Stability of Rock Armoured Beach Control Structures

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Report SR 289
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Summary

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Control of shingle beaches for coast protection and/or sea defence may require use of rock groynes or shore-detached breakwaters. Experience from coast protection schemes on shingle beaches along the south coast of England however suggests that some control structures, including rock groynes and rubble revetments, have suffered greater damage than had been expected. This damage seems to have been most closely associated with steeper beach slopes, m > 1:10.

This report summarises information from recent research studies on the stability / damage behaviour of rock armour on 4 general beach control structures:

a) simple 1:2 rubble sea wall slope;
b) rock bastion or roundhead groyne;
c) inclined crest rock groyne;
d) L-shaped rubble groyne adapted from b) above.

Sections of this report describe prediction methods for the determination of armour sizes on rock groynes of various configurations. They identify the main types of rock armoured beach control structures; the design methods available to calculate armour size; and the main limitations in their use. The results of new research studies are used to modify armour design methods, and thus to suggest new methods to estimate the effect of steep beach slopes.

The report is aimed at coastal engineers who require information on the analysis / design of rock armoured beach control structures. It summarises data from a number of previous studies by HR Wallingford and co-workers, and includes results from recent wave basin tests from the companion report IT 413 (Ref 4).

These reports do not address the response of the beach to the structures, as those aspects are discussed in detail by Coates and co-workers (Refs 1-3).

For further information on the studies covered by this report, please contact Professor N.W.H. Allsop or Mr R.J. Jones of the Coastal Group at HR Wallingford.

Notation

A_c	Main armour crest freeboard relative to SWL
A_e	Area eroded around SWL
a, b	Empirically derived coefficients
B	Structure width
D_{n50}	Nominal particle diameter defined $(M/\rho_r)^{1/3}$ or $(M/\rho_c)^{1/3}$
H_b	Breaking wave height, derived as a function of bed slope, water depth, and wave period
H_{max}	A maximum wave height, general 1.8-2.0H_s
H_s	Significant wave height, average of highest one-third of all wave heights
$H_{1/10}$	Wave height, average of highest one-tenth of all wave heights
h	Water depth
h_s	Water depth seaward of toe of structure
h_t	Toe depth, or depth of water at toe of rubble mound
K_D	Hudson stability coefficient
K_{RR}	Hudson stability coefficient for rip-rap
M	Armour unit mass
M_{50}	Median mass
$N_{d\%}$	Number of unit displaced, expressed as a % of units in the area
N_s	Stability number, defined $H_s/\Delta D_{n50}$
N_z	Number of waves in a storm, record or test
P	Notional permeability factor
R*	Dimensionless freeboard defined $R_c/(T_m\sqrt{gH_s})$
R^*_p	Dimensionless freeboard defined $R_c/(T_p\sqrt{gH_s})$
R_c	Structure crest freeboard relative to SWL
S_d	Dimensionless damage to a mean profile, defined A_e/D_{n50}^2
s_m	Mean wave steepness, defined $2\pi H_s/gT_m^2$
s_p	Peak wave steepness, defined $2\pi H_s/gT_p^2$
t_a	Armour layer thickness
T_m	Mean wave period
T_p	Peak wave period (usually offshore)

α(alpha)	Structure slope angle to the horizontal
β(beta)	Angle of wave attack, relative to the normal to structure
$\mu(x)$	Mean of x
ξ_m	Mean Iribarren Number, defined $\tan\alpha/s_m^{1/2}$
ξ_p	Peak Iribarren Number, defined $\tan\alpha/s_p^{1/2}$
Δ	Relative buoyancy density, (ρ_r/ρ_w)-1 or (ρ_c/ρ_w)-1
ρ_r	Rock density (Kg/m^3)
ρ_w	Water density (Kg/m^3)
$\sigma(x)$	Standard deviation of x

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1 Introduction

1.1 Purpose of the research

The role of shingle beaches as part of a coastal defence system increasingly requires the use of control structures to assist in the maintenance and retention of the beach. A wide range of structures may be used in this role, including groynes, breakwaters, sills, and revetments.

In recent years, rock armoured groynes and/or breakwaters have become more popular in the UK as means of containing or controlling shingle beaches than timber, steel or concrete groynes. Rock armoured groynes or breakwaters are relatively permeable to wave action, being constructed as a mound or slope of rubble material protected by layers of armour units. They are designed to dissipate incident wave energy as waves run up onto and into the porous slope, giving low wave reflections, and reduced wave-induced near-shore currents.

On a rubble mound groyne or breakwater, the principal structural variable set by stability requirements is the armour size. Many other structural dimensions, and aspects of construction practice, are influenced strongly by the armour size. If under-sized, rock armour will experience excessive armour movement and/or displacement, leading in turn to deterioration of the armour layer, erosion of fill material, and/or excessive wave overtopping. A number of methods may be used to calculate the stable armour size, but these methods are only valid over defined ranges of conditions and specific configurations.

Recent experience from coast protection schemes on shingle beaches along the south coast of England has suggested that many beach control structures, including both rock groynes and rubble revetments, have suffered substantially greater damage than might have been expected. This damage seems to have been most closely associated with steeper beach slopes, m > 1:10, and was of sufficient frequency to prompt Ministry of Agriculture, Fisheries and Food (MAFF) to support an investigation into the stability / damage characteristics of rock armoured beach control structures.

This report summarises design methods available to determine the size / mass of the armour needed to protect such structures from direct wave attack. Companion reports by Coates & Lowe (Ref 1), Coates (Ref 2) and Coates & Simm (Ref 3) address the response of shingle beaches to these structures. New results from three-dimensional (3-d) physical model tests by Jones & Allsop (Refs 4 & 5) describe armour response on four typical rock armoured structures constructed on a 1:7 slope shingle beach, and are used here to develop modifications to standard design formulae. These and other modifications have been tested against measurements at full scale from the field.

1.2 Outline of the report

The main structure types in use in the UK and their stability characteristics are reviewed in Chapter 2. The principal methods available to calculate the stability of rock armour against wave attack are described in Chapter 3. The results from the new 3-d model tests are summarised in Chapter 4, and measurements / observations of damage from site are discussed in Chapter 5. The main conclusions of the studies, recommendations for analysis / design, and for future research studies are drawn in Chapter 6.

2 Types of structure

2.1 Types of rock armoured beach control structures

The general configurations of offshore breakwaters or groynes used as beach control structures are discussed by Coates and co-workers (Refs 1-3), and in the CIRIA / CUR manual on rock armoured structures (Ref 7). Six example structure types are shown schematically in Figure 1.

The simplest use of rock in a beach control structure is probably the addition of rock to an existing timber, steel or concrete piled groyne to reduce wave reflections and hence wave-induced currents (Fig 1a). This may however only be useful if the existing groyne lengths and spacings are appropriate, and the existing structures are of sufficient residual life. In practice, full replacement by rock groynes at wider spacings may often be preferred to modifying existing solid groynes.

A rock bastion or CIRIA Type 2 groyne (Fig 1b) is shorter and broader than other groyne types. Its outer roundhead will generally be placed above high water, shown in Figure 2. Bastion groynes will tend to dissipate wave energy around the enlarged outer end, as well as along the trunk section.

An inclined or CIRIA Type 1 groyne follows the beach slope, and is of relatively constant cross-section along its length, Figure 2. A greater length of groyne can be constructed for a given volume of rock, but the effect of the groyne on local wave conditions and beach movement at any particular water level may be less than for a roundhead groyne. Type 1 or Type 2 groynes represent the extremes of the range of simple configurations. Many practical examples combine features from both types.

The bastion or Type 2 groyne may be extended at the outer end to give a "hammer-head" or "L-shaped" groyne (Fig 1c). The extended arms give increased zones of shelter behind the outer limb(s), and may offer significant reductions in wave reflections and wave-induced currents. In some cases the extensions may be taken further to give the "fishtail" groyne (Fig 1d), generally a larger structure whose root and up-drift arm act as conventional bastions. The down-drift arm is often aligned with crests of the predominant wave attack direction, acting to diffract waves around the groyne.

The final developments in this sequence are the partially-attached or detached breakwaters (Figs 1e-f) whose use is discussed by Coates & co-workers (Refs 1-3), and for which design rules are discussed fully in the CIRIA / CUR rock manual and SPM (Refs 7, 8).

2.2 Review of UK examples

In the first phase of the work in this study, six practical examples of these types of structures were identified, and the principal structural parameters are discussed by Jones & Allsop (Refs 4 & 5). It appeared initially that there were considerable variations in design of these structures, but a number of common features applicable to most structures may be summarised.

Most beach control structures have been designed to allow wave overtopping during storms. Bastions or Type 2 groynes around southern England are generally 50 to 100m long. The crest level of a roundhead groyne is generally set below or at the height of the beach storm ridge at the site. Side slopes are generally kept steep, typically at 1:1.5 or 1:2.

Low-level or inclined groynes (Type 1) are often longer than Type 2 groynes, depending on beach slope and tidal range. Side slopes are usually kept within 1:1.5 - 2.5. The crest width is normally equivalent to 3 armour stones, and the armour is usually 2 layers thick if separate from the core.

On small structures, a single grading of rock is often used. The omission of any bedding layer implies that some settlement of the groyne may be tolerated. For larger structures involving more design effort and capital cost, the use of core, underlayers, and armour layers may prove to be more economic than allowing for repeated "top-ups" of rock to compensate for settlement. Settlement of a larger structure may be more important, and a geotextile and/or granular filter system will often be required to reduce the risk of structure and beach materials mixing. Standard filter rules are discussed in detail in the CIRIA / CUR rock manual (Ref 7).

3 Methods to calculate armour stability

3.1 General philosophy

On a rubble structure, the principal parameter set by stability requirements is the armour size, or unit mass, and this influences many of the other dimensions on the structure. If the armour is under-size, it will experience excessive movement / displacement leading to deterioration of the structure, loss of crest level, and hence of efficiency in retaining the beach material. Armour on a rubble mound structure subjected to wave impact and drag forces will resist displacement by mobilising:

a) its own self-weight;
b) interlock with adjoining armour units;
c) friction against adjoining armour units.

It is important to note that an individual armour unit within a well-constituted armour layer will resist substantially greater wave forces than can a solitary unit. It is known that even large solitary units can be moved considerable distances by wave impact forces from relatively moderate wave attack. So once detached from the armour layer, an individual unit will make little further contribution to the overall stability of the system. On a sand or shingle beach, loose units will quickly tend to sink into the beach, making little or no contribution to the performance of the beach / structure. Even for relatively flexible structures such as rock groynes or near-shore breakwaters, it is therefore important to limit armour movement to a small proportion of the armour.

The forces applied by wave action vary with the principal wave characteristics, H_s and T_m, the structure slope angle, cot α, and the local angle of wave attack, β. The resistance forces are also influenced by the structure slope angle; the plan configuration; the size or unit mass of the armour unit, its shape, density of the armour material; and its position on the slope.

Rock armour varies considerably in unit size and shape. The construction of armour layers using this material will give rise to variations in placement density and attitude, and hence in unit interlock and friction. Waves also vary randomly in both height and period. Both disturbing and resistance forces are therefore variable in time and space. This does not allow the use of simple deterministic methods with the same levels of confidence as applied to other areas of structural design.

The displacement of a (small) proportion of the armour units on a rubble groyne or breakwater will however be of relatively little consequence to the overall performance of the structure. The design approach has therefore been to identify the limiting median armour size for a given (acceptable) level of armour displacement. A common limit used in design calculations is displacement of 0-5% of the armour units in the area of armour considered. Simple empirical formulae are then used to calculate the armour size given by the median unit mass M_{50} or nominal unit diameter D_{n50} required for the chosen level of damage at the design wave height.

3.2 Design methods for simple sections

In the design of rock armoured structures, the median armour unit size required to resist direct wave attack therefore constitutes the most important parameter to be determined. Design methods for rock armour focus principally on the calculation of the median armour unit mass, M_{50}, or the nominal median stone diameter, D_{n50}, defined in terms of M_{50} and ρ_r as the cube root of the armour unit volume. It is important to note that the nominal diameter is not an effective diameter, and cannot therefore be measured by any simple process, other than by weighing.

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

The most commonly used armour prediction methods may be summarised:

a) the Hudson formula as used in the Shore Protection Manual, SPM (Ref 8);
b) CIRIA 61, based on studies by Thompson & Shuttler, but now included in the method by Van der Meer, see discussion by Allsop (Ref 10);
c) Van der Meer's equations (Ref 9, and in Ref 7).

Since the original derivation of Van der Meer's formulae, results of further random wave studies in UK and the Netherlands have been used to derive modifications and extensions. These particularly cover: the influence of rock particle shape; stability of thinner armour layers: the effect of wide armour gradings; and the influence of wave attack in shallow water. These further developments are discussed further in sections 3.4 and 3.5 below.

<u>Hudson's formula</u>

Hudson developed a simple expression for the minimum armour weight required for a given wave height. This expression may be re-written in terms of the median armour unit mass, M_{50}, and the wave height, H:

$$M_{50} = \rho_r H^3 / (K_D \cot\alpha\ \Delta^3) \tag{2}$$

where ρ_r density of rock armour (Kg/m^3);
Δ buoyant density of rock, = (ρ_r/ρ_w)-1;
ρ_w density of (sea) water;
α slope angle of the structure face;

and K_D is a stability coefficient to take account of the other variables. For wide-graded rock armour known as rip-rap, values of a coefficient K_{RR} are substituted for K_D.

Values of K_D were derived from model tests using regular waves with permeable sections subject to no overtopping. A range of (regular) wave heights and periods were studied. In each case, the value of K_D chosen was that corresponding to the wave condition giving worst stability. Some re-arrangement of the armour was expected, and values of K_D have been suggested for "no damage" where up to $N_{d\%}$=5% of the armour units may be displaced.

Developments of the Hudson equation

It is sometimes convenient to define a single stability number to substitute for ($K_D\cot\alpha$) in eqn (2); to use the significant wave height H_s rather than H; and to work in terms of the median nominal armour unit diameter, D_{n50}. The Hudson equation can then be re-arranged in terms of the stability number $N_s = H_s/\Delta D_{n50}$:

$$H_s/\Delta D_{n50} = (K_D\cot\alpha)^{1/3} \tag{3}$$

In the 1973 edition of the Shore Protection Manual (SPM), values given for K_D for rough, angular stone in 2 layers on a breakwater trunk were: K_D=3.5 for breaking (plunging) waves, and 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 eqns (2) or (3) with H_S=H.

By 1984 the advice from the US Army Corps of Engineers was more cautious. The SPM now recommended that "the design wave height ... should usually be the average of the highest 10 percent of all waves", $H_{1/10}$=H, and values of K_D were revised (Ref 8). 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 mass by about 3.5! Since 1984, these changes have been recognised as over-conservative, although no formal change had been published by the US Army Corps of Engineers by 1995.

The practice recommended by this author if other methods based on random wave testing are not available, is to use H_s=H with values of K_D based on the results of model tests using random waves.

The Hudson formula does not of itself give any information on the level of damage, or of its development with increasing wave height. Information is however available in the SPM that allows the derivation of a similar equation relating damage $N_{d\%}$ to the relative wave height. Taking the damage level S_d=0.8$N_{d\%}$, a damage formula based on eqn (3) may be written:

$$H_s/\Delta D_{n50} = a\,(K_D\cot\alpha)^{1/3}\,S_d^{\,b} \tag{4}$$

where values of the coefficients a=0.70 and b=0.15 have been suggested by Van der Meer based on results of tests in UK and the Netherlands for rock armour (Ref 9).

Van der Meer's formulae

Van der Meer derived new formulae to calculate armour damage which include the effects of random waves, storm duration, a wide range of core / underlayer permeabilities, and distinguish between plunging and surging wave conditions. The data used to derive the formulae included that used previously in CIRIA 61. For plunging waves:

$$H_s/\Delta D_{n50} = 6.2\, P^{0.18}\, (S_d/\sqrt{N_z})^{0.2}\, \xi_m^{-0.5} \quad (5a)$$

and for surging waves:

$$H_s/\Delta D_{n50} = 1.0\, P^{-0.13}\, (S_d/\sqrt{N_z})^{0.2}\, \sqrt{\cot\alpha}\, \xi_m^{P} \quad (5b)$$

where the parameters not previously defined are:

- P notional permeability factor
- S_d design damage number = A_e/D_{n50}^2
- A_e erosion area from profile
- N_z number of waves
- ξ_m Iribarren number = $\tan\alpha/s_m^{1/2}$
- s_m wave steepness for mean period = $2\pi H_s/gT_m^2$

and the transition from plunging to surging waves is calculated using a critical value of ξ_m :

$$\xi_m = (6.2\, P^{0.31}\, (\tan\alpha)^{0.5}\,)^{1/(P+0.5)} \quad (5c)$$

Recommended values of the damage parameter, S_d, are given below for initial damage, equivalent to $N_{d\%} \approx 0$-5%, intermediate damage, and failure. Failure is assumed when the filter or underlayer is first exposed.

Slope	Damage, S_d		
	Initial	Moderate	Failure
1 : 1.5	2	-	8
1 : 2	2	5	8
1 : 3	2	8	12
1 : 4-6	3	8	17

A range of core / underlayer configurations were used in the test programme, each with an armour layer thickness, $t_a = 2.2D_{n50}$. To each of these configurations, a value of the permeability factor, P, was assigned, Figure 3. 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 important to note that the value of P significantly influences the damage experienced, or the armour size required to limit damage. Values of P have not been established analytically, so the engineer should explore the sensitivity of the calculations to any assumptions made. In particular, values of $P \geq 0.4$ should not be used unless the structure is relatively open and permeable to wave action.

3.3 Angled attack and roundheads

Oblique attack

The design methods for armour stability described above are based on hydraulic model tests in wave flumes where the wave attack is normal to the test section, and where the model section is of relatively simple form. Unfortunately most beach control structures are subject to angled wave attack; they include zones of considerable curvature; and are often designed for quite high levels of wave overtopping. Each of these changes from the "idealised" cases for which the "standard" prediction methods have been derived will have different effects, and corrections to the simple prediction formulae must be estimated.

Oblique wave attack on a simple trunk section is generally expected to give the same or less damage than normal attack on the same slope, rather as wave run-up levels and wave forces on armoured slopes tend to reduce under oblique attack. A simple modification in the use of the Hudson or Van der Meer formulae is to reduce the slope angle $\cot\alpha$ in the calculation by $1/\cos\beta$, where the angle of obliquity β is measured from the normal.

This may however give unsafe results in some cases. Tests by Galland (Ref 15) give results of armour damage measurements on rock armour subjected to long-crested waves at approach angles from $\beta=0°$ to 75°. The results are subject to some qualification, but seem to indicate that for damage, $S_d<2$, there is relatively little reduction in damage to rock slopes for wave attack angle up to about 60°. At $\beta=75°$ damage was however significantly reduced. Tests using short-crested waves reported by Canel & de Graauw (Ref 16) gave more confusing results. Generally the trend appears to be for slightly less damage as the degree of spreading increases from 0° up to 20° for $\beta=0°$. At $\beta=30$ or 45°, damage increases with increasing directional spreading.

These studies suggest that armour sizes on structures attacked by long- or short-crested waves should not be reduced until $\beta>60°$. It is however unlikely that many beach control structures will be subject to significant short-crested wave attack at the design condition, particularly as the design wave condition for most beach control structures will probably be depth-limited, and hence strongly refracted towards $\beta=0°$ at the shoreline. It should be noted that most groynes are however placed normal to the shoreline, so wave attack along the groyne will often fall in the range of relative obliquities $\beta=60$ to 90°,

<u>Roundheads</u>

Curvature of the structure in plan may increase local wave velocities (concave bends or junctions), or may reduce the support and interlock generated between adjoining armour units (convex curves and roundheads). No reliable generic studies have been reported on the stability under random waves of rock armour on roundheads, although it is well established that armour on some zones of a roundhead will be less stable than along a trunk section. The Shore Protection Manual (Ref 8) suggests reduced values of K_D for structure heads under breaking waves, from which a relative increase in armour unit mass may be estimated:

Structure	K_D	Relative increase in M_{50}
Trunk, slope 1:1.5-3.0	2.0	1.0
Roundhead, slope 1:1.5	1.9	1.2
Roundhead, slope 1:2.0	1.6	2.0
Roundhead, slope 1:3.0	1.3	3.6

Vidal et al (Ref 14) tested the stability of a cubic armoured breakwater head and trunk, and concluded that the mass of the armour units on the roundhead should be 1.3 - 3.8 times greater than the mass of the trunk armour units, depending on the level of damage permitted, consistent with the factors estimated above. They observed that the zone of minimum stability at the head of the structure is defined by a 60° sector from the direction normal to the wave front, and that unlike the trunk, armour units lost from the roundhead do not create any protection, but are carried away from the head by the momentum of the waves.

3.4 Effect of particle shape, layer thickness, and grading

The formulae derived by Hudson and Van der Meer were based on the tests of structures protected by 2 layers of rock armour of generally cubic shape. Further wave flume studies by HR Wallingford and Queen Mary College (Ref 11)have identified some influences of armour particle shape and of layer thickness, and derived modifications to Van der Meer's formulae (Refs 10-12).

Effect of rock particle shape

Five different armour shapes were tested by Bradbury et al (Ref 11), representing different stages or forms of quarried and degraded armour: fresh; equant; tabular; semi-round; and very round. Fresh crushed rock, used in most model studies is typical of the random shape of much quarried rock. As in many design specifications, flat or slabby rock was excluded from this category. The fresh rock also provided a control condition which allowed comparison to be made with previous studies.

Equant or nearly cubic rock is typical of rock produced using specialised quarry production techniques, particularly where the natural jointing and bedding allows production of equant blocks. Tabular or slabby rock, with maximum to minimum dimension ratio in excess of 2, is often excluded completely from specifications, but is typical of much quarried limestone or other bedded material.

The effects of armour degradation in the marine environment by abrasion was examined by using two grades of rounded rock, prepared by tumbling the rock in a concrete mixer. Rounding to a weight loss of 5-10%, semi-round, and 20-25%, very round, represented materials which have been subject to varying degrees of abrasion.

The influence of the block shape on armour damage was described by modifications to eqns (5a) and (5b). The coefficient 6.2 in equation (5a), was replaced by C_{pl}, and the coefficient 1.0 in equation (5b) was replaced by C_{su}. The modified formulae become, for plunging waves:

$$H_s/\Delta D_{n50} = C_{pl}\, P^{0.18}\, (S/\sqrt{N_z})^{0.25}\, \xi_m^{-0.5} \qquad (6a)$$

and for surging waves:

$$H_s/\Delta D_{n50} = C_{su}\, P^{-0.13}\, (S/\sqrt{N_z})^{0.25}\, \sqrt{\cot\alpha}\, \xi_m^{P} \qquad (6b)$$

where coefficients C_{pl} and C_{su} calculated for the shapes tested, and for t_a=1.6D_{n50}, are given:

	C_{pl}	C_{su}
Fresh	6.32	0.81
Equant	6.24	1.09
Semi-round	5.96	0.99
Very round	5.88	0.81
Tabular	6.72	1.30

The values of C_{pl} and C_{su} indicate higher stability for block shapes other than very round rock. As the results are only based on a limited number of tests (1:2 slope and impermeable core), the modified formulae should be used with care. A safe approach is to use Van der Meer's equations un-modified, except for very rounded rock. This shape of rock armour is used occassionally in ports around the Baltic, where glacially rounded "sea stones" can be found in profusion, but very round rock is usually excluded from specifications in the UK and elsewhere.

Armour layer thickness

Conventional two layer armour construction is generally expected to give a layer thickness of t_a=2.0 to 2.2D_{n50}. It is however possible to place two layers of armour to form a rather thinner layer. In the tests by HR / QMC (Refs 10-12), measurements of layer thickness on rock armour gave results contrary to those expected from the more commonly used methods. In each case, the thickness measured was less than the thickness calculated using the SPM method (Ref 8), using k_Δ=1.1 for a 2 layer thickness of armour, nominally t_a=2.2D_{n50}.

The procedure adopted by HR / QMC for armour layer construction in this study differed from those studies where the armour layer is formed to full depth in a single operation moving up the slope. In contrast armour was placed on the slope to cover the underlayer in a single stone thickness. The second layer was then added, typical of some methods used for two layer armouring to sea walls and breakwaters. The average thicknesses measured in this study of t_a=1.6D_{n50} represents a 30% reduction in layer thickness on earlier studies, and probably a similar reduction of the total armour volume.

Careful consideration of the permeability factor P and its use in Van der Meer's equations suggests that damage would be expected to increase with any reduction in depth of permeability. Analysis of the damage measurements gave an indication of the influence of layer thickness by a simple adjustment of the power coefficient of $(S_d/\sqrt{N_z})$ in eqns (5a) & (5b) from 0.2 to 0.25, giving a better description of the damage for armour layers of thickness t_a=1.5-1.7D_{n50}. The modified equations then become, for plunging waves:

$$H_s/\Delta D_{n50} = C_{pl}\, P^{0.18}\, (S_d/\sqrt{N_z})^{0.25}\, \xi_m^{-0.5} \tag{7a}$$

and for surging waves:

$$H_s/\Delta D_{n50} = C_{su}\, P^{-0.13}\, (S_d/\sqrt{N_z})^{0.25}\, \sqrt{\cot\alpha}\, \xi_m^{P} \tag{7b}$$

Initially these changes seem to be counter-intuitive, in that the rate of damage with wave height reduces from H^5 to H^4. For realistic cases however, values of $S_d/\sqrt{N_z}$ vary between 0.15 and 0.03, where the effects of the modifications are to increase damage with decreasing layer thickness. This may be illustrated by considering the size of rock on a 1:2 slope of P=0.4 against waves of H_s=2m, T_m=6s, N_z=1500 waves. For the "standard" or thicker layer, t_a=2.2D_{n50}, an armour unit mass of M_{50}=0.85 tonne is needed for a damage level S_d=2. If the armour is laid to form the thinner layer, t_a=1.6D_{n50}, an armour unit mass of M_{50}=1.3 tonne is needed for the same damage level.

Wide graded rock armour

In the studies by Van der Meer and others, rock armour gradings of D_{85}/D_{15}=1.25 and 2.25 had been tested with layer thickness of t_a=2.2D_{n50}. A grading of D_{85}/D_{15}=1.25 represents a narrow graded rock armour carefully selected by individual unit mass. A grading of D_{85}/D_{15}=2.25 represents wide graded material usually termed rip-rap. Each of these gradings are however substantially narrower than any grading produced by blasting in the quarry, and during the compilation of the CIRIA / CUR rock manual it was argued that there might be significant cost savings by using rather wider gradings than D_{85}/D_{15}=2.25 . A series of hydraulic model tests by Allsop (Ref 12) were therefore conducted to explore the performance of $D_{85}/D_{15} \leq 4.0$.

Gradings used in earlier studies had generally conformed to a straight line log-linear grading, but quarry production curves seldom approach a straight line. A more typical form for armour rock is given by the Schuman equation. Two batches of material with a grading of D_{85}/D_{15}=4.0 were prepared, one to the Schuman equation, and the other to a log-linear form. The two gradings gave the same sizes at 85% and 15% exceedance levels, but differed substantially at the 50% level. The log-linear grading, having a smaller median size was constructed to a layer thickness of t_a=2.7D_{n50}. Both wide gradings were placed in bulk, rather than by individual units.

Results from these tests showed more scatter than those on conventional gradings, perhaps due to the wide variation in armour size along the sample length. There was a noticeable preferred movement of the small fraction of rock, resulting in reduced support to the larger rocks in the armour layers, and damage at higher wavae heights was more abrupt than with narrow gradings. Analysis of damage to the alternative wide gradings prepared to the log-linear and Schuman gradings indicated no significant differences when the median size, D_{n50} was used to describe the armour size. Scatter in the results did not permit further modification to design formula, but the potential for wide variations in (local) damage suggests that such wide gradings should not be adopted without further site specific studies.

3.5 Effect of depth-limited wave attack, or of low-crest levels

Depth-limited wave attack

In the methods used above, the significant wave height H_s has been used in the stability equations. In shallow water conditions the distribution of the wave heights will deviate from the Rayleigh distribution (truncation of the curve due to wave breaking). Further tests on a 1:30 slope foreshore by Van der Meer suggested that $H_{2\%}$ might be a better value of wave height in shallow water in describing the effect of depth-limited wave conditions, rather than H_s. Van der Meer re-arranged eqns (5a) and (5b) in terms of $H_{2\%}$, so that for plunging waves:

$$H_{2\%}/\Delta D_{n50} = 8.7\ P^{0.18}\ (S/\sqrt{N})^{0.2}\ \xi_m^{-0.5} \tag{8a}$$

and for surging waves:

$$H_{2\%}/\Delta D_{n50} = 1.4\ P^{-0.13}\ (S/\sqrt{N})^{0.2}\ \sqrt{\cot\alpha}\ \xi_m^{P} \tag{8b}$$

These equations can be used where the value of $H_{2\%}$ can be calculated, or derived from model tests. Where wave heights are Rayleigh distributed, eqns (8a) and (8b) give the same results as eqns (5a) and (5b), but for depth-limited conditions the ratio of $H_{2\%}/H_s$ will be smaller, and the armour size required for stability may therefore be reduced.

There is however a potential problem with the use of these equations to reduce the size of armour required. In cases where the design wave condition is significantly depth-limited, it is probable that other combinations of wave condition and water level of lower return period will also give the same or very similar inshore wave heights. The frequency with which this design case is met will therefore be substantially increased, in turn significantly increasing the frequency with which small levels of damage will accumulate over the life of the structure.

Low-crest sections

The design methods for armour stability described earlier are based on hydraulic model tests in wave flumes where the test sections extended upwards to a level that allowed relatively little wave overtopping. Lower crest levels may allow increased wave overtopping, reducing the chance of damage to the armour on the front face, but increasing (potential) damage on the crest and rear face. The SPM (Ref 8) suggests that low-crest breakwaters subject to heavy overtopping may need increased armour sizes on the rear face, but does not comment on the influence that allowing significant levels of overtopping will have on the stability of the front face armour. The CIRIA / CUR manual (Ref 7) however gives a simple correction factor, f_i, to be applied to the size of the armour on the front face when described by its nominal diameter, D_n:

$$f_i = 1 / (1.25 - 4.8R^*_p) \qquad (9)$$

which is valid over the range $0<R^*_p<0.052$ where $R^*_p = (R_c/H_s)(s_p/2\pi)^{0.5}$.

4 Wave basin tests on beach control structures

4.1 Aim of tests

The aim of the physical model tests described by Jones & Allsop (Refs 4 & 5) was to quantify the stability of rock armour on 3 or 4 shingle beach control structures and hence to check the application of the design methods discussed in Chapter 3 to such structures. Three control structures were tested: a low-level or inclined crest groyne (Type 1); a high level roundhead groyne (Type 2); and a high level "L" shaped groyne. Plans and sections of these three structures are shown in Figures 4 and 5. A 1:2 rock armoured sea wall slope was also tested under direct wave attack, $\beta=0°$, as a comparison. The results from these tests were used to check the application of the simple methods for "standard" structures discussed in Chapter 3, and/or to be used directly by engineers designing or analysing these types of structures.

The tests described by Jones & Allsop (Ref 4) were conducted in the Roundhead Test Basin at HR Wallingford. The different model sections were constructed in pairs on a beach slope of 1:7. Each structure was profiled and photographed before any waves were run to provide control measurements against which damage could be quantified, and was then re-surveyed and photographed after each test. Testing was considered complete when the structures were damaged to such an extent that total re-building was necessary.

The principal measure of armour response was the comparison of surface profiles of the armour before and after each test, used to calculate armour damage values S_d. Example profiles on the Type 2 roundhead groyne are shown in Figure 6.

4.2 Model test results

The test results were surprising as nearly all of the profiles showed significantly more damage than predicted by conventional formulae. Some local increases in damage had been expected, as discussed in 3.3 above, but not the consistently greater damage found here.

Initial analysis focused on the 1:2 sea wall slope, and equivalent section on the L-shaped groyne, but even these simple configurations showed substantially greater damage than predicted by Van der Meer's formulae. Additional data on damage to rubble groynes were obtained from the CEPYC laboratory in Madrid under an EU MAST project. Some analysis of these had been presented by Baonza & Berenguer (Ref 17), and their test data were further analyzed by Allsop & Franco (Ref 18) under MAST project G6-S. These data suggested that the sea bed slope might be the significant factor in increasing the armour damage, probably by modifying the kinematics of waves breaking at and onto the structures tested.

<u>Simple sea wall and related sections</u>

Damage results for the sea wall section and the front face of the L-shaped groyne are summarised in Figure 7, using axes based on Van der Meer's plunging wave formula, eqn (5a). This prediction using P=0.5 significantly under-predicts the measurements, and a modified coefficient was developed for eqn (5a) to describe the effect of the 1:7 bed slope on increasing armour damage. The plunging waves equation may then be written:

$$H_s/\Delta D_{n50} = 4.8P^{0.18} (S_d/\sqrt{N_z})^{0.2} \xi_m^{-0.5} \qquad (10a)$$

This equation was derived to fit the measurements on the 1:2 sea wall slope, and then compared with the damage on the front face of the L-shaped groyne with relatively good agreement. A similar increase for surging waves is suggested:

$$H_s/\Delta D_{n50} = 0.77P^{-0.13} (S_d/\sqrt{N_z})^{0.2} \sqrt{\cot\alpha}\, \xi_m^{P} \qquad (10b)$$

The Van der Meer formulae were derived for structures in relatively deep water, and take into account the effect of structure slope on stability through the surf similarity parameter. The results of these studies indicate that a further parameter may be required to describe the effects of modifications as waves approach the structure over steep bed slopes. The simple modifications to the formulae suggested here are not however intended to reflect the complex nature of the changes to the waves or the response, but simply to allow a first estimate of the increased damage to be calculated.

<u>Types 2 roundhead groynes</u>

Damage to the roundhead or bastion groyne Type 2 varied spatially, depending both upon the severity of (local) wave attack, and on the resistance to movement afforded by the local structure geometry. The net effect of these influences was that the most severe damage was not on the seaward faces of the structure 0° ± 90°, but in the transition zone from 90° to 135°. In this zone the armour is least able to resist the oblique jet of water passing over and around the slope at ± 90° to the structure axis. The zone with least damage was, as expected, that running up the beach back from the 135° sector.

On the "L" shaped section the more severe damage covered 45° to 135° on the outer sector. Once the effect of the steep beach slope had been accounted for, the test results suggest that Van der Meer's simple prediction method as modified in eqn (10a-b) adequately describes the average level of damage, as shown in Figure 7 for the front face of the L-shaped groyne.

Folllowing the discussion on stability of roundheads in Chapter 3, it would be expected that curved parts of the structures might experience greater damage than the simple sections discussed above, and this is illustrated by the damage sustained on the curved parts of the L-shaped groyne shown in Figure 8. The damage on these sections was greater than predicted by eqn (10a), and analysis suggests that a further increase in armour mass by 1.5-1.75 may be required to ensure that the required design damage level can be achieved. These increases are however still consistent with those suggested for breakwater roundheads in Chapter 3.

Type 1 inclined groyne

Damage to the inclined groyne, Type 1, varied along its length, with the location of greatest damage depending on wave height and period. The mean level of damage taken over the active length of the groyne, and derived by averaging the erosion areas from each profile, fits the general prediction given by eqn (10a), and is summarised in Figure 9.

Peak values of (local) damage along the length of the groyne however reached twice the mean, often at the wave run-up and run-down limits along the groyne, and this is illustrated in Figures 10 and 11 which show local levels of damage plotted against position along the groyne from the landward end for increasing relative wave heights. For $H_s/\Delta D_n50$ up to 1.72, damage only exceeds S_d=5 at 1000 waves over small regions. For $H_s/\Delta D_n50$=2.16, however, damage over most of the length of the structure has exceeded this criterion.

4.3 Discussion on damage

The influence of angled attack on these types of structures was not studied in the HR tests, but some results were derived from the CEPYC tests with waves 10° off the structure axis, β=80° to the structure trunk. Those results suggest that damage may increase on the seaward side, and decrease on the sheltered side, but that the overall level of damage is not significantly changed. This suggests that the conclusions from these studies can be applied to structures under direct and (slightly) oblique attack.

The design event for the stability of these structures will probably be of 1:30 - 50 year return period. Storms of this severity will inevitably generate longer wave periods which will in turn be influenced more significantly by wave refraction, thus approaching the shoreline at smaller relative angles than more frequently occurring conditions. The obliquity of wave attack at the shoreline is therefore likely to decrease with increasing severity of the storm.

The studies at HR and CEPYC considered Type 2 groynes armoured with a single (median) armour size. The influence of waves around these strongly 3-dimensional structures varies widely, and damage to the armour therefore varies significantly over the different zones surveyed. The most severely damaged parts of Type 2 groynes were around the 90° - 135° sectors, and around the 45° - 135° sectors on the "L" shape. Here damage was generally consistent with the discussion on roundheads in Chapter 3. Where larger armour is more expensive than smaller rock, it may be economic to armour the outer end and transition of a bastion groyne with larger material, reducing this protection on the main trunk.

Typical levels of damage that might be tolerated by rock armoured structures are in the range S_d=2 to 8. In general however, beach control structures differ markedly from sea walls or breakwaters in that the consequence of excessive armour movement is less immediate, and may be significantly less severe. The opportunity to restore crest level or side slopes on these structures simply by adding more armour is also more easily available than for many sea wall or breakwaters. It is probable therefore that the level of damage S_d that may be permitted under the design storm may be rather greater than the level permitted for most sea walls or breakwaters where damage is typically limited to S_d=2. The results of these tests suggest that a design damage level of S_d=6 might be used with eqns (10a) or (10b) for large beach control structures where damage to the toe or side slopes will not immediately lead to erosion of the crest.

The results of this work were not intended to be applied to breakwaters in deeper water, and none of the tests considered here specifically addressed these structures. The general levels of damage around the curved parts of the L-shaped groyne do however confirm the armour mass increases suggested in Chapter 3 for roundheads.

5 Measurements of stability performance in the field

5.1 Initial analysis of damage

Rock revetment

During the initial analysis of beach control structures, there were indications that some structures had suffered more damage than expected. Profile line measurements on a rock revetment on Hurst Spit illustrate levels of damage suffered by some structures. This temporary revetment was constructed with a thin armour layer of D_{n50} = 0.9-1.0m, and rock of density ρ_r = 2500kg/m^3. The revetment has been monitored by New Forest District Council, with five profiles at 100 metre intervals along the structure, and levels expressed to a local datum, LD. Significant damage was experienced at Profile lines 2 and 4, shown in Figure 12, between October and December 1989.

Wave analysis suggests that the storm wave condition between October and December 1989 might have been given by H_s = 2.2m, T_m = 7.7s, with a storm duration equivalent to N_z = 3000 to 5000 waves. The survey measurements at Profile 2 in Figure 12 show significant erosion of the revetment crest, a form of damage not described by the design methods discussed earlier. Damage to Profile 4 of the revetment above the level of significant shingle beach movement was however of the general form expected from previous laboratory tests. Analysis of profile changes above 2mLD suggest that the damage was equivalent to S_d = 6.8. This may be contrasted with damage predicted by eqn (5a), of S_d = 0.8. The modified eqn (10a), takes account of the steep beach slope, and the predicted damage increases to S_d = 2.8. Then adjusting the calculation to take account of the reduced armour thickness as in eqn (7a), the predicted damage increases further to S_d = 5.3. This is still smaller than that calculated from Profile 4, but probably greater than that experienced at other profiles, and gives reasonable confidence in the application of the methods discussed in Chapters 3 and 4..

In contrast to the high levels of damage monitored at Hurst revetment, the rock revetment at nearby Barton-on-Sea, armoured with a more conventional two layer thick armouring, has shown no signs of wave induced damage. The use of Van der Meer's formulae to describe the performance of the rock revetment at Barton-on-Sea appears to be appropriate for this site for the range of wave conditions measured.

Rock groynes

Analysis of the performance of rock groynes at full scale presents significant practical problems. Profiling of the structures is extremely difficult, especially on the roundheads which may be at least partially submerged at all states of the tide. Photographic methods of analysis present safer and more reliable opportunities for structure analysis. To date some qualitative analysis of ground level and high level aerial photography has been carried out on rock groynes at Milford-on-Sea and Barton-on-Sea, and important observations have been made which help to support observations in the laboratory.

The groynes at both sites are Type 1, armoured with similar sizes of rock and subject to very similar nearshore wave climates, but have performed quite differently under similar storm conditions. Both groups of groynes have suffered some damage, but those at Milford-on-Sea have performed significantly worse. Photographs show significant damage to the groynes, and inspection of the site at Milford-on-Sea suggests that the groynes reached failure under storm conditions. Rock armour was strewn widely about the beach and no clearly recognisable groyne geometry was visible after the storms. The groynes at Barton-on-Sea exhibited considerably less damage under severe storm events. Photographs indicate concentrations of damage at the junction of the roundhead with the groyne trunk on the down-drift side of the groyne. Wave conditions during storms resulted in wave breaking on the structures and wave overtopping. Storms of greater severity offshore, but at lower water levels, have resulted in no visible damage to the structures.

On initial examination the two groyne systems appear to have very similar hydrodynamic and structure variables. Closer examination of the sites however has demonstrated that the approach bathymetry is much steeper seawards of the groynes at Milford-on-Sea than at Barton-on-Sea. The approach bathymetry from the structure toe to approximately 200m offshore at Milford-on-Sea is 1:25, but at Barton-on-Sea the equivalent beach slope is substantially shallower at 1:90. This again supports the conclusions of the model studies.

The performance of these groyne systems had therefore given cause for concern. Recent beach management schemes for both areas provided the opportunity to examine these structures further in site specific physical models. Satisfactory designs allowing for low damage levels ($S_d < 3$) have been achieved for Barton-on-Sea. The stability performance of groynes on the steep foreshore at Milford-on-Sea have however provided significant design problems. Even allowing for considerable increases in the armour size and flattening of the roundhead slopes, it was not possible to achieve a stable structure design in the hydraulic model tests given the practical constraints of rock armour size and construction procedures. In such situations, the acceptance of higher damage and hence increased maintenance may provide a more economic solution unless alternative types of structure can be used.

5.2 Future field measurements

The uncertainty in design for stability, and lack of suitable empirical formulae for the design of rock groynes present some problems. Model test results and site monitoring together have resulted in some concern on the economies of the design of these structures. With this in mind, a carefully coordinated field measurement programme supported by MAFF has been designed to compliment model study results and provide additional guidance for design and maintenance. A more detailed programme of monitoring by HR Wallingford and New Forest District Council, which includes those structures previously discussed, commenced in December 1994.

Measurements of damage to rock revetments will be carried out using laboratory techniques applied to the field. The measurement programme includes measurement of waves and tides as well as structure response. The rock groynes will be monitored using low level vertical aerial photography taken from a ground controlled ballon, or from conventional aerial photographs. Analysis will be by photographic and profile comparison methods based on those commonly used for laboratory studies.

6 Conclusions and Recommendations

6.1 Conclusions

A series of hydraulic model tests have been performed using random waves to investigate the degree of armour damage sustained by a number of beach control structures. Damage measured in these tests has been compared with that predicted by formulae derived for "standard" breakwater or revetment construction. The following conclusions may be drawn:

1. Two standard formulae are available to calculate rock armour sizes needed to resist damage by wave action, the Hudson and van der Meer formulae. The Hudson formula, developed from tests using regular waves, gives a simple expression for the minimum armour weight required for a given wave height. Van der Meer developed prediction formulae to calculate armour damage based on tests using random waves. The formulae include the effects of a range of core / underlayer permeabilities, and distinguish between plunging and surging wave conditions.

   The coefficient K_D, used with the Hudson formula, has been derived for a variety of structure configurations including trunks and roundheads. No similar extension of the Van der Meer formulae have been derived. The mass of an armour unit on or near a roundhead may need to be increased by factors up to or even in excess of 2.

2. The stability of armour on rock armoured beach control structures has been shown to depend critically on the local sea bed slope. Tests performed by CEPYC identified the relative effect of beach slope on damage, but were unable to quantify the effect. Tests by HR on a shingle beach slope of 1:7 indicated a clear dependence of armour damage on beach slope, and were used to develop modified formulae for use with steep beach slopes, based on the Van der Meer formulae, modified in eqns (10a) and (10b). The changes to the coefficients are equivalent to increasing the mean armour mass by a factor of 2.2 to maintain armour stability.

3. The influence of angled wave attack on beach control structures was studied by CEPYC. The tests demonstrated that at a relative angle of 80°, armour damage may increase on the seaward side, and decrease on the sheltered side, but overall damage was not significantly altered. It is probable that the conclusions from both the HR and CEPYC tests may therefore be applied to structures under either direct or slightly angled wave attack.

4. Damage to the inclined groyne, Type 1, varied along its length, with the location of greatest damage depending on wave height and period. The mean level of damage taken over the active length of the groyne, and derived by averaging the erosion area for each profile, fits the general prediction given by eqn (10). Peak values of (local) damage often reached twice the mean, often at the wave run-up and run-down limits along the groyne.

5. Damage to the roundhead or bastion groyne, Type 2, also varied spatially. The most severe damage was in the transition zone from 90° to 135°. In this zone the armour is least able to resist the oblique jet of water passing over and around the slope at ± 90° to the structure axis. The least damage zone was, as expected, that running up the beach back from the 135° sector. On the "L" shaped section the more severe damage covered 45° to 135° on the outer sector. Damage on these severely attacked sections was greater than predicted by the (modified) design formula, eqn (10), but consistent with the requirement for a further increase in armour mass by 1.5-1.75 that would be expected for a roundhead.

6. Three different groyne structures were tested in the study. It was not possible to say whether an individual groyne suffered more damage than any of the others. The relative merits of constructing a Type 1, a Type 2, an "L" or "T" shaped groyne should be decided on influences other than damage levels ie beach topography, the intended impact on the beach, tidal ranges, ease of maintenance and construction cost etc.

7. This study was completed on structures built in relatively shallow water. The general conclusions may be applicable to near-shore structures, and the damage results from the "L" shaped groyne may be used to deduce armour sizes required for the ends of near-shore detached breakwaters.

6.2 Recommendations

1. This study identified some unusual effects, and led to some unexpected conclusions. The evidence from this study and the MAST G6-S project are not sufficient on their own to justify new general formulae. It is recommended that the performance of other structures behind steep (beach) slopes should be reviewed to identify whether a more general effect may lead to problems.

2. It is clear that the form of wave breaking on steep beach slopes typical of UK shingle beaches (1:7 to 1:10) may be rather different than on the typical beach slopes for which most design methods have been developed (generally 1:50 to 1:100, occasionally 1:30). It is recommended that wave flume tests measure the form and process of wave breaking for typical UK sea states on a number of sea bed slopes: perhaps 1:30 to give a control and compare with previous data; 1:10 and 1:7 to give responses typical of UK shingle beaches slopes. Measurements should then be made of armour

displacement on a rock armoured slope, and perhaps of wave impact pressures on a seawall or revetment slope.

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Figures

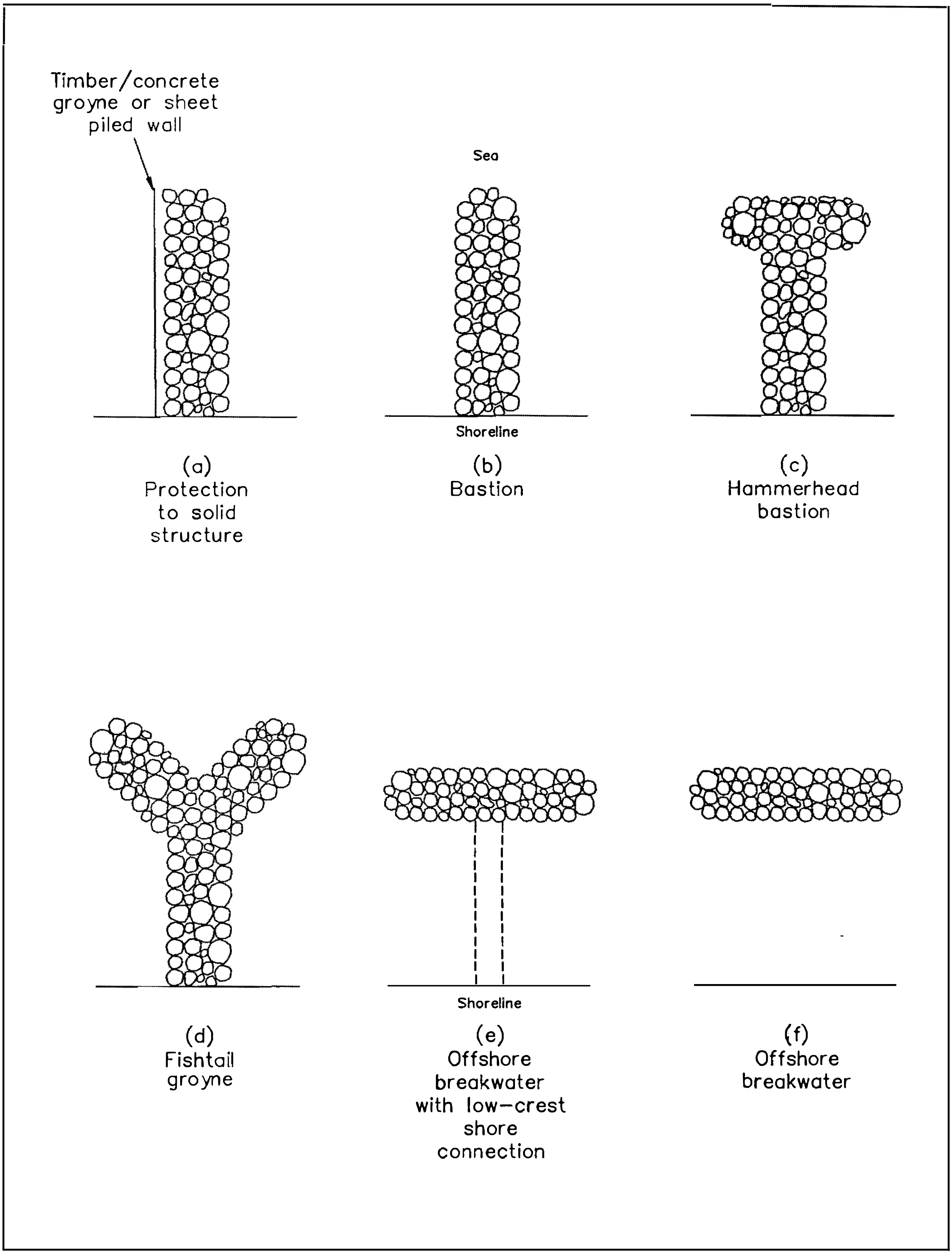


Figure 1 Types of beach control structures from simple groynes to near-shore breakwaters

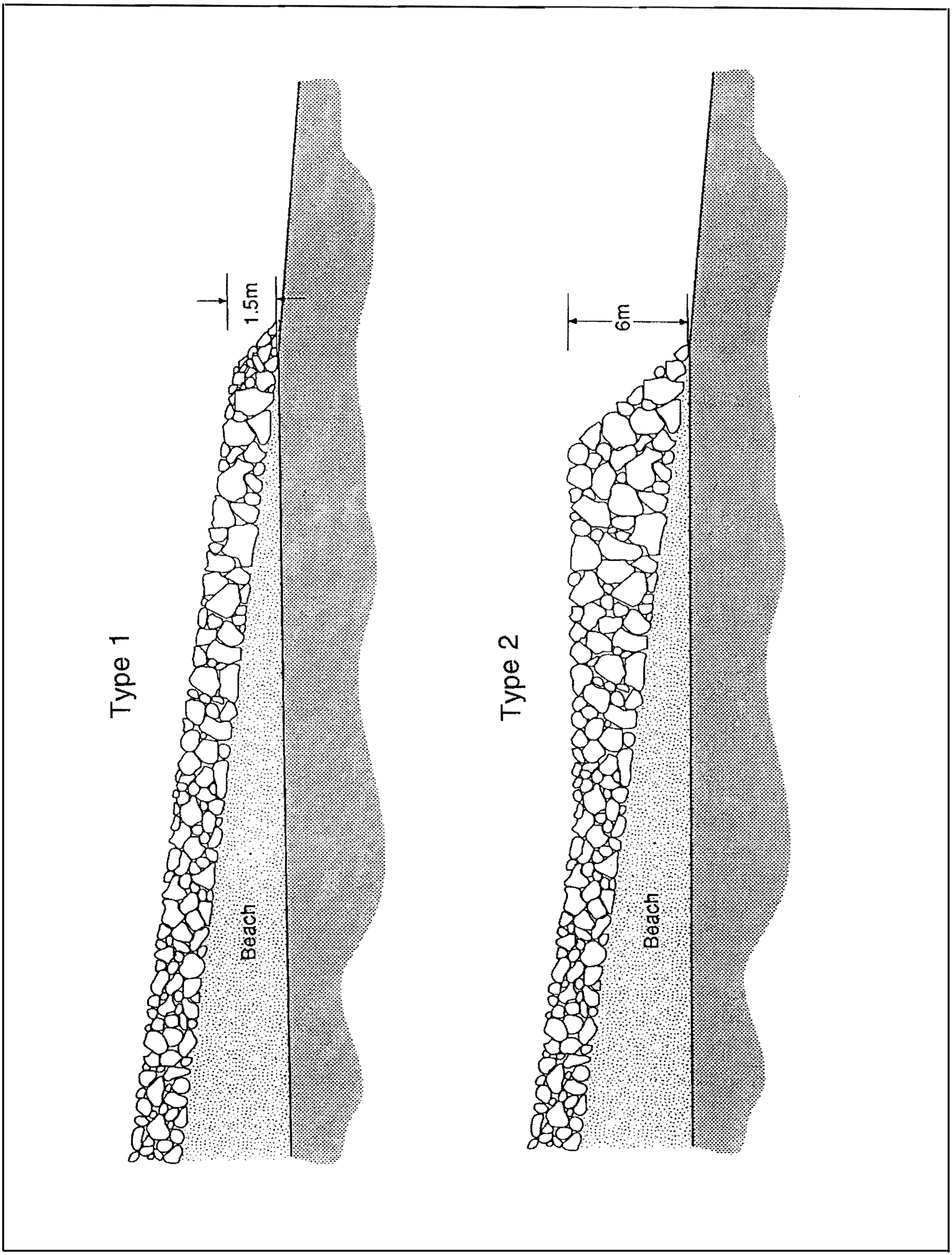


Figure 2 Types and profiles of rock armoured groynes

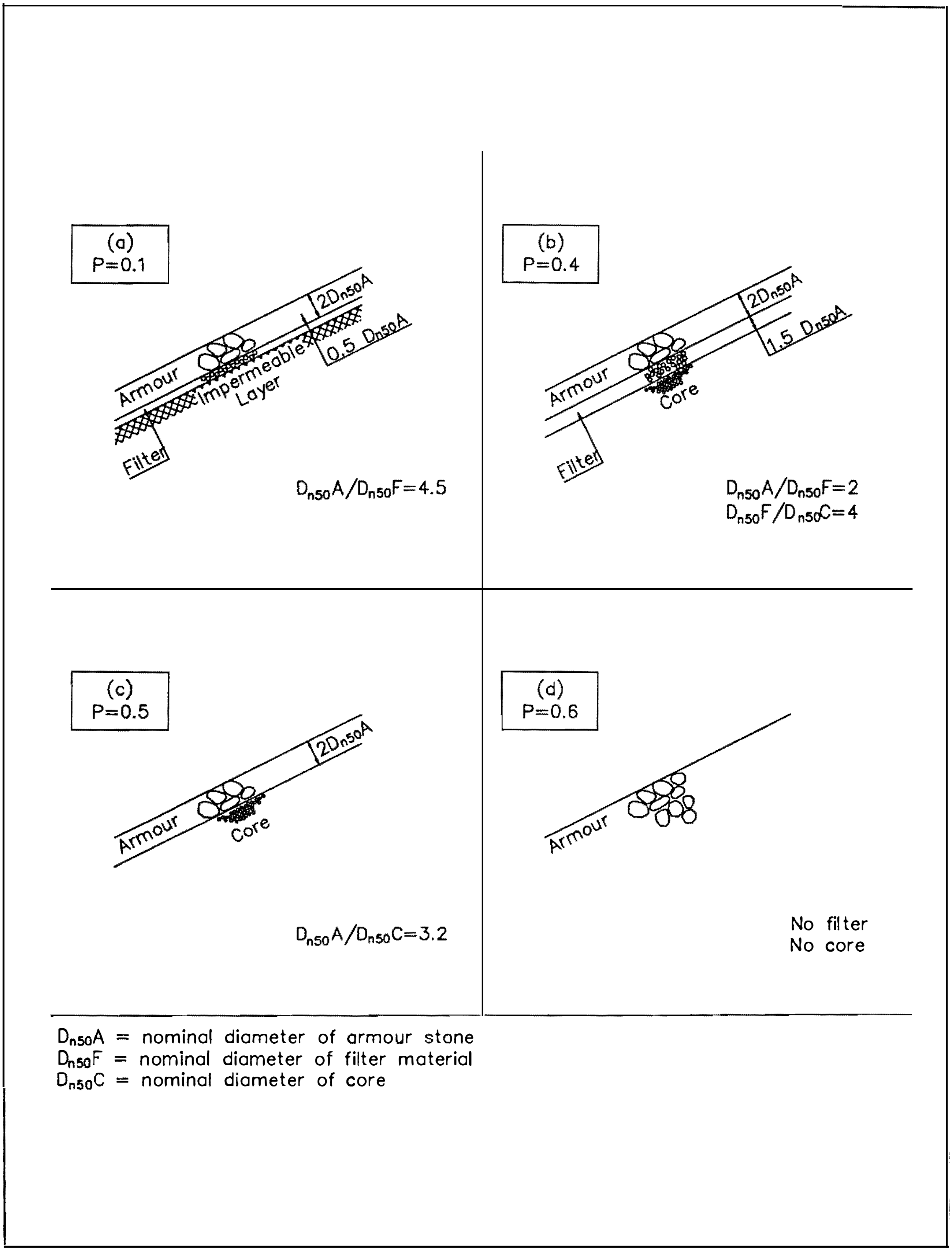


Figure 3 Van der Meer's permeability factor

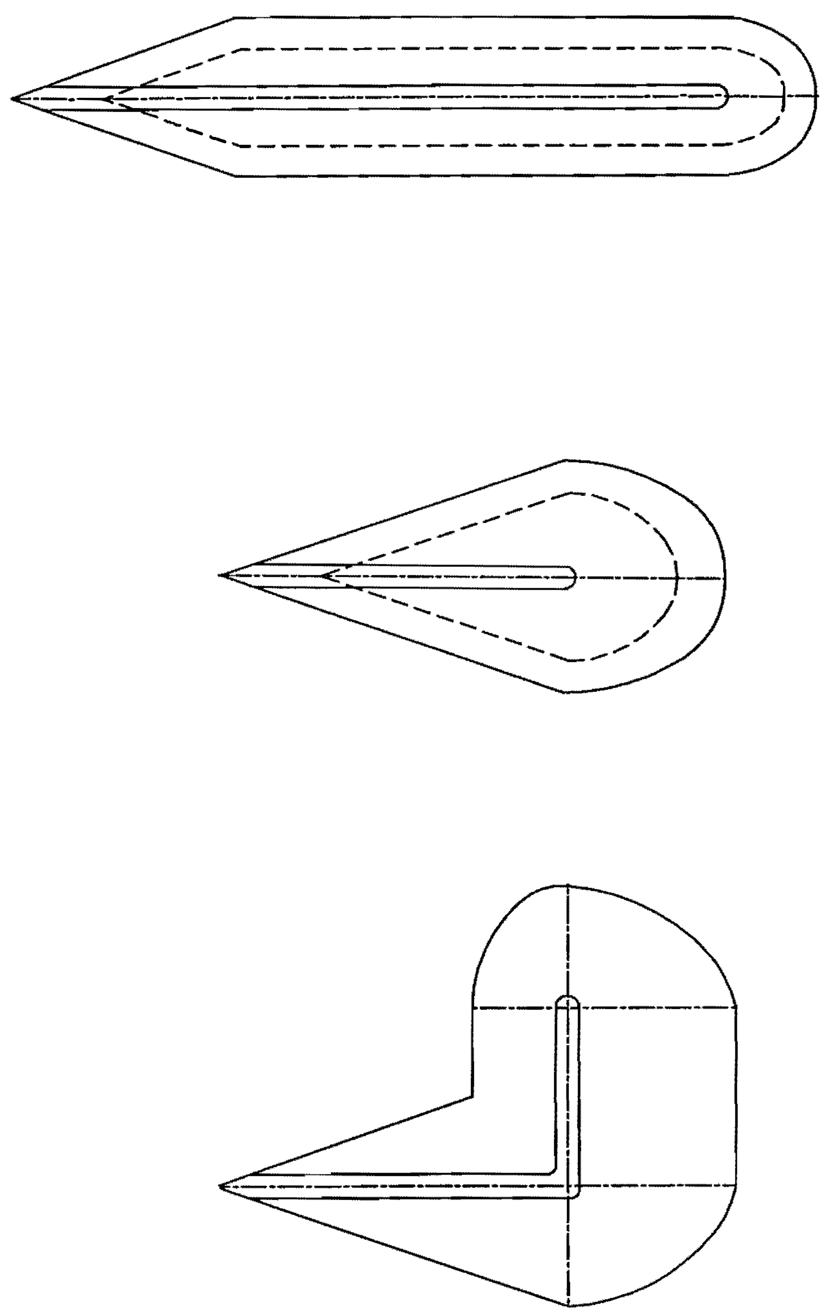


Figure 4 Type 1, Type 2, and L-shaped groynes, plan

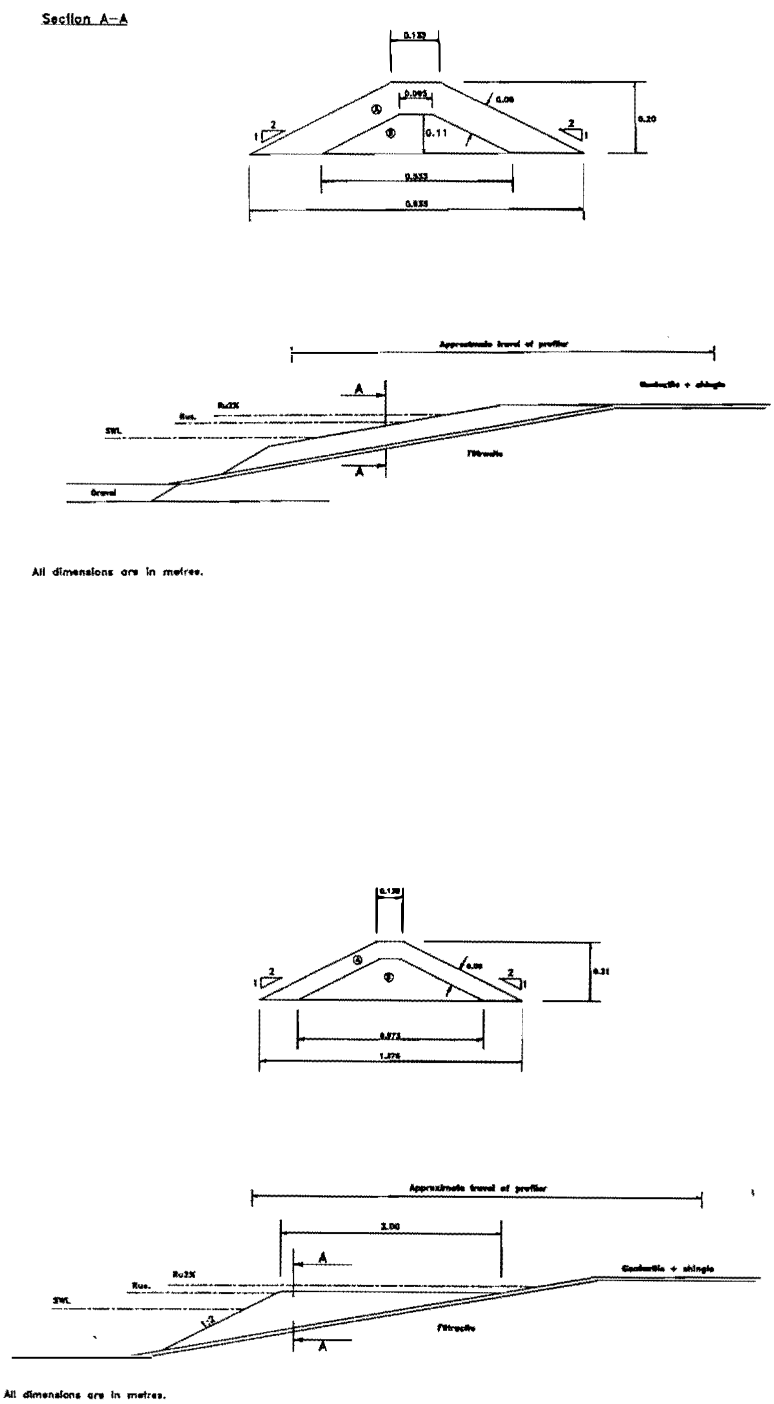


Figure 5 Type 1 and Type 2 groynes, sections

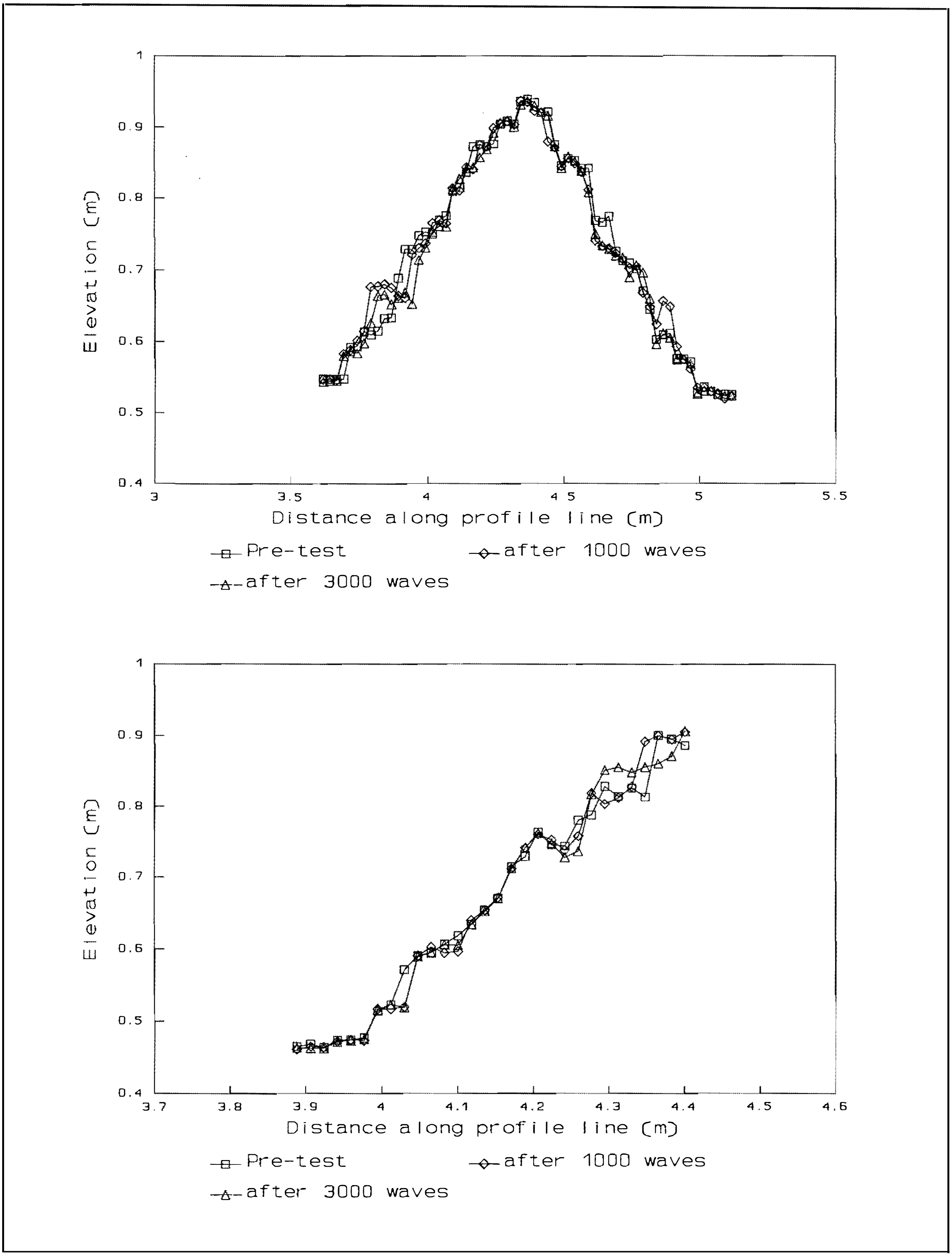


Figure 6 Example damage on Type 2 groyne

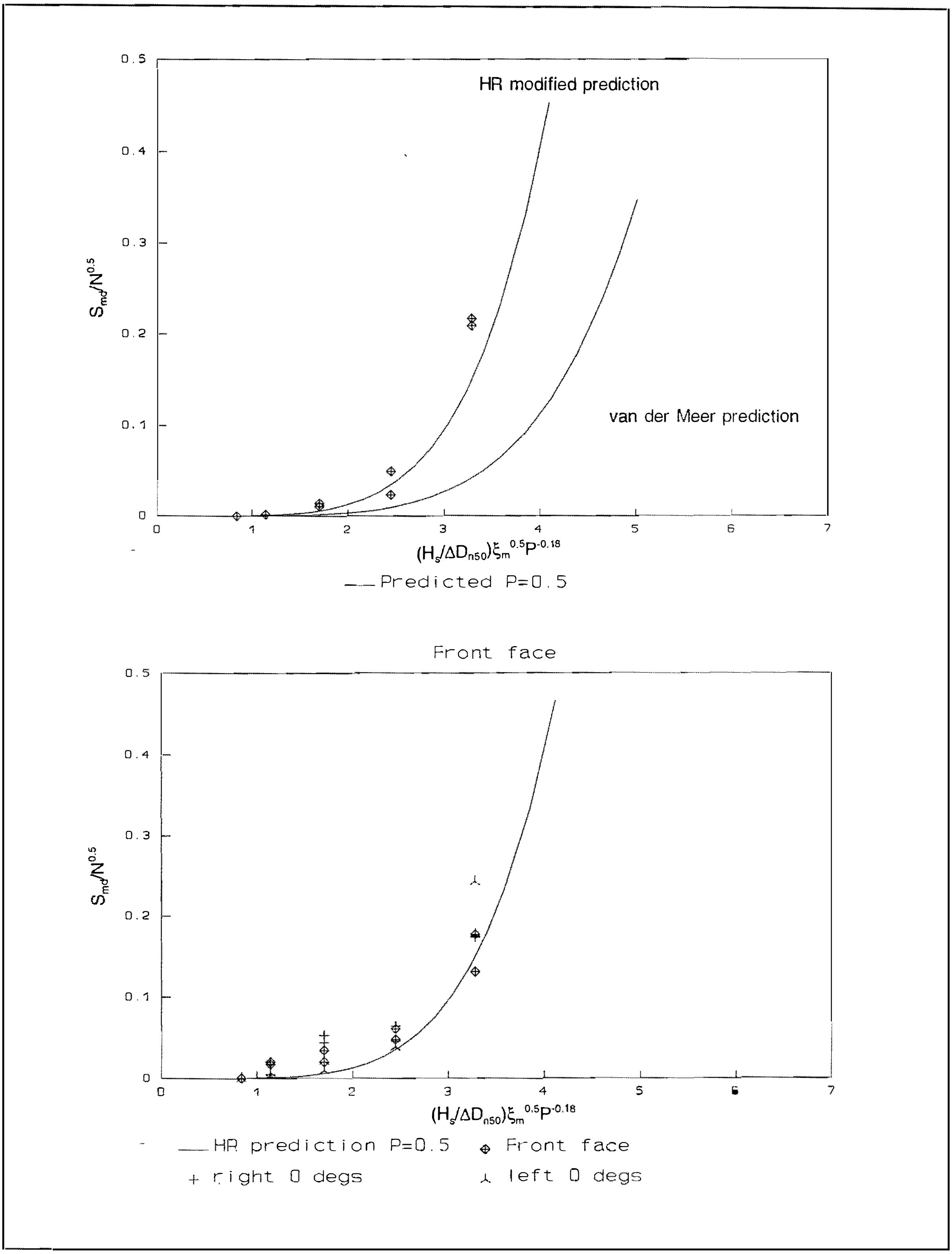


Figure 7 Damage to 1:2 sea wall and front face of L-shaped groyne

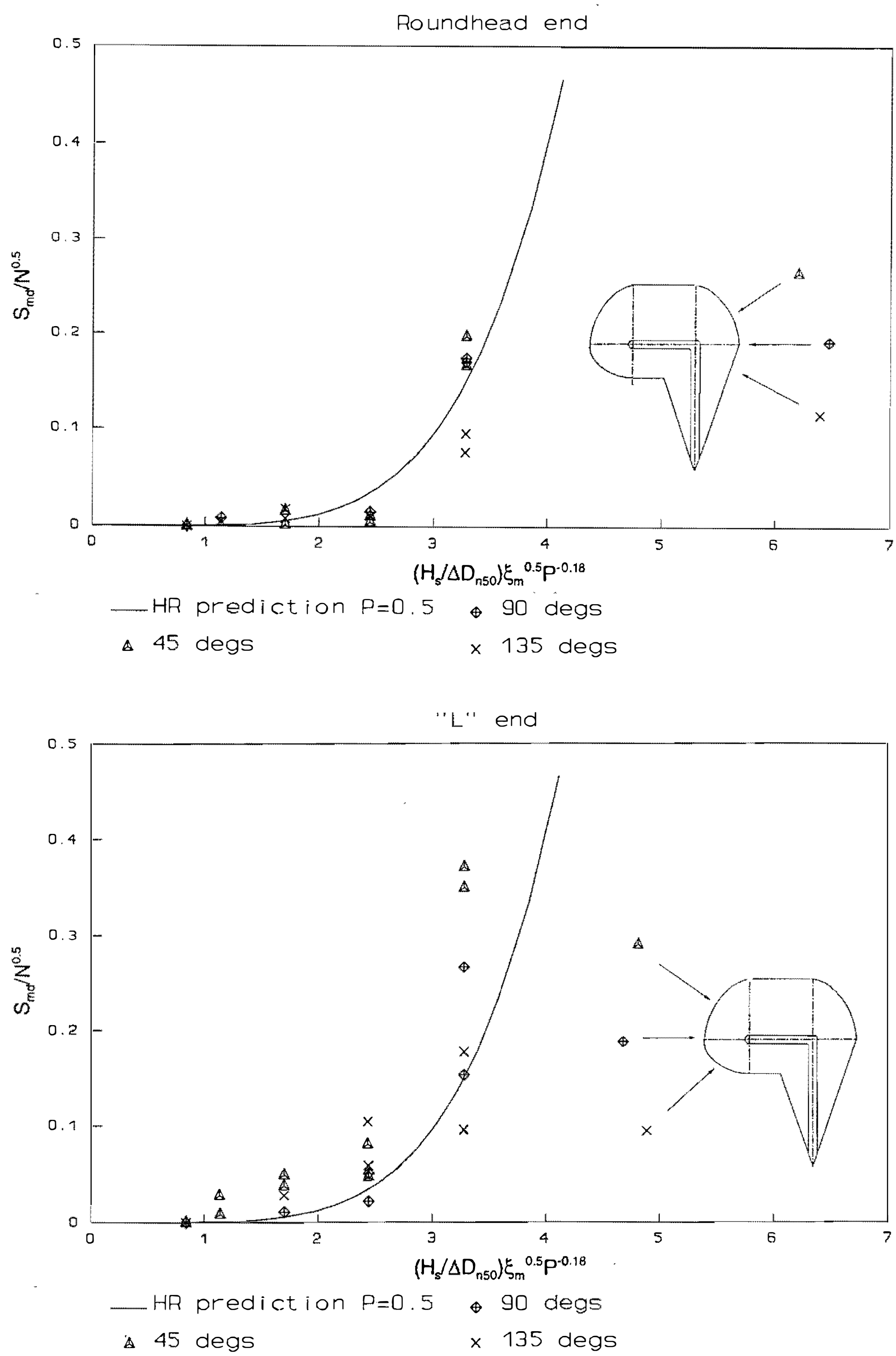


Figure 8 Damage to curved parts of L-shaped groyne

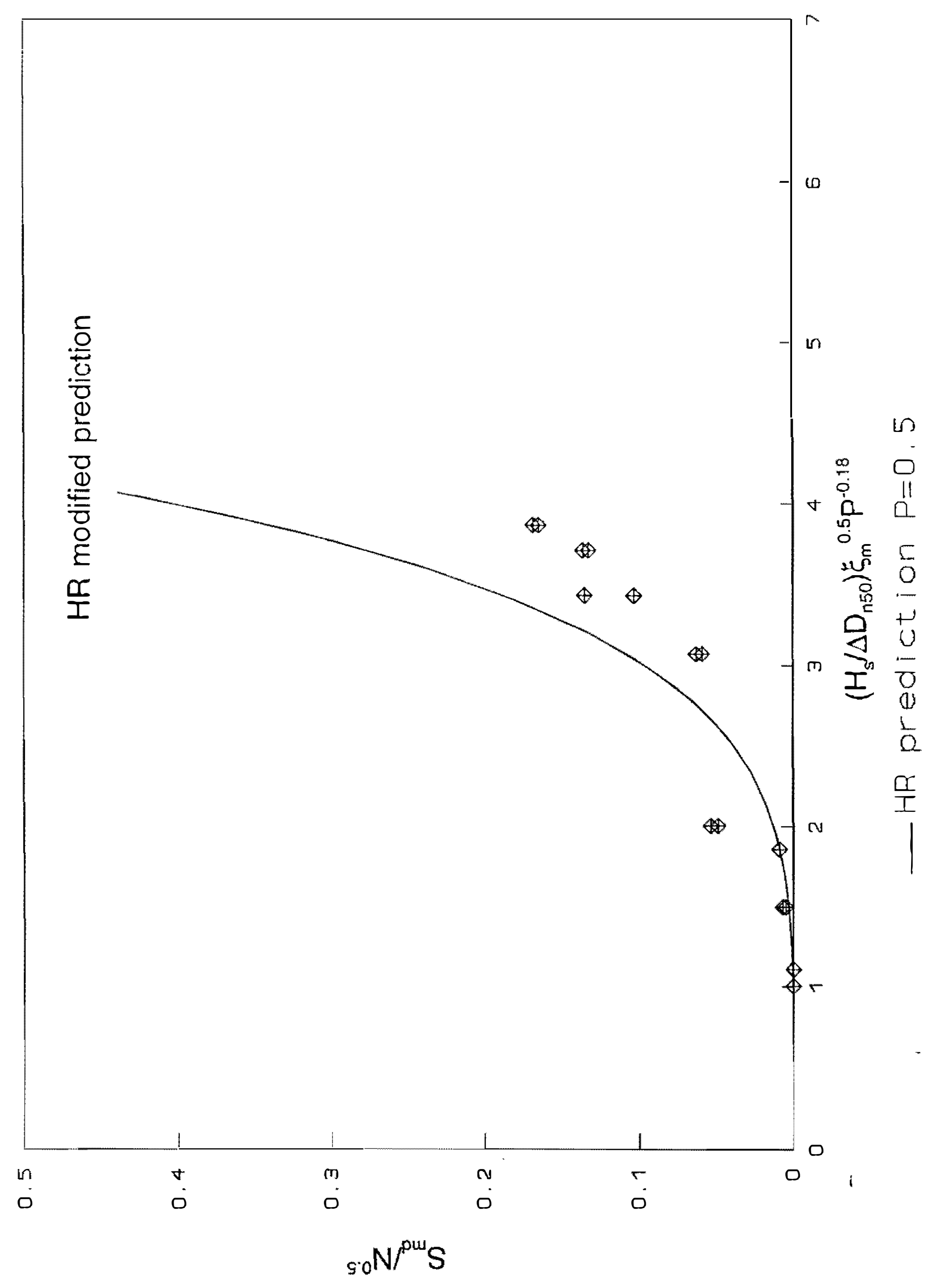
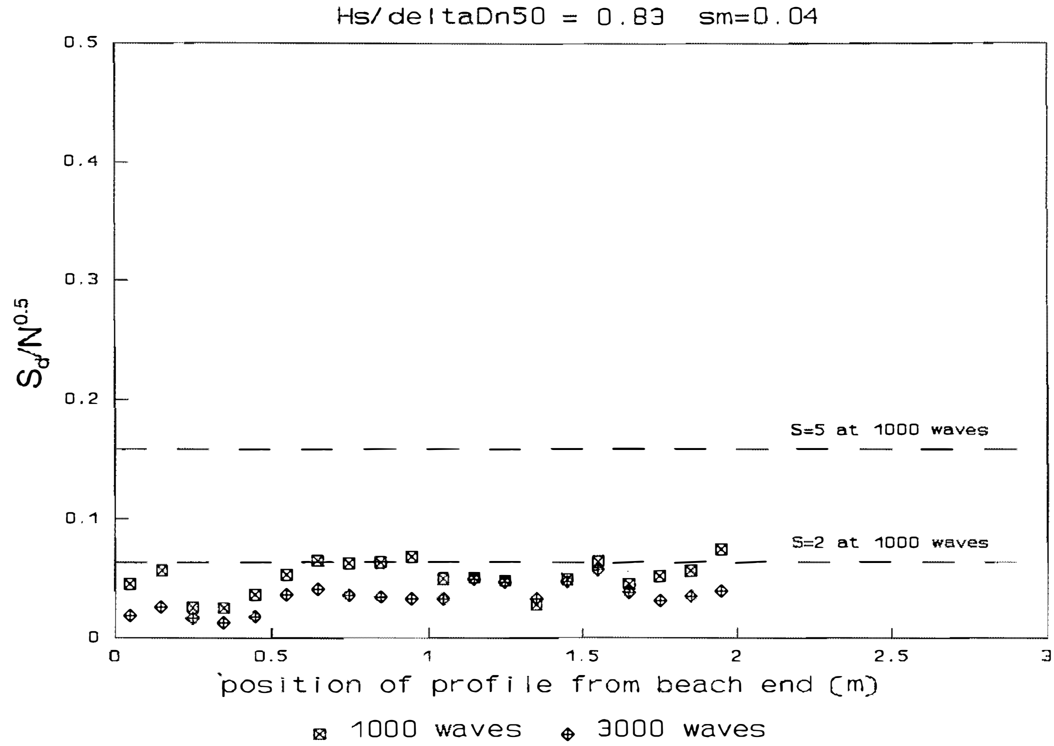


Figure 9 Summary of damage to Type 1 groyne



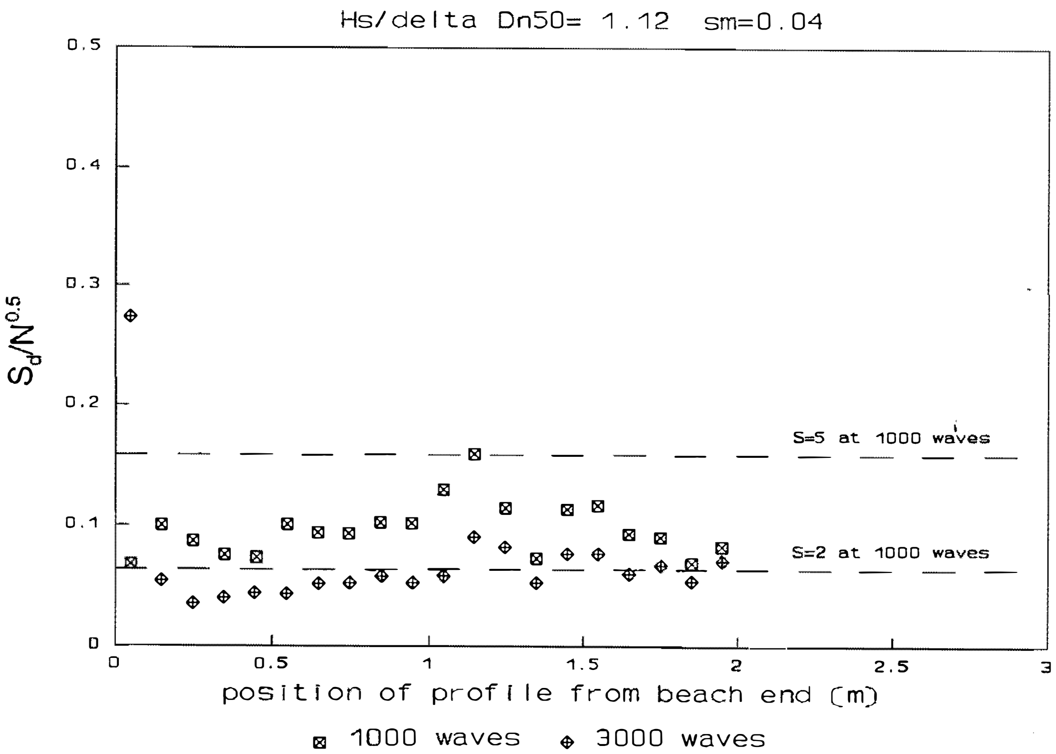


Figure 10 **Damage along Type 1 groyne, s_m=0.04, $H_s/\Delta D_{n50}$=0.83, and $H_s/\Delta D_{n50}$=1.12**

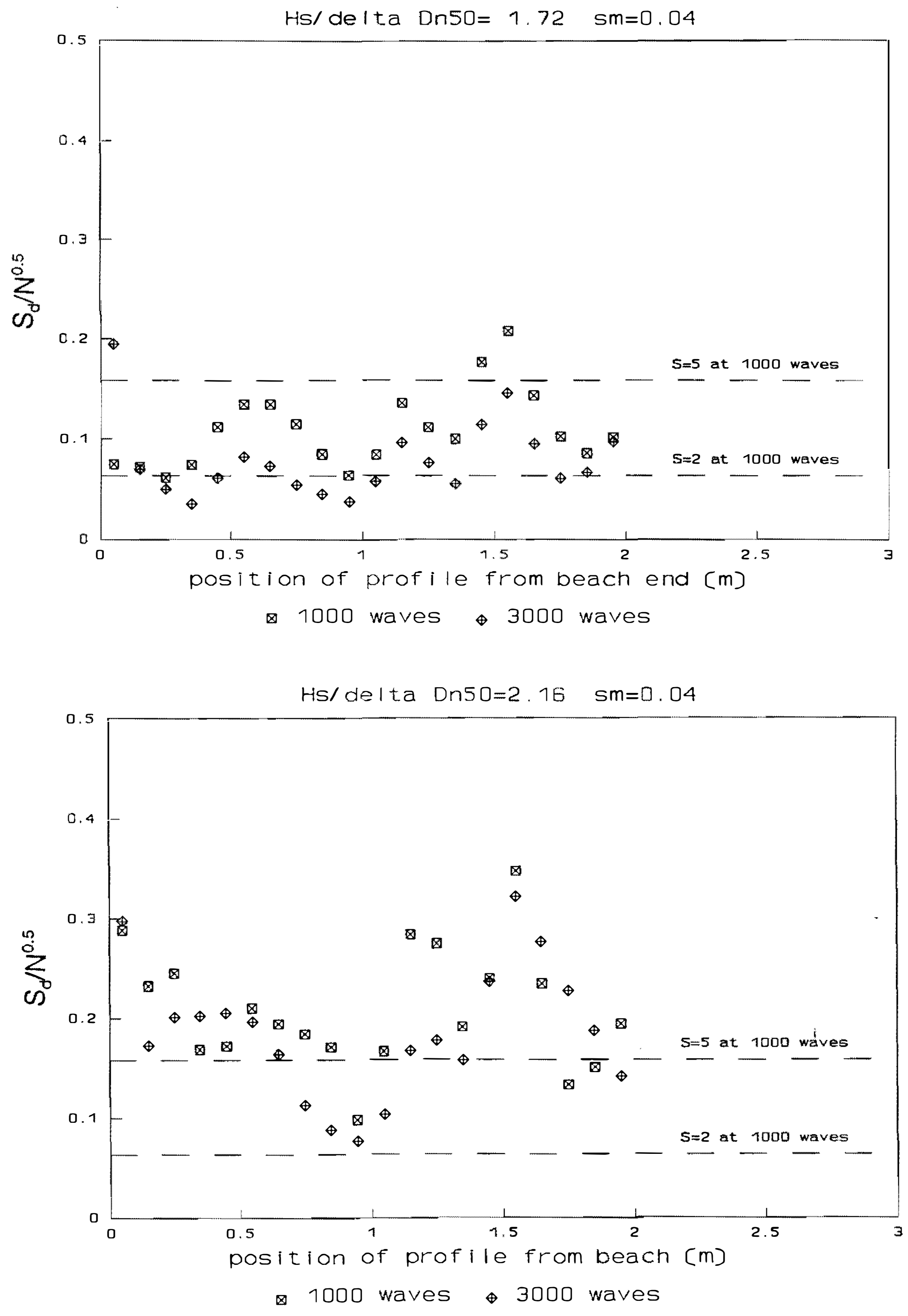


Figure 11 **Damage along Type 1 groyne, s_m=0.04, $H_s/\Delta D_{n50}$=1.72, and $H_s/\Delta D_{n50}$=2.16**

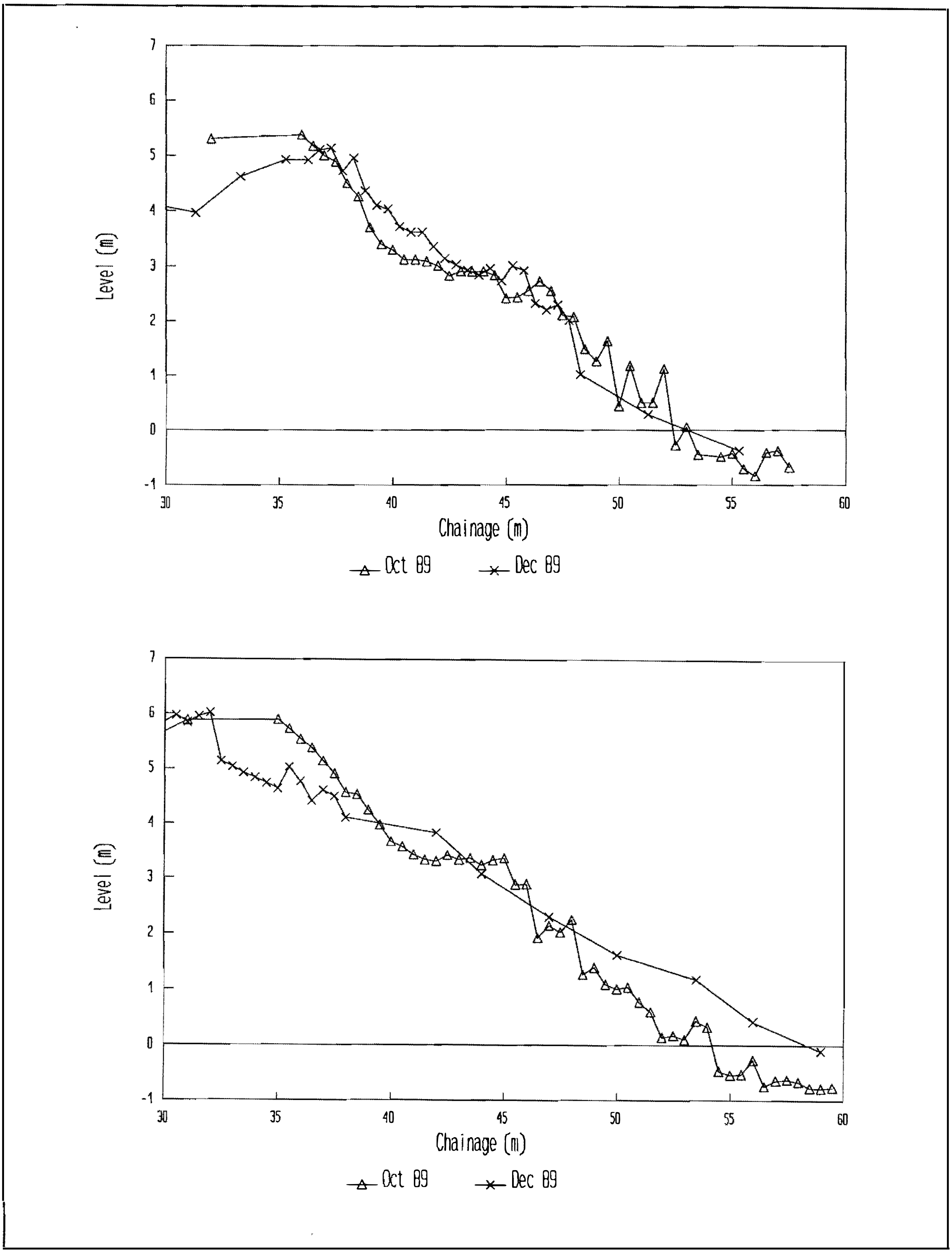


Figure 12 Damage to rock revetment at Hurst Spit, profiles 2 and 4