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STABILITY OF OVERTOPPED ROCK ARMOURED BREAKWATERS

by

D.N. FOSTER AND S.P. KHAN

REPORT NO. 161
February, 1984

WATER RESEARCH LABORATORY

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8. ABSTRACT

Design of breakwaters is largely based on guidelines developed from model tests and prototype experience of non-overtopped structures. It is generally recognised that an overtopped breakwater designed according to these procedures may be unsafe and that special attention needs to be paid to the crest and the leeward face. In practice this is achieved by rigorous model testing of each particular design.

The present study was aimed at identifying the primary variables influencing the stability of an overtopped breakwater and to take a first step towards providing more realistic design guidelines. These objectives were only partially achieved and the study has possibly raised more questions than it has answered. The investigation shows that the parameters influencing the stability of an overtopped structure are far more complex than those pertaining to a non-overtopped structure and that design guidelines based on the latter have little relevance and in many instances may lead to an unsafe design.

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PREFACE

The study reported herein was supported by a financial grant from AMSTAC (Australian Marine Science and Technology Advisory Council) under the general heading of "Towards design guidelines for overtopped breakwaters".

Much of the work was undertaken by Mr. Shahid Khan as partial requirement for the Master of Engineering Science Degree at The University of N.S.W. under the supervision of Professor D.N. Foster.

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CONTENTS

1. INTRODUCTION

The majority of breakwaters and revetments constructed in Australia are of the rubble mound type of construction, designed according to U.S. Army Corps of Engineers practice as set out in S.P.M. (1976). These guidelines are based on extensive model testing of breakwaters with zero or a moderate amount of overtopping. For such structures damage tends to be concentrated at the still water level on the seaward face.

There are obvious economic advantages if the crest of the breakwater (and consequently the volume of material) can be reduced by lowering the crest elevation. Indeed in areas subject to high storm tides (as applies over much of the coastline of Australia in the low latitudes) it is not practical to design for no overtopping under extreme conditions.

For an overtopped breakwater the zone of major damage under a particular set of design wave tide and surge conditions can shift from the seaward face to the crest and/or the leeward face requiring even larger armour than that required to resist the direct wave attack on the seaward face.

At the present time there are almost no guidelines for the design of an overtopped structure and extensive physical model testing is required as indicated in the list of references.

As the physical environment, types of units, and method of construction vary for each installation it is not possible to directly compare the results of these studies or to deduce any general design guidelines.

The objective of the present study is to examine some of the factors which influence the stability of a rock armoured breakwater subject to overtopping under wave action.

2. STABILITY OF A NON-OVERTOPPED BREAKWATER

Before proceeding to look at the stability of an overtopped structure the basis of design for a non-overtopped breakwater is briefly examined. For a more detailed discussion the reader is referred to Foster (1980).

The design of most rubble mound structures is based on the commonly referred to Hudson equation (Hudson, 1959) which was developed from the earlier work of Irrabaren (1938). The basis of this equation is as follows.

Removal of units from the face of the breakwater will occur when the wave forces exceed the restraining forces from submerged weight, friction and interlocking. Assuming that drag forces dominate over inertial forces or that the effects of fluid acceleration can be included in the drag coefficient the displacement force F_D may be written in the form:

$$F_D = K_1 \rho_f D^2 V|V| \tag{1}$$

where ρ_f = fluid density

V = a characteristic velocity in the vicinity of the rock which may be assumed to be proportional to the water particle velocities in the incident wave

D = characteristic rock dimension defined by $(M/\rho)^{1/3}$

M = mass of the unit

K_1 = coefficient to take account of the drag coefficient, shape, sheltering and the complex flow conditions.

For breaking waves in shallow water wave celerity and water particle velocities are related to the wave height H through the expression

$$V \propto \sqrt{gH} \tag{2}$$

Substituting in equation (1) yields

$$F_D = K_2 \rho_f gHD^2 = K_2 \rho_f gH \left(\frac{M}{\rho}\right)^{2/3} \tag{3}$$

where K_2 = coefficient incorporating the variables included in K_1 plus the relationship between velocity in the wave and local velocity in the vicinity of the armour.

Considering the restraining forces the submerged unit weight W_s is given by

$$W_s = (\rho - \rho_f)\, g \frac{M}{\rho} \tag{4}$$

where ρ = density of unit

If it is assumed that interlocking and friction are proportional to the submerged weight the restraining force

$$F_R = K_3 (\rho - \rho_f)\, g \frac{M}{\rho} \tag{5}$$

where K_3 = coefficient to allow for friction and interlocking.

Equating equations (3) and (5) and combining coefficients

$$\frac{\rho H^3}{\Delta^3 M} = N_s^3 \tag{6}$$

where $\Delta = \dfrac{\rho - \rho_f}{\rho_f}$ = submerged relative density

and N_s = coefficient to take account of all the unknowns.

The cube root of the L.H.S. equation (6) is known as Hudson's Stability Number.

This could have been derived from dimensional analysis. However, the equational approach gives some insight into the dynamics on which Hudson's equation is based and the many factors influencing the magnitude of N_s.

To proceed further the value of N_s must at present be determined empirically in the laboratory or the prototype. By using rock of uniform size and undertaking model tests in relatively deep water at the toe Hudson (1959) undertook studies to isolate the effect of wave period and breakwater slope. Surprisingly it was found that the coefficient was independent of wave period, a result which has since been questioned by other researchers (see Per Bruun 1977 and Price 1978) although there is ample evidence that at least for rock the effect is secondary. For rubble mound slopes varying between 1 in 1.25 and 1 in 5 Hudson found that N_s was closely proportional to $(\cot \alpha)^{1/3}$ giving rise to the final form of Hudson's equation

$$M = \frac{\rho H^3}{K_D \Delta^3 \cot \alpha} \tag{7}$$

Since the value of the coefficient K_D is very sensitive to the amount of damage that is allowed, it has been commonly referred to as the damage coefficient. As well as allowable damage it also includes all the unknown variables (except slope and period) which influence the stability of the breakwater. These include:

i. armour type and shape

ii. number of layers

iii. armour placement

iv. friction and interlocking

v. water depth

vi. breakwater geometry

vii. angle of attack

viii. size and porosity of underlayers

ix. offshore topography

x. wave spectra and wave grouping.

Experimental values of the damage coefficient for a non-overtopped or moderately overtopped rock armoured breakwater are shown in Figure 1. The wide scatter of results reflects variations in placement and the definition of percentage damage used by different investigators.

Despite its simplicity and shortcomings the Hudson equation has worked surprisingly well over the past 20 years since it was first introduced.

3. STABILITY OF AN OVERTOPPED BREAKWATER

The interaction of overtopping water with armour units on the back-slope and/or the crest is quite different from that for a non-overtopped structure on which the Irrabaren or Hudson design equations are based. Some possible failure modes have been described by Walker, Palmer and Dunham (1975). Several others have been defined in the course of the present study.

The complexity and variability of the physical factors influencing the stability of an overtopped breakwater are shown schematically in Figure 2 and are described briefly below.

When the water level in front of the structure is sufficiently high such that the wave run-up exceeds the elevation of the core, seepage will take place resulting in uplift forces over the crest and on the leeward face. Should these forces exceed the restraining forces provided by the submerged weight of the unit, friction and interlocking, then units will be displaced leading to failure (Figure 2a).

As the water level is further increased (or the crest elevation reduced) overtopping increases and weir flow (Plate 1) occurs down the back face resulting in inertial, drag and lift forces adding to the seepage forces.

Observations made during the present tests indicated that under weir flow velocities on the leeward face are complex. For most of the tests an hydraulic jump formed at the still water level which dissipated much of the energy and below the still water level forces on the back face and bed were greatly reduced (Figure 2b). However, for several of the tests no hydraulic jump resulted, the flow taking place as a submerged jet with little energy dissipation (Figure 2c). This jet resulted in displacement of units well below the water line and undoubtedly would have caused scour in a movable bed model (Figure 2c).

Weir flow did not always occur. Under certain combinations of wave characteristics and water depth the inertia of the overtopping jet is sufficient for the flow to separate from the crest resulting in a free jet impacting on the leeward face or directly on the water surface as illustrated in Figure 2d. If the water depth is small the jet impact may penetrate to the bed causing bed scour as indicated in Figure 2e.

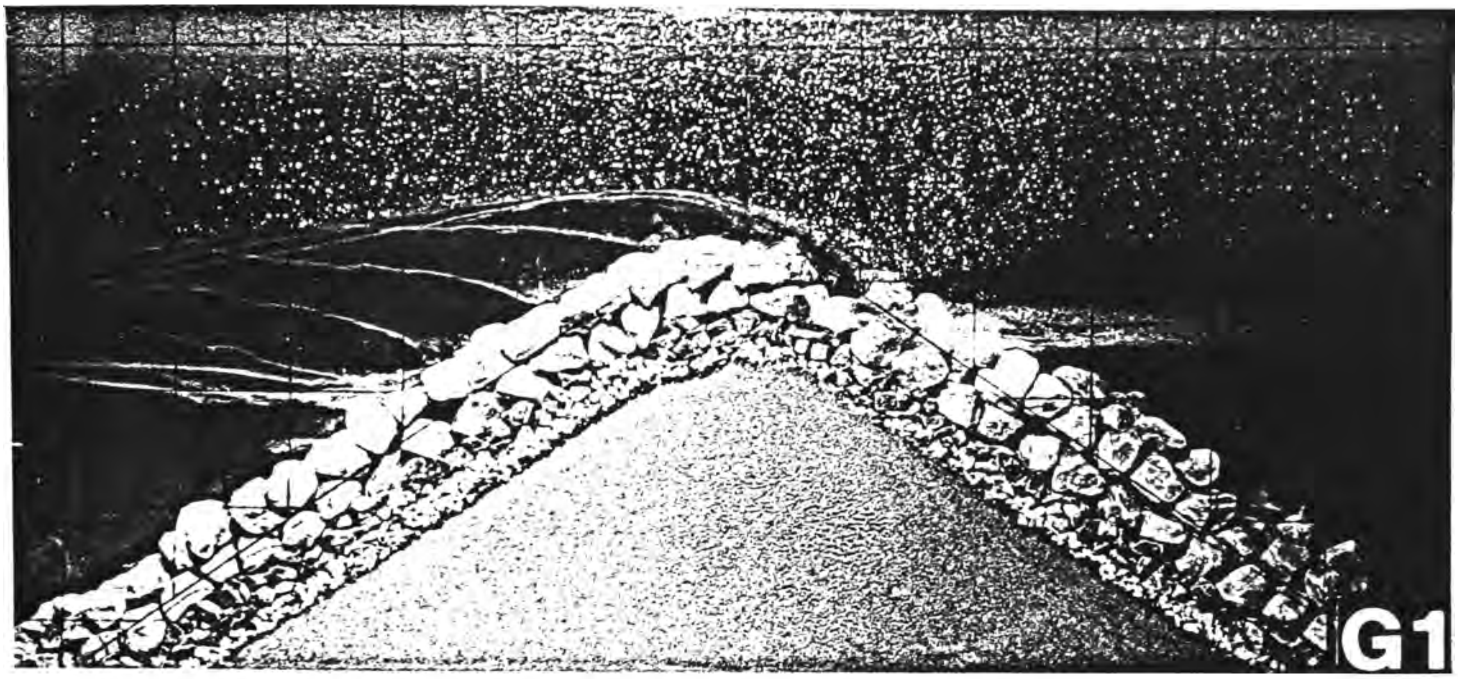
At still higher water levels the wave front over the crest steepens and may break as shown in Figure 2f. Backflow over the crest causes considerable interference with the incident wave and physical conditions are very complex and depend critically upon the phasing between the backflow and incident wave. Under these conditions steep pressure gradients can occur at the wave front resulting in local failure.

For a deeply immersed structure the wave form is modified over the crest, forces are predominantly inertial, lift and drag as a result of the wave induced velocities and accelerations in the vicinity of the crest (Figure 2g).

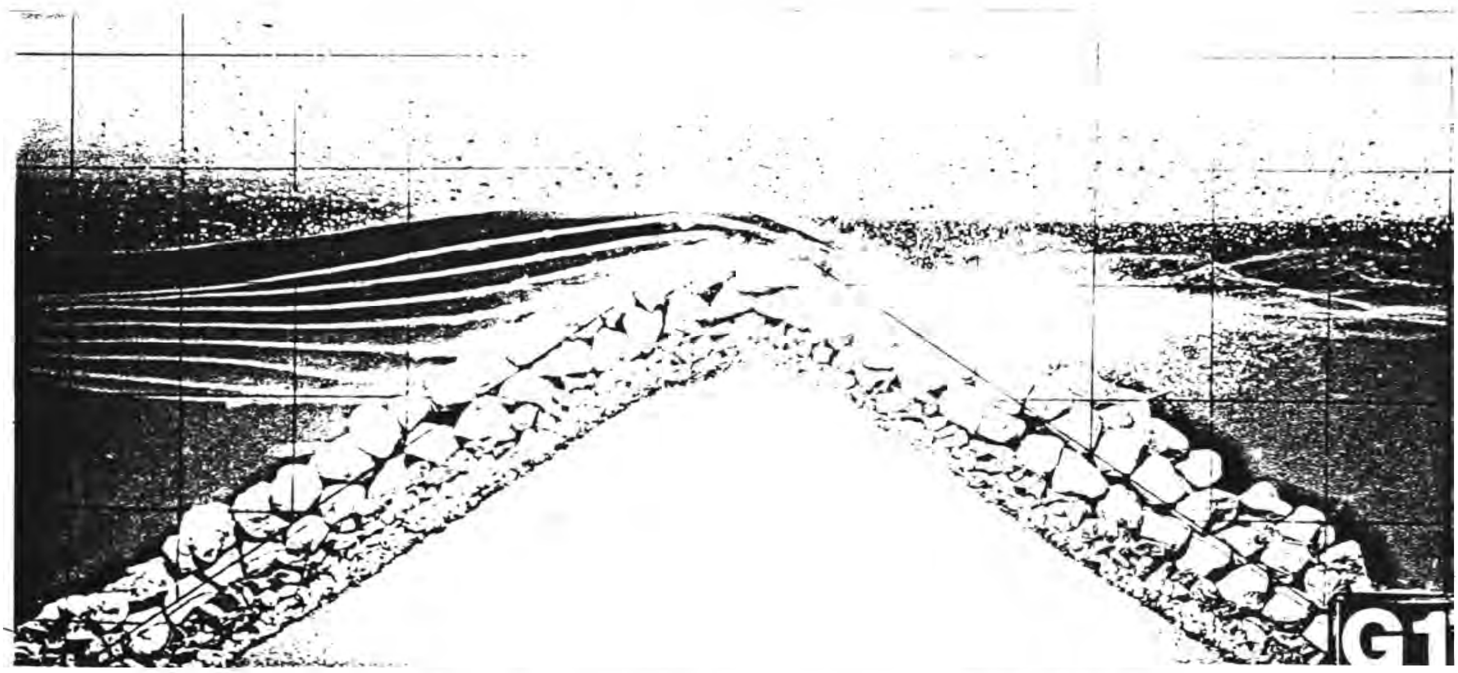
Because of the variation in water level, wave height and wave period, an overtopped breakwater may be subjected to several of these modes in any one storm.

4. DESIGN PRACTICE

The variables governing armour stability of the crest and back face of an overtopped breakwater are far more complex than the seaward face for which most design guidelines have been developed. At the present time there are no accepted design procedures and model testing of each particular site is necessary. Based on several model tests undertaken by the U.S. Corps of Engineers, Walker (1975) has suggested that as a general rule "the back-slope armour units should equal or exceed the size of the seaward-slope units when crest elevation is less than 0.7 times the design wave height but may be less when the crest elevation is greater than 0.7 times the wave height". However, the over-riding clause is added "In as much as the governing factors are not yet clearly defined, model studies to determine back-slope stability requirements are recommended whenever the crest height permits substantial overtopping".



(a) First half wave period - wave run-up



(b) Second half wave period - wave run-down

TYPICAL WAVE MOTION OVER BREAKWATER

STROBOSCOPIC PHOTOGRAPHS AT
1/16 sec. INTERVALS - WAVE PERIOD 2 secs.

PLATE 1.

No guidelines are given in the paper as to what is meant by design wave height and no account is taken of the variability of wave run-up and overtopping with type of unit and physical factors such as water depth, offshore slope, wave period and wave grouping, porosity of the armour layer etc.

As there is little published information on the stability of overtopped breakwaters it is worthwhile to repeat the conclusions and some possible corrective actions given in the paper by Walker et al (1975).

Corrective Action

The problems attendant upon back-slope stability arise from several sources. They include failure due to internal pressures, impact of the overtopping jet, entrapment of pressures due to concrete caps, and scour of the heel by the overtopping waves. Various measures that have been devised to avoid these problems are discussed below:

Alternative 1

The most common practice, which needs no illustration, is simply to strengthen the back-slope. This may be accomplished by better placement and careful seating of stones, by use of larger stones, or by providing heel protection. The latter is essential for low-height breakwaters constructed in shallow water.

Alternative 2

A less common measure is to steepen the back-slope to allow the jet of water to pass over the back-slope, as illustrated in Figure 3. This has not always worked, as in the case of the Hilo Harbour breakwater.

Pressure transmitted through voids in the structure may cause more damage with the steeper slope. At the other extreme, in cases where the back-slope is flattened, more area is subjected to the impact of the overtopping jet and the damage may actually be increased. A 1:1.5 back-slope appears to be a reasonable compromise for design purposes.

Alternative 3

Increasing the crown width, Figure 4, reduces the amount of wave dissipation on the back-slope. This may be an important aspect of design of low breakwater. Wider crests also reduce transmitted wave heights. The widening of the crest of a structure allows some of the energy to be absorbed and to percolate into the structure, thereby reducing the jet impact.

Alternative 4

A massive concrete cap has been provided in some instances. This is supposed to enhance overall stability, but cases reported by Magoon (1975) indicate that the underlying stone may be washed out from under the cap until it finally collapses under its own weight. Construction of the cap often produces the unwanted effect of transmitting pressures through the breakwater, as shown schematically in Figure 5. Although a cap may enhance stability of the crest, it may decrease stability on the back-slope. Vents are generally placed in the cap to release the pressure build-up; however, the effectiveness of the cap venting system has not yet been demonstrated.

Conclusions

1. The many variables in the breakwater design render the study of breakwater back-slope stability difficult.

2. Based on model experiments and prototype experience, the back-slope may be subjected to more damage than the front slope.

3. Variations in structure slopes, crest widths, armour unit size and seating may enhance back-slope stability. A more economical design may evolve through consideration of back-slope stability as a specific objective of future hydraulic-model studies of overtopped breakwaters.

4. Waves that have the highest rate of overtopping and break just at the structure appear to damage the back-slope more severely.

5. The design wave for the back-slope is not necessarily that used for design of the seaward slope.

6. These conclusions are general and at best precautionary. More study of the subject is needed, and more prototype as well as model study data should be acquired to develop reliable criteria for back-slope design.

One of the corrective measures suggested by Walker (1975) is to steepen the back face such that the overtopping jet clears the back face and impinges upon a water cushion. To achieve a similar objective the Department of Harbours and Marine, Queensland, have suggested the use of an apron on the leeward crest to deflect the overtopping jet beyond the back-face (DHM, 1981).

It is clear that this particular corrective method is limited to the type of flow conditions illustrated in Figure 2(d) and as pointed out by Walker (1975) has not always worked. For seepage and weir flow (Figures 2a and 2b) the increased slope will increase downslope gravity forces and reduce the stability of the breakwater.

5. DESIGN PARAMETERS

The basic parameters influencing the stability of an overtopped breakwater are:

Physical

i. wave height

ii. wave period

iii. wave grouping

iv. storm duration

v. storm tide level

vi. water depth

vii. angle of wave attack

viii. porosity

Geometric

i. type of armour unit

ii. breakwater dimensions

iii. crest width

iv. back-slope

v. offshore topography.

Construction Methods

Construction methods have a very significant effect on the stability of any breakwater whether it be overtopped or non-overtopped. For example the stability of a rock armoured breakwater can be increased some ten fold by placement of the rock with its long axis perpendicular to the slope (Figure 1) thereby insuring careful interlocking and friction between units [Lording and Scott (1971), Brown (1978)]. Such practice is normally possible only above still water level which is the area of maximum wave forces for an overtopped structure. Consequently construction methods and supervision can play an important role in increasing the stability of an overtopped structure. However, a word of warning needs to be added. This increased stability is achieved by careful placement and keying of units in such a manner that drag and lift forces are reduced and friction and interlocking between units is increased. The failure of a single unit may lead to a break of this bond resulting in catastrophic failure and consequently a higher factor of safety is normally required.

6. ROCK STABILITY UNDER STEADY FLOW

Before proceeding to investigate the stability of an overtopped breakwater subject to wave attack it is worthwhile to examine the factors influencing the stability of a similar structure under steady flow.

The design of a non-scouring channel is normally based on the critical tractive stress at the bed as given by the relationship:

$$\tau_c = \rho_f \, g \, y \, S_c \tag{8}$$

where τ_c = bed shear at initiation of movement

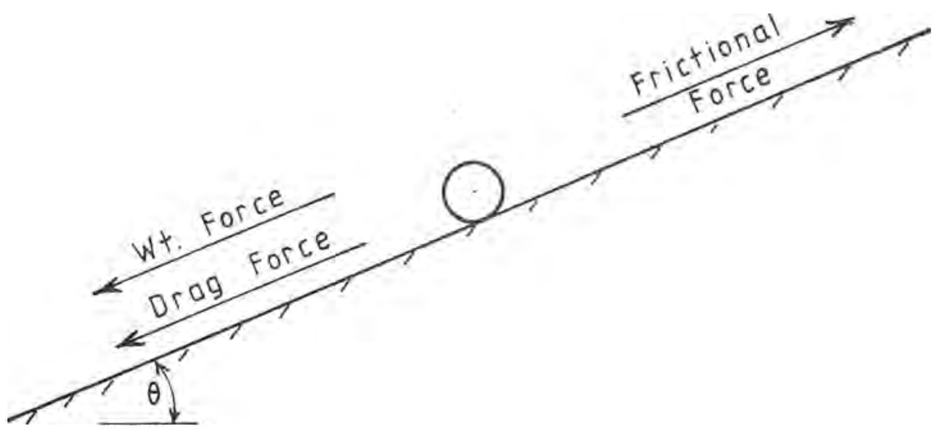
ρ_f = fluid density

g = gravitational acceleration

y = flow depth

S_c = slope of the energy gradient at the initiation of movement

If the bed shear at which the particles move on a horizontal bed is τ_{cH} then for a sloping bed the shear stress τ_c' at which movement commences will be reduced below τ_{cH} because of the component of the particle weight down the slope. Consider the channel shown in the diagram below.



White (1940) has suggested that drag forces can be approximated by the expression

$$F_D = \frac{\tau_o' D^2}{r}$$

where τ_o' = bed shear
D = particle diameter
r = packing factor

If it is assumed that at critical conditions the sum of the downslope weight and drag forces are just balanced by the maximum frictional force then at the commencement of movement

$$\frac{\tau_c' D^2}{r} + W \sin \theta = W \cos \theta \tan \phi$$

where W = submerged weight
ϕ = angle of repose of the bed material
θ = bed slope

For a horizontal bed $\tau_c' = \tau_{cH}$ and the expression reduces to

$$\frac{\tau_{cH} D^2}{r} = W \tan \phi$$

Rearranging terms and dividing we have

$$\frac{\tau_c'}{\tau_{cH}} = \cos \theta - \frac{\sin \theta}{\tan \phi}$$

$$\text{or } \tau_c' = \tau_{cH} (\cos \theta - \frac{\sin \theta}{\tan \phi}) \qquad (9)$$

which gives an expression for the shear at which bed movement will commence in a sloping wide channel in terms of the critical shear stress on a horizontal bed.

In order to apply Equation 9, information is required on angle of repose ϕ. Extensive tests by the U.S.B.R. have shown that ϕ depends on the size of stone and whether they are angular or rounded. The results of these tests are plotted in Figure 6.

A discharge formula commonly used for open channels is the Manning equation. From this equation the critical energy slope is given by:

$$S_c = \frac{V_c^2 n^2}{y^{4/3}} \qquad (10)$$

where V_c = critical velocity - ms^{-1}

y = flow depth - m

n = Manning roughness parameter.

Substituting for S_c in Equation (8) and assuming ρg = 10,000 (salt water), the relationship between critical velocity and critical bed shear (SI units) is given by the expression:

$$\tau_c' = \frac{V_c n^2}{y^{1/3}} \times 10^4 \qquad (11)$$

or

$$V_c = \frac{(\tau_c')^{1/2} y^{1/6}}{100 n}$$

To apply this equation, information is needed on the magnitude of the critical tractive stress τ_c' or τ_{cH} and the roughness parameter n.

For fully developed rough turbulent flow in a wide channel it can be shown (Keulegan, 1938, Chow, 1959) that the Manning roughness parameter is related to the flow depth y and the rock size D through the expression -

$$n = \frac{(y/D)^{1/6}}{21.9 \log (12.2 \ y/D)} \qquad (12)$$

Equation (12) can be expressed in the simpler dimensional form

$$n = \frac{D^{1/6}}{K} \qquad (13)$$

Where D is particle size in metres and K is a coefficient which is a function of y/D. Values of the coefficient K for a range of values of y/D are shown in Table 1.

TABLE 1. Variation of Coefficient K in the Equation $n = D^{1/6}/K$ with y/D ratios where D is in metres

y/D	K	y/D	K
1	19.5	5	24.4
2	22.2	6	24.9
3	23.3	8	25.2
4	24.1	10	25.5

For steep slopes Iwagaki (1954) suggests that K and n may also be a function of the Froude number. However, the data is too sparse and variable to draw any definite conclusions. Foster (1969) tested the stability of rock on a slope of 1 to 13 at Froude numbers varying between 1.0 and 2.0. Using the same form of equation as proposed by Lane and Carlson (1953) the results indicated values of K between 18 and 20 which is in close agreement with equation 17 below.

For most practical purposes K can be assumed to be effectively constant. For graded rock several empirical equations of this form have been suggested by various researchers.

Strickler (1923) $$n = \frac{D_{50}^{1/6}}{24.1} \qquad (14)$$

Keulegan (1938) $$n = \frac{D_{50}^{1/6}}{25.4} \qquad (15)$$

Irmay (1949) $$n = \frac{D_{max}}{26.6} \qquad (16)$$

Lane and Carlson (1953) $$n = \frac{D_{25}^{1/6}}{21.1} \qquad (17)$$

Where D_x refers to mesh size in metres of the sieve on which x per cent of the material is retained.

For practical design it may therefore be assumed that the Manning roughness is a function of the rock size in accordance to the following relationship:

$$n = D_{50}^{1/6}/K \qquad (18)$$

Where K has a value of approximately 25 with D in metres.

For a rock armoured channel and fully developed rough turbulent flow the critical tractive stress (τ_{cH}) on a horizontal bed is closely approximated by the well known Shields relationship:

$$\frac{\tau_{cH}}{\rho g \Delta D} = 0.056 \tag{19}$$

Where Δ is the submerged relative density and the other terms are as previously defined.

Assuming a value for salt water of ρg = 10,000 and Δ = 1.65 this collapses to the very simple expression:

$$\tau_{cH} = 924 \text{ D where D is in metres} \tag{20}$$

Lane and Carlson (1953) undertook tests in fresh water using graded rock with a D_{50} varying between 0.02 and 0.08 m and channel slopes of 1:1,000 and 1:100 and arrived at the following relationship:

$$\tau_{cH} = 943 \ D_{25} \tag{21}$$

Foster (1969) undertook similar tests on a slope of 1 to 13 which after adjusting for slope resulted in the following relationship:

$$\tau_{cH} = 1{,}000 \ D_{25} \text{ to } 1{,}500 \ D_{25} \tag{22}$$

On the basis of these test results, a reasonable estimate of the critical shear stress on a horizontal bed of graded rock can be taken as:

$$\tau_{cH} \sim 1{,}000 \ D_{50} \tag{23}$$

Where τ_{cH} is in Newton´s per square metre and D_{50} is in metres.

The approximate design equations for stability of rock of a relative density of 2.65 submerged in seawater are therefore:

$$n = \frac{D_{50}^{1/6}}{25} \tag{24}$$

$$\tau_{cH} = 1,000\ D_{50} \tag{25}$$

$$S = \frac{V^2 n^2}{y^{4/3}} \tag{26}$$

$$\tau_c' = \tau_{cH}\ (\cos\theta - \frac{\sin\theta}{\tan\phi}) = 10,000\ y\ S \tag{27}$$

These equations are dimensional using the following units:

Length - Metres
Force - Newtons
Time - Seconds

The above equation can be combined to determine the velocity V_m at the initiation of motion

$$V_m = 7.90\ D_{50}^{1/3}\ y^{1/6}\ (\cos\ \theta - \frac{\sin\ \theta}{\tan\ \phi})^{1/2} \tag{28a}$$

or for a typical value of ϕ equal to 40 degrees (Figure 6).

$$V_m = 7.90\ D_{50}^{1/3}\ y^{1/6}\ (\cos\ \theta - 1.2\ \sin\ \theta)^{1/2} \tag{28b}$$

For <u>uniform</u> flow the stable slope is related to the discharge and the stone size through the expression:

$$\frac{\cos\ \theta}{(\sin\ \theta)^{0.7}} - \frac{(\sin\ \theta)^{0.3}}{\tan\ \phi} = \frac{1.45\ q^{0.6}}{(D_{50})^{0.9}} \tag{29a}$$

or for ϕ equal to 40 degrees:

$$\frac{\cos\ \theta}{(\sin\ \theta)^{0.7}} - 1.2\ (\sin\ \theta)^{0.3} = \frac{1.45\ q^{0.6}}{(D_{50})^{0.9}} \tag{29b}$$

This equation can be combined with the standard head discharge relationship for flow over a weir:

$$q = C_D\ H^{3/2} \tag{30}$$

to estimate the available heads where C_D is the coefficient of discharge.

6.1 Previous Studies

There is very little data to test the validity of equations (28) and (29) for steep slopes. Foster (1969) undertook laboratory tests on a 1:13 slope for rock with D_{25} sizes varying between 65 and 110 mm.

Posey (1957) undertook both laboratory and field tests of overtopped road embankments for rock sizes of 40 to 280 mm and slopes of 1 on 2, 1 on 4 and 1 on 6. The results were presented in terms of maximum head over the road against minimum rock size required on the downstream embankment and consequently can not be compared directly with equation 29. If it is assumed that the coefficient of discharge is 1.6 (S.I. units), the angle of friction is 40^o and the relative density of the rock is 2.65 then an approximate comparison can be made.

Figure 7 shows a comparison of the results of both Foster (1969) and Posey (1957) with that predicted theoretically by equation 29. Despite some experimental scatter (particularly in the results of Posey, part of which may be due to the assumptions made above) the agreement is reasonable.

A further point of interest in the paper of Posey (1957) is that he noted that damage increased with decreasing tailwater depth and that whilst it may have been expected that the hydraulic jump at the junction would increase the tendency to scour this was found not to be true. Carl E Kindsvater in discussion of Posey's paper also noted that resistance to damage can be increased by raising the tailwater level.

6.2 Application to Typical Breakwater Sections

Armour mass and slopes commonly used in rock armoured breakwaters vary between 1 and 10 tonnes and 1 on 1.5 to 1 on 5 respectively. Critical velocities, flow rates and heads (assuming a discharge coefficient of 1.6) can be calculated from equations 28 and 29. Results are plotted in Figures 8 and 9.

It is clear from these figures that for low tailwater and uniform flow on the downstream face very little overtopping can be allowed if damage is to be prevented.

7. BREAKWATER TESTS

7.1 Testing Facilities

All tests were undertaken in the Water Research Laboratory's 0.9 m wide by 1.5 m deep by 35 m long wave flume. The basic model arrangement is shown in Figure 10. Waves were generated by a standard oscillating paddle. A recirculating pumping arrangement was used to equalise water levels on each side of the breakwater and to measure the flow past the breakwater as a result of overtopping. Incident, reflected and transmitted wave heights were measured using a capacitance wave gauges and discharge were measured using a standard orifice metre.

All tests were undertaken on the basic two layer breakwater section shown in Figure 11. The characteristics of the materials used were:

Primary Armour

Mass 100 $\pm$ 10 grams
Ratio of maximum to minimum dimensions < 2:1
Relative density 2.65
Natural angle of repose $40^{\circ} \pm 5^{\circ}$
Equivalent spherical diameter 0.0416 m.

Secondary Armour

Mass 10 to 15 grams
Relative density 2.65

Core

Sand with a median grain size of 0.05 mm.

All tests were undertaken for seaward and leeward slopes of 1 on 1.5 which corresponds to that commonly used for breakwater construction in Australia.

7.2 Test Results Steady Flow

As discussed in Section 6.2 theory and measurements indicate that for steady uniform flow a breakwater would be unstable for all but very minor overtopping. For the rock used in the model the limit conditions can be calculated from equation 29 (assuming a coefficient of discharge of 1.6) as:

$$q_{max} = 0.00045\,\mathrm{m^2s^{-1}}$$

$$H_{max} = 0.0043\,\mathrm{m}.$$

However, as suggested by Kindsvater in the discussion of the paper by Posey (1957) these limits will possibly be larger for the high tailwater levels associated with breakwaters. The present tests were undertaken to investigate this aspect.

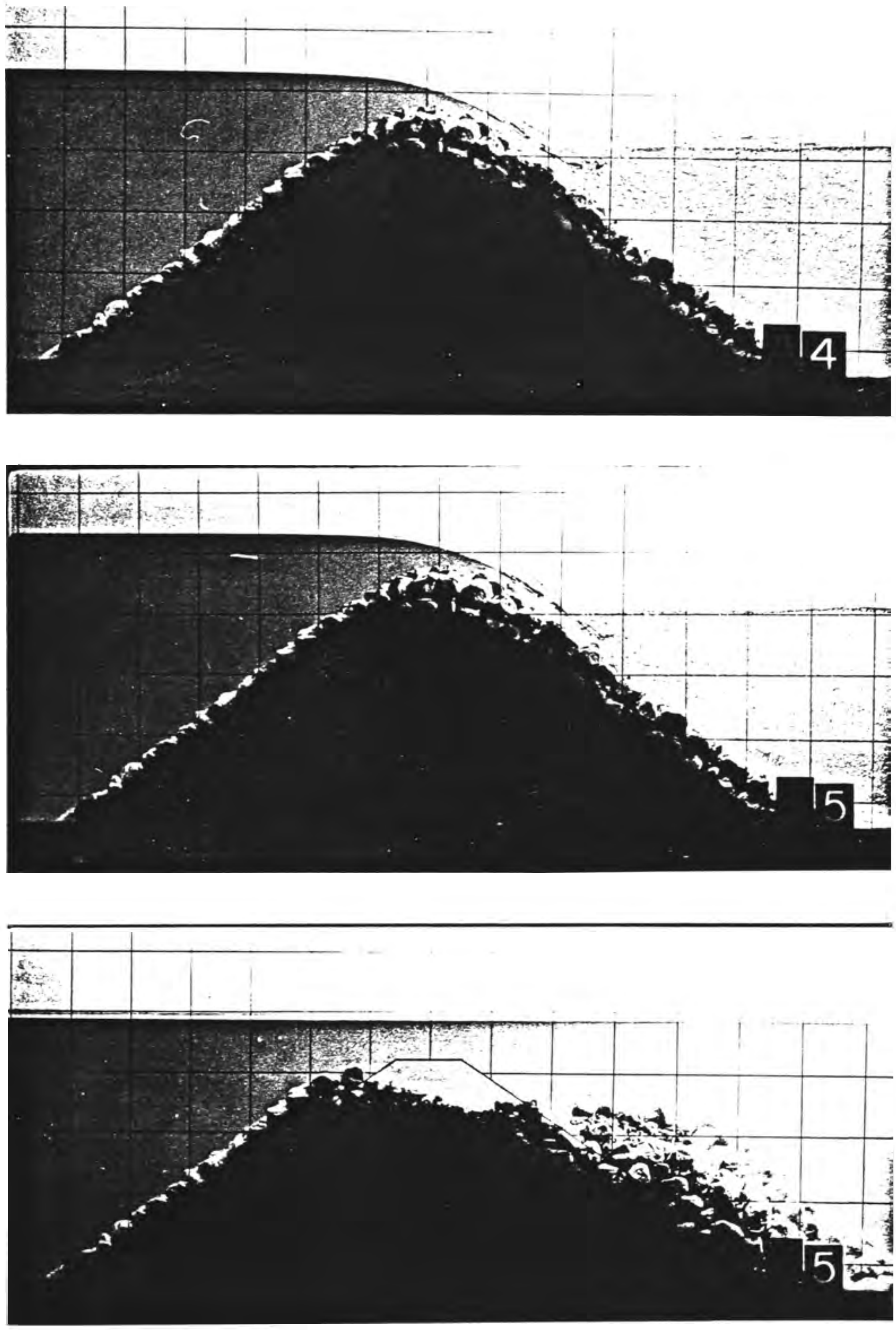
Starting from equal upstream and downstream heads the control valve was set and the pump turned on. The tailwater level was then gradually reduced until failure occurred. The test was repeated for different flow rates and settings of the control valve. In most of the tests failure was sudden and catastrophic. (See Plate 2). Test results are given in Table 1 and are plotted in Figures 12 and 13. Some of the scatter in these curves result from the difficulty of accurately defining the crest level which was taken as the average level of the upper rock surface.

The results indicate that the overtopping level and the flow rate resulting in instability are dependent upon the tailwater level. For example at a tailwater level of zero relative to the crest, failure occurred at an upstream head of approximately 88 mm and a discharge of $0.042\,\mathrm{m^2s^{-1}}$. For a tailwater level 50 mm below the crest the corresponding values are 40 mm (total head difference 90 mm) and $0.013\,\mathrm{m^2s^{-1}}$. Over the range of the tests the difference between upstream and downstream water levels was approximately constant at 90 mm for critical conditions.

For the range of tailwater levels tested, critical heads at failure were higher than that predicted from Equation 29 for uniform flow conditions but would approach each other at a tailwater level of approximately 90 mm below the crest.

The implication of these results in relation to a breakwater is that the stability can be expected to be a function of factors which influence the level of overtopping (e.g. slope, wave height, wave period, water depth, offshore slope, etc.) and the relative level of the overtopping to the still water level as determined by tide and storm surge.

During the tests it was observed that the hydraulic conditions at the junction of the overflow and the still water level are complex. At times an hydraulic jump resulted with a marked reduction in velocities below water. However, on other occasions the flow occurred as a submerged jet with little reduction in flow velocities over the entire back face which undoubtedly would have scoured a movable bed. Some doubts must therefore be raised as to the safety of reducing armour mass below water as is commonly adopted in breakwater practice.



FAILURE OF A RUBBLE MOUND SLOPE
UNDER STEADY FLOW

PLATE 2.

TABLE 2. Steady State Stability Tests

Test No.	Upstream Head m × 10^2	Downstream Head m × 10^2	Discharge $m^2 s^{-1}$	Damage
1	9.5	1.7	0.0447	Nil
2	9.6	1.0	0.0460	Failure
3	10.0	5.0	0.0471	Nil
4	10.2	3.8	0.0523	Nil
5	10.5	2.1	0.0567	Nil
6	10.7	-0.1	0.0574	Failure
7	11.5	5.9	0.0604	Nil
8	8.0	3.5	0.0319	Nil
9	8.2	2.3	0.0350	Minor
10	7.5	-1.0	0.0280	Minor
11	7.7	-2.5	0.0317	Failure
12	10.5	5.5	0.0578	Nil
13	10.6	4.9	0.0574	Nil
14	10.7	4.5	0.0583	Nil
15	6.9	1.2	0.0257	Nil
16	7.3	-1.3	0.0293	Nil
17	7.5	-1.5	0.0308	Extensive
18	7.6	2.0	0.0389	Nil
19	7.8	1.4	0.0404	Nil
20	8.0	2.5	0.0422	Nil
21	8.3	1.0	0.0449	Nil
22	8.6	0.5	0.0480	Nil
23	5.9	0.7	0.0278	Nil
24	6.5	-1.6	0.0314	Nil
25	6.6	-2.4	0.0389	Failure
26	5.2	-1.2	0.0253	Nil
27	5.8	-4.5	0.0322	Extensive

7.3 Base Test

Before proceeding to studies of an overtopped structure base tests were undertaken for a non-overtopped structure using the same model materials to be used in the overtopping studies. All tests were undertaken for non-breaking offshore wave conditions, a breakwater slope of 1 to 1.5 and a constant wave period of 1.70 seconds. Parameters measured were the equivalent deep water wave height, percentage damage, wave run-up, wave run-down and depth at the toe of the structure.

Test results are summarised in Table 3. Comparison of the results with other researchers are shown on Figures 1, 14 and 15.

It is difficult to directly compare results between different investigations because of the subjective methods used in the measurement of percentage damage, wave run-up, wave run-down and interlocking between units. In the present tests percentage damage is defined as the number of units dislodged relative to the total number of units between the limits of wave run-up and wave run-down; wave run-up and wave run-down was based on subjective judgement; and the placement of units can be described as random but careful to achieve a near uniform two layer cover on a slope of 1 to 1.5.

TABLE 3. Base Tests for Two Layer Non-Overtopped Breakwater - Non Breaking Wave

Mass of Primary Armour	=	100 ± 10 gms
Mass of Secondary Armour	=	10 to 15 gms
Breakwater slope	=	1 on 1.5
Relative Density of Armour	=	2.65
Wave Period	=	1.70 sec

Test No.	Water Depth (m)	Deep Water Wave Height (m)	Run-Up (m)	Run-Down (m)	Percentage Damage	Hudson Damage Coefficient (K_D)
1B	0.233	0.104	0.125	0.065	0	4.42
2B	0.233	0.116	0.147	0.071	0	6.14
3B	0.230	0.110	0.155	0.060	0	5.23
4B	0.219	0.120	0.169	0.057	13	6.80
5B	0.243	0.118	0.167	0.073	8	6.46
6B	0.240	0.118	0.168	0.074	4	6.46
7B	0.240	0.123	0.174	0.076	4	7.32
8B	0.240	0.125	0.182	0.080	5	7.68
9B	0.240	0.131	0.206	0.076	21	8.84
10B	0.243	0.135	0.195	0.081	7	9.68
11B	0.243	0.135	0.185	0.075	4	9.68
12B	0.258	0.120	0.162	0.078	8	6.80
13B	0.258	0.123	0.185	0.075	11	7.32

Figure 1 indicates that the stability of the model armour used in the present tests is generally in accord with that found by other researchers. Wave run-up and wave run-down show (Figures 14 and 15) some variation with the values given in S.P.M. (1976) for rubble mounds and graded rip-rap slopes. However, this difference is partly explained by the lower depth to wave height ratios used in the present tests (2.0 to 2.2) and the difficulty of defining wave run-up and wave run-down on a rubble mound slope.

Within the range of experimental accuracy it can be concluded that results of the base tests were generally in accord with the findings of others.

7.4 Overtopping Tests - Test Series I

In these tests the wave period and height of the breakwater wave kept constant at 1.70 seconds and 0.335 m respectively. For a fixed water depth relative to the crest the wave height was increased in increments until failure resulted. A range of water depths were tested from a near non-overtopped structure to a deeply submerged structure. Parameters measured were still water level relative to the crest, equivalent deep water wave height of the incident wave, level of overtopping to the crest, wave run-down relative to still water, transmitted wave height, average flow rate over the structure, the degree and location of damage. Test results are summarised in Table 4.

Damage as a function of wave height, water level and level of overtopping is plotted in Figures 16 and 17. As is usual in breakwater tests considerable variability in damage occurred from one test to another as a result of the difficulty of achieving similar interlocking between units for each individual test. Despite this variability a distinct pattern is noticed.

TABLE 4. Test Results for Overtopped Breakwater - Test Series I

Test No.	SWL	H_o'	t	OT	RD	HT	q	Damage
1.01	-92	94	0.5	25	63	5	2.1	Negligible
1.02	-65	83	0.5	30	80	15	4.2	Negligible
1.03	-33	87	0.5	40	82	40	6.6	Negligible
1.04	+03	91	0.5	70	88	60	13.7	Substantial
1.05	+25	101	0.5	85	110	60	19.2	Minor
1.06	-65	107	0.5	45	80	40	5.1	Negligible
1.07	-34	94	0.5	57	77	40	8.8	Negligible
1.08	+06	111	0.5	83	91	60	17.2	Minor
1.09	-91	104	0.5	33	64	10	3.3	Negligible
1.10	-45	103	0.5	52	78	60	7.9	Minor
1.11	-03	108	0.5	90	86	90	21.0	Substantial
1.12	+01	115	0.5	125	108	100	32.9	Major
1.13	-61	109	0.5	50	74	45	6.6	Minor
1.14	-31	120	0.5	67	78	80	12.0	Minor
1.15	-03	114	0.5	95	102	105	23.5	Substantial
1.16	+23	122	0.5	120	98	110	32.4	Substantial
1.17	+50	123	0.5	160	105	110	38.1	Substantial
1.18	-59	120	0.5	50	76	40	8.1	Substantial
1.19	-18	118	0.5	85	117	110	18.5	Substantial
1.20	+06	129	0.5	100	121	120	30.4	Major
1.21	-123	101	0.25	15	52	25	3.0	Substantial
1.22	-45	98	1.0	50	90	60	12.0	Substantial
1.23	+03	132	1.58	100	108	120	29.7	Major
1.24	+35	126	0.75	135	100	120	36.0	Substantial
1.25	-55	106	1.0	55	100	80	13.7	Major
1.26	-25	138	0.87	105	98	110	22.8	Major
1.27	+08	141	0.5	135	107	110	31.5	Major
1.28	+52	149	0.5	210	127	120	41.5	Minor
1.29	+52	152	0.25	240	107	120	39.6	Minor
1.30	+32	153	0.58	240	97	130	36.4	Substantial
1.31	+14	150	1.17	185	95	115	30.6	Major
1.32	+33	131	0.50	135	118	110	38.1	Substantial
1.33	+33	150	0.58	155	118	120	42.5	Major
1.34	+08	133	0.25	95	93	100	29.7	Major
1.35	+25	149	2.00	125	110	100	38.7	Major
1.36	+32	143	1.17	160	117	110	40.1	Substantial
1.37	+05	134	0.58	130	120	100	32.4	Major
1.38	+07	113	1.50	110	96	110	27.4	Major
1.39	-31	95	0.33	60	86	80	13.7	Substantial
1.40	-31	101	1.75	75	96	90	15.2	Major
1.41	-36	107	2.00	85	91	90	13.7	Substantial
1.42	-36	124	1.75	78	95	100	21.0	Substantial
1.43	-39	121	2.00	78	114	100	19.8	Major
1.44	-58	124	0.08	70	97	100	16.6	Major
1.45	-37	143	0.08	80	118	110	-	Failure
1.46	-52	99	0.50	70	71	80	10.1	Minor
1.47	-80	101	1.00	57	81	50	7.9	Minor
1.48	+44	156	1.00	180	104	120	48.9	Major
1.49	+12	144	0.08	120	97	120	-	Failure
1.50	-24	118	0.33	80	93	100	21.9	Major
1.51	-59	102	1.50	56	78	80	11.5	Major
1.52	-46	103	1.33	73	79	80	13.6	Substantial
1.53	-75	130	0.91	52	86	56	9.4	Minor
1.54	+29	163	1.50	175	99	120	43.4	Major
1.55	+08	194	0.08	142	128	140	39.1	Failure
1.56	-18	137	0.08	100	107	100	28.6	Failure
1.57	-47	109	0.75	69	88	80	15.9	Major
1.58	-73	109	1.50	43	92	64	12.2	Major
1.59	-73	107	0.62	64	82	72	10.1	Minor
1.60	+36	143	1.17	153	121	110	42.6	Major
1.61	+15	350	0.50	146	110	110	34.0	Major
1.62	-11	324	0.92	100	94	90	23.1	Major
1.63	-41	294	1.50	65	80	90	14.8	Substantial

Legend

1. SWL refers to the still water level relative to the crest in mm.

2. H_o' is the equivalent deep water wave height of the incident wave in mm.

3. t is the duration of the test in hours.

4. OT is the overtopping level relative to the crest in mm.

5. RD is the rundown level relative to SWL in mm.

6. HT is the height of the transmitted wave in mm.

7. q is the average discharge past the breakwater in litres per second per metre.

Consider a wave height which results in zero or only partial damage for a non-overtopped structure (say 10 to 12 cm for example). As the water and overtopping levels are increased there is initially a substantial decrease in stability and increase in damage. Initiation of damage occurred at an overtopping level of approximately 3.5 cm and major damage or failure when the overtopping level reached approximately 4.5 cm. With further increase in water levels damage is not substantially increased and eventually a water level is reached when damage ceases.

Tests under steady flow (Section 7.2) indicated that damage was initiated when the head difference was approximately 9 cm (Figure 11) and that this damage tended to occur catastrophically. In the present tests the head difference was somewhat higher and the damage more gradual.

7.5 Overtopping Tests - Test Series II

The objective of these tests was to examine the influence of wave period on the stability of an overtopped breakwater. The height of the breakwater was maintained constant at 0.45 m. Wave periods tested were

1.34, 2.20 and 2.69 secs.

In these tests the wave paddle setting was held constant and the water level very slowly increased (or decreased). The parameters measured, as a function of time, were equivalent deep water incident wave height, still water level (SWL) relative to the crest, overtopping level, transmitted wave height and the degree of damage. Test results are presented in Figures 18(a) to 18(d) for 1.34 sec period, Figures 19(a) to 19(j) for 2.20 sec period and Figures 20(a) to (i) for 2.69 sec period.

As for the still water tests at a wave period of 1.70 secs (Test Series I) the degree of damage was dependent not only on the wave height but also upon the SWL relative to the crest and the overtopping level.

A summary of the test results, using the same format of Figures 16 and 17 for the still water tests, are presented in Figures 21, 22 and 23. It should be noted that the time scale between these figures varies and damage was often catastrophic with complete failure occurring over a very short time span.

8. PROTOTYPE APPLICATION OF RESULTS

Results presented in this report are deliberately given in actual dimensions, as the correct formulation of the relevant dimensionless parameters and their inter-relationship are not clear to the authors at this stage. However, the test conditions have been carefully chosen to correspond to typical Australian conditions when based on normal hydraulic model practice using Froudian scaling. An example of such scaling is given below.

Assuming:

i. The relative density of the armour in the model and the prototype are equal.

ii. Model testing is undertaken in fresh water whilst the prototype will be in sea water with a relative density of 1.025.

iii. Shape of the model and prototype units are geometrically similar.

iv. Gravity forces dominate (Froude scaling).

It is a simple exercise to show that under these conditions the length scale (L_R) is related to the mass scale (M_R) by:

$$M_R = 0.955\ L_R^3 \tag{31}$$

and the time scale T_R is related to the length scale by:

$$T_R = \sqrt{L_R} \tag{32}$$

These scaling parameters can be used for the transfer of the results given in this report to typical prototype conditions as illustrated by the following examples:

Example (a)

Parameter	Model	Prototype	Scale
Armour Mass	100 gms	5 tonnes	50,000
Linear Scale			37.41 (Equation 31)
Time Scale			6.12 (Equation 32)
Wave Period	1.34 sec	8.2 sec	
	1.70 sec	10.4 sec	
	2.20 sec	13.5 sec	6.12
	2.69 sec	16.5 sec	
Linear Dimensions	100 mm	3.741 m	
eg. wave height,	10 mm	0.374 m	37.41
water level,	1 mm	0.037 m	
overtopping etc.			

Example (b)

Parameter	Model	Prototype	Scale
Armour Mass	100 gms	2 tonnes	20,000
Linear Scale			27.56 (Equation 31)
Time Scale			5.25 (Equation 32)
Wave Period	1.34 sec	7.0 sec	
	1.70 sec	8.9 sec	
	2.20 sec	11.6 sec	5.25
	2.69 sec	14.1 sec	
Linear Dimensions	100 mm	2.760 m	
	10 mm	0.276 m	27.56
	1 mm	0.028 m	

Further it is of interest to consider a typical design example as given below:

Design Conditions

Wave Height	3.65 m
Wave Period	8 to 16 sec
Water Depth	10 m.

Assuming a relative density of the armour rock of 2.65, the mass of primary armour required on a slope of 1 on 1.5 can be calculated from Hudson equation using a design K_D value of 4.0 in accordance to the recommended methods outlined in S.P.M. (1973).

$$M = \frac{\rho H^3}{K_D \Delta^3 \cot \alpha}$$

$$= \frac{2.65 \times 3.65^3}{4.0 \times (1.625)^3 \times 1.5}$$

$$= 5.0 \text{ tonnes}$$

Model scales applying to the results of the present study are therefore as given in Example (a) above.

Mass	50,000
Length	37.41
Time	6.12

Reference to Table 3 indicates that for a non-overtopped structure there would be no damage for a 4.30 m wave (0.115 m model), and major damage for a 4.86 m wave (0.13 m model). The difference between the design wave height of 3.65 m and the model results reflects some conservatism or factor of safety in choosing a design K_D value of 4.0. For the non-overtopped structure these results are not significantly influenced by wave period.

If the crest of the structure is now reduced to allow for overtopping Figures 17, 21, 22 and 23 can be used to estimate the effect on the stability of the structure.

For a wave period of 8.2 sec (1.34 model), Figure 21 indicates that there is no loss of stability irrespective of the water level.

For a wave period of 10.4 sec (1.70 model), Figure 17 indicates that accelerated damage may occur if the water level is within 3.4 m of the crest (0.09 model) and the overtopping exceeds 1.1 m (0.03 model) over the crest. For water levels in excess of 3 m below the crest (0.08 model), the wave height for no damage is reduced to 3.4 m (0.09 model), except if the crest is submerged by more than 1.9 m (0.06 model).

For a wave period of 16.5 sec (2.69 model), Figure 23 indicates that accelerated damage may occur if the water level is within 6.7 m of the crest and overtopping levels exceed 0.07 m over the crest. However, the wave condition for no damage at all water levels is essentially the same as that indicated by the tests for a 10.4 second period.

To ensure no damage to the crest of an overtopped breakwater over the range of wave periods tested and with a similar factor of safety as applying to the seaward face, the mass of the crest armour would need to be increased by a factor proportional to the no damage wave heights cubed or $(11.5/9.0)^3 = 2.1$; that is from 5 to approximately 10 tonnes.

9. DISCUSSION OF RESULTS

i. Current practice in the design of a rubble mound breakwater, subject to moderate overtopping, is to use the design guidelines developed for a non-overtopped structure with the primary armour being carried over the crest and down the leeward face to still water level.

The present tests indicate that (particularly for the longer period waves within the spectra) such a design may be unsafe if the breakwater is subject to moderate overtopping or conservative for an immersed structure.

Insufficient tests have been undertaken to define with certainty the conditions separating stability and instability of an overtopped structure, however, they do indicate that the following parameters play an important role:

a. wave height

b. wave period

c. water level relative to the crest

d. height of overtopping

e. phasing between the incident wave and backflow over the crest

f. breakwater and offshore geometry.

ii. Both theoretical analysis and experimental data indicate that for typical breakwater geometries and rock sizes very little overtopping is permissible under steady flow conditions. This stability can be increased by raising the tailwater level and flattening the leeward face. Under wave action the test results indicate that the stability is somewhat higher than that for steady flow but tend to approach more closely to steady flow conditions as the wave period is increased.

iii. Failure of a non-overtopped rock armoured breakwater is relatively gradual and progressive as the wave height is increased. For an overtopped structure damage tends to occur more rapidly and in the case of steady flow or the longer period waves is often sudden and catastrophic. Consequently greater care is needed in choosing the design conditions.

iv. The stability of a non-overtopped rock armoured breakwater is not strongly affected by wave period. This is not the case for an overtopped structure. The present tests indicate that the stability is effected by wave period in at least two ways.

The more obvious is that for a given wave height, and still water level, wave run-up, overtopping levels, duration of overtopping and head differences are increased all of which tend to increase structural damage.

A second effect is related to the interference which occurs between the incident wave and backflow over the crest. This interference results in local steepening of the wave and high local pressure gradients giving rise to localised damage. Depending upon the phasing, damage is concentrated at locations extending from the seaward to the leeward face. It is obvious that this phasing is also dependent on other factors such as water level, height of the incident wave geometry of the structure and the distribution of the waves within the wave spectra.

v. For random placed rock armour the test results indicate that for a moderately overtopped structure the rock size over the crest may have to be increased by a factor of about two in order to ensure a strength equivalent to the seaward face for the same wave conditions. A similar result was noted by Raichlen (1972) for Tribar armouring.

In practice this added strength is often achieved by careful construction methods to increase interlocking or special placement techniques (such as placement of the long axis perpendicular to the slope, thereby reducing the displacement force, whilst simultaneously increasing the mass surcharge resisting displacement).

However, it should be noted that if the bond between special placement units is lost, sudden and catastrophic damage may occur.

vi. Below, the low water level on the leeward face, it is common practice to reduce the rock size. For the majority of the conditions experienced in the present tests this practice would appear to be justified as a hydraulic jump occurs at the interface between the still water level and the structure which dissipates much of the energy of the overtopping water. However, in several of the tests (both under steady flow and wave action) a submerged jet was observed which penetrated over the full extent of the back face and for a considerable distance along the seabed. Under these conditions there was little energy dissipation and undoubtedly in a movable bed model severe erosion at the heel of the structure would have occurred.

vii. General design rules such as decreasing the back-slope armour units if the crest elevation is greater than 0.7 times the wave height (Walker, 1975) should be treated with caution and accepted only after confirmation by model testing as also recommended by Walker (1975).

The present tests indicate that for the longer period waves this design rule is unsafe.

viii. The present study indicates that for a moderately overtopped structure the highest damage is likely to result when overtopping occurs with low tailwater levels. Consequently long period swell may be more damaging than larger shorter period storm waves; a condition which has also been noted by Per Bruun (1978).

Consequently the design wave for the back-slope is not necessarily that used for the design of the seaward face.

10. CONCLUSIONS

The objectives of this study was to examine some of the factors which influence the stability of a rock armoured breakwater subject to overtopping under wave action. The results indicate that the variable governing armour stability of an overtopped structure are far more complex than those applying to the seaward face and current design guidelines do not ensure the safety and stability of the crest and the backface of an overtopped structure.

More study is needed before reliable design criteria can be established and at the present time all proposals should be subjected to rigorous model testing. Such testing should cover a much wider range of test conditions than those applying to a non-overtopped structure, as long period swell may be considerably more damaging than the higher shorter period storm waves normally adopted for the design of a non-overtopped breakwater.

Model testing should at least include the following parameters:

i. The full range of wave heights;

ii. The full range of wave periods;

iii. The full range of water depths resulting from tide and storm surge;

iv. Variation in wave spectral density and phasing;

v. The interaction between these variables.

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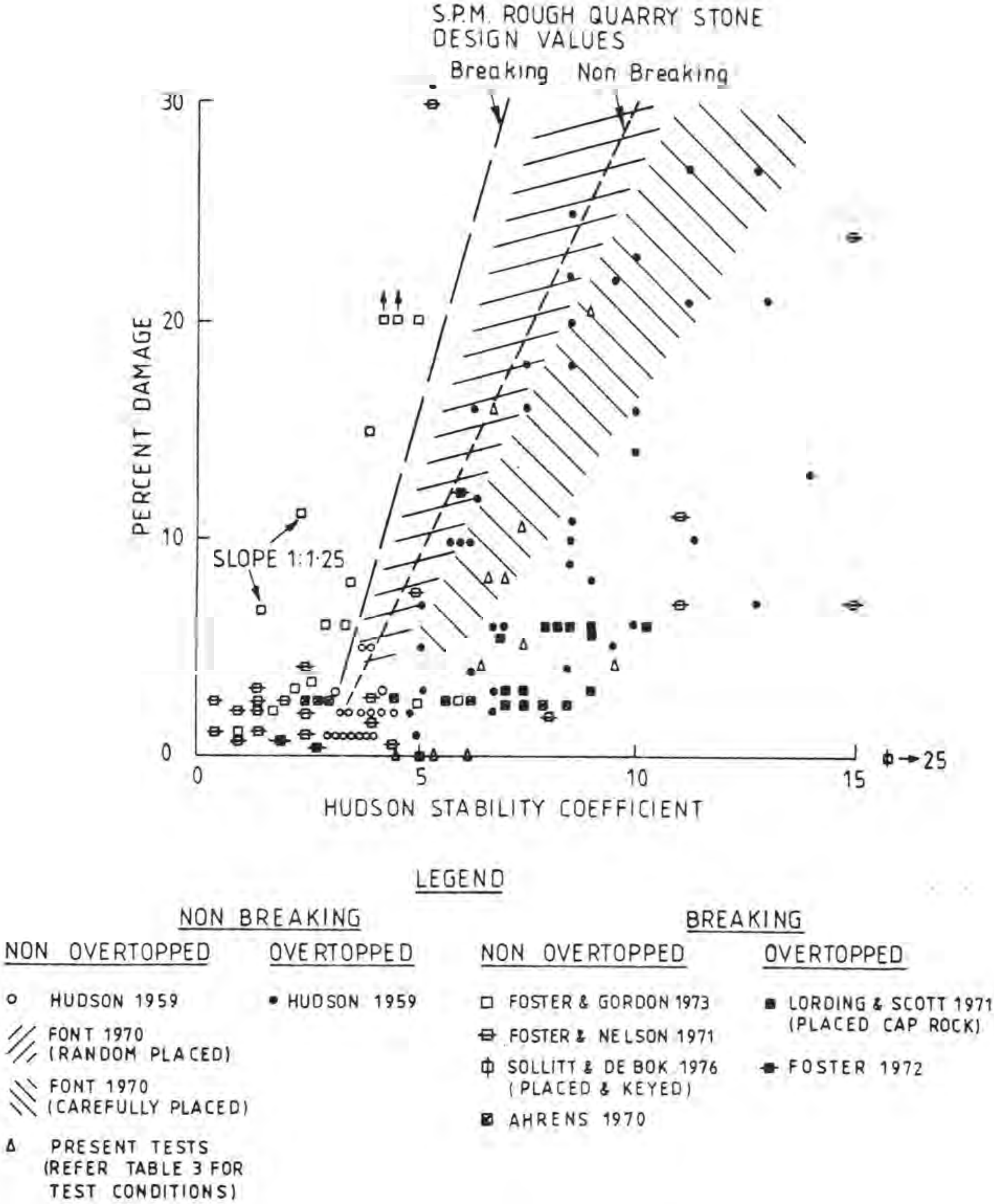


HUDSON STABILITY COEFFICIENT
FOR QUARRY STONE BREAKWATERS

FIGURE 1

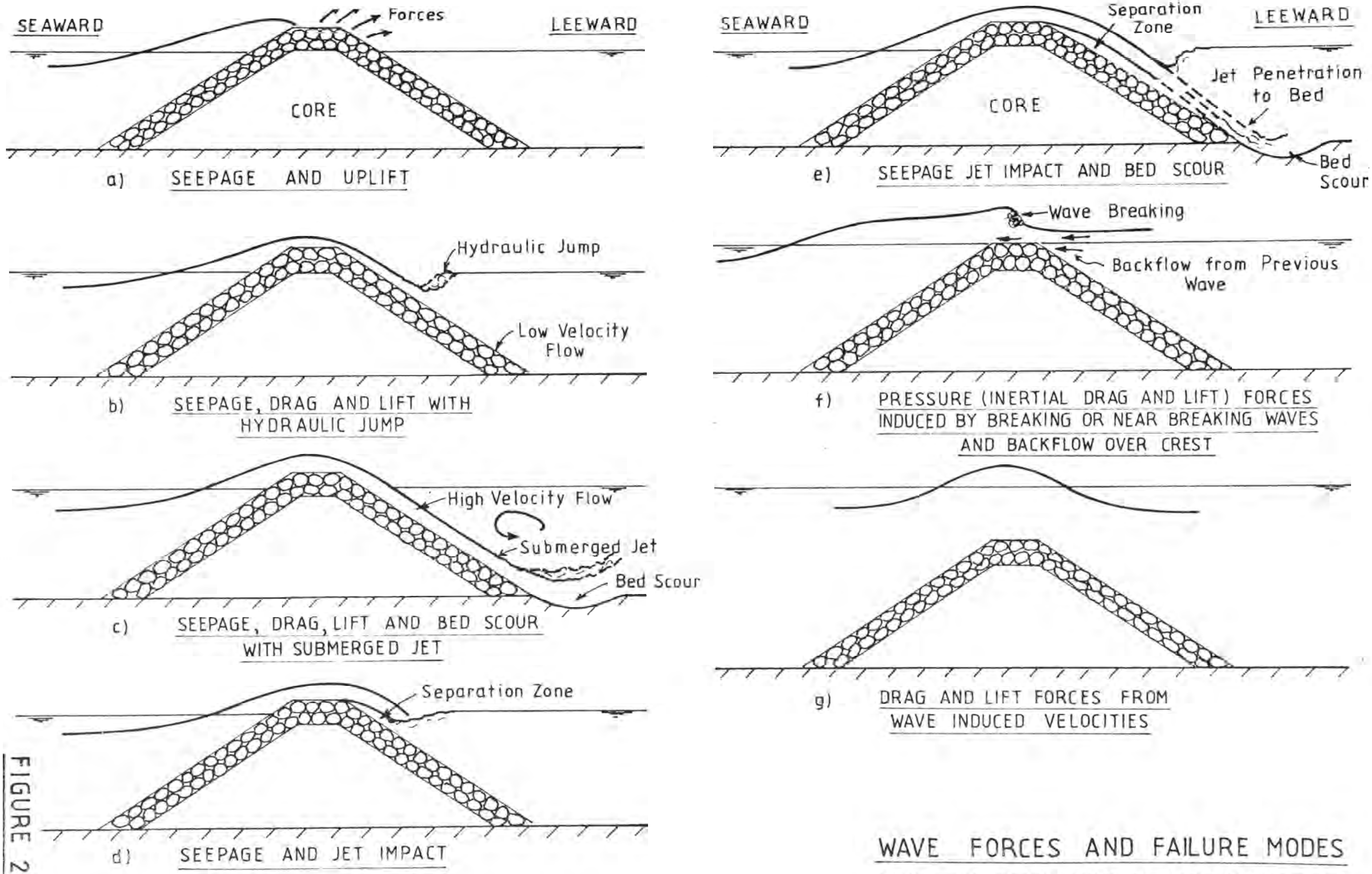
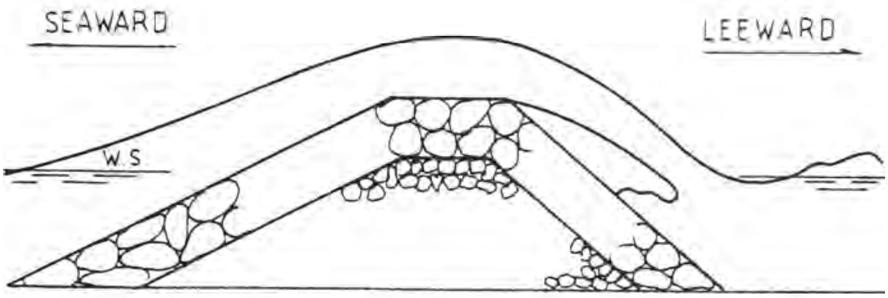


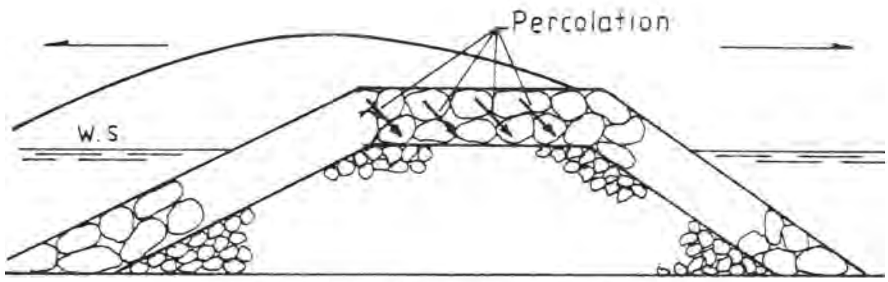
FIGURE 2.

WAVE FORCES AND FAILURE MODES FOR AN OVERTOPPED BREAKWATER



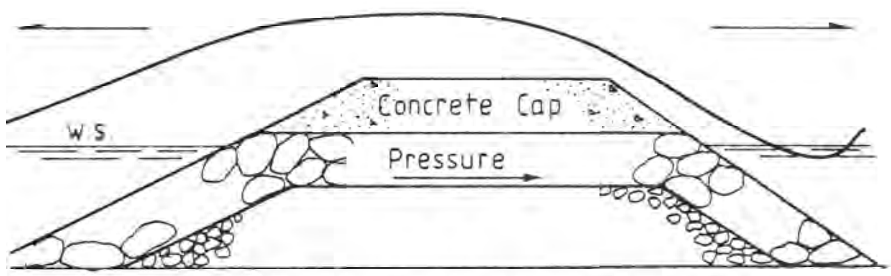
STEEPEN BACK SLOPE UNTIL JET CLEARS

FIGURE 3



INCREASE WIDTH OF CROWN

FIGURE 4

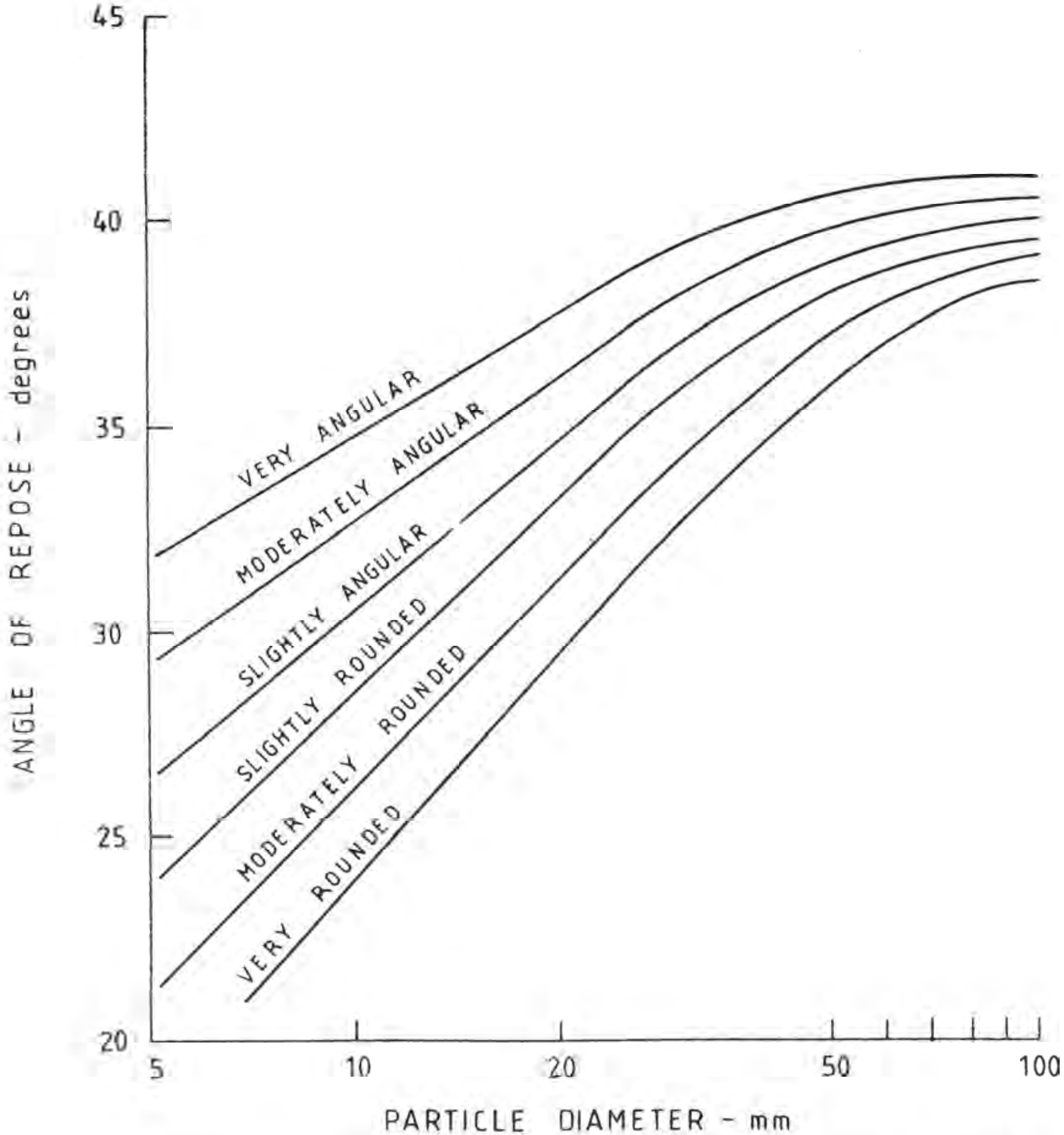


STABILISE WITH CONCRETE CAP

FIGURE 5

(After Walker 1975)

FIGURE 3, 4 & 5.



ANGLE OF REPOSE OF NON COHESIVE MATERIALS
(AFTER U.S. BUREAU OF RECLAMATION)

FIGURE 6

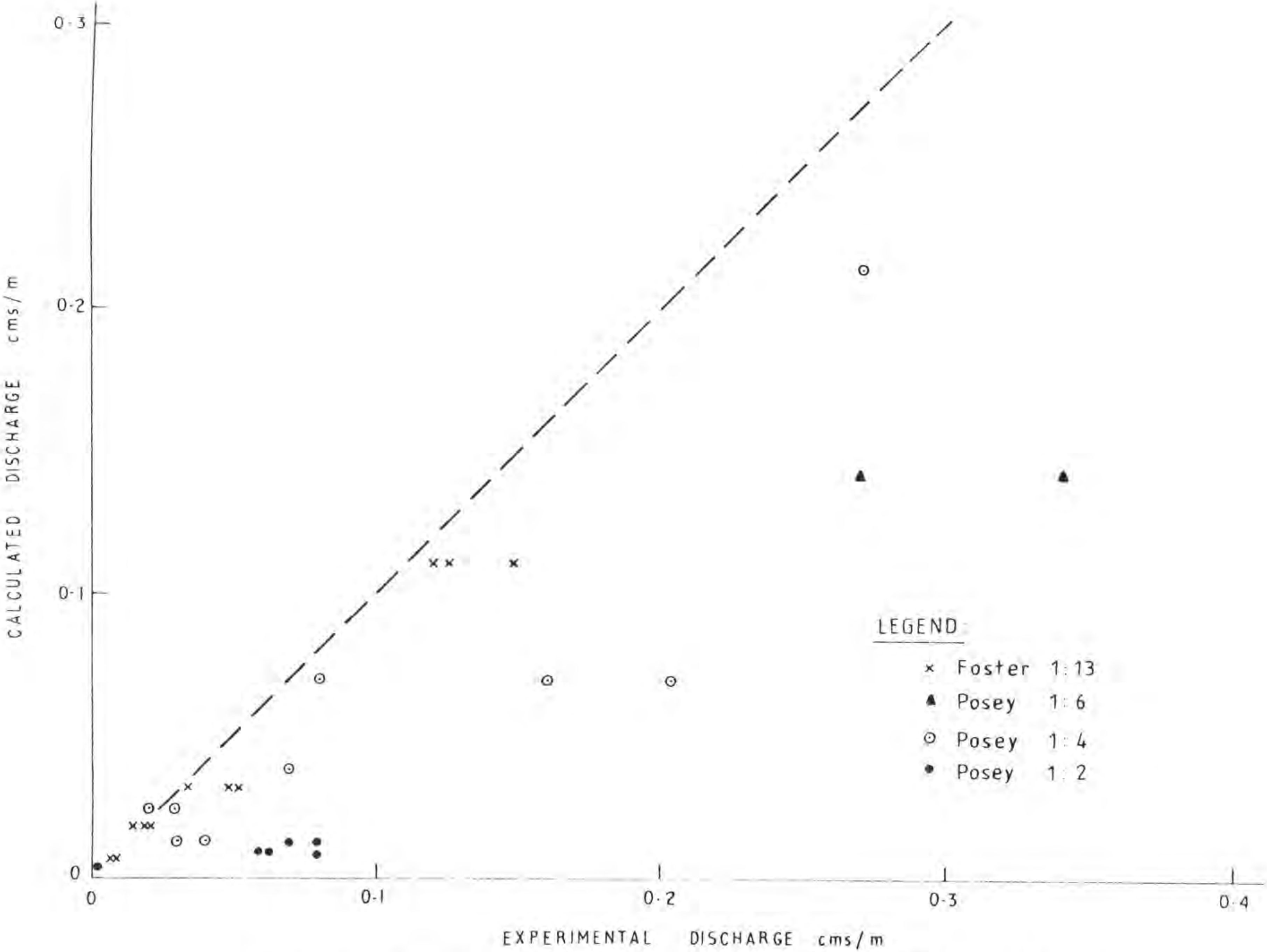
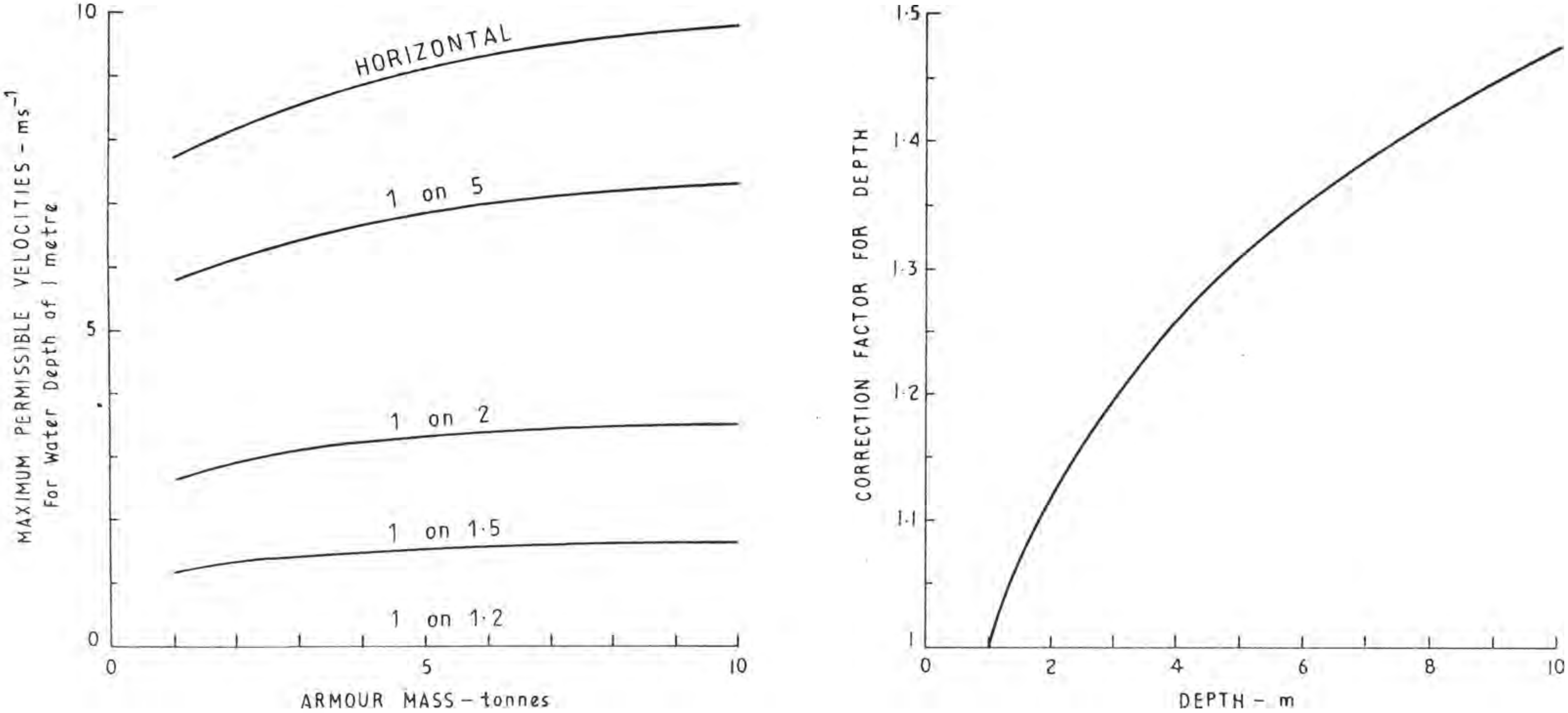


FIGURE 7.

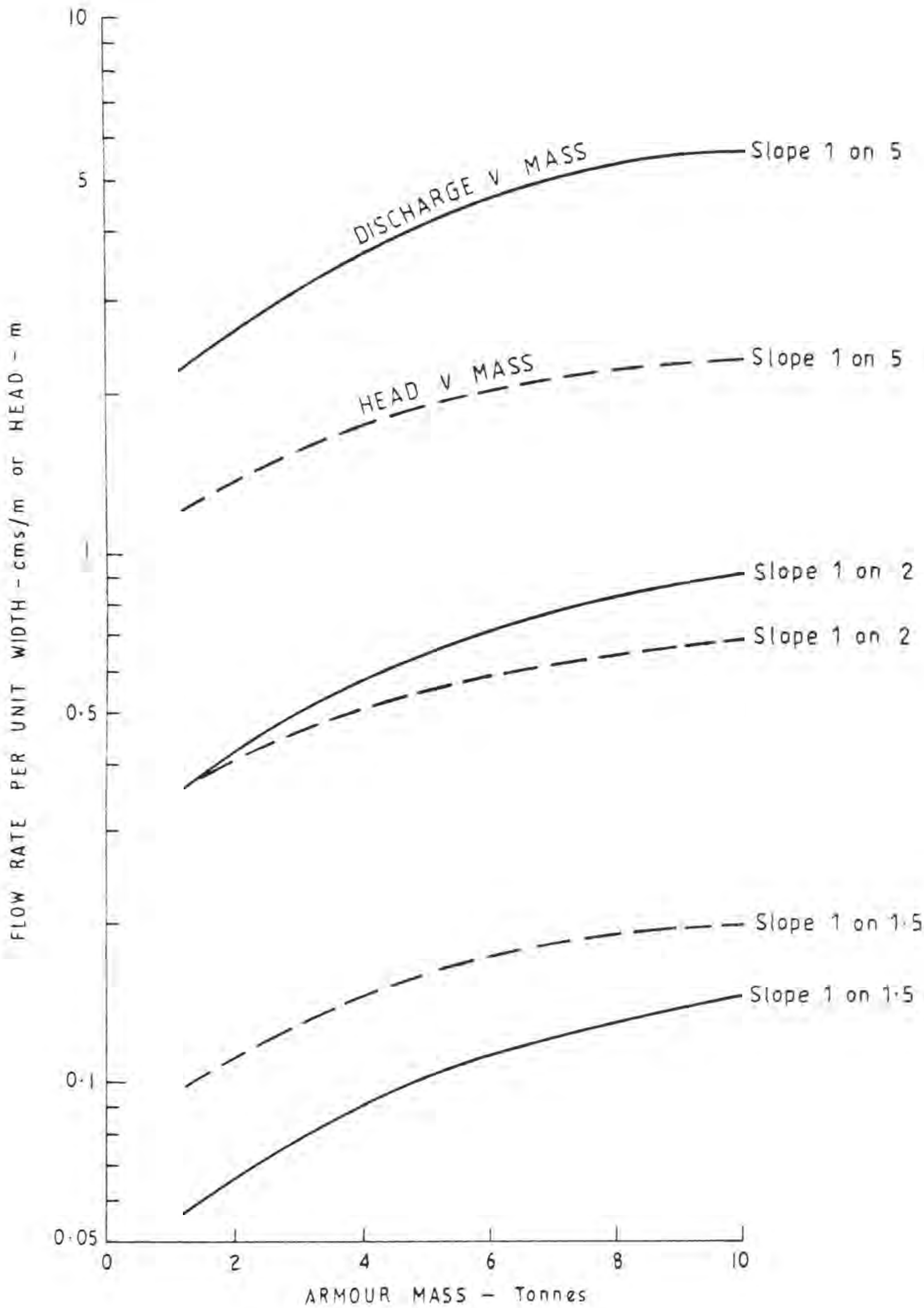
COMPARISON BETWEEN PREDICTED CRITICAL FLOW RATES AND EXPERIMENTAL RESULTS FOR STEADY UNIFORM FLOW



MAXIMUM PERMISSIBLE VELOCITIES FOR ROCK ARMOUR —

STEADY FLOW (For $\phi = 40^\circ$ and relative density = 2·65)

FIGURE 8.



STABLE DISCHARGES AND HEADS FOR STEADY UNIFORM FLOW ON BACKFACE AND WEIR DISCHARGE COEFFICIENT OF 1·6

FIGURE 9.

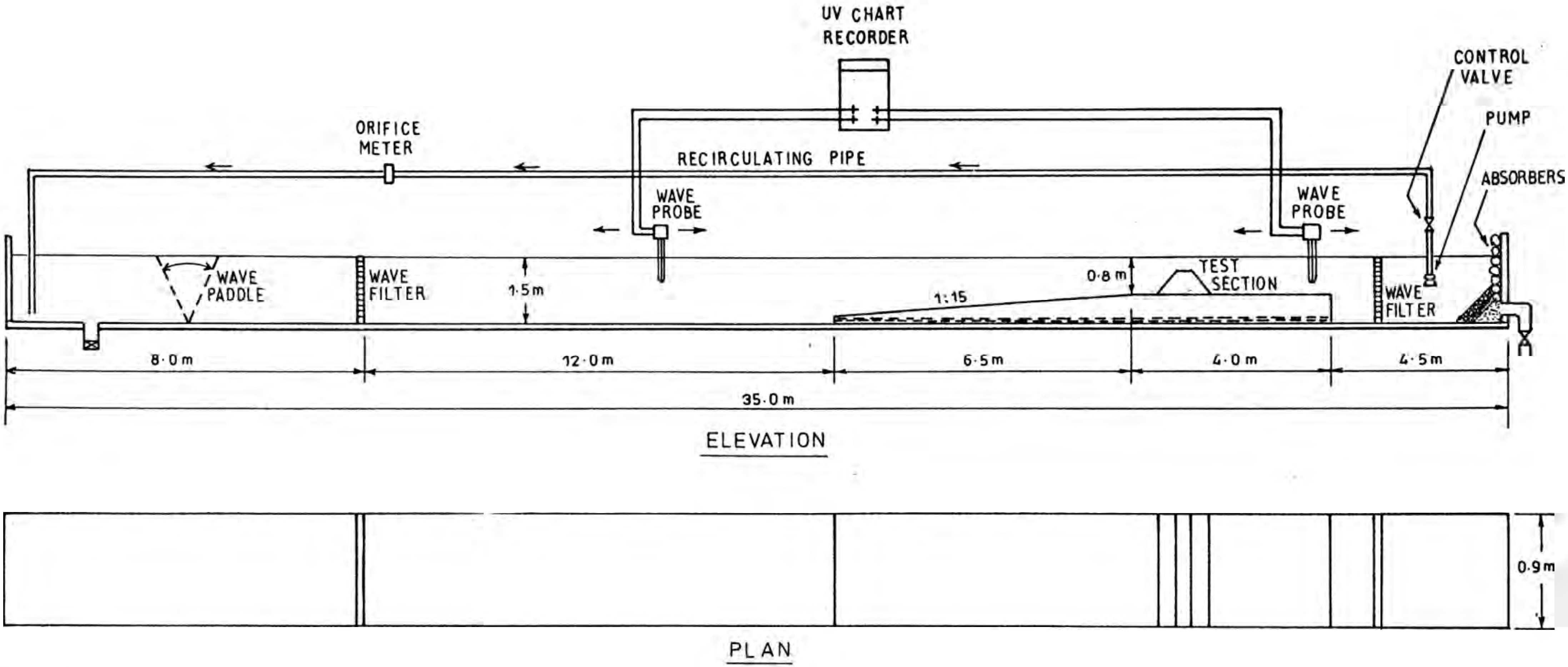
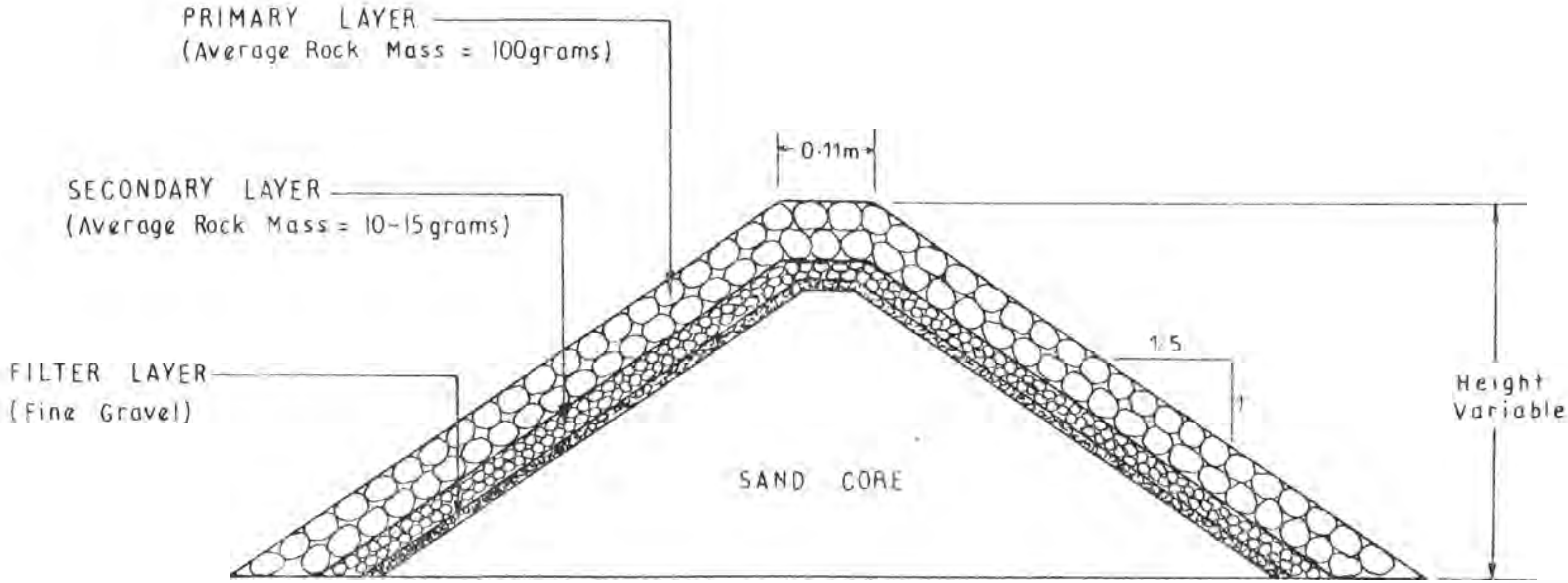


FIGURE 10.

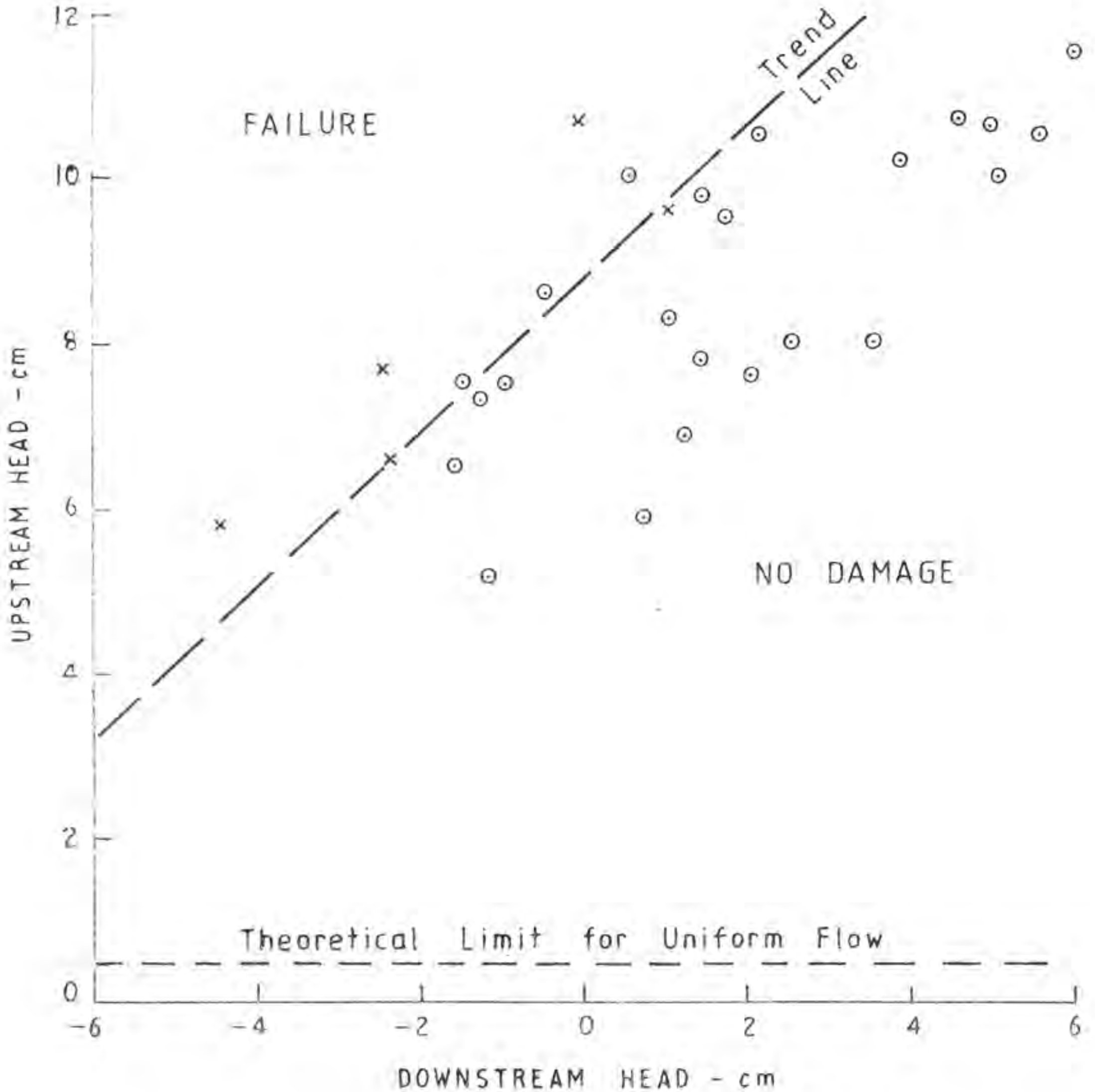
MODEL LAYOUT IN THE WAVE FLUME

(Not to scale - Schematic only)



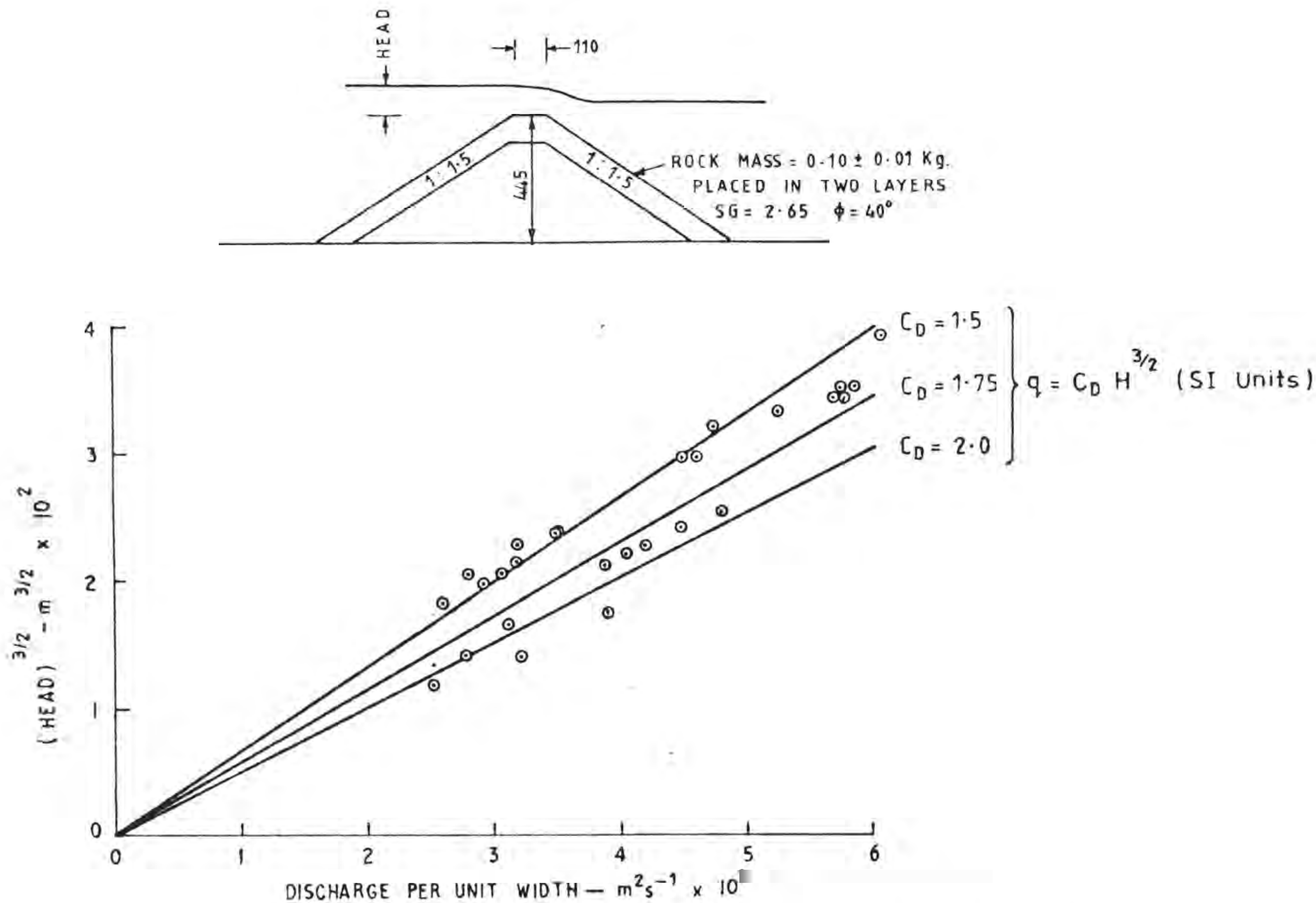
CROSS-SECTION OF BREAKWATER MODEL

FIGURE 11.



CREST DAMAGE UNDER STEADY FLOW

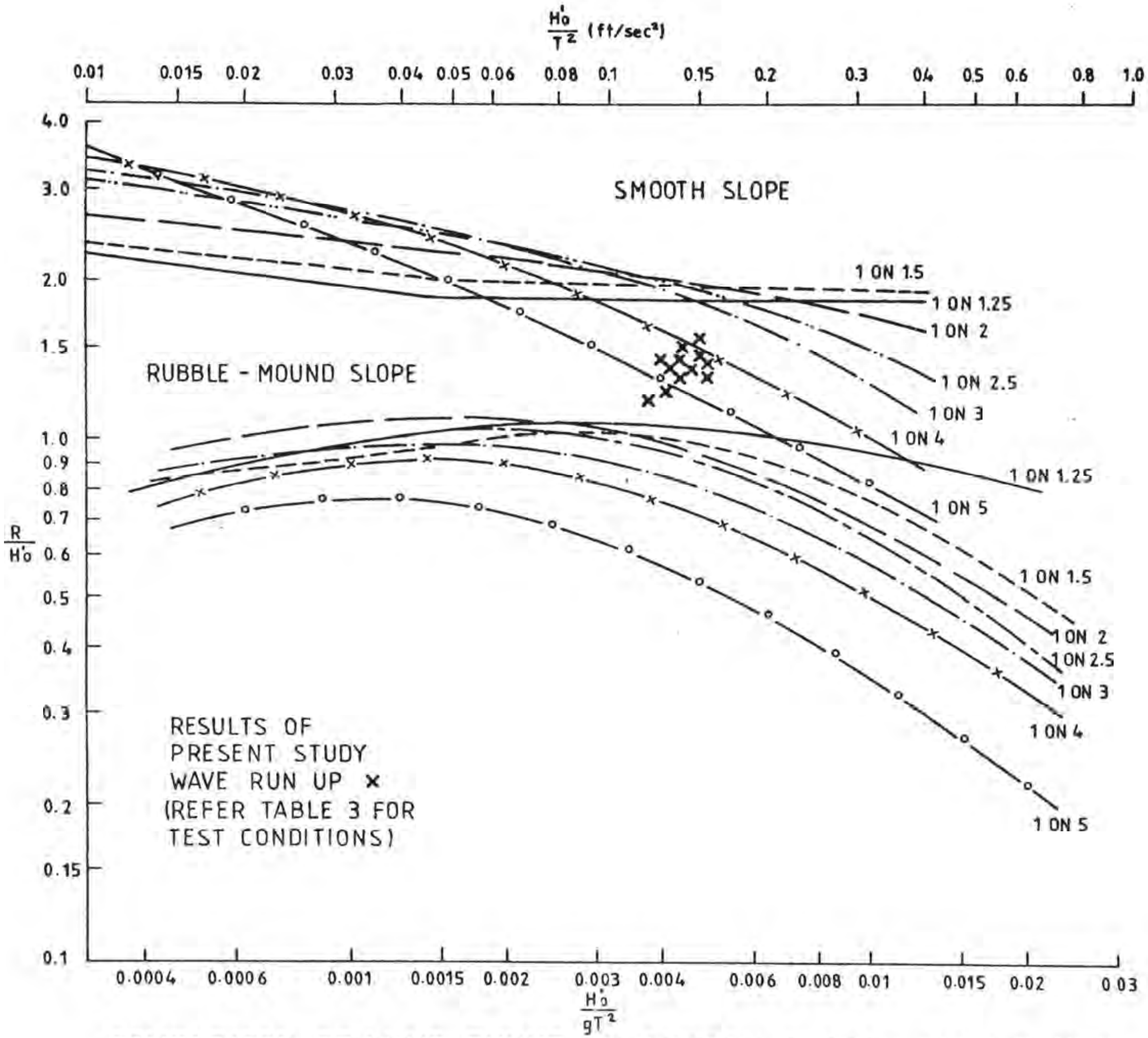
FIGURE 12.



DISCHARGE HEAD RELATIONSHIP - STEADY FLOW

(Submergence Ratios 0 to 0·5)

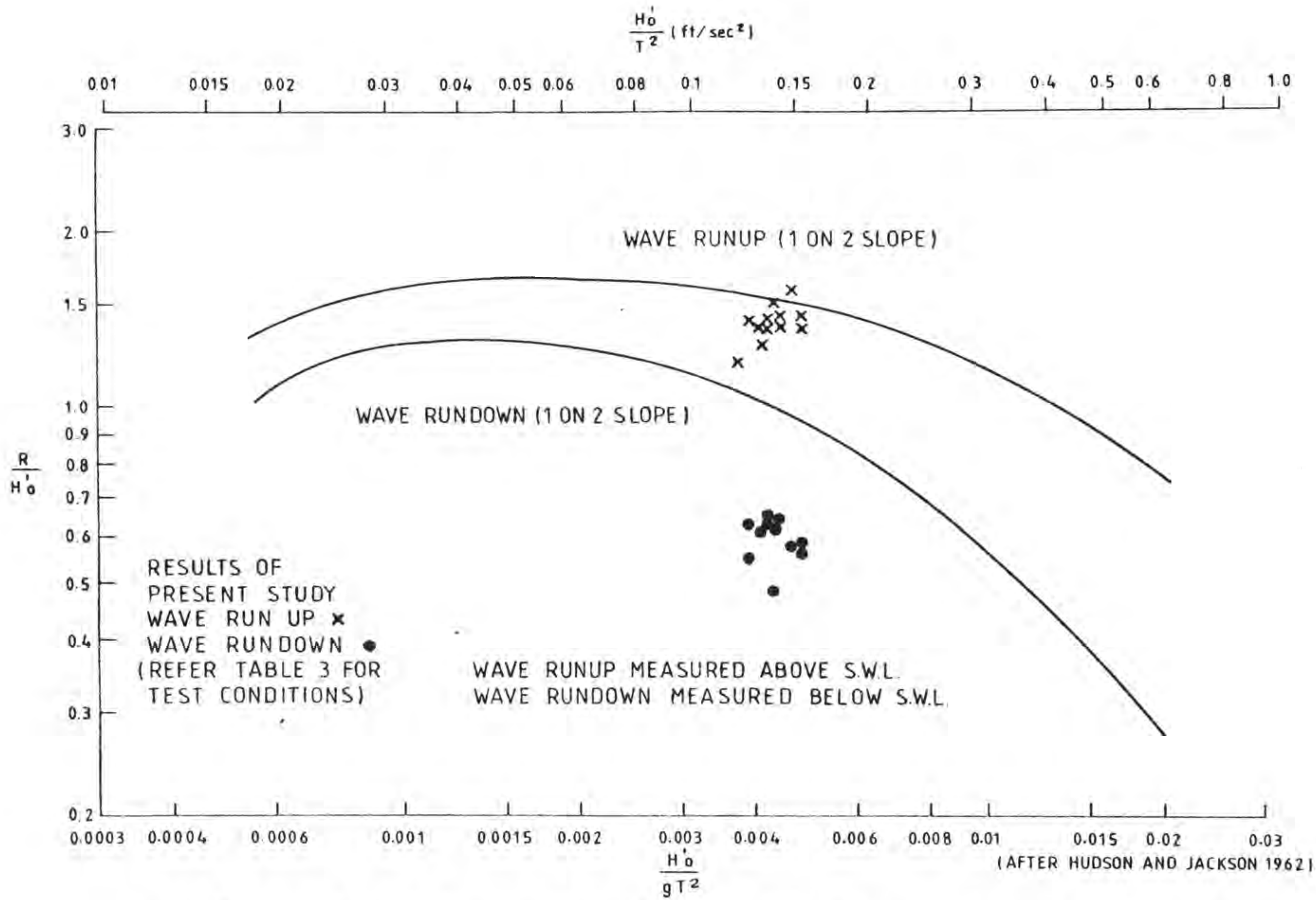
FIGURE 13.



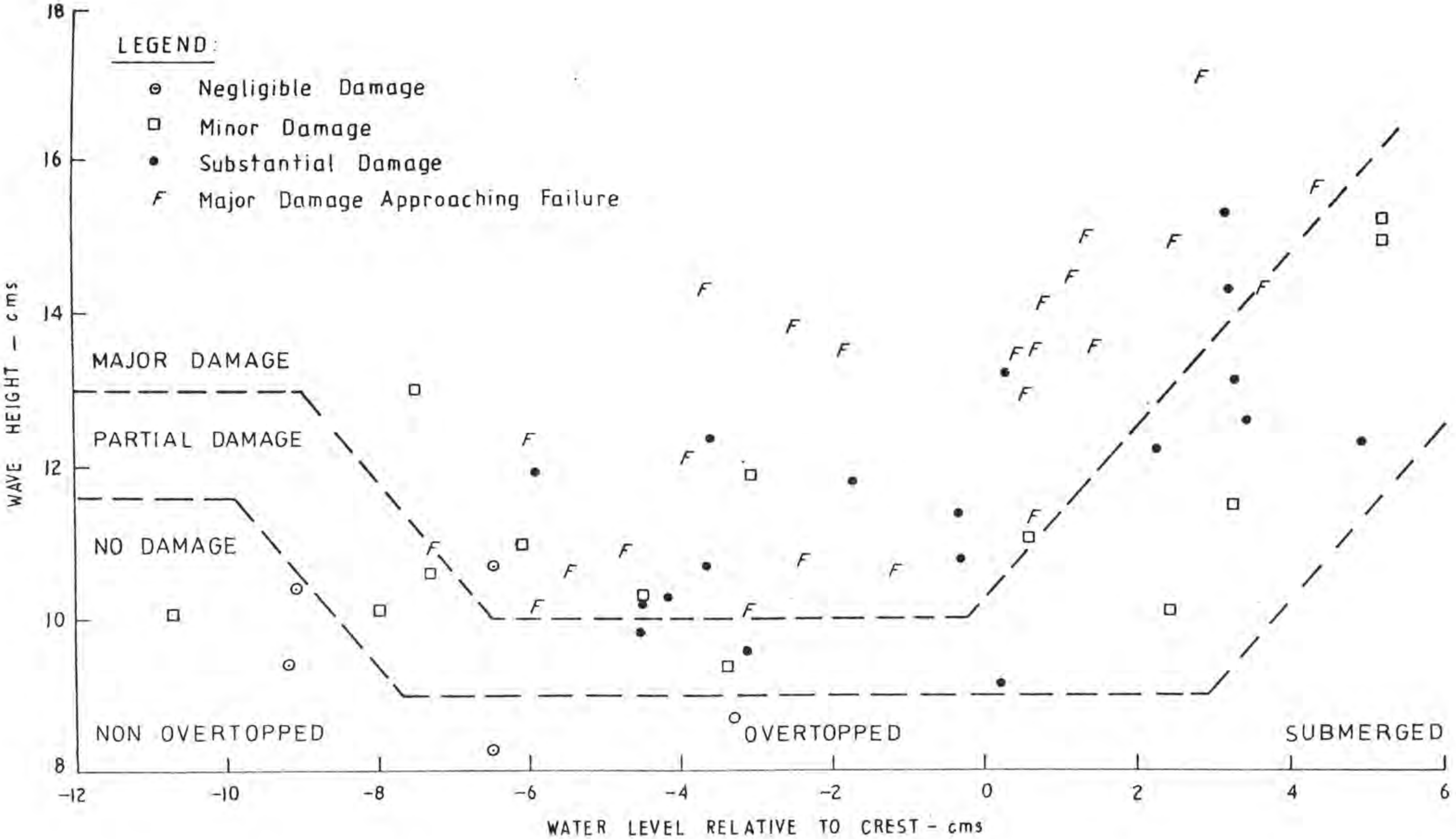
COMPARISON OF WAVE RUNUP ON SMOOTH SLOPES WITH RUNUP ON PERMEABLE RUBBLE SLOPES (DATA FOR ds/H'o > 3.0)
(REPRODUCED FROM S.P.M. 1976)

FIGURE 14

FIGURE 15



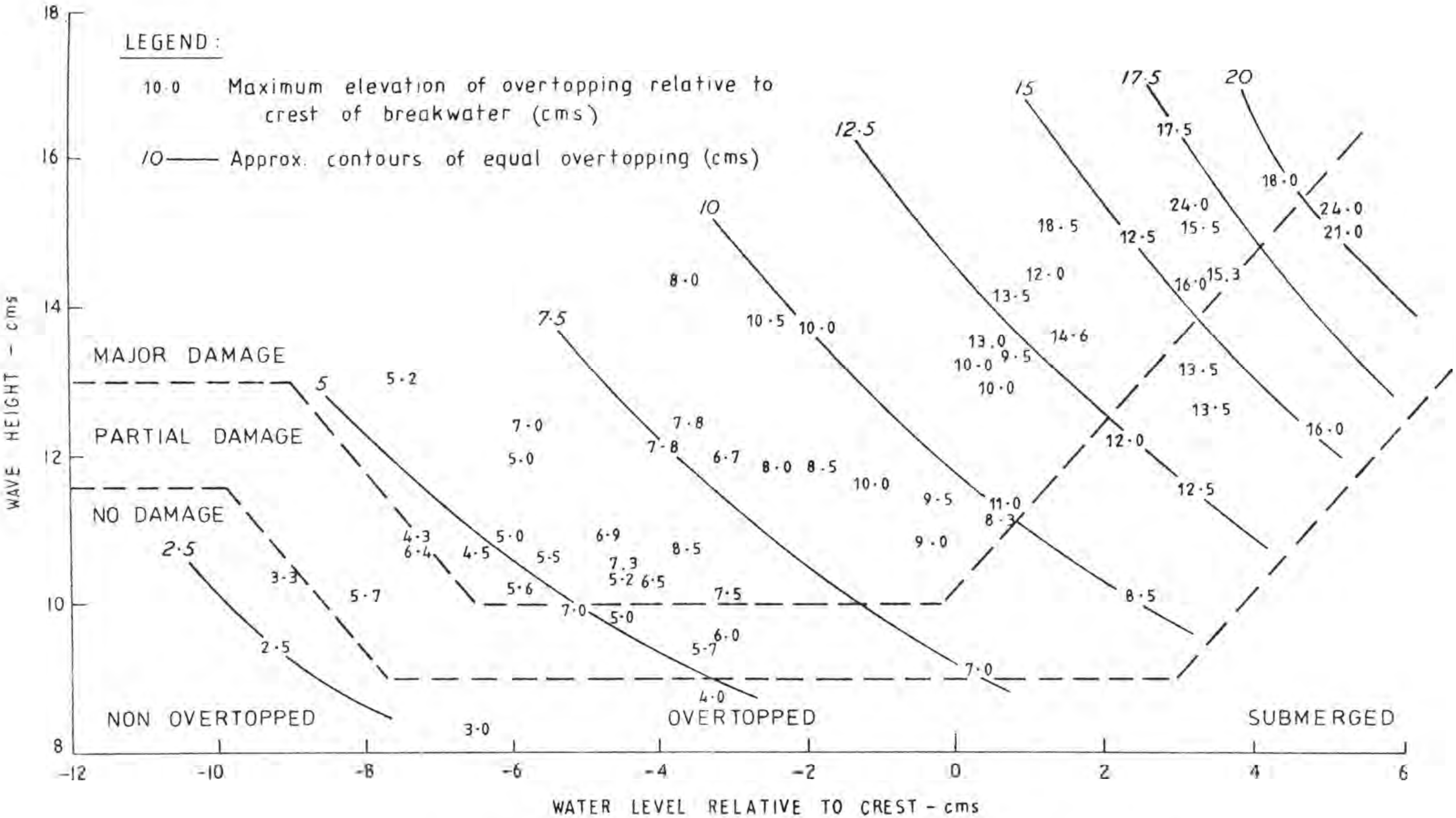
WAVE RUNUP AND RUNDOWN ON GRADED RIPRAP, 1:2 SLOPE IMPERMEABLE BASE, VERSUS H_o'/gT^2 (data for $d_s/H_o' > 3{\cdot}0$)
(REPRODUCED FROM S.P.M. 1976)



STABILITY OF OVERTOPPED BREAKWATER

WAVE PERIOD 1·70 SEC.

FIGURE 16.



VARIATION OF OVERTOPPING LEVEL WITH WAVE HEIGHT AND WATER DEPTH - WAVE PERIOD 1·70 SEC.

FIGURE 17.

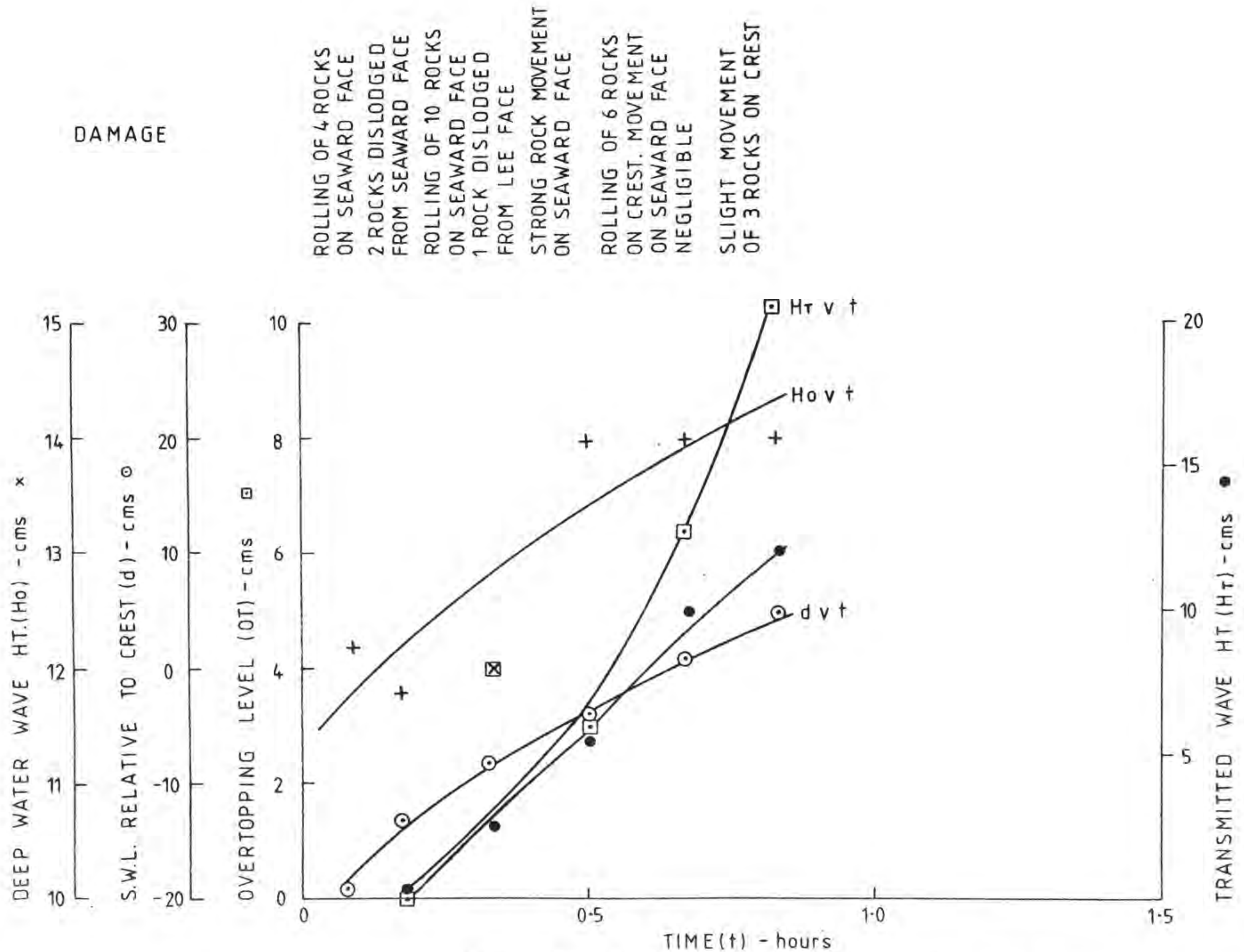
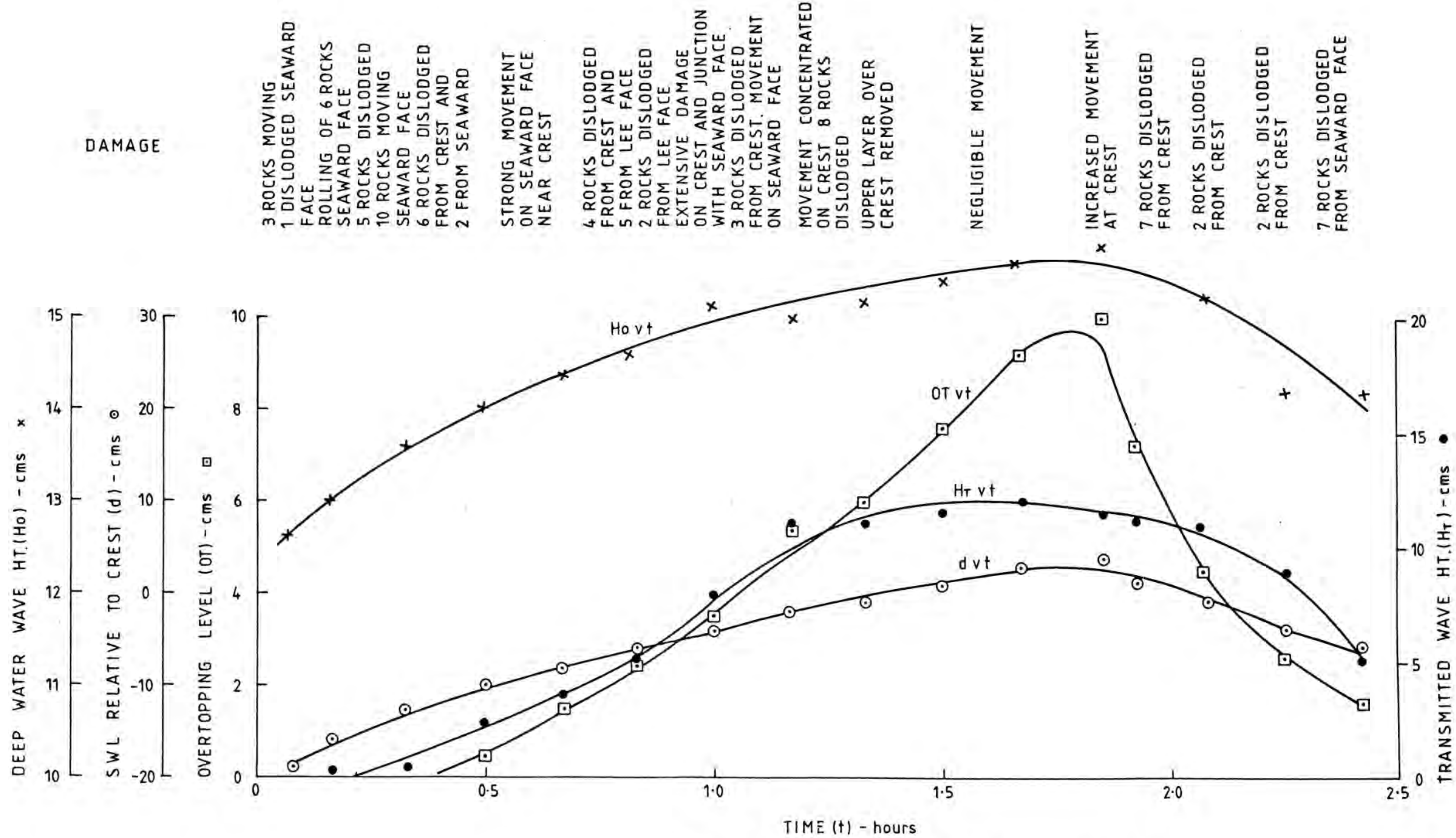


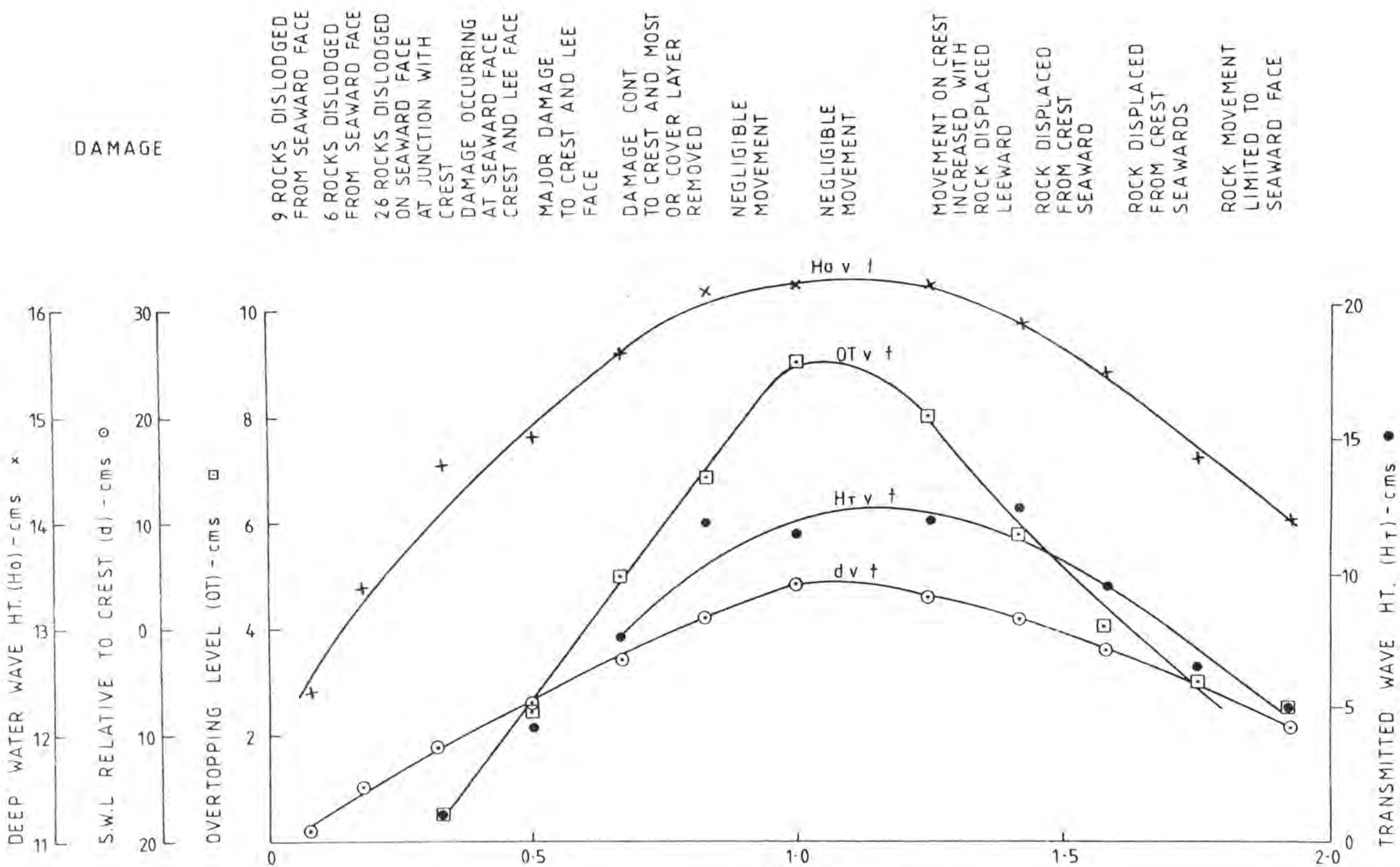
FIGURE 18 (a)

STABILITY TESTS - WAVE PERIOD 1·34 sec.



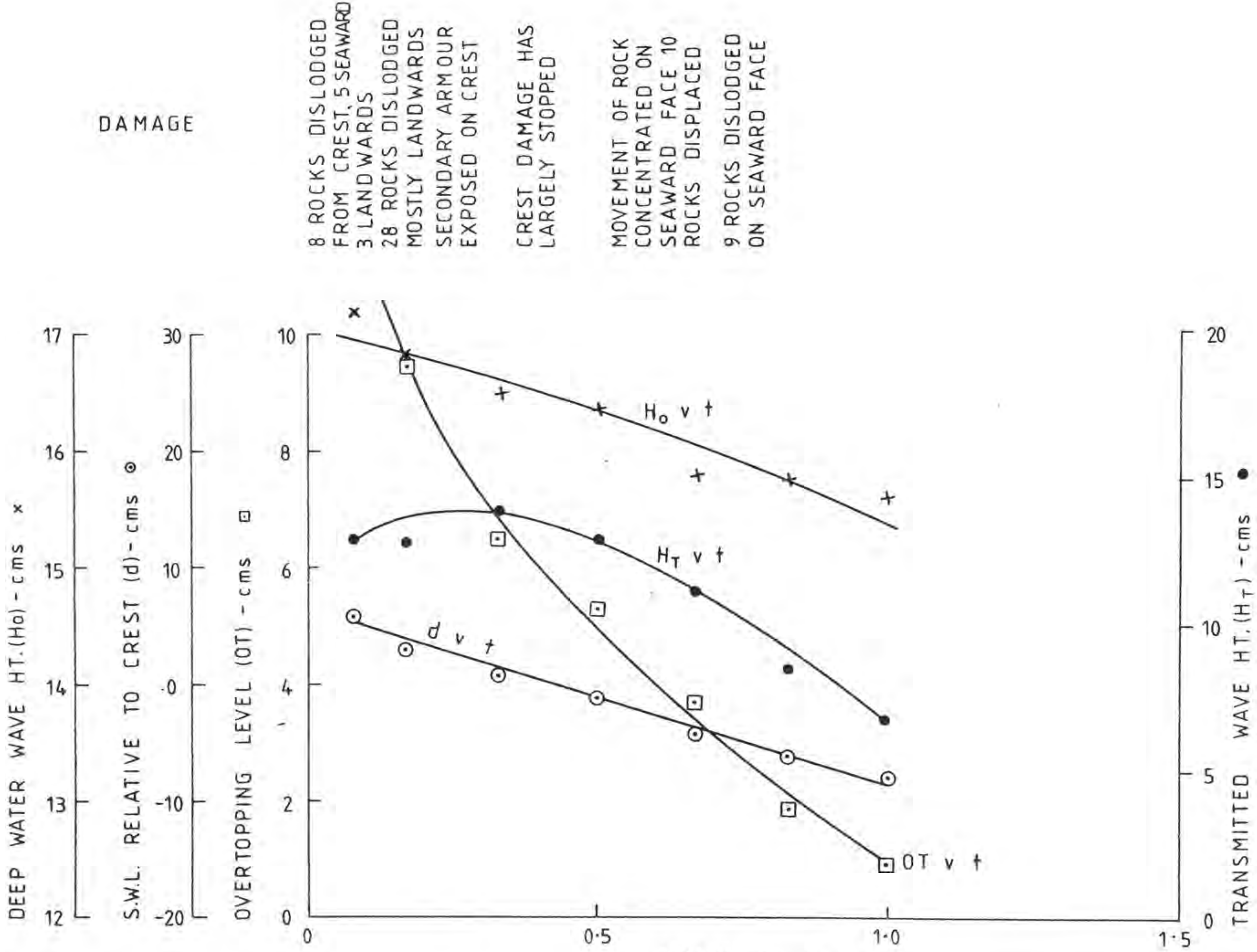
STABILITY TESTS - WAVE PERIOD 1·34 sec.

FIGURE 18(b)



STABILITY TESTS WAVE PERIOD 1·34 sec.

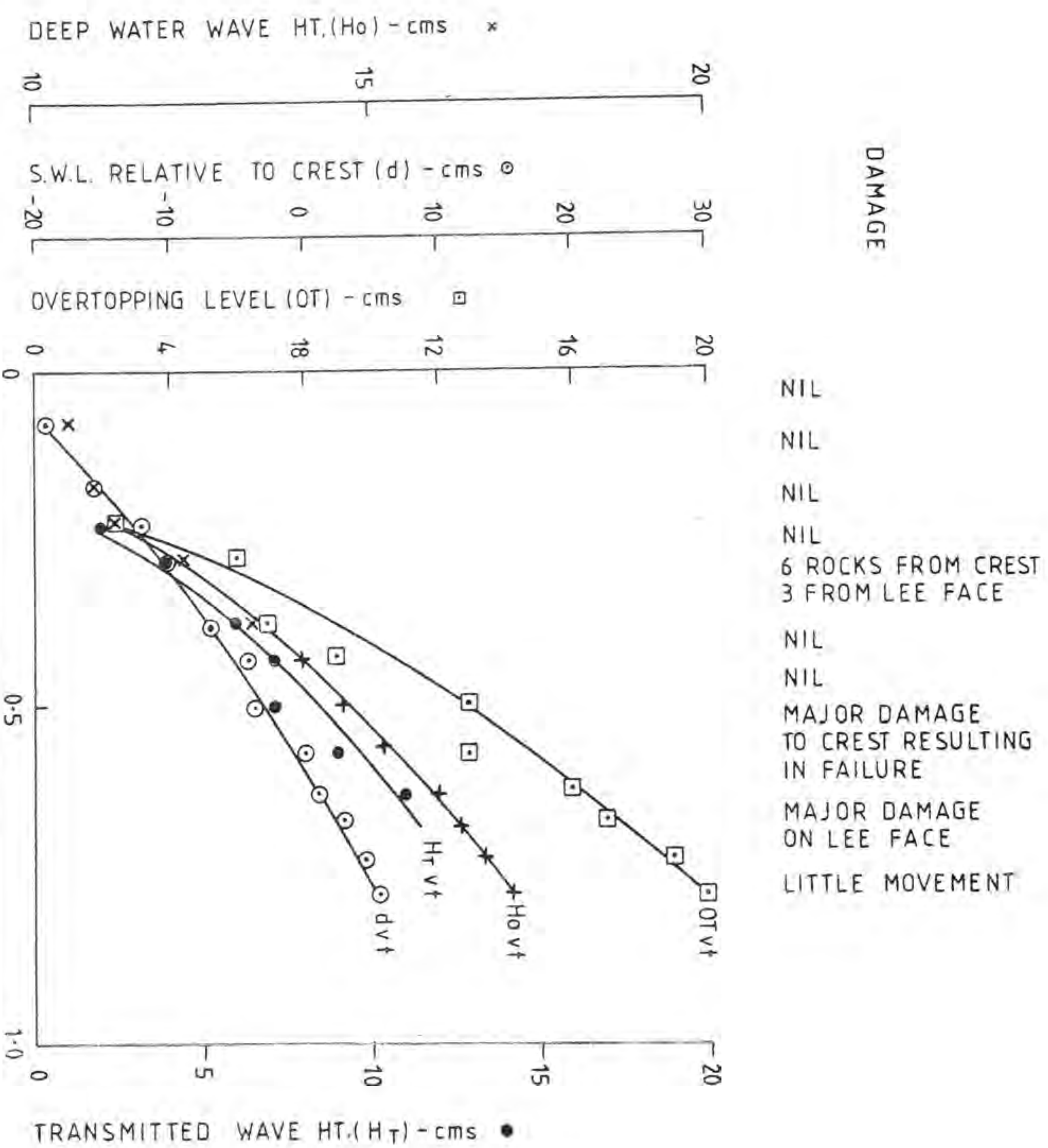
FIGURE 18(c)



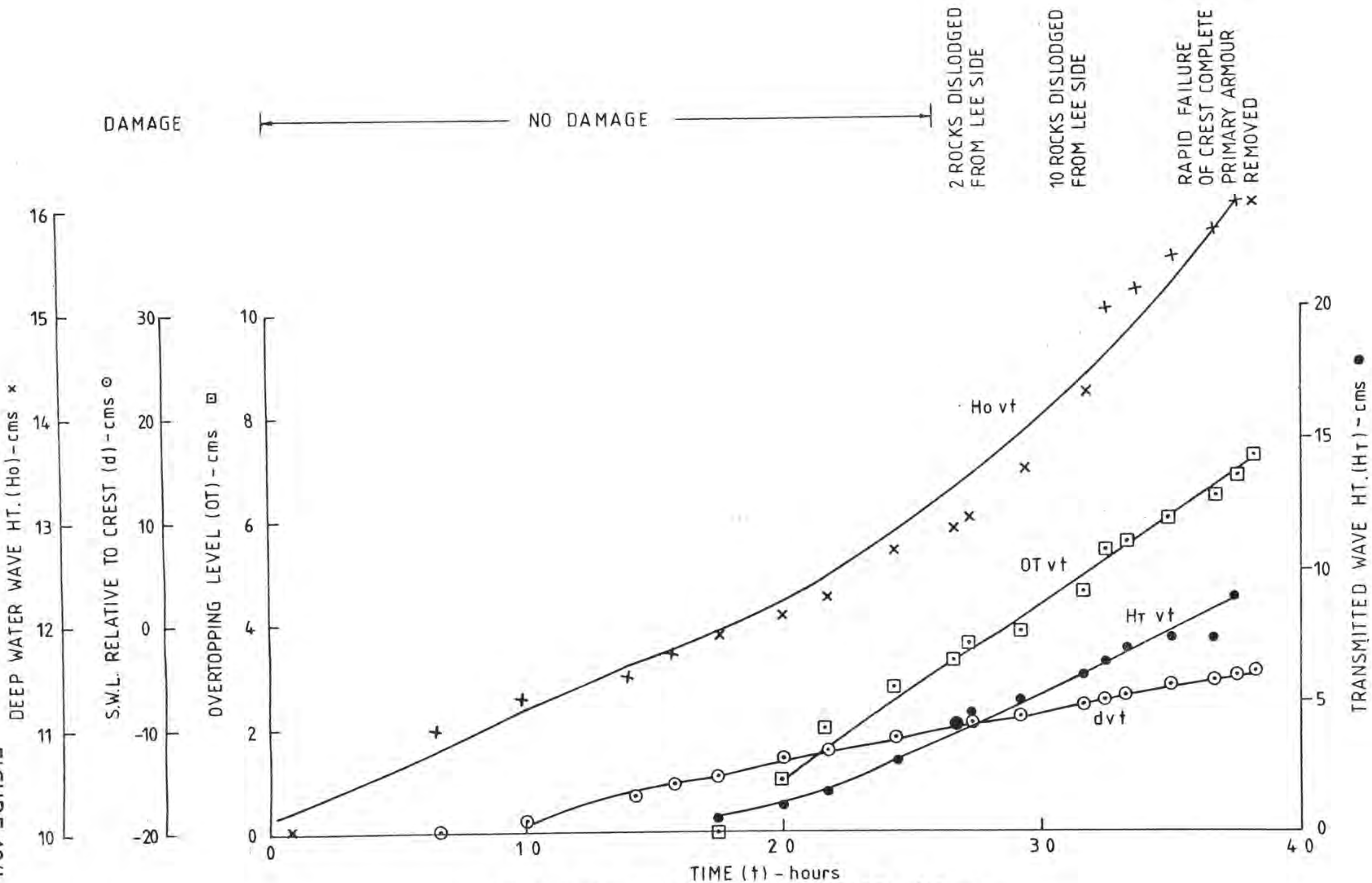
STABILITY TESTS WAVE PERIOD 1·34 sec.

FIGURE 18(d)

FIGURE 19(a)

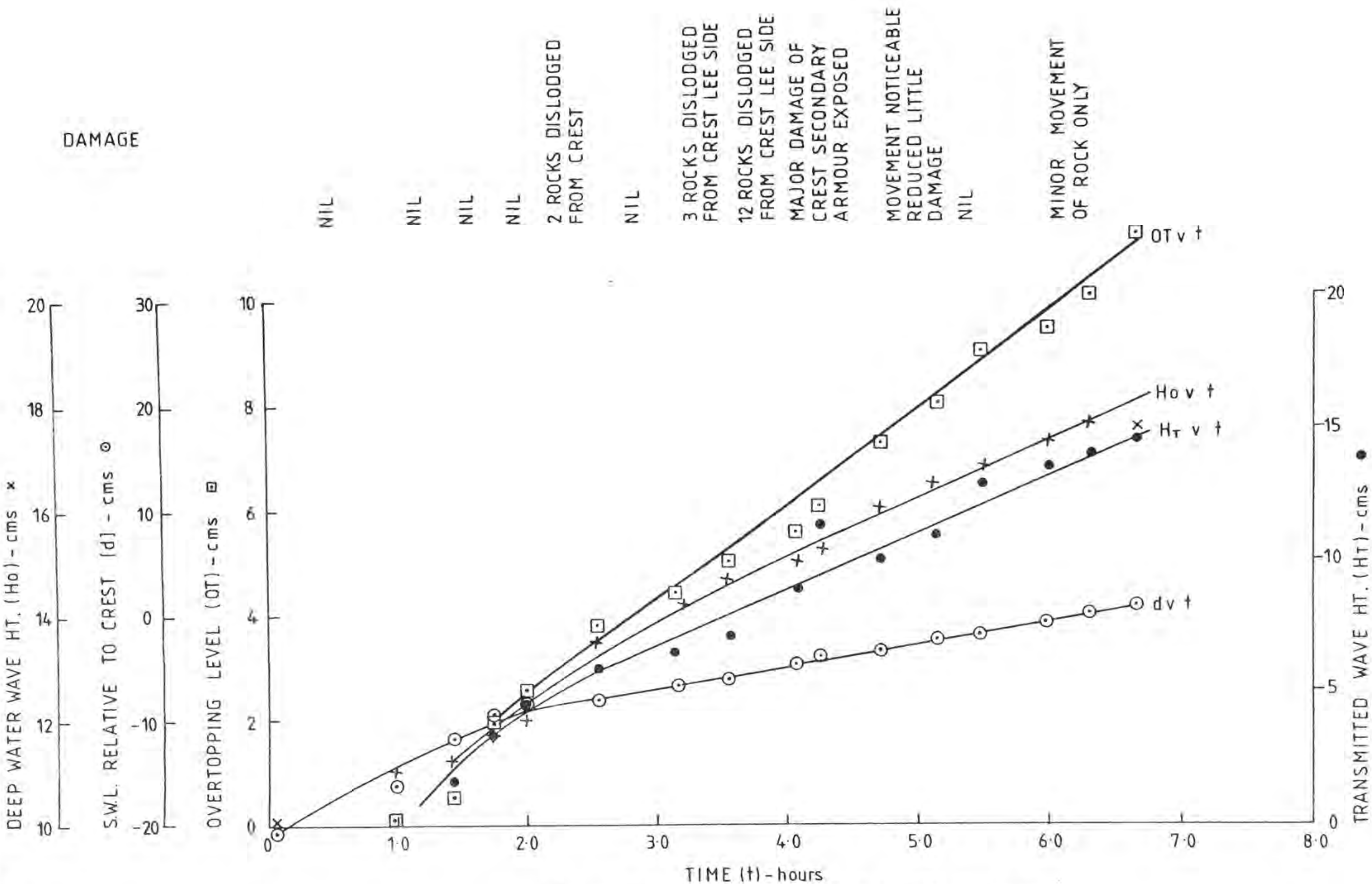


STABILITY TESTS - WAVE PERIOD 2·2sec.



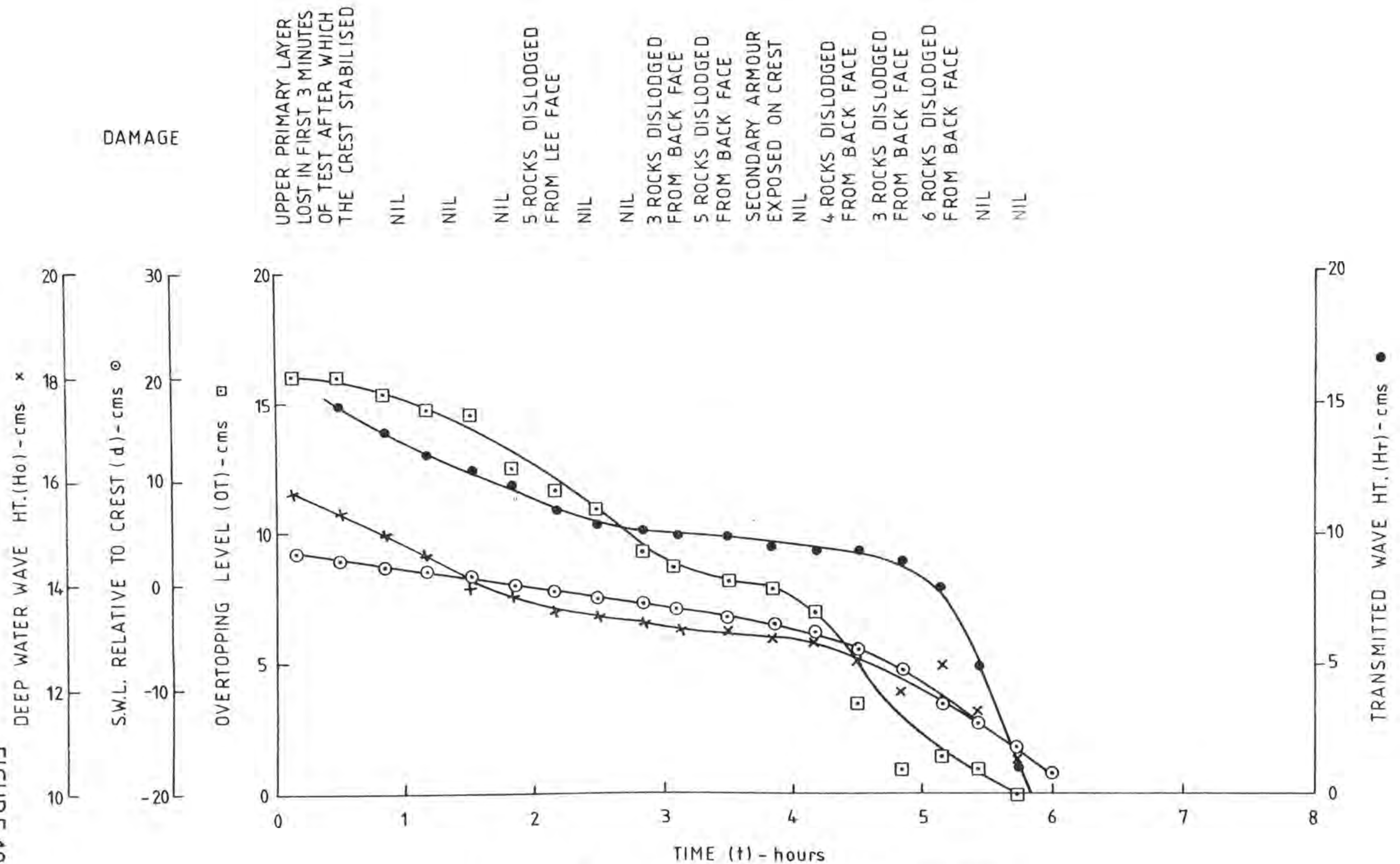
STABILITY TESTS - WAVE PERIOD 2·2 sec.

FIGURE 19(b)



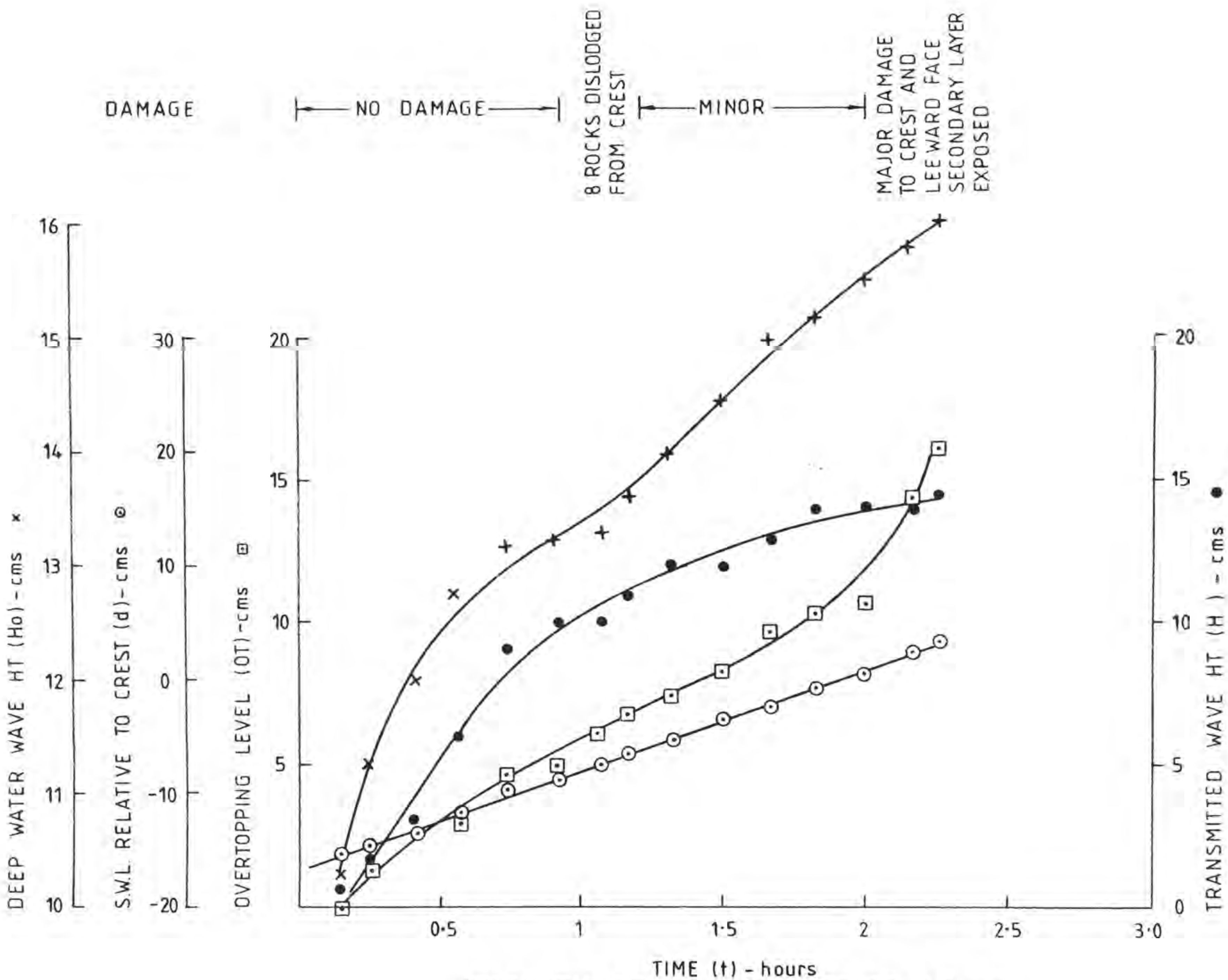
STABILITY TESTS - WAVE PERIOD 2·2 sec.

FIGURE 19(c)



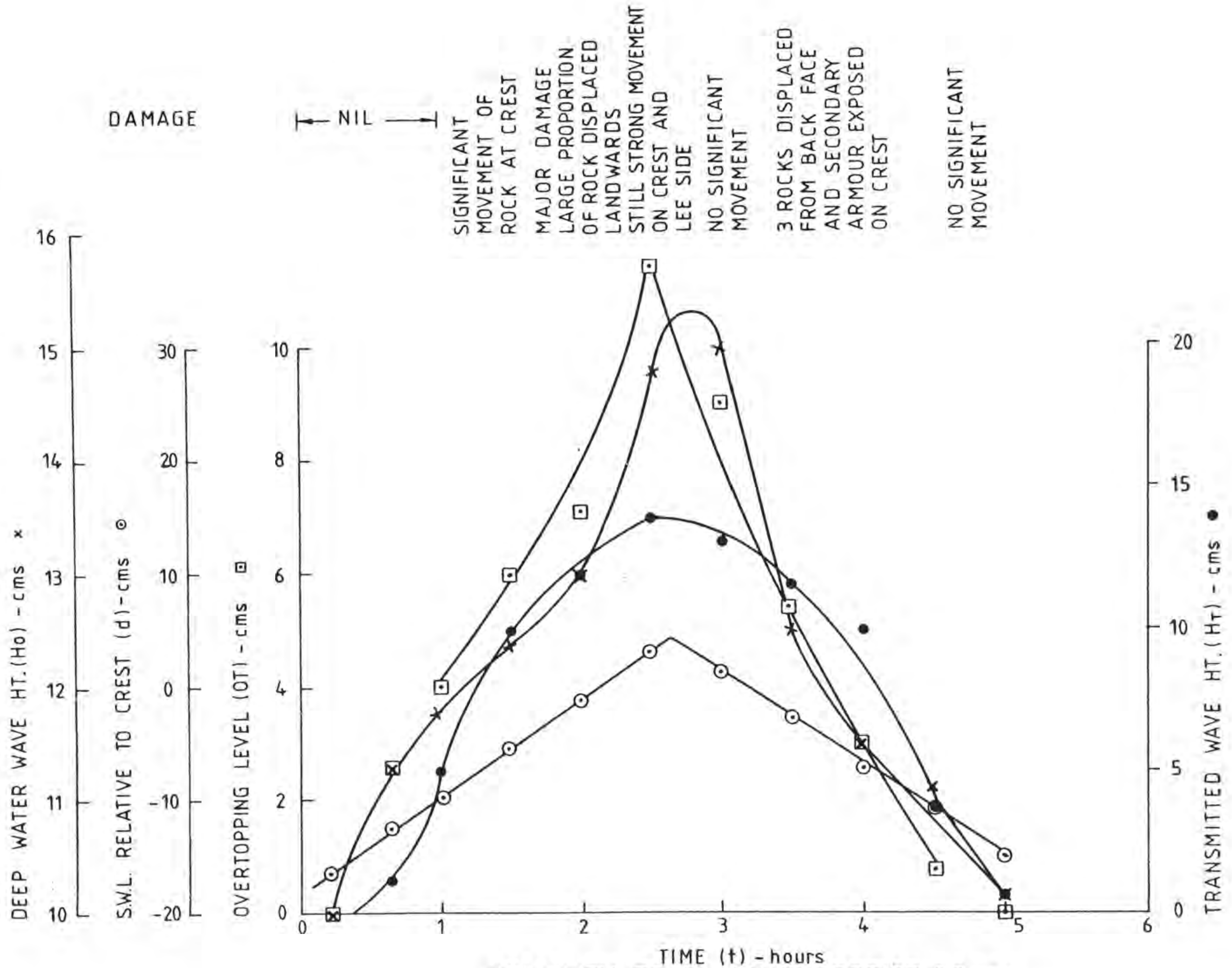
STABILITY TESTS - WAVE PERIOD 2·2 sec.

FIGURE 19(d)



STABILITY TEST - WAVE PERIOD 2·2 sec.

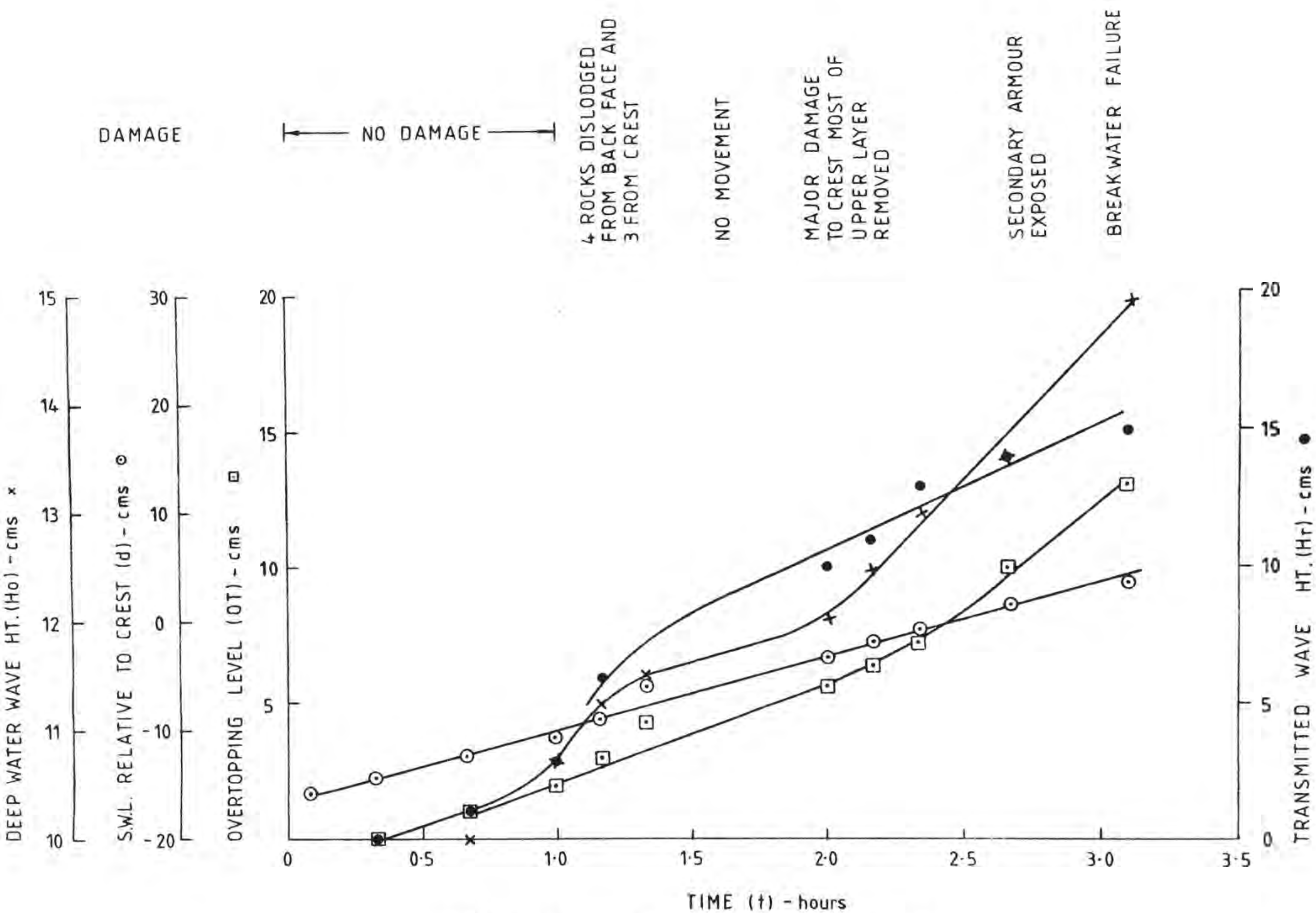
FIGURE 19(e)



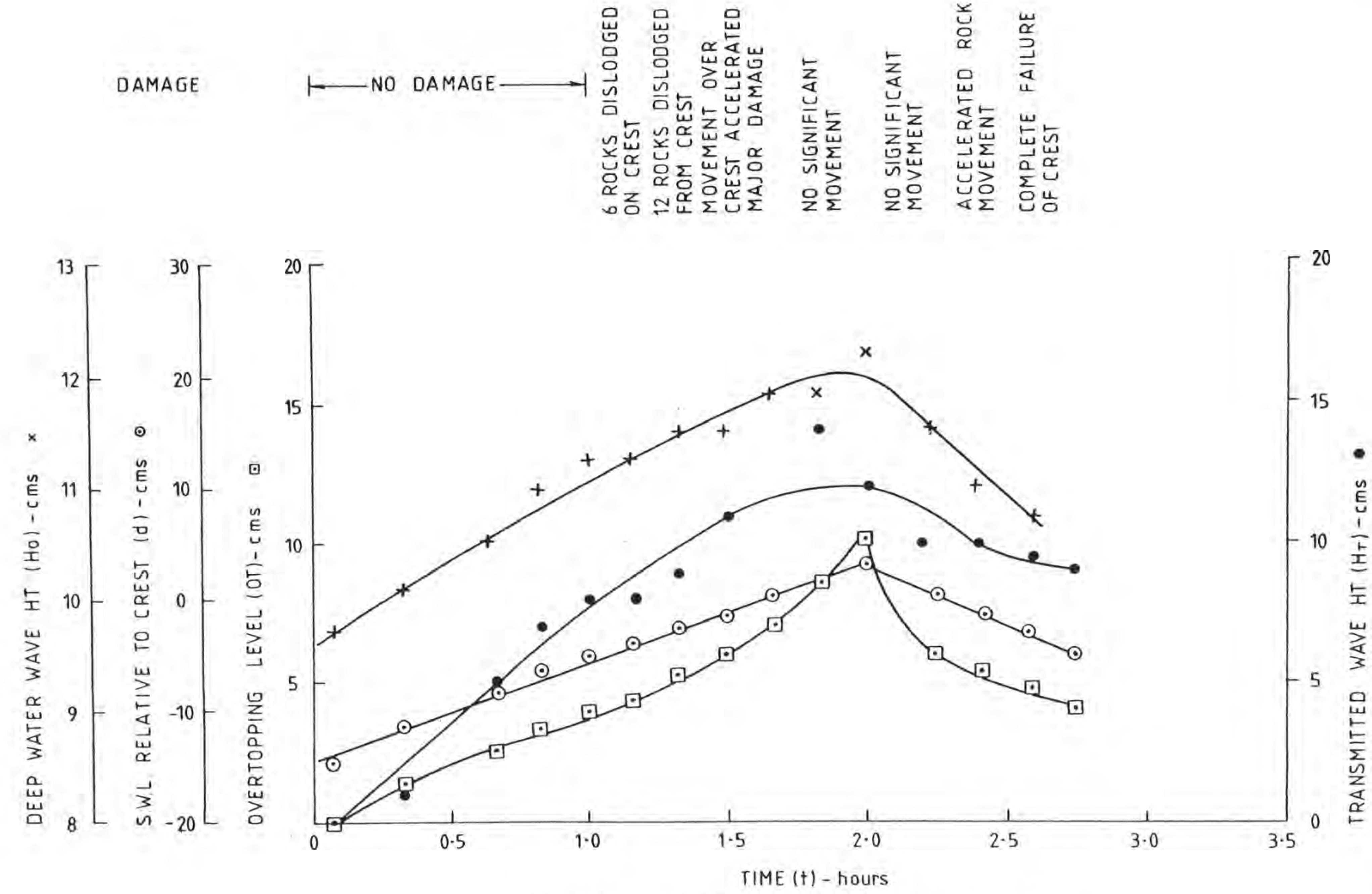
STABILITY TEST - WAVE PERIOD 2·2 sec.

FIGURE 19(f)

FIGURE 19(a)

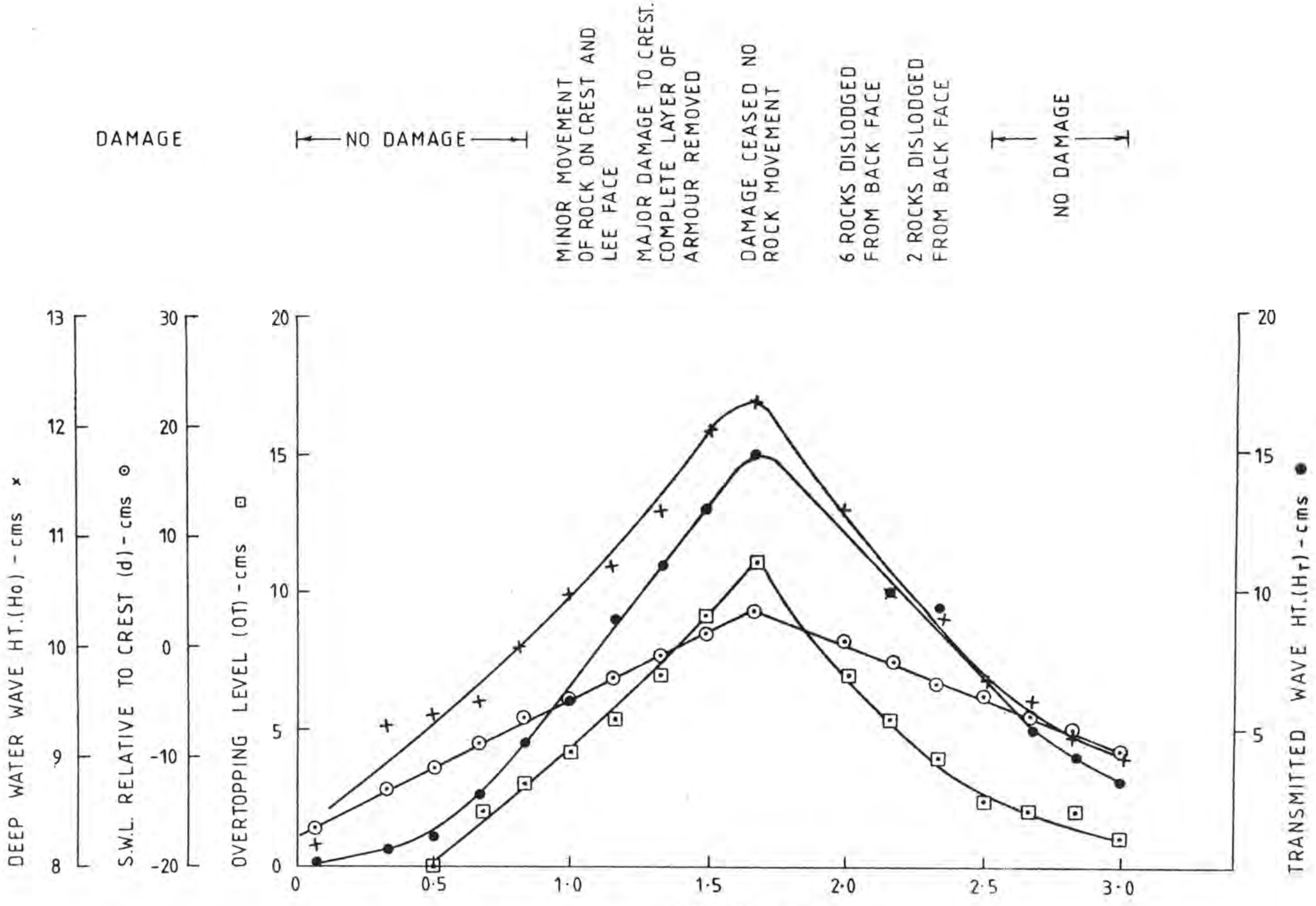


STABILITY TEST - WAVE PERIOD 2·2 sec.



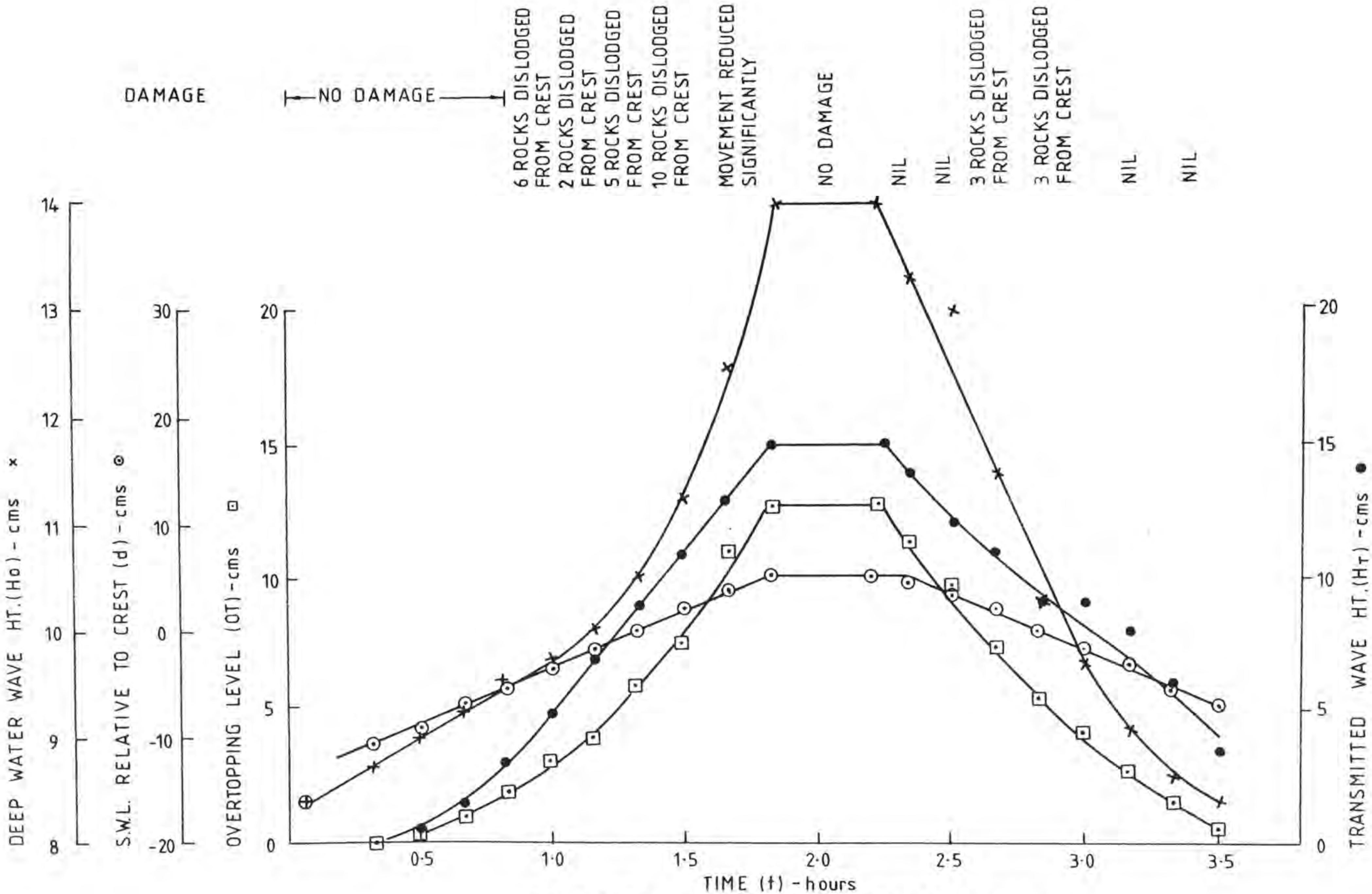
STABILITY TEST - WAVE PERIOD 2·2 sec.

FIGURE 19(h)



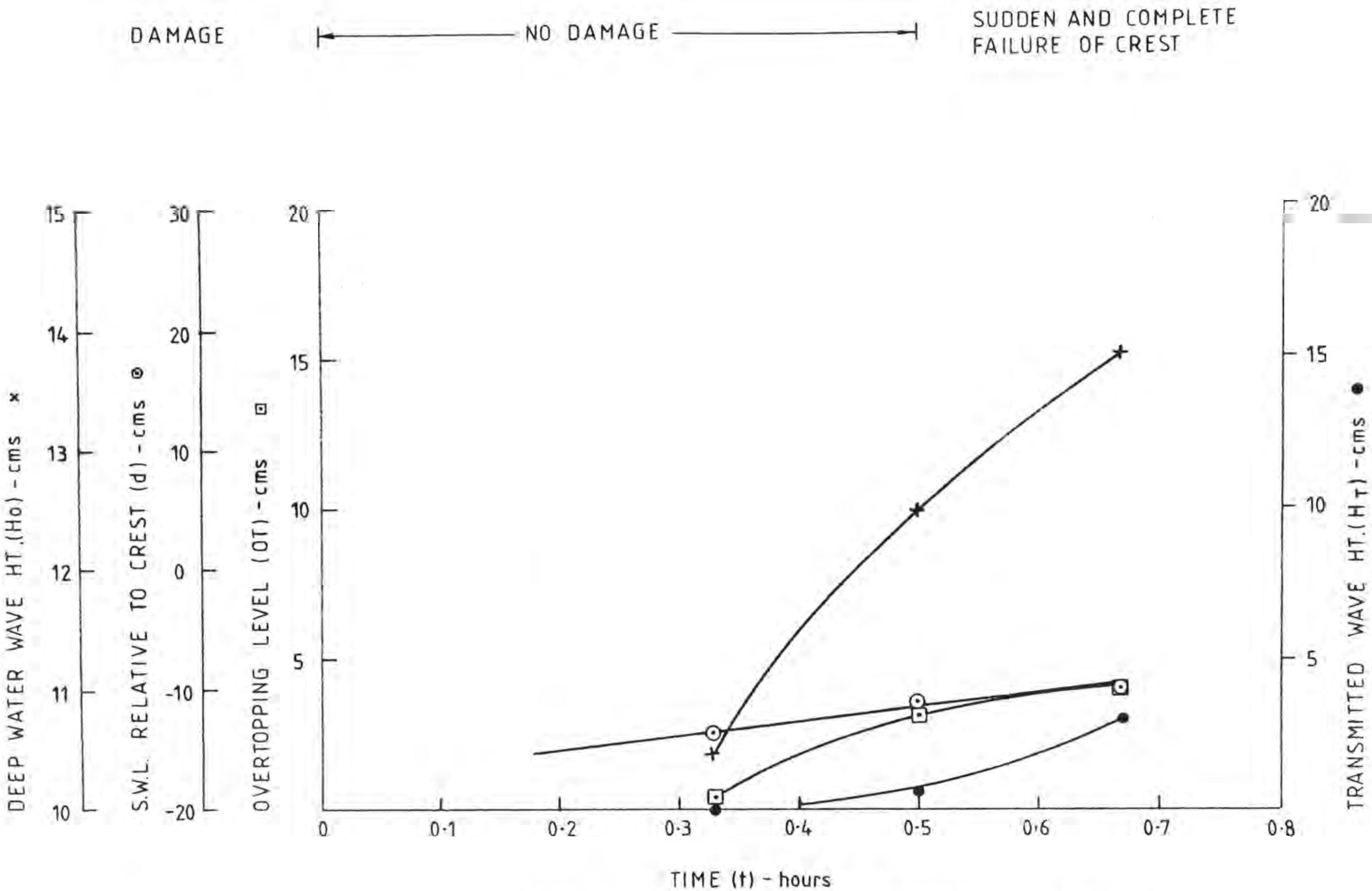
STABILITY TEST - WAVE PERIOD 2·2 sec.

FIGURE 19(i)



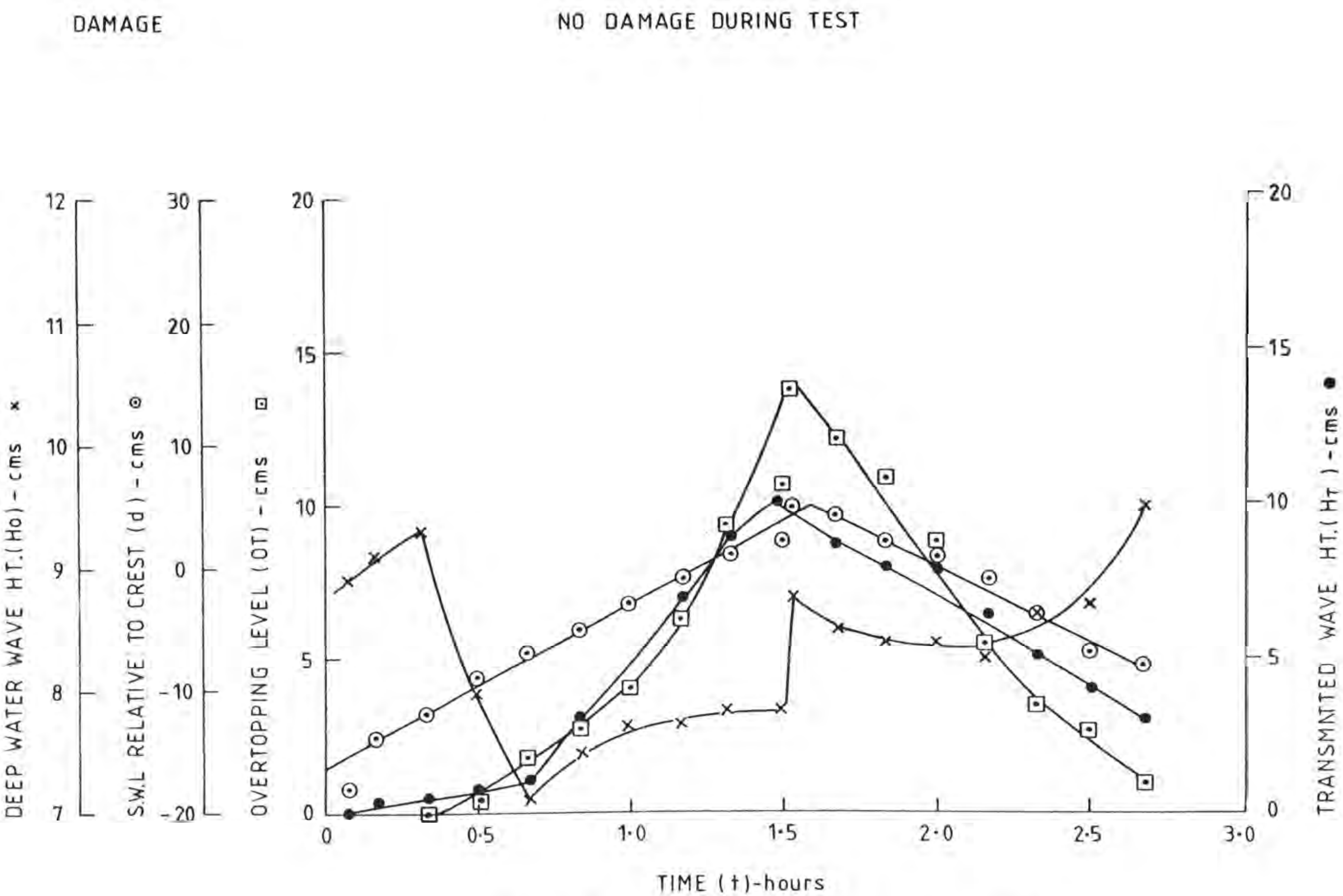
STABILITY TEST - WAVE PERIOD 2·2 sec.

FIGURE 19(i)



STABILITY TEST - WAVE PERIOD 2·69 sec.

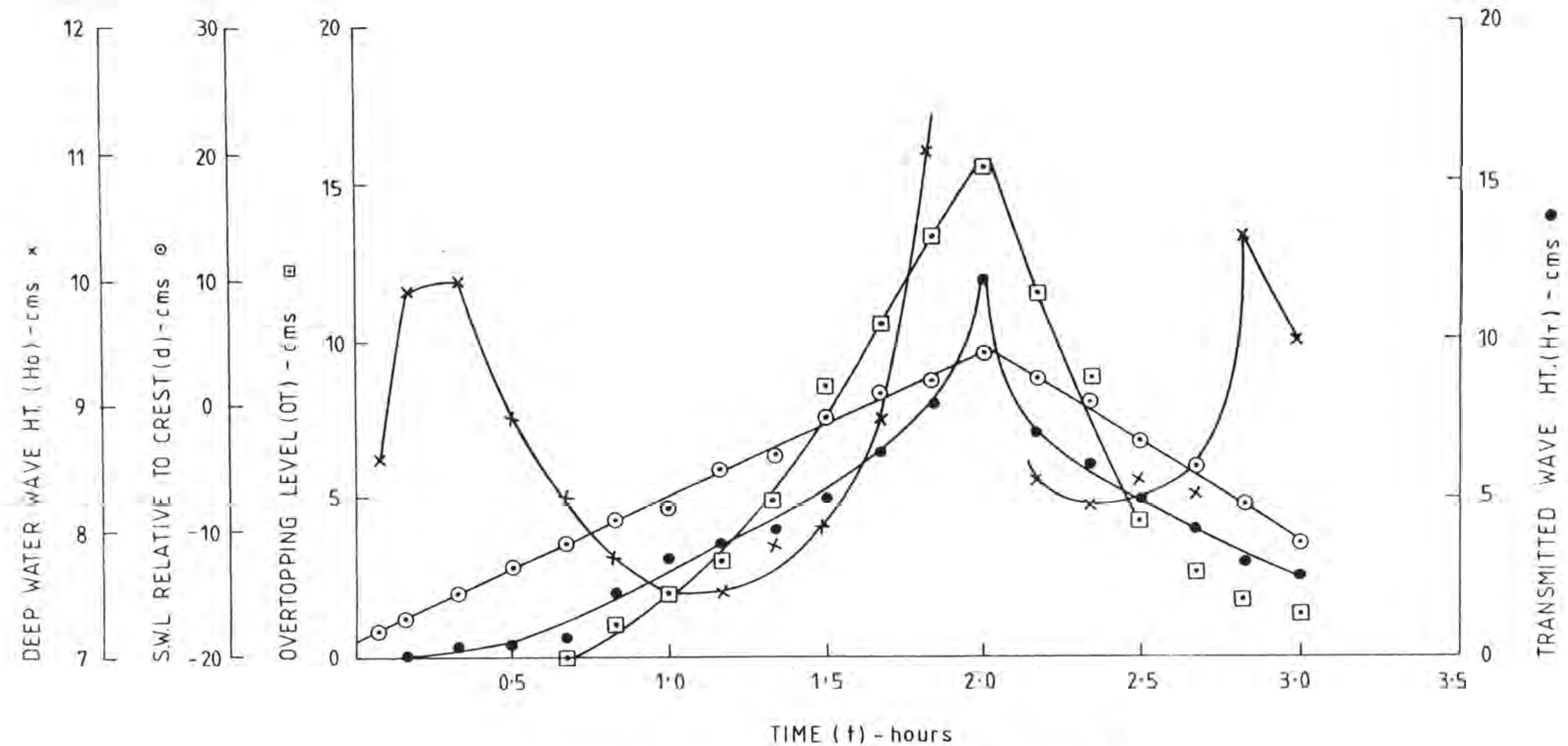
FIGURE 20(a)



STABILITY TEST - WAVE PERIOD 2·69 sec.

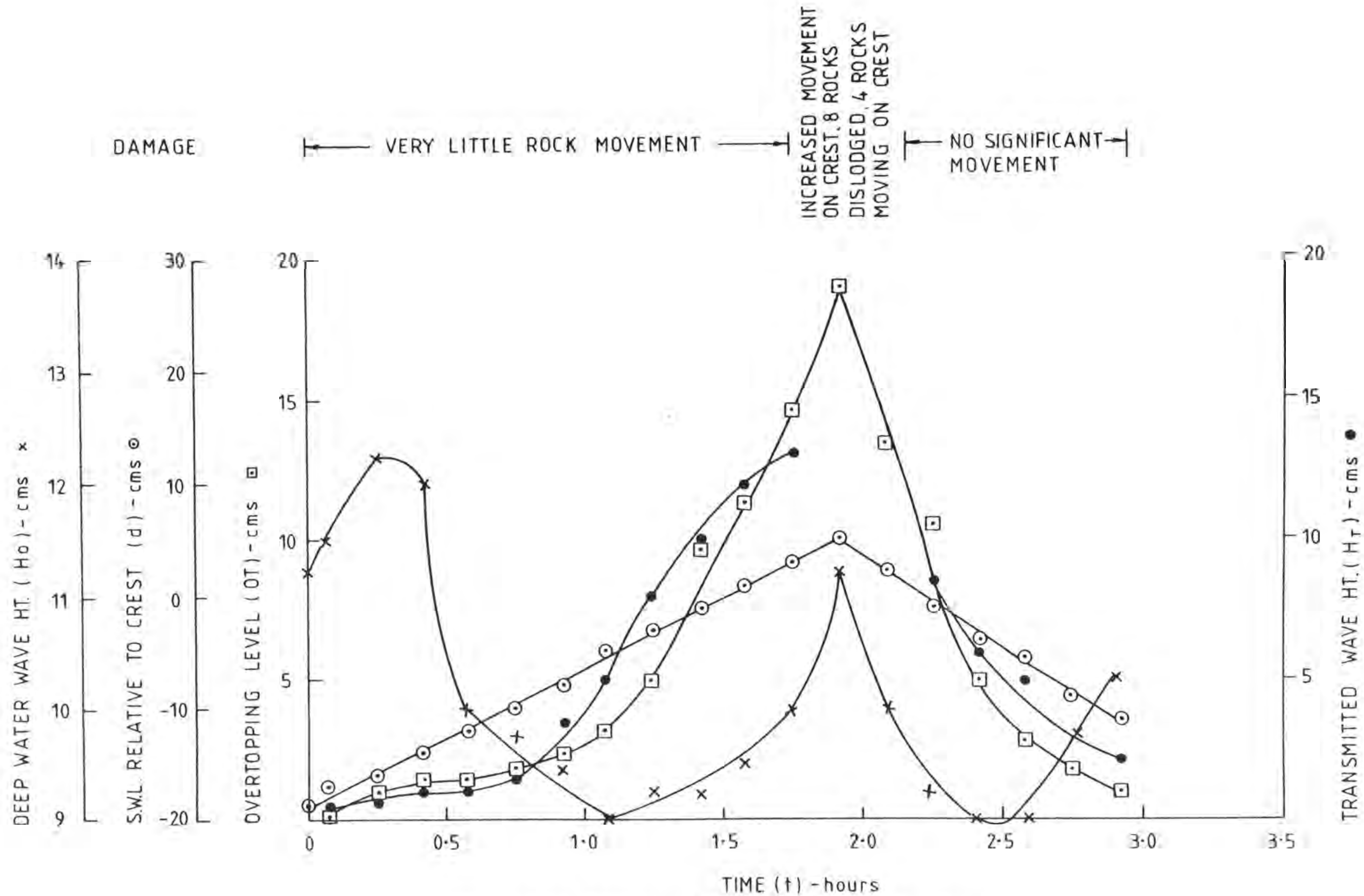
FIGURE 20(b)

DAMAGE NO SIGNIFICANT DAMAGE DURING TEST



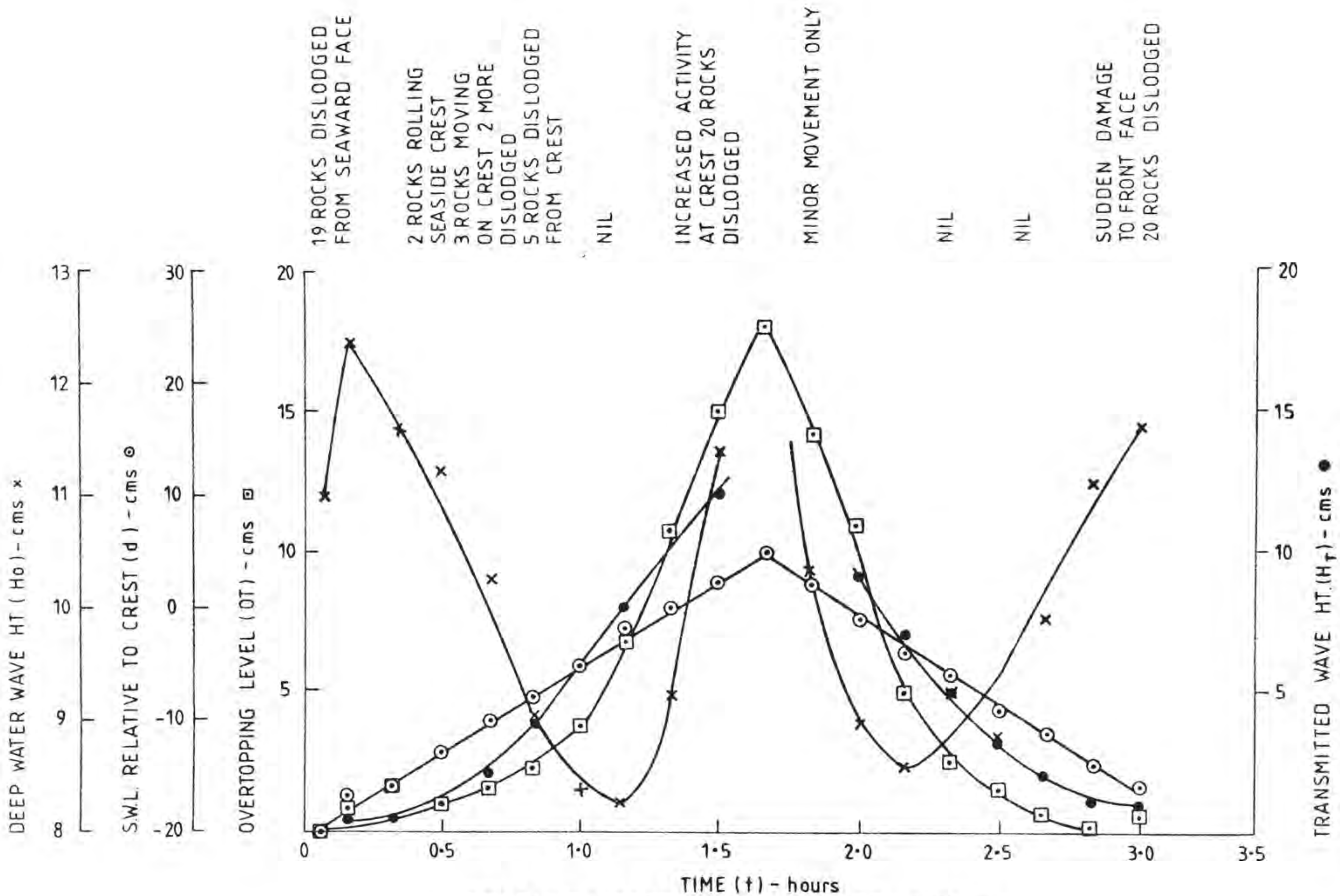
STABILITY TEST - WAVE PERIOD 2·69 sec.

FIGURE 20(c)



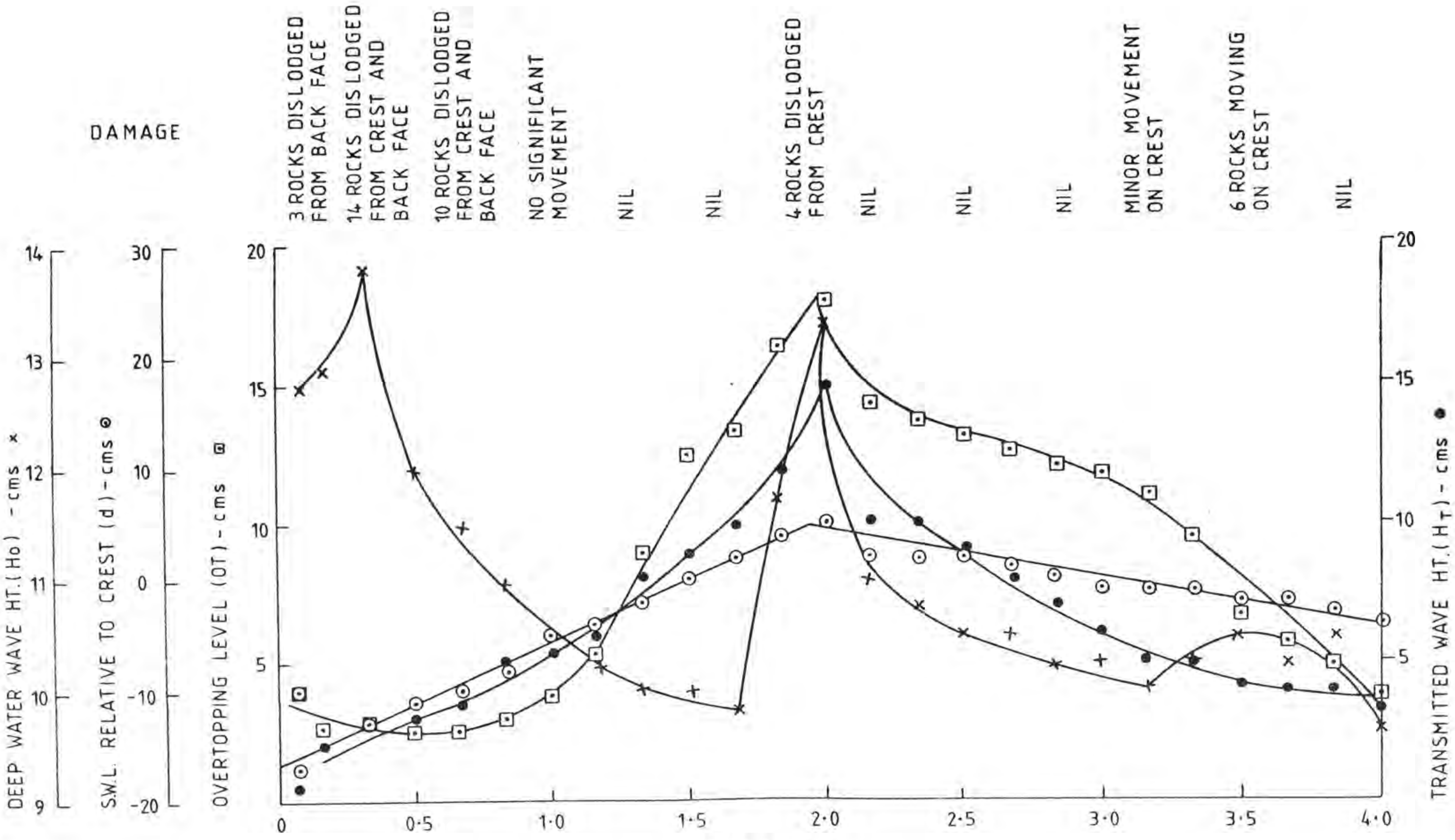
STABILITY TEST - WAVE PERIOD 2·69 sec.

FIGURE 20(d)



STABILITY TEST - WAVE PERIOD 2·69 sec.

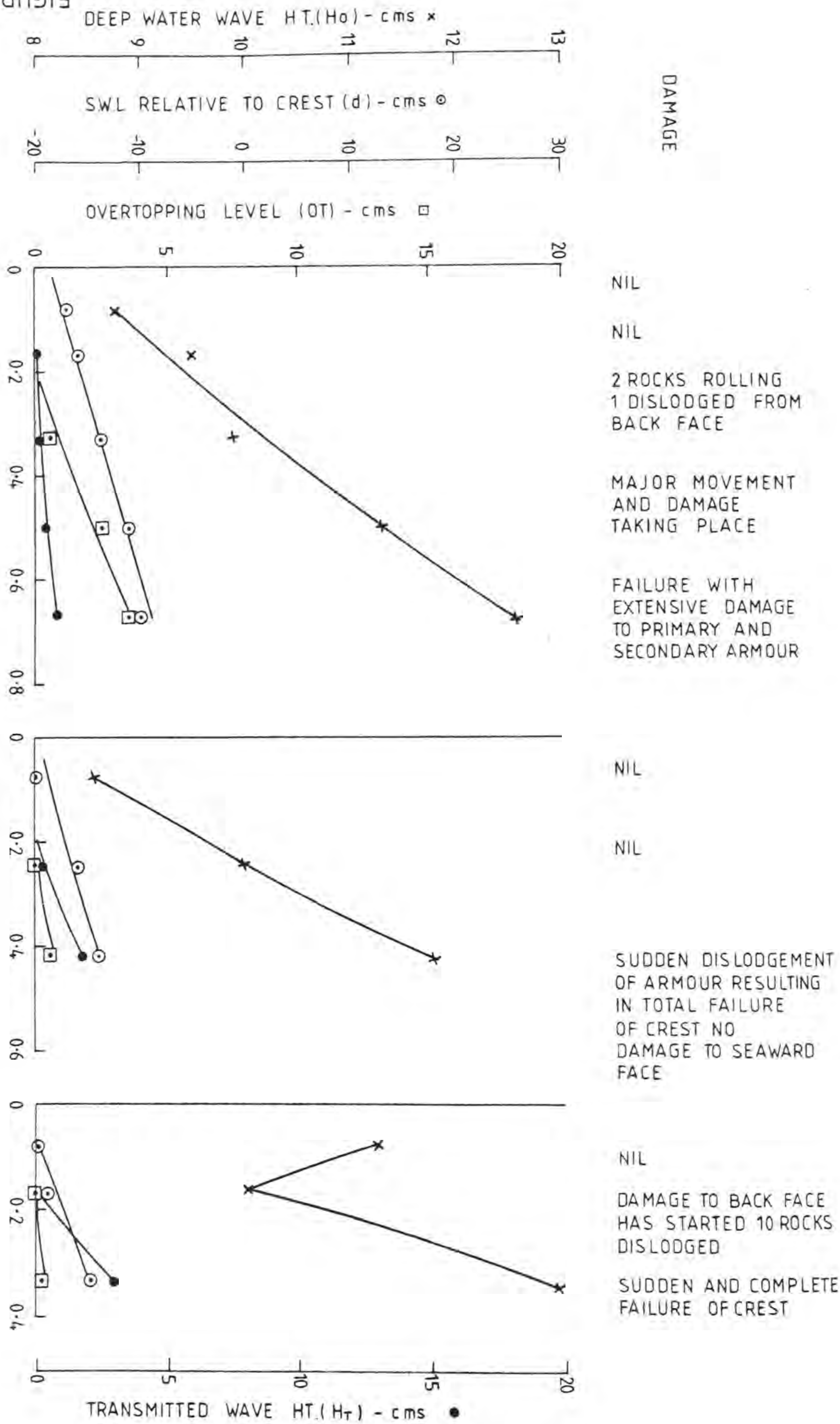
FIGURE 20(e)



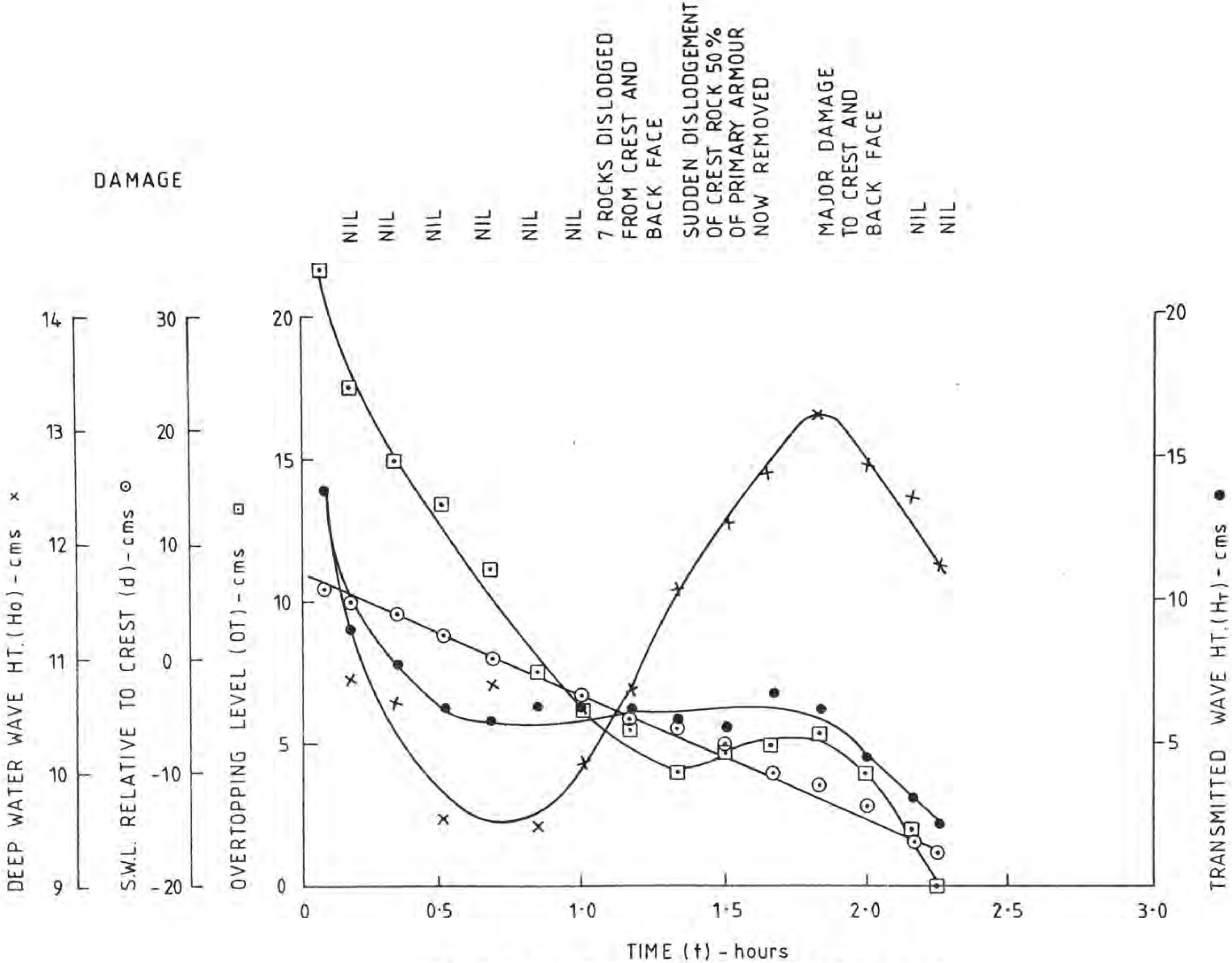
STABILITY TEST - WAVE PERIOD 2·69 sec.

FIGURE 20(f)

FIGURE 20(g)

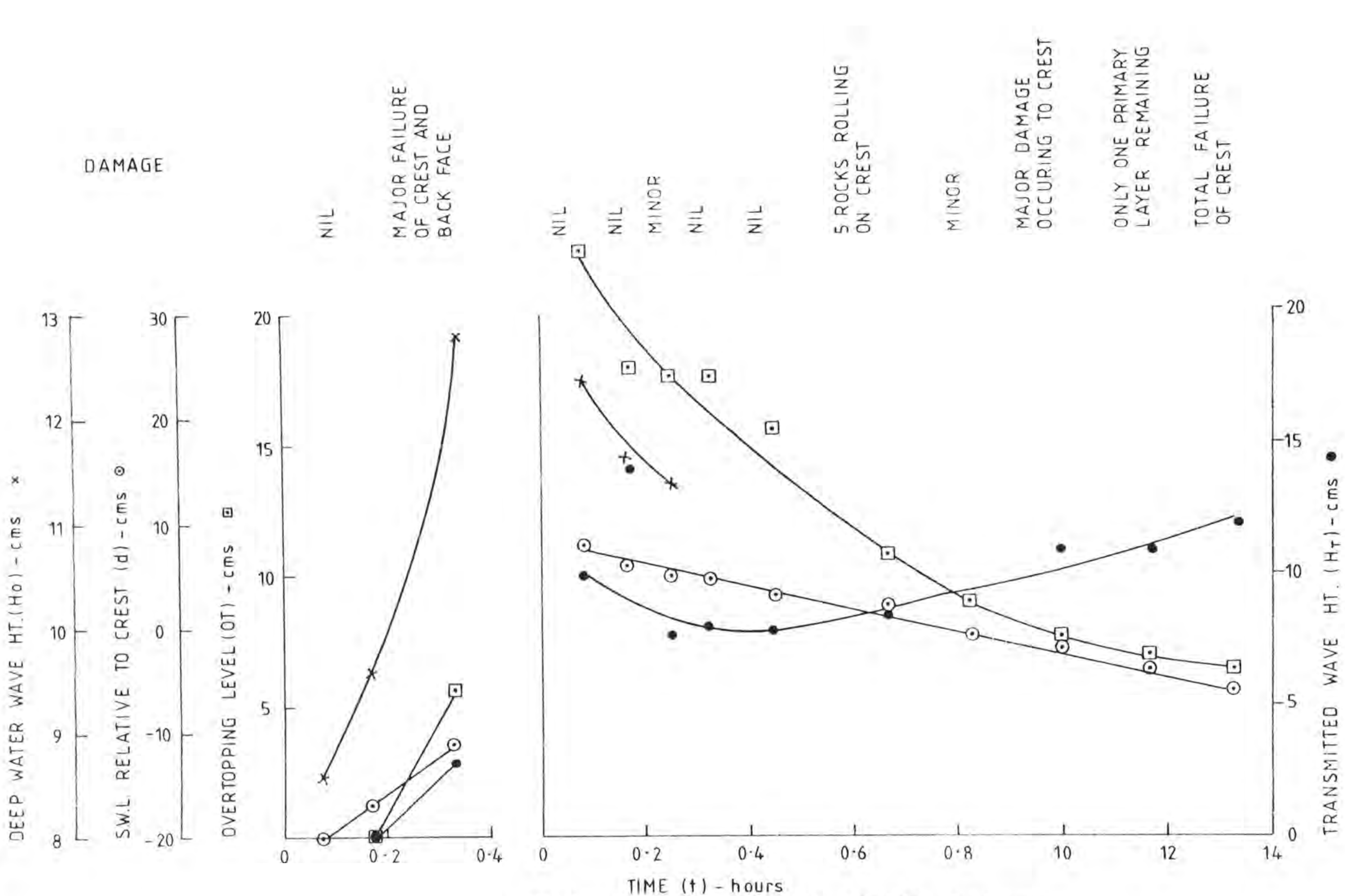


STABILITY TEST - WAVE PERIOD 2·69 sec.



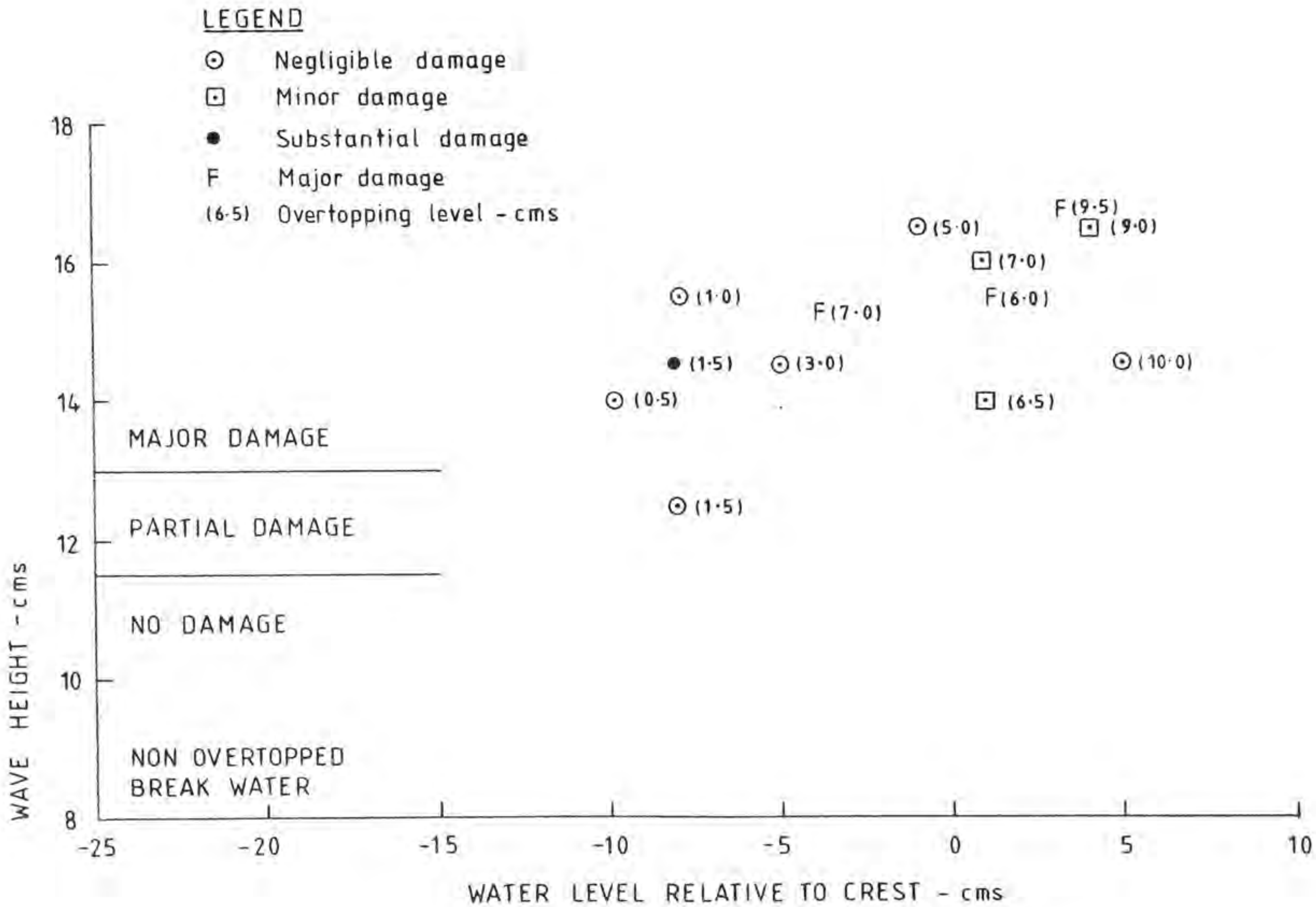
STABILITY TEST - WAVE PERIOD 2·69 sec.

FIGURE 20(h)



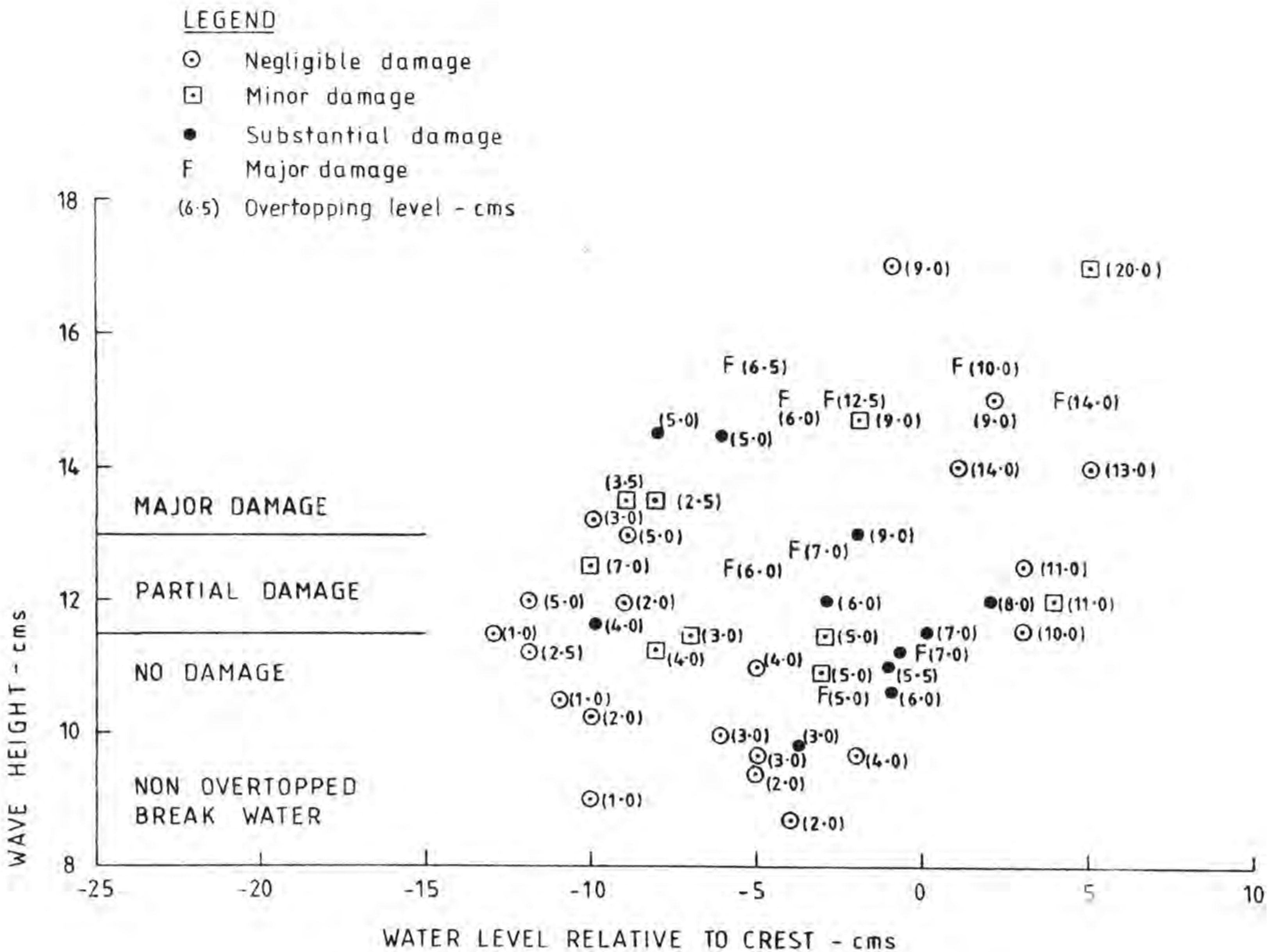
STABILITY TEST - WAVE PERIOD 2·69 sec.

FIGURE 20(i)



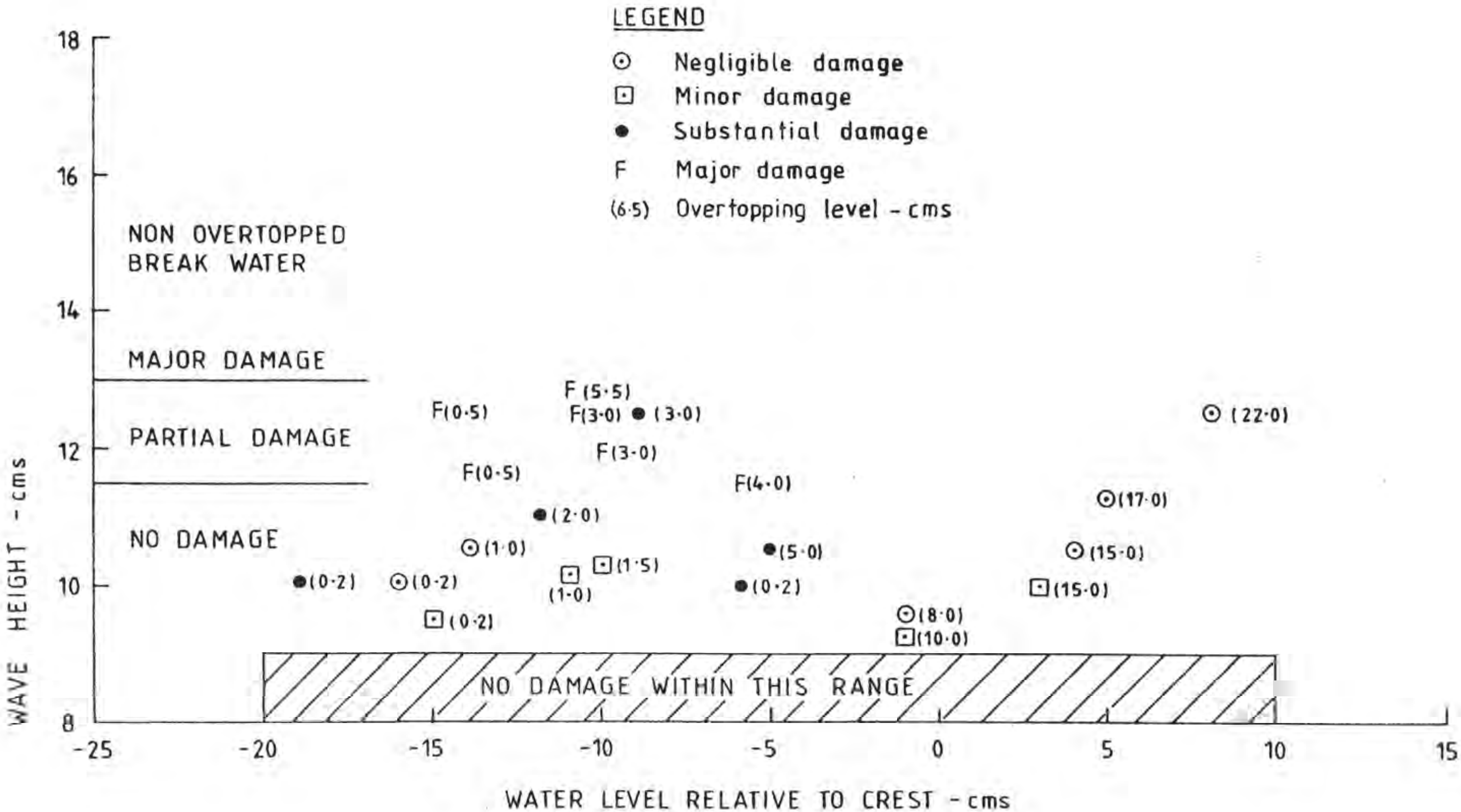
CREST DAMAGE - WAVE PERIOD 1·34 sec.

FIGURE 21



CREST DAMAGE-WAVE PERIOD 2·2 sec.

FIGURE 22



CREST DAMAGE - WAVE PERIOD 2·69sec.

FIGURE 23