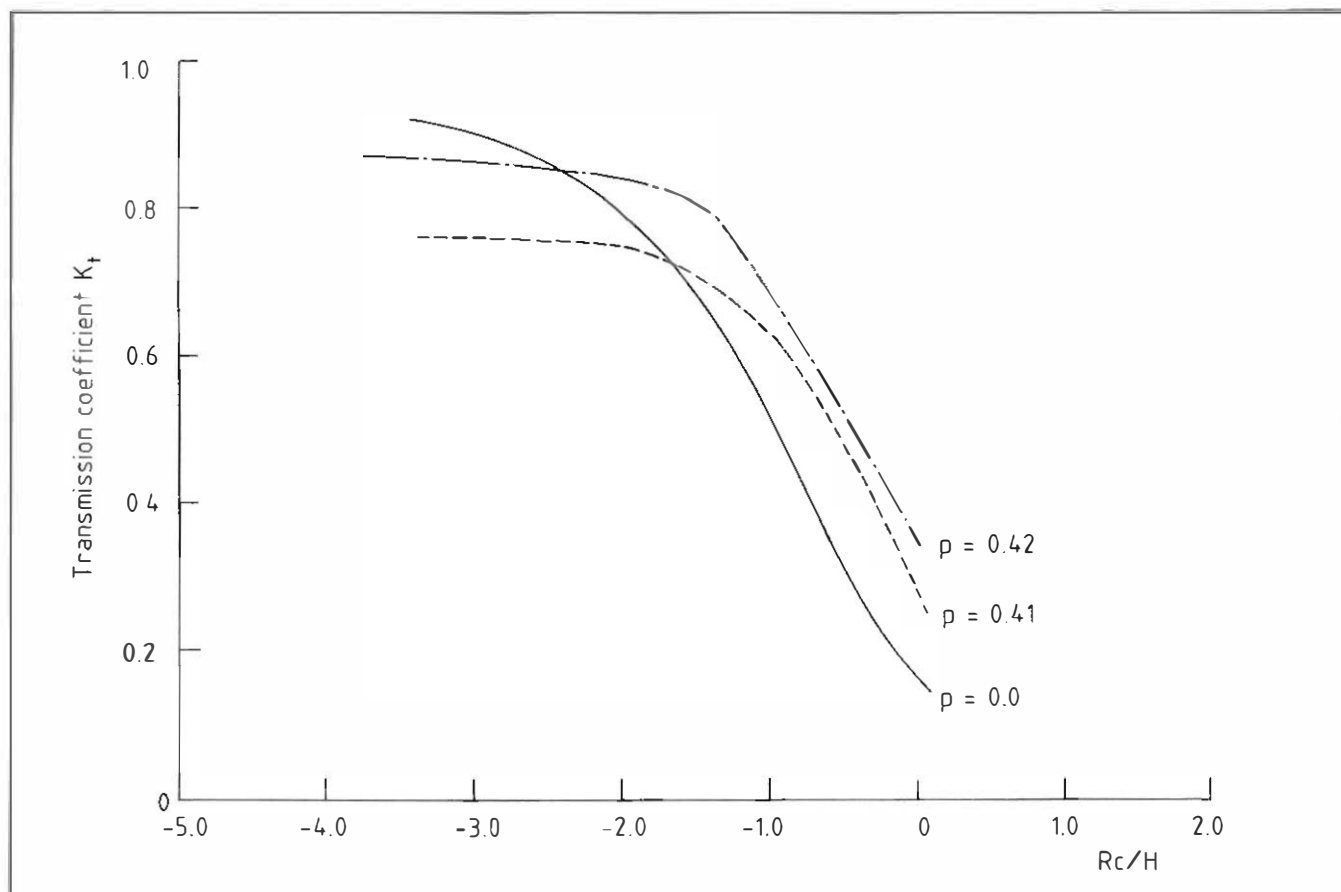


Fig 3.11 Effect of porosity on the wave transmission coefficient - Data from Reference 27

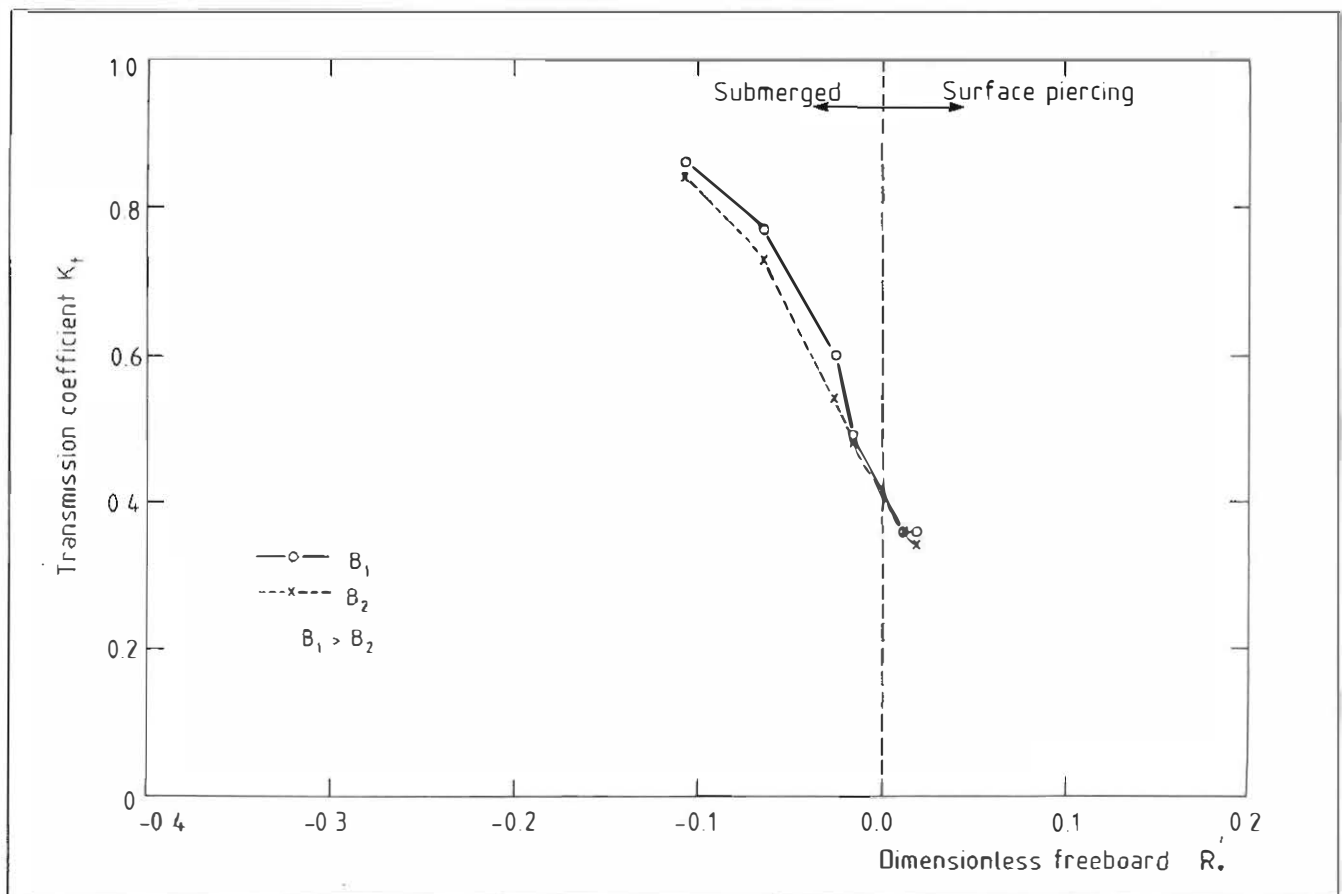


Fig 3.12 Effect of breakwater crest width on wave transmission - Reference 39

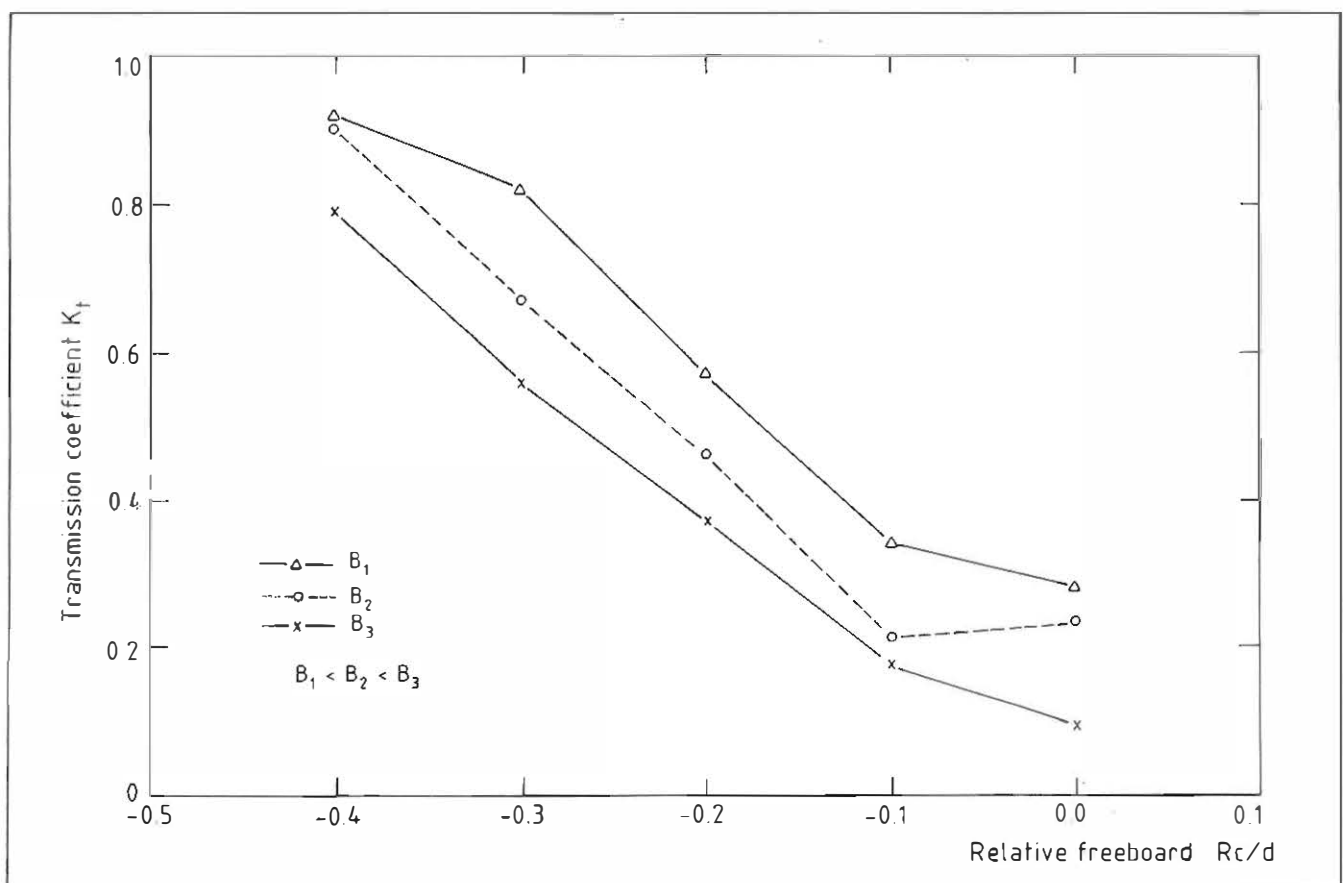


Fig 3.13 Effect of breakwater crest width on wave transmission - Reference 27

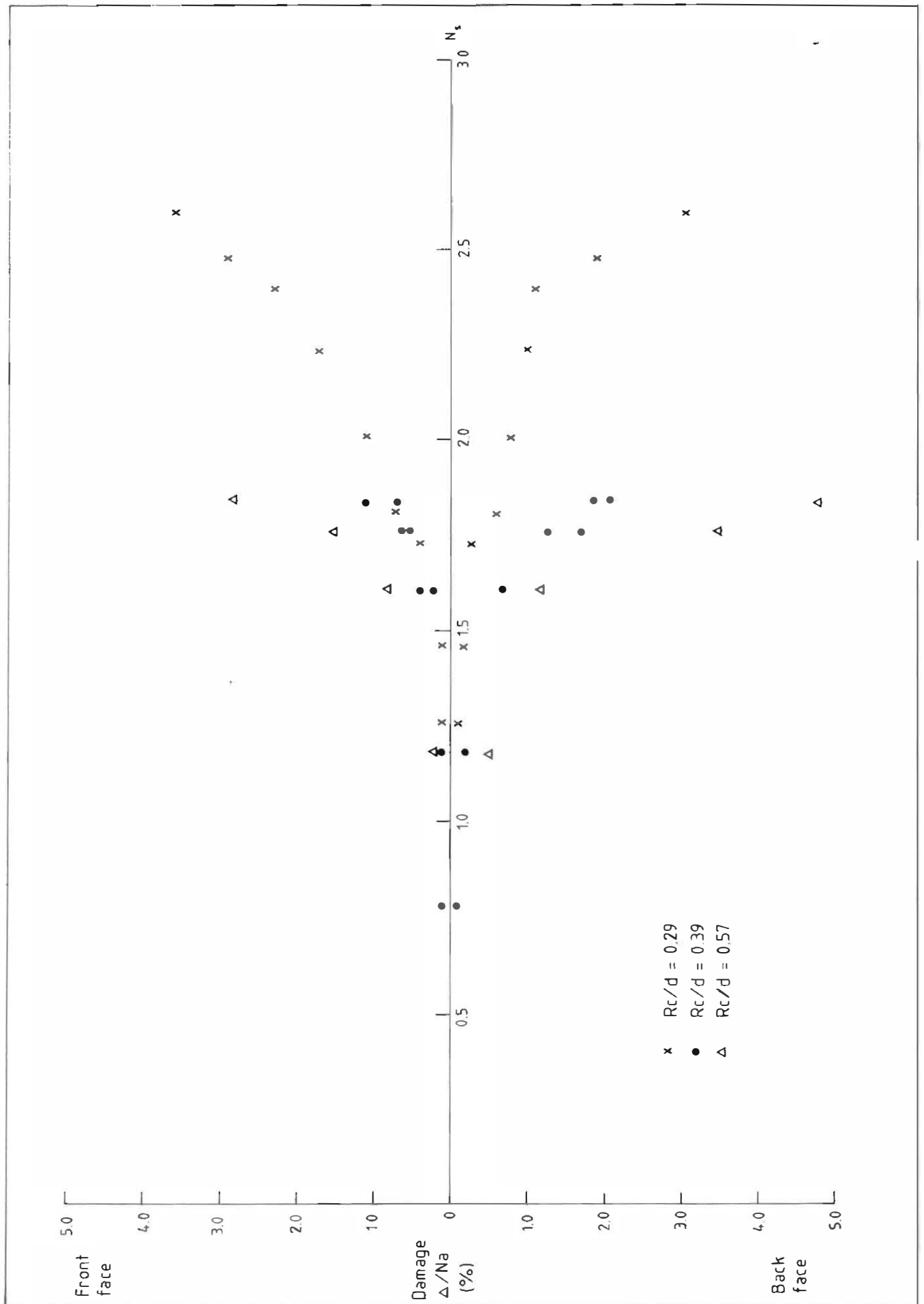


Fig 4.1 Front and back face damage as a function of N_s -long waves⁽³⁰⁾
 ($T_p = 19.0s$)

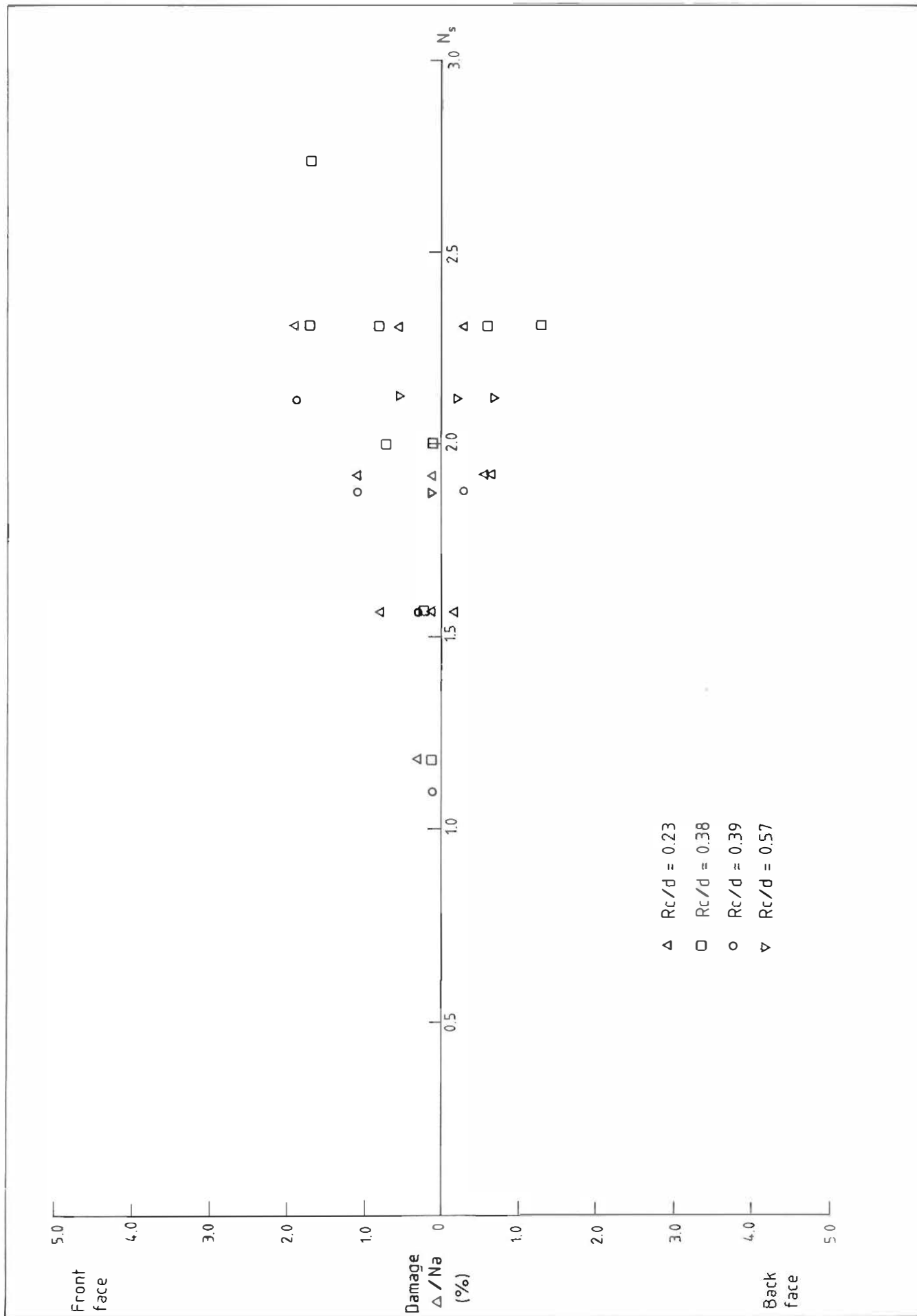
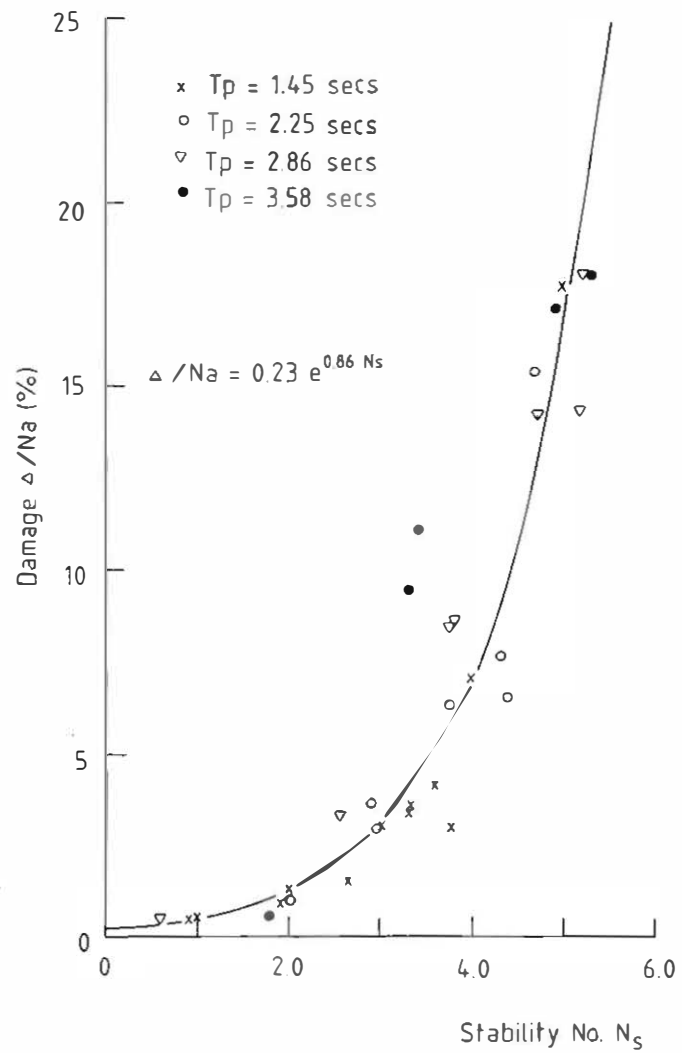


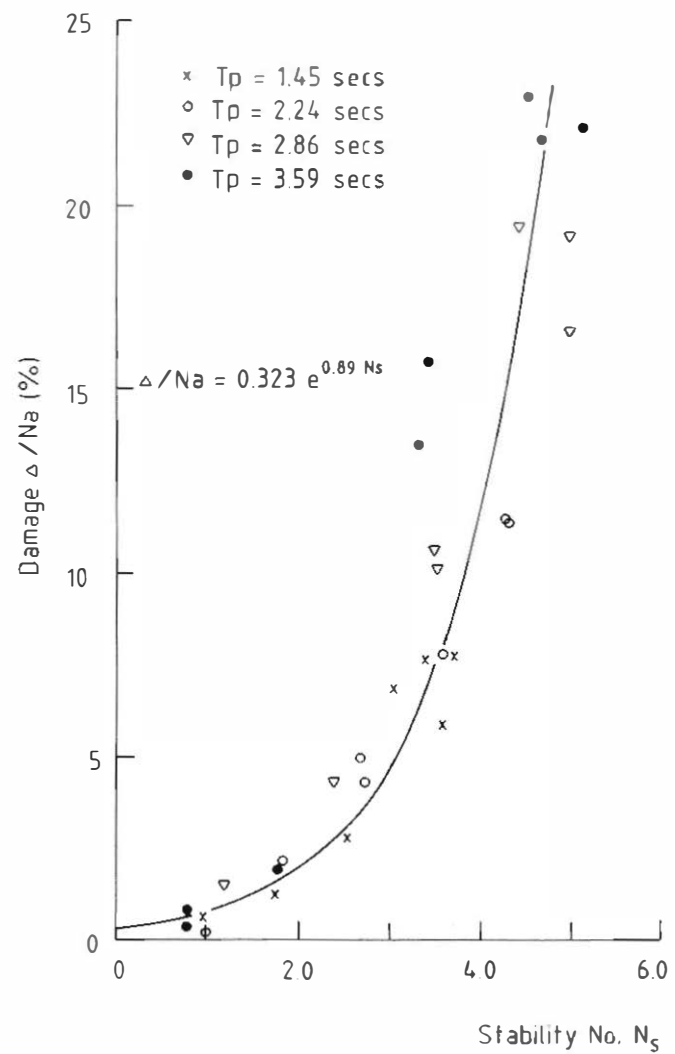
Fig 4.2 Front and back face damage as a function of N_s -short waves⁽³⁰⁾
 $(7.0s < T_p < 13.0s)$

Fig 4.3b Damage as a function of Hudson's stability number, N_s



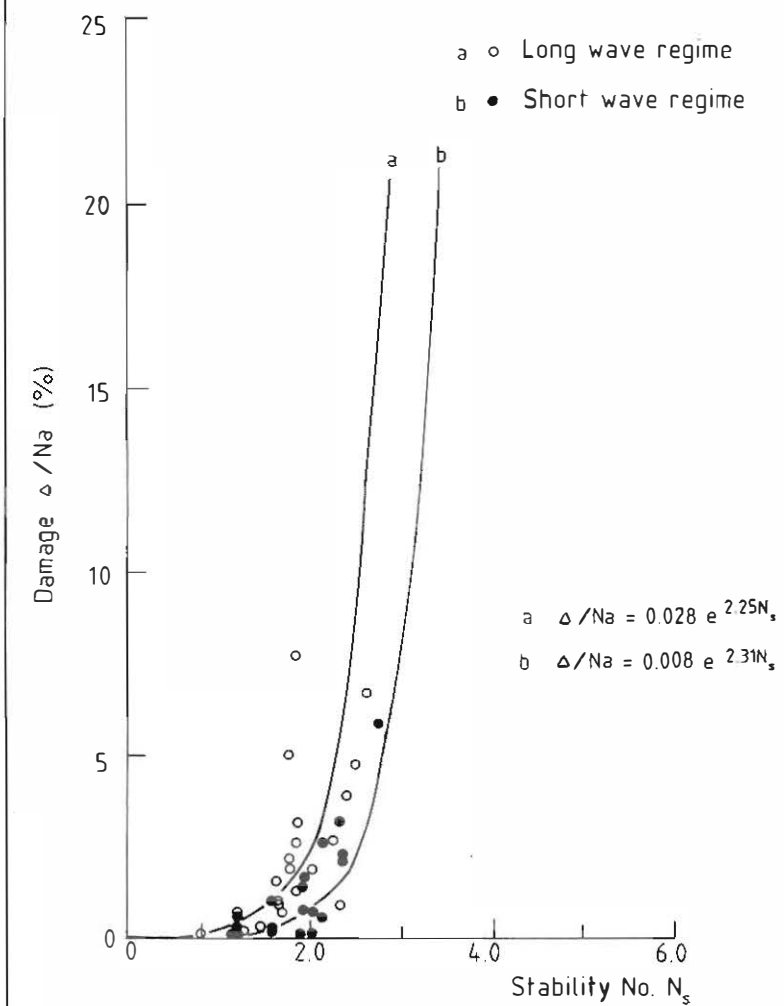
Data from Reference 38 ($R_c/d = 0.0$)

Fig 4.3a Damage as a function of Hudson's stability number, N_s



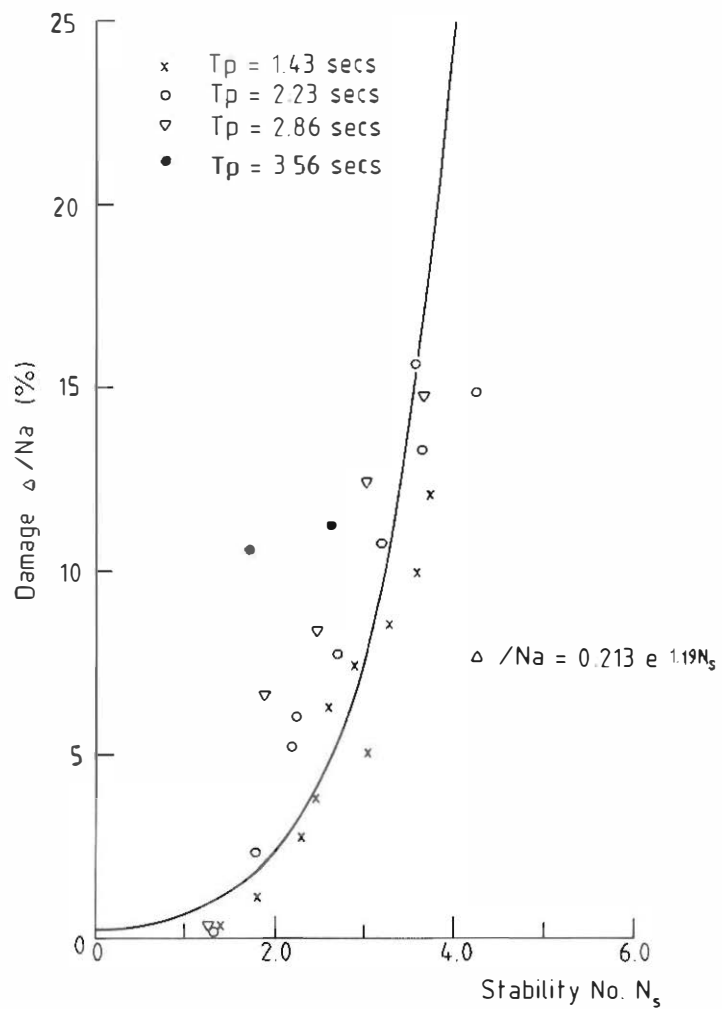
Data from Reference 41 ($R_c/d = 0.2$)

Fig 4.4b Damage as a function of Hudson's stability number, N_s



Data from Ref 30

Fig 4.4a Damage as a function of Hudson's stability number, N_s



Data from Ref 38 ($R_c/d = 0.4$)

Fig 4.5b Damage as a function of the spectral stability number, N_s^*

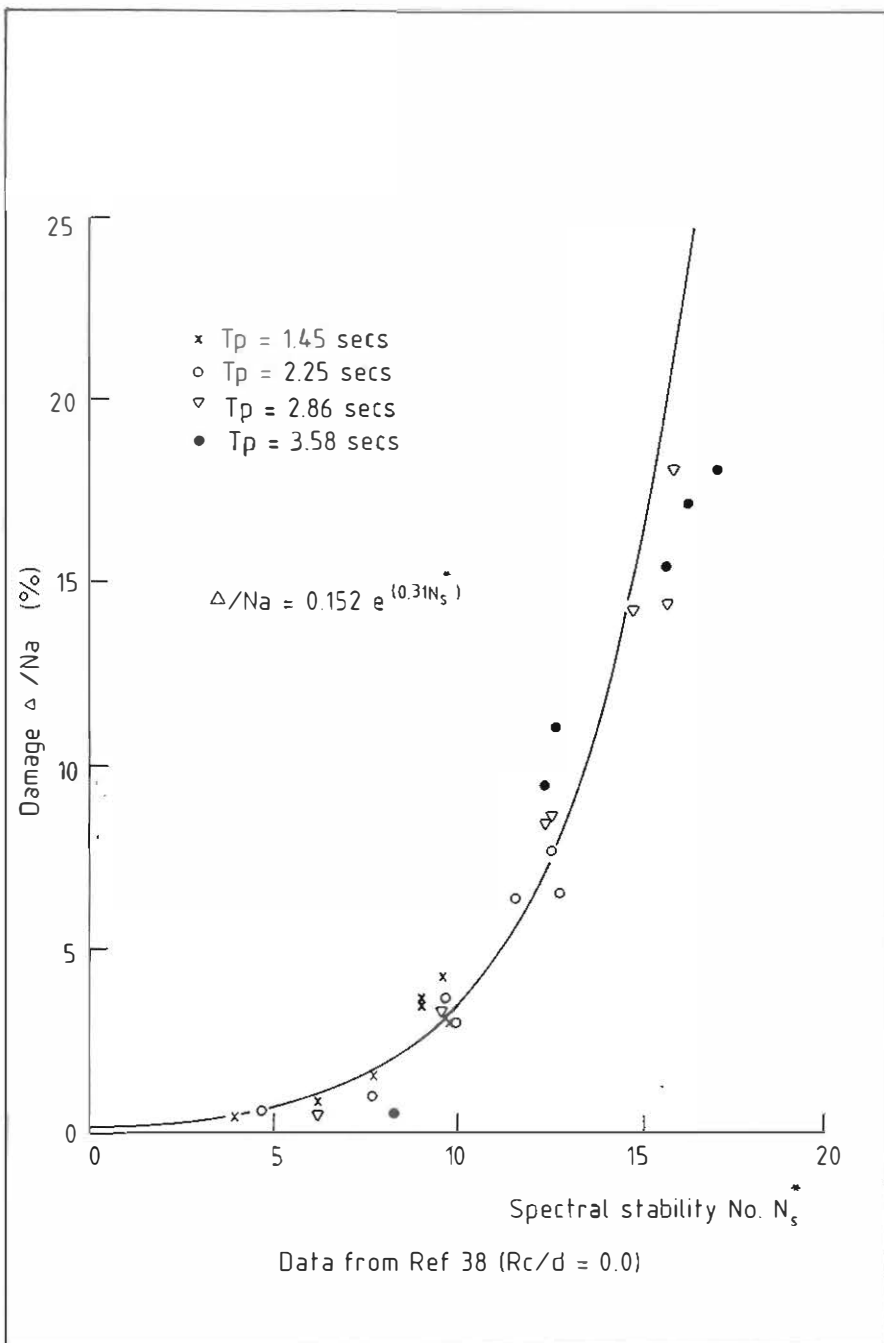


Fig 4.5a Damage as a function of the spectral stability number, N_s^*

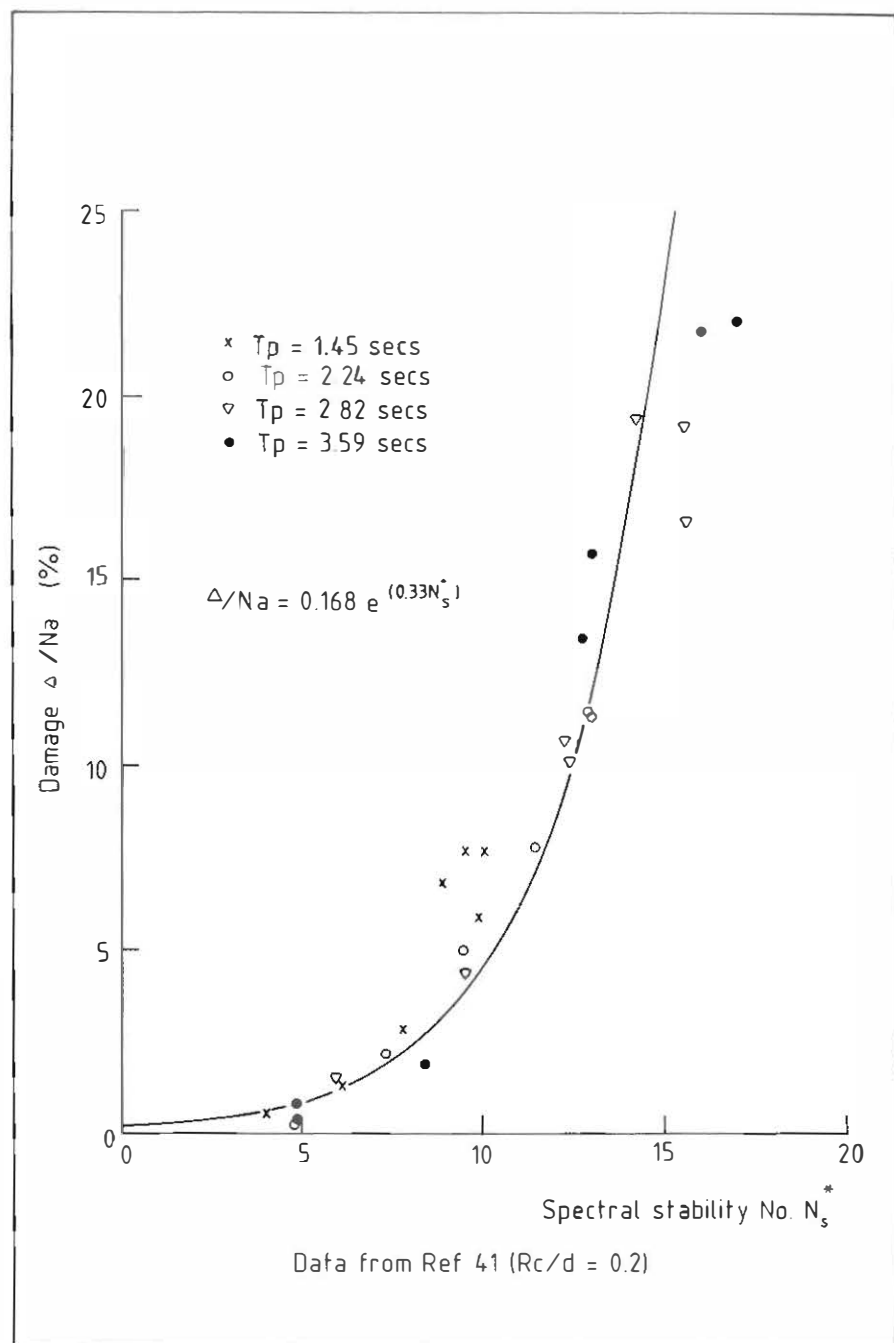


Fig 4.6b Damage as a function of the spectral stability number, N_s^*

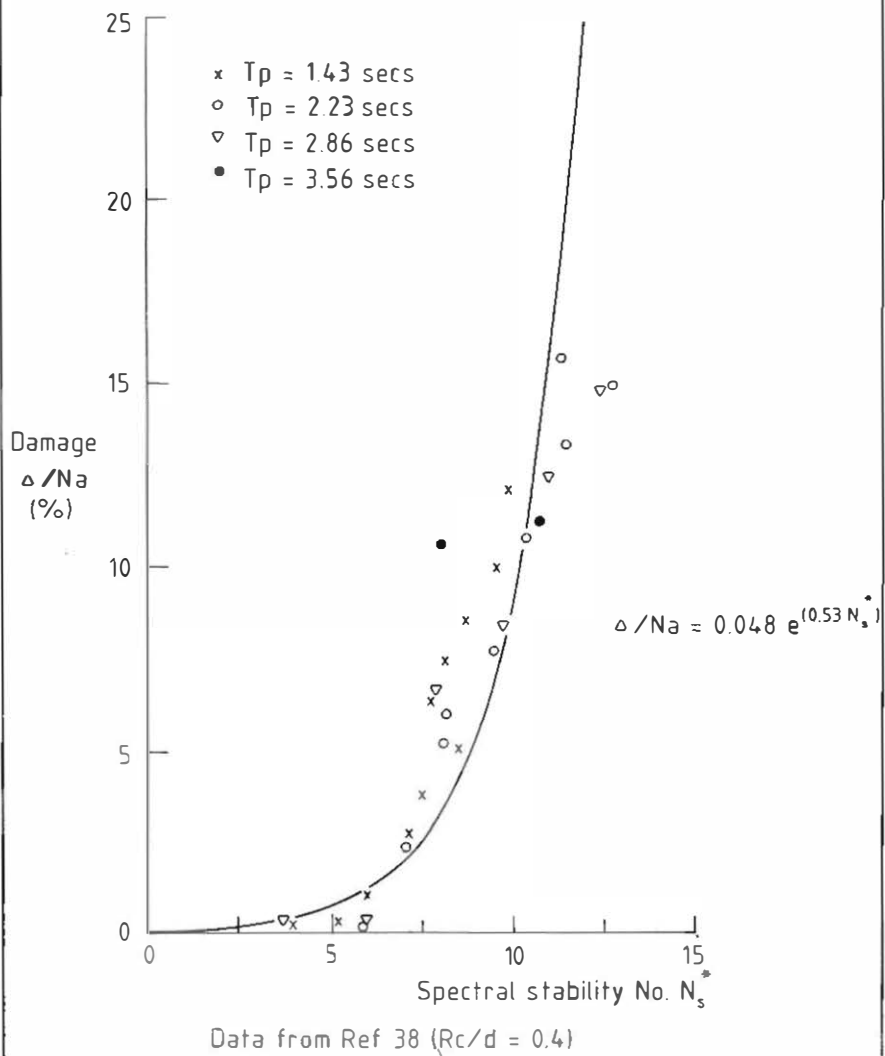
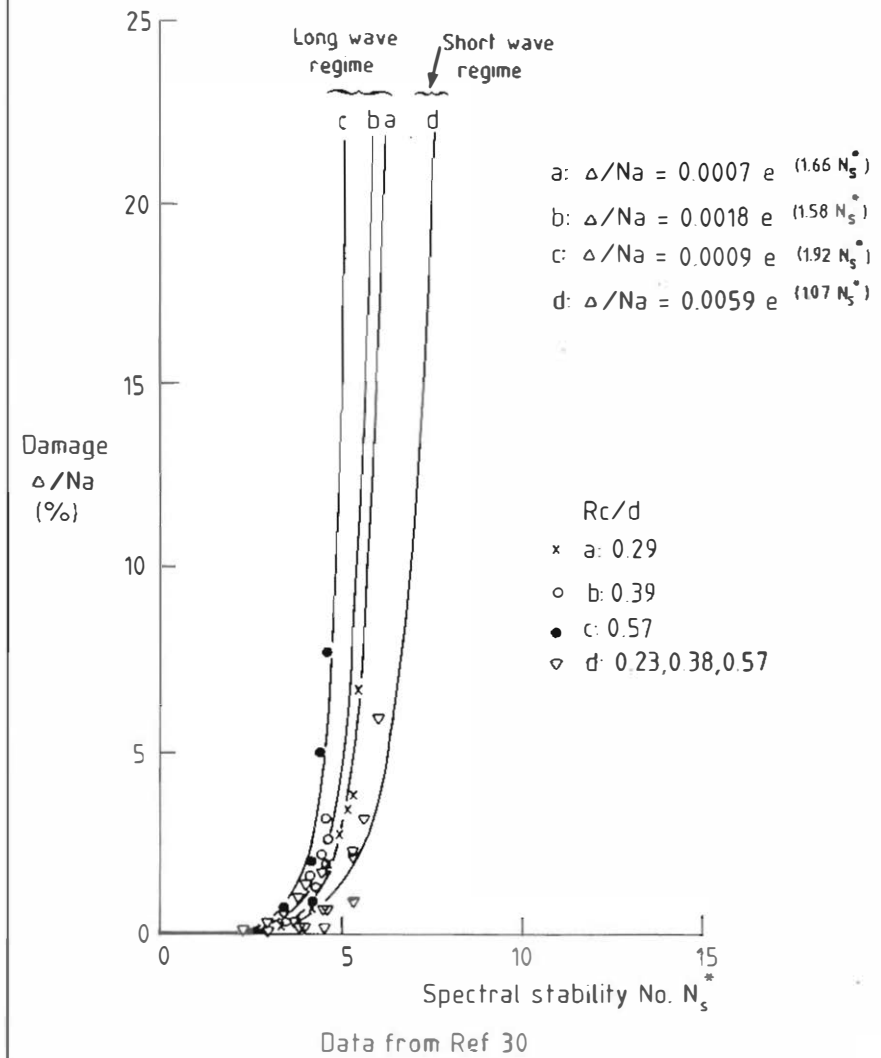


Fig 4.6a Damage as a function of the spectral stability number, N_s^*



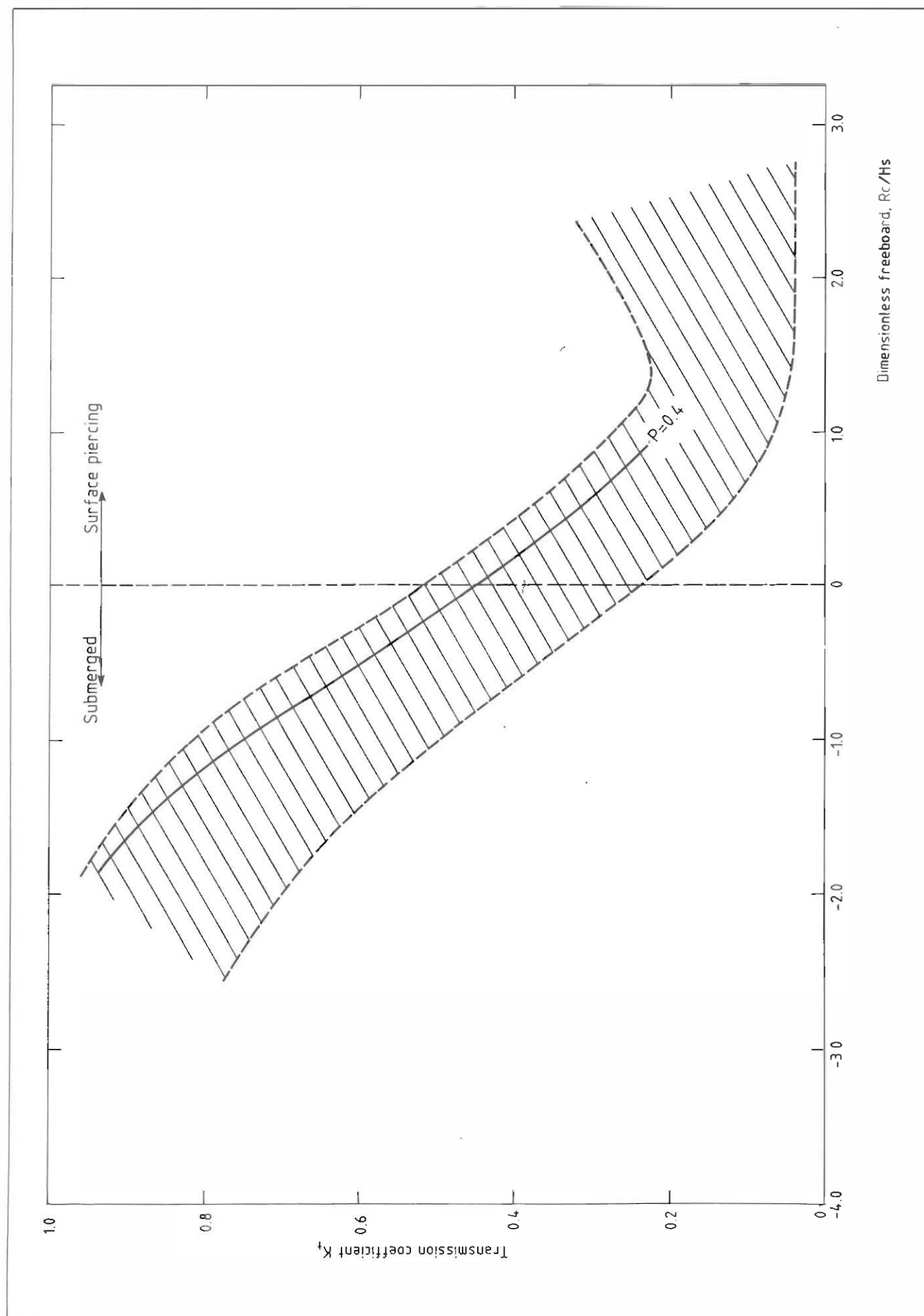


Fig 5.1 Wave transmission envelope for low crest breakwaters as a function of R_c/H_s

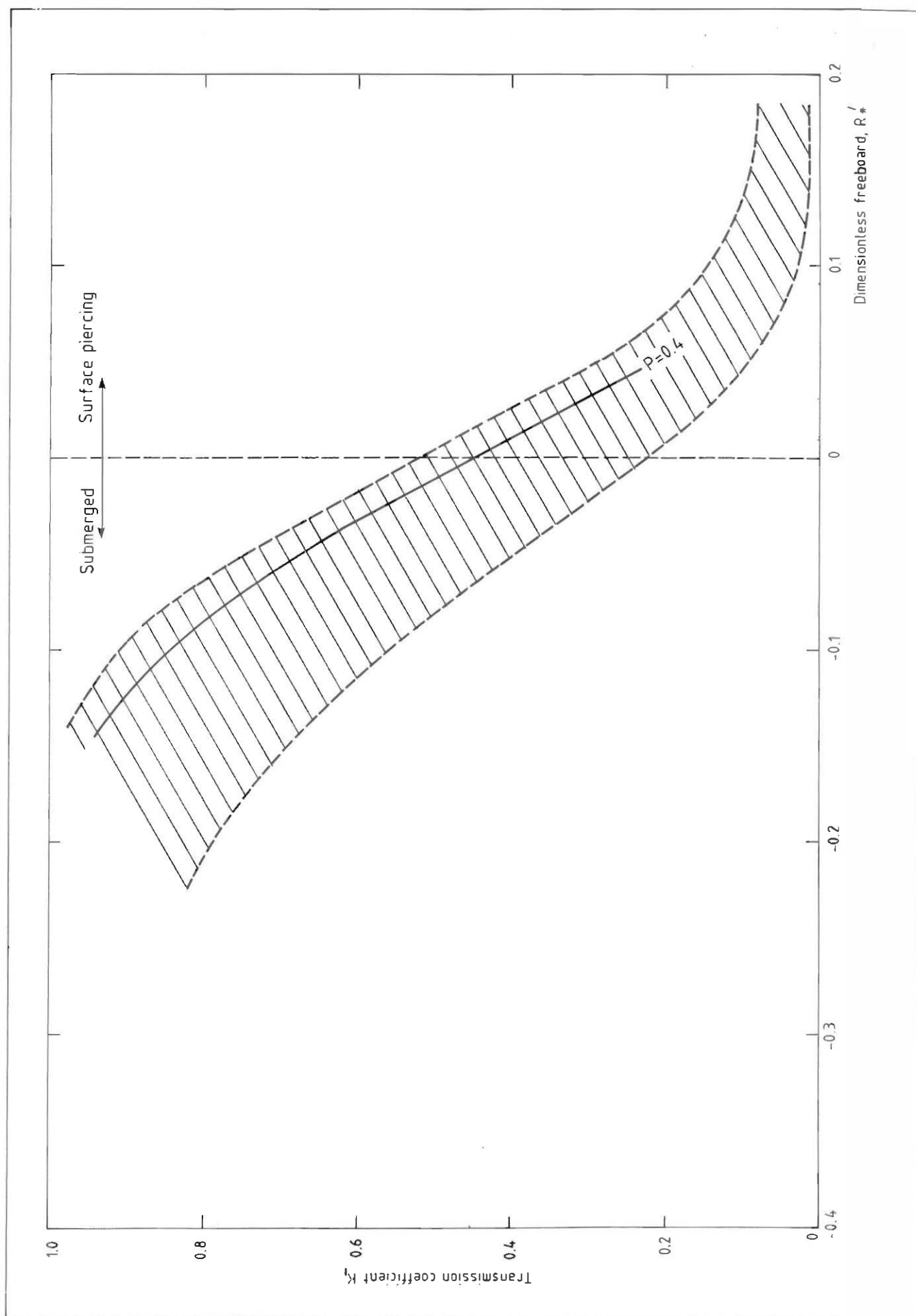


Fig 5.2 Wave transmission envelope as a function of R^*

Fig 4.5b Damage as a function of the spectral stability number, N_s^*

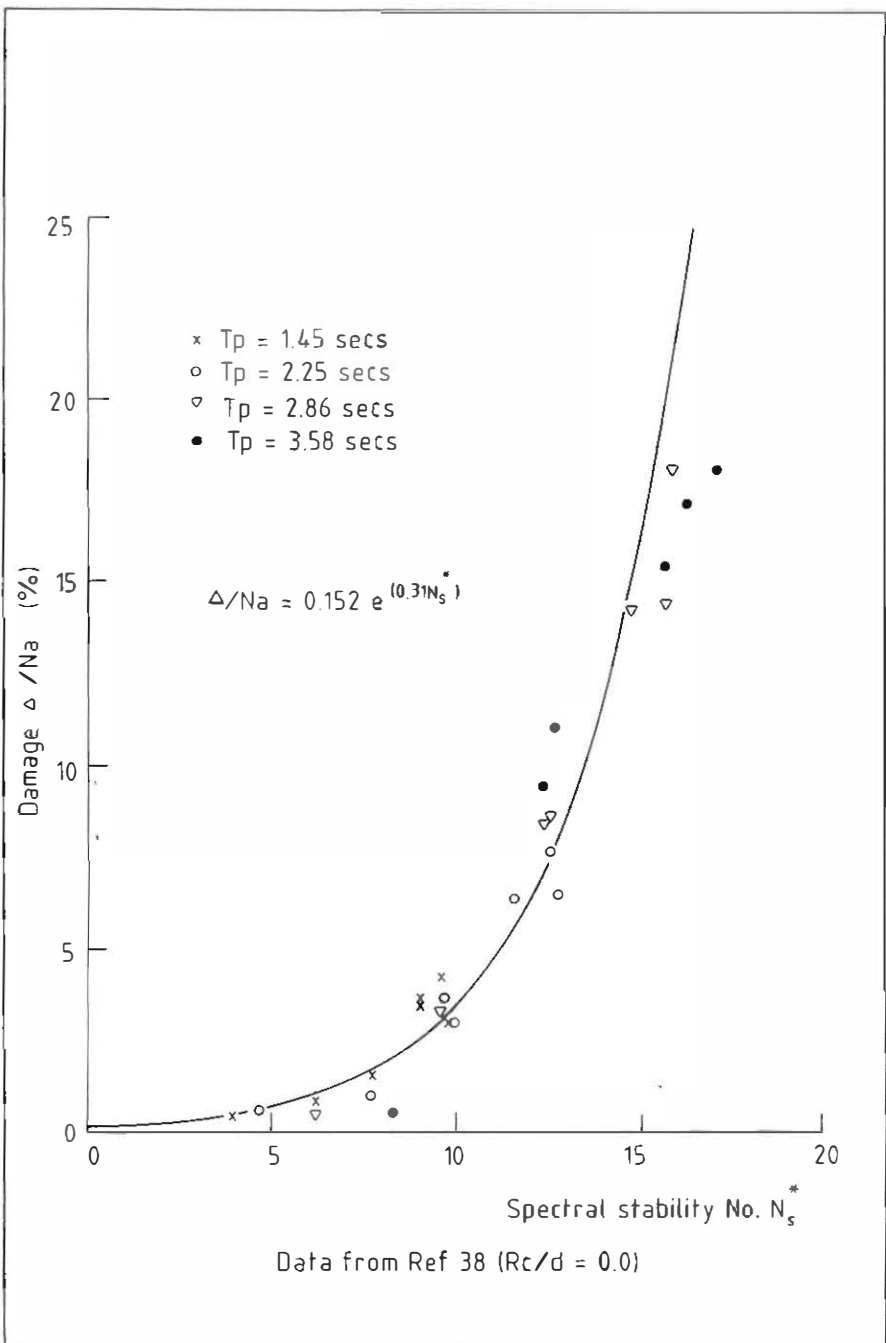


Fig 4.5a Damage as a function of the spectral stability number, N_s^*

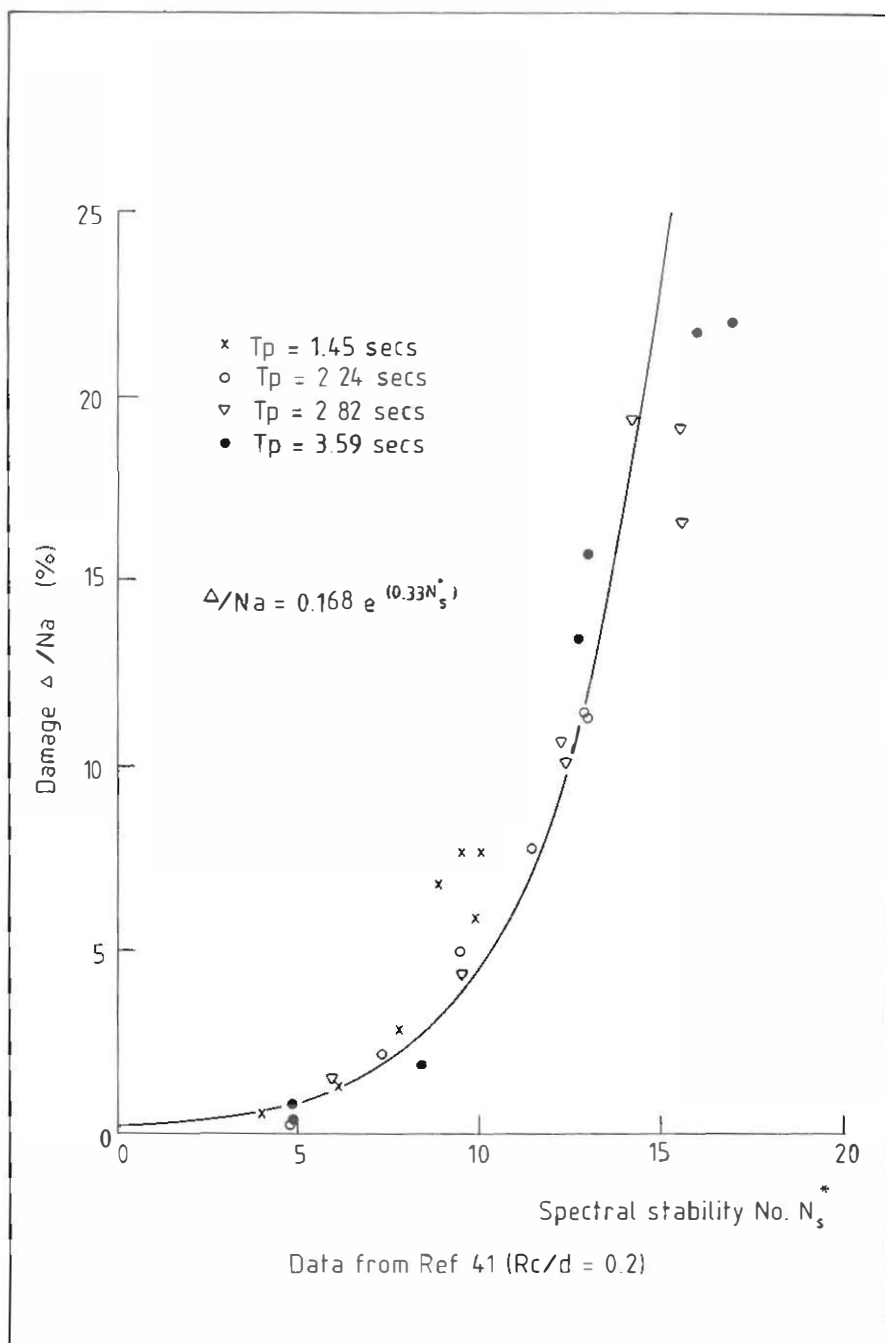


Fig 4.6b Damage as a function of the spectral stability number, N_s^*

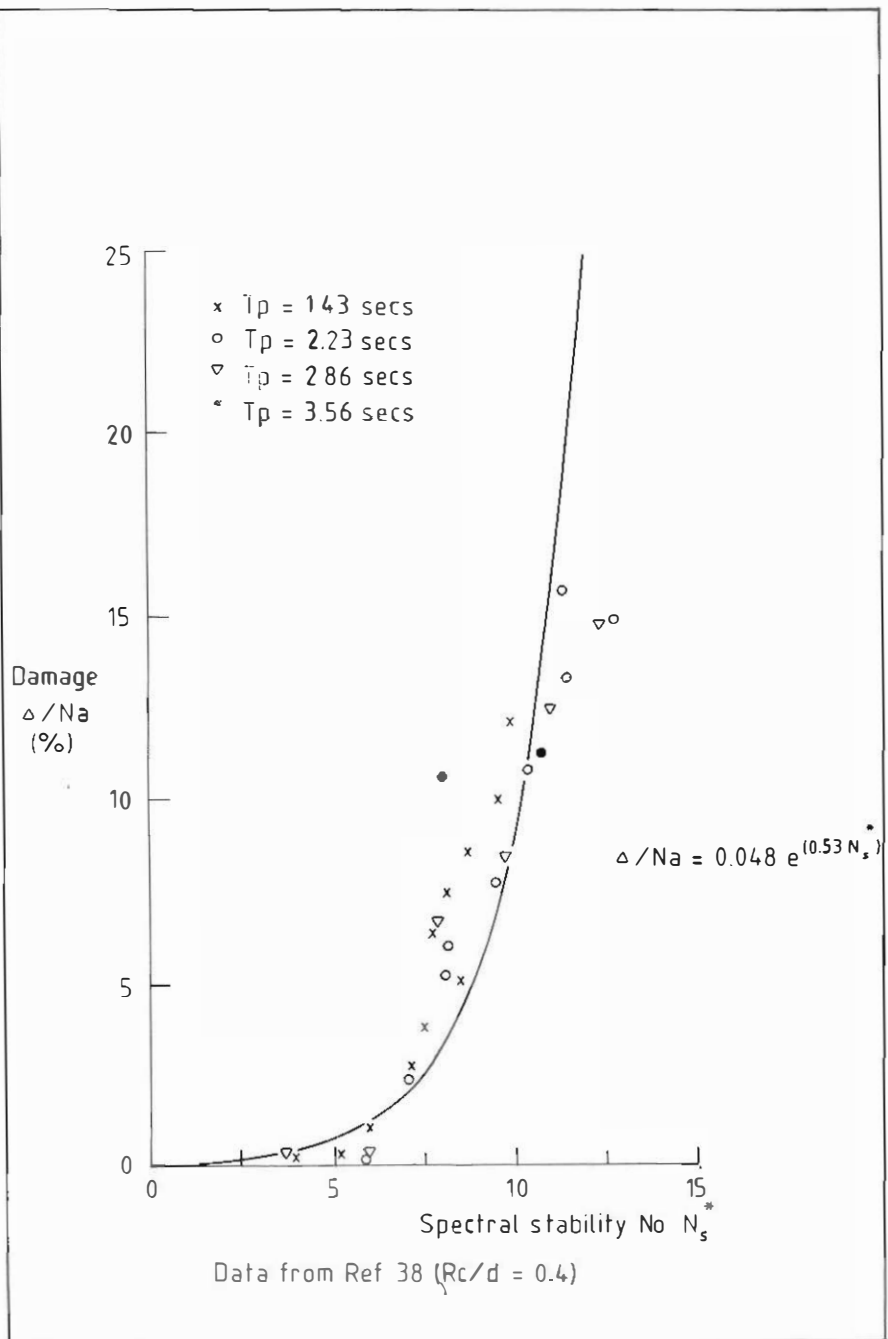
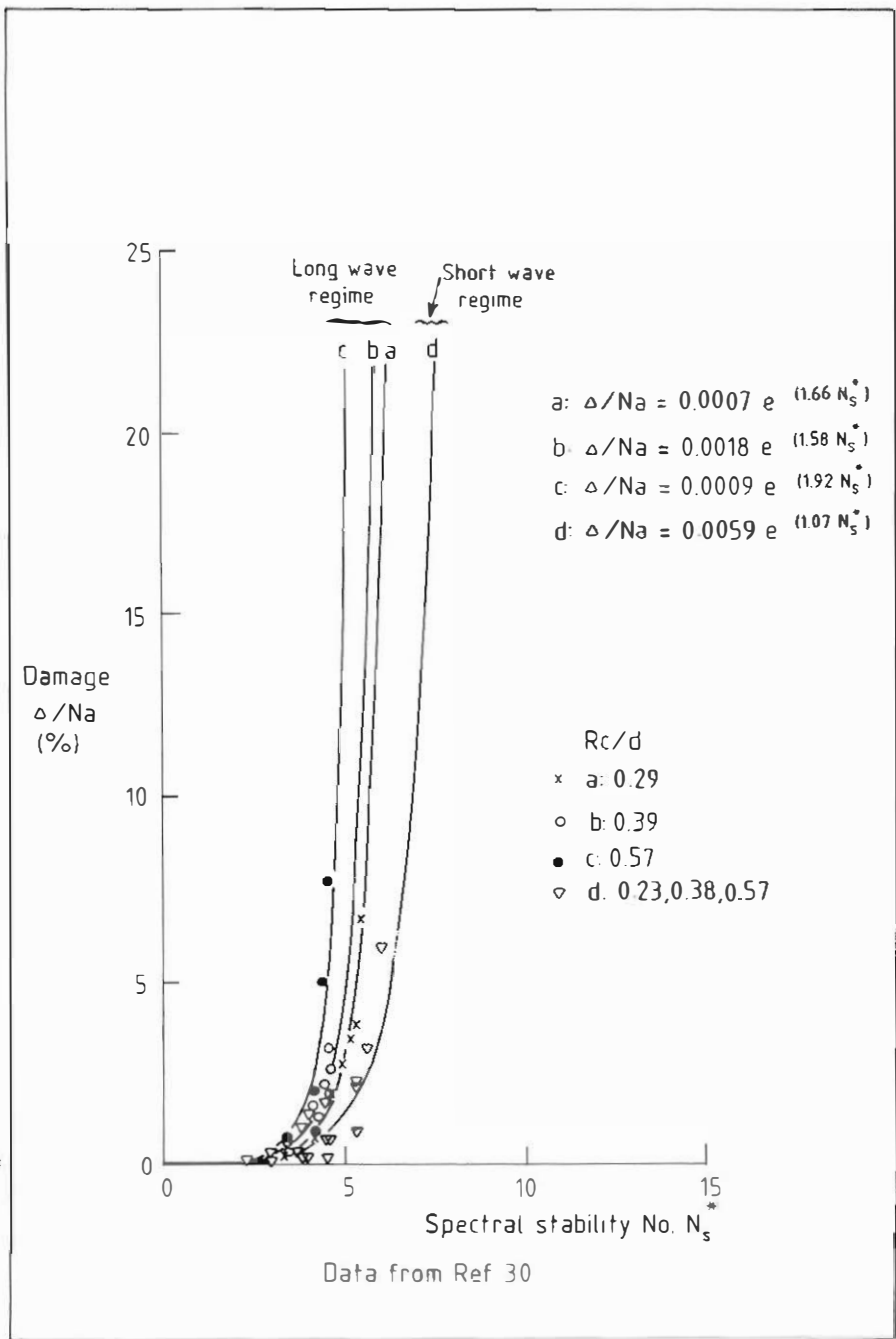


Fig 4.6a Damage as a function of the spectral stability number, N_s^*



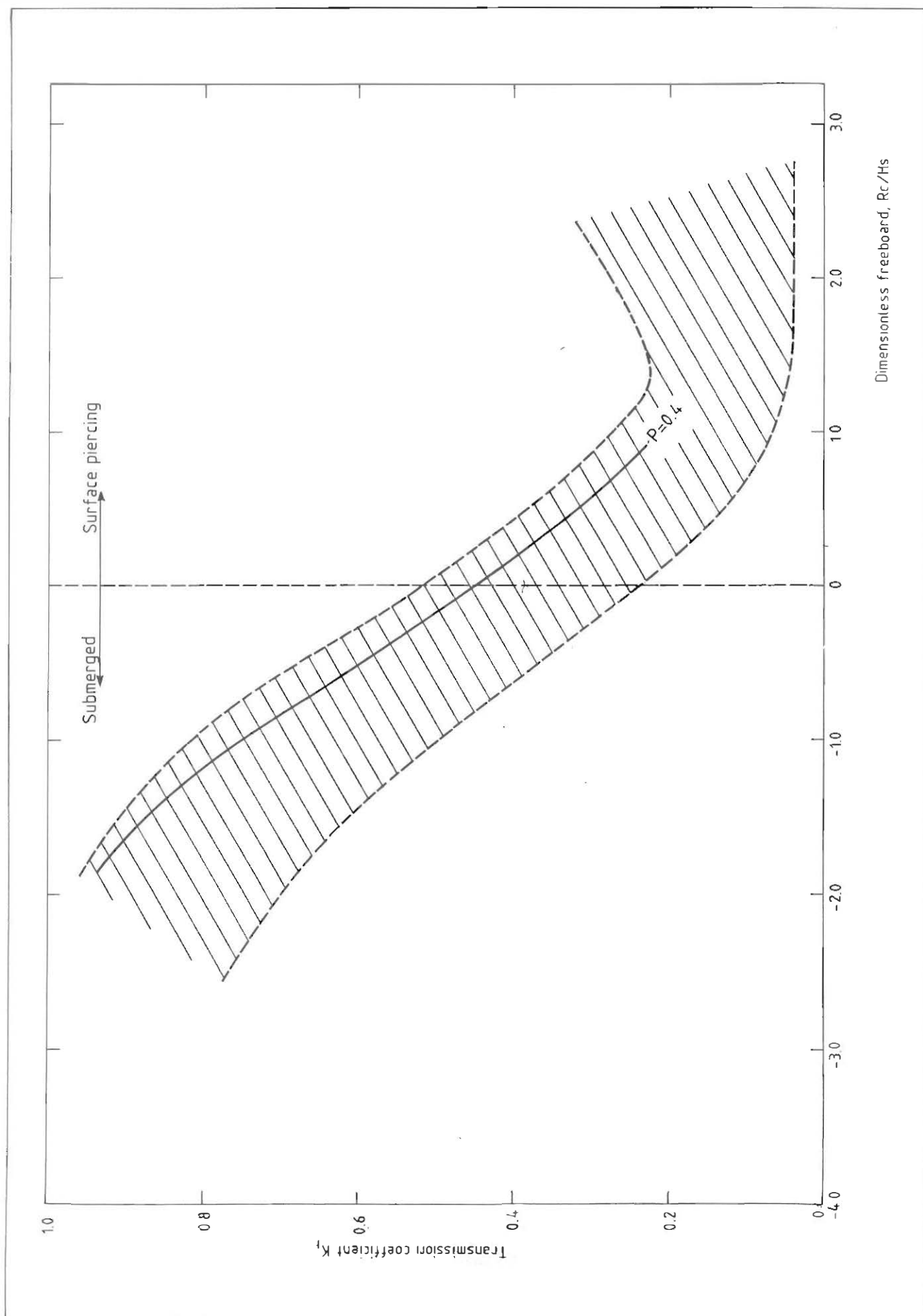


Fig 5.1 Wave transmission envelope for low crest breakwaters as a function of R_c/H_s

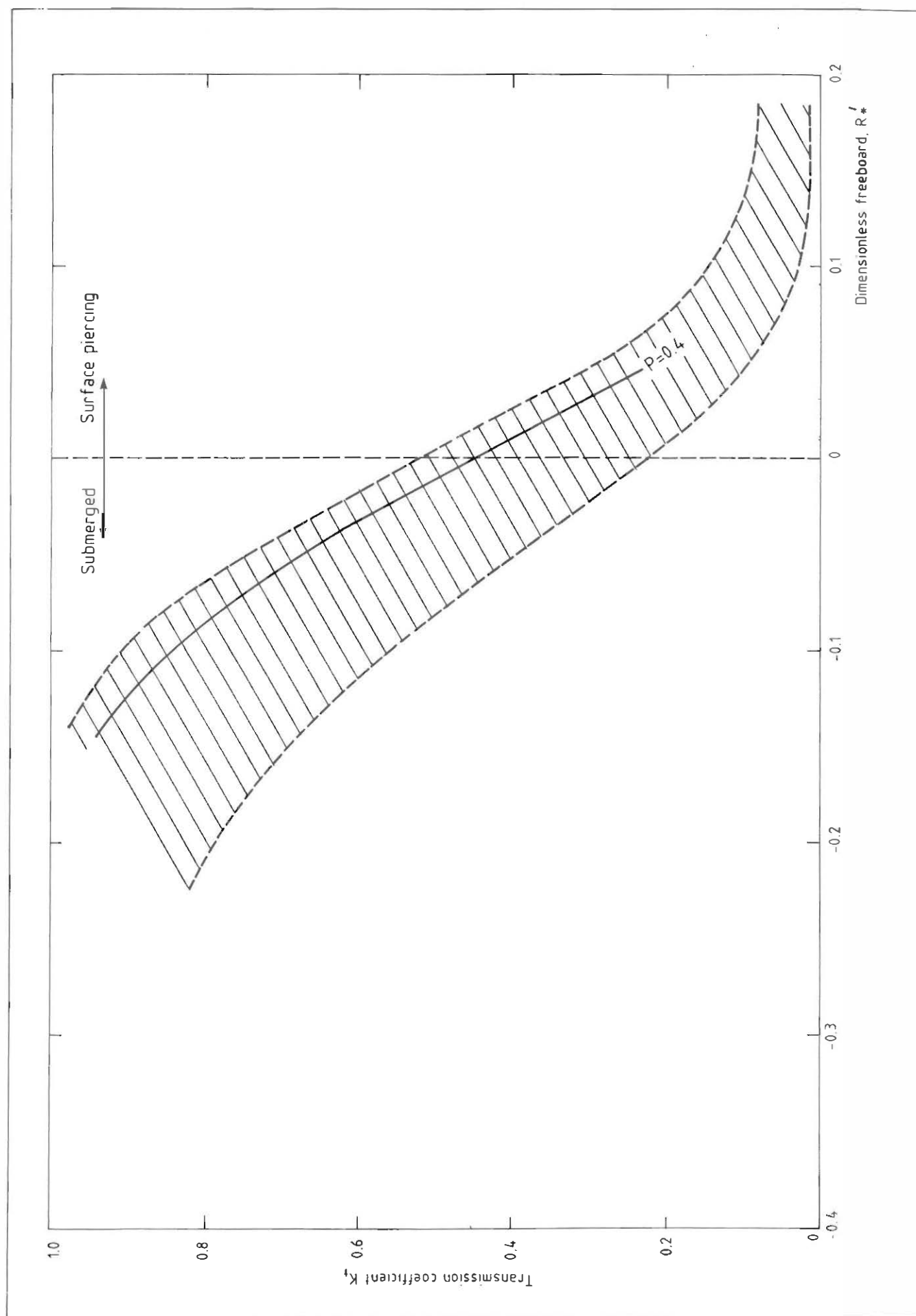


Fig 5.2 Wave transmission envelope as a function of R_*'