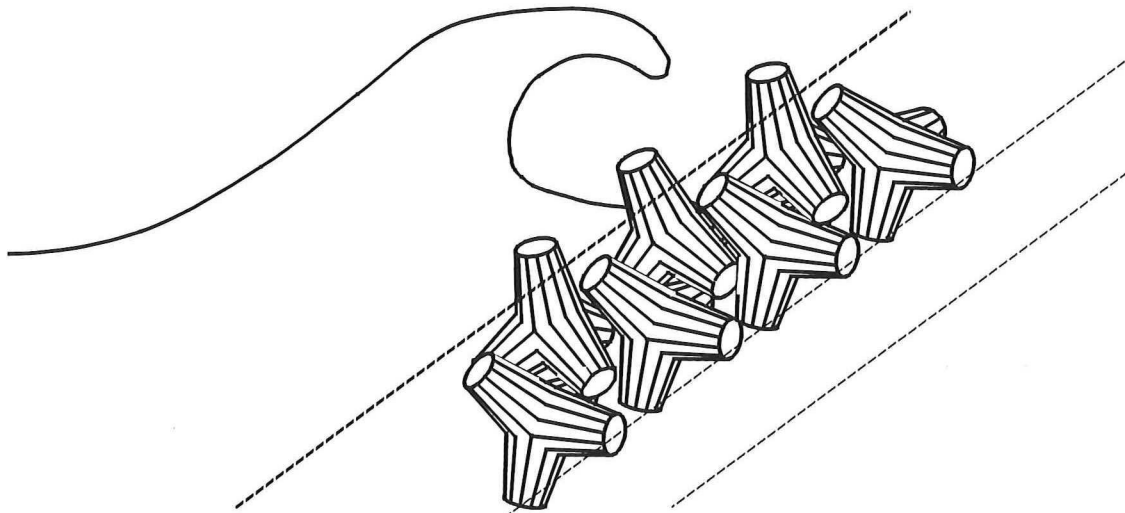


Stresses in tetrapod armour units exposed to wave action

August 1994

C.P. van Nes



Preface

This report is the result of a study, accomplished at Delft University of Technology, Faculty of Civil Engineering in order to get the degree of Master of Science.

I would like to express my gratitude to, first of all, the MAST II project, which made it possible for me to accomplish this study.

I also would like to thank Delft Hydraulics for making it possible to perform the small scale model tests. I have enjoyed working there with the people of the laboratory.

Furthermore, the tests would not have been possible without the help of Aalborg University Center. Therefore, I would like to thank prof. H.F. Burcharth and dr. Z. Liu both for their help.

Finally, or should I say last but certainly not the least, I would like to thank the members of my thesis committee : prof.ir. K. d'Angremond, dr.ir. J.W. Van der Meer and ir. G.J. Schiereck for their advise and support during this study.

Peter van Nes
Delft, August 1994

Executive summary

In the late seventies, begin eighties a number of large breakwaters was severely damaged. The armour layer of these breakwaters consisted of slender concrete armour units, like dolosse or tetrapods. It appeared that one of the main reasons of failure of these breakwaters was breakage of the armour units. Obviously, the mechanical strength of the armour units had been exceeded.

An extensive research program has been set up under the name of "*Rubble mound breakwater failure modes*". This research is part of the European MAST II project, (MARine Science and Technology) in which a number of universities and hydraulic institutes, from various countries in Europe, are participating.

In this study an analysis concerning the static and quasi-static portion of the tensile stresses inside tetrapod armour units is presented. The data has been obtained from a series of small scale model tests. Stresses have been measured using a load-cell technique developed by CERC (Coastal Engineering Research Centre) in association with AUC. (Aalborg University Center)

In general, the stress signal can be divided into three parts. Firstly, a static part, i.e., stresses caused by the weight of the armour units. Secondly, a quasi-static part can be distinguished. Quasi static stresses originates from the motion of the water around the armour units. Thirdly, a dynamic part can be identified caused by the concrete to concrete collisions.

The obtained stress signal has been processed using a preliminary analysis. This analysis was similar to a simple surface water wave analysis, resulting in the maximum value of the quasi-static stress within each stress wave. These maximum values were used in a statistical analysis.

For the tested area of the breakwater only the following parameters appeared to have influence on the stress distribution inside a leg of a tetrapod :

- significant wave height : H_s
- water depth in front of the breakwater : h_{toe}

The parameters investigated which appeared to have no influence on the stress distributions were :

- the fictitious wave steepness, s_{op}
- the location of the tetrapod
- the orientation of the instrumented leg of the tetrapod.

The stress distributions can be described using a Log Normal distribution. The average of these Log Normal distributions increases with increasing wave height. The standard deviation of the distribution decreases with increasing wave height.

However, because large differences between subsequent test runs have been observed under identical conditions, the randomness of the process involved must have large influence on the variation in stress level. As the number of repetitions for each of the combinations of the parameters involved, i.e. H_s , h_{toe} , s_{op} , location and orientation, was rather small, it was not possible to derive trends between all individual variables and the accompanying stress distributions.

This means that reliable conclusions, concerning the influence of the individual parameters on the stress distributions, can only be drawn after performing large number of tests.

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List of variables

a	[m/s ²]	= acceleration of an armour unit
C_1	[-]	= constant used in AUC preliminary analysis
C_2	[-]	= constant used in AUC preliminary analysis
C_3	[-]	= constant used in DUT preliminary analysis
C_4	[-]	= constant used in DUT preliminary analysis
D	[m]	= characteristic diameter of armour unit
D_n	[m]	= nominal diameter of armour unit $(M/\rho_a)^{(1/3)}$
f_s	[Hz]	= sample frequency
f_n	[Hz]	= natural frequency
g	[m/s ²]	= acceleration of gravity
H	[m]	= wave height
H_s	[m]	= significant wave height (highest 1/3 of the waves)
$H_{2\%}$	[m]	= wave height exceeded by 2% of the waves
h_{toe}	[m]	= water depth at toe
K_D	[-]	= damage coefficient
M	[kg]	= mass of armour unit
M_y	[Nmm]	= orthogonal bending moment
M_z	[Nmm]	= orthogonal bending moment
N	[-]	= number of waves
N_{od}	[-]	= damage level
N_{omov}	[-]	= number of movements
$N_{o,imp}$	[-]	= number of impacts
s	[-]	= wave steepness
s_{om}	[-]	= fictitious wave steepness based on T_m
s_{op}	[-]	= fictitious wave steepness bases on T_p
s_p	[-]	= local wave steepness based on T_p
S_T	[N/mm ²]	= maximum tensile stress of concrete
T	[Nmm]	= torque
T_m	[s]	= average wave period
T_p	[s]	= peak period
W	[N]	= weight of an armour unit
W_b	[mm ³]	= modulus of strain gauged cross section
$\cot \alpha$	[-]	= slope angle
Δ	[-]	= relative density of armour units $((\rho_a/\rho_w)-1)$
Δt	[s]	= duration of an impact
γ	[-]	= enhancement factor JONSWAP spectrum
λ_L	[-]	= scaling factor for length
λ_{Ea}	[-]	= scaling factor for elasticity
$\lambda_{\rho a}$	[-]	= scaling factor for mass density of armour units
$\lambda_{\rho w}$	[-]	= scaling factor for mass density of water
λ_σ	[-]	= scaling factor for stresses
ξ	[-]	= breaker parameter
μ_{LN}	[-]	= dimensionless average of Log Normal Fit
ρ_a	[kg/m ³]	= mass density of armour units

ρ_w	[kg/m ³]	= mass density of water
σ	[N/mm ²]	= normal stress
σ_{LN}	[-]	= standard deviation of Log Normal Fit
σ_T	[N/mm ²]	= maximum principal tensile stress
σ_{impact}	[N/mm ²]	= impact stress
σ_{quasi}	[N/mm ²]	= quasi static stress
$\sigma_{\text{quasi, max}}$	[N/mm ²]	= maximum of σ_{quasi} within a period
σ_{local}	[N/mm ²]	= average stress over a few wave periods
σ_{overall}	[N/mm ²]	= average stress over entire test run
σ_{static}	[N/mm ²]	= stress due to weight of tetrapods
τ	[N/mm ²]	= shear stress
ψ	[-]	= shape factor of accelerations

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List of abbreviations

AUC	=	Aalborg University Center
CERC	=	Coastal Engineering Research Centra
CUR	=	Center for civil engineering Research and codes
DH	=	Delft Hydraulics
DUT	=	Delft University of Technology
LN	=	Log Normal Distribution
MAST	=	MARine Science and Technology
SWL	=	Still Water Line

1 Introduction

1.1 General

Generally, breakwaters change coastal phenomena in some way. The most obvious purpose of a breakwater is to provide protection against waves. This protection might be provided for the approach channel or for the harbour itself.

As a result of economic developments, the increasing size of ships, etc., harbours had to be expanded as well in order to maintain adequate facilities. The approach channels were extended seaward. Breakwaters had to be built in deeper water as a result of this development. To withstand the large wave forces acting on the breakwaters in deep water areas, very large armour units have been used on rubble mound breakwaters .

In the late seventies, begin eighties, a number of rubble mound breakwaters was severely damaged. (e.g., Sines, Figure 1.1) The armour layers of these breakwaters consisted of armour units with a slender shape and a high K_D factor (see chapter 2), like dolosse or tetrapods, and were made of unreinforced concrete.

Analysis showed that one of the main reasons of these failures of rubble mound breakwaters are breakage of concrete armour units. The design was actually based on extrapolation of experience from smaller or less exposed breakwaters.

New design methods, which not only take into account the hydraulic stability but the structural stability as well, are therefore needed. Especially for the slender armour unit types which are more vulnerable to breakage than the massive armour units. (Figure 1.2) In this study it is tried to establish a design method for tetrapods which includes the structural stability of these armour units.

The study is performed by C.P. van Nes, student at Delft University of Technology (DUT) and presented here as his Master's thesis, under guidance of prof.ir. K. d'Angremond (DUT), dr.ir. J.W. van der Meer (Delft Hydraulics) and ir. G.J. Schiereck (DUT). In December 1993, the author performed a series of tests at DH to investigate the structural stability of tetrapods.

1.2 Aim of the research

Armour layers are usually designed by means of a preliminary design based on stability formulae (Hudson, 1953; Van der Meer, 1988) followed by a detailed study in a scale model. The strength of these individual units is not to scale in such a model; they are much too strong. Consequently, there is a possibility that the prototype armour blocks are hydraulically stable but structurally unstable as a result of such design methods.

Structural damage to individual blocks can escalate into hydraulic damage of the whole armour layer. (Figure 1.3) It is necessary to design coastal structures that take into

consideration not only the hydraulic stability of the armour units but also the structural strength of the armour units.

Therefore, a design method has to be established which gives a relation not only between the governing wave parameters and the weight of the individual elements but also a relation between these wave parameters and the structural strength of the individual elements. (Figure 1.4)

Two lines of research can be identified on this subject. The first one, the CUR C70 investigation, concentrated on the movements or rocking of the armour units. These movements result into concrete to concrete collisions. A description was derived to calculate the stresses originating from these collisions. (CUR C70, see chapter 2)

The other research concentrates on describing the internal stresses of armour units by measuring stresses directly inside the armour units. Such a design method is currently established for dolosse, performed at Aalborg University Center. (see chapter 2)

1.3 Outline

Firstly, a few of the most important developments in breakwater design will be mentioned. With this in mind the model test program for the determination of stresses in tetrapods will be discussed.

Next, the results of the model tests will be treated. The results are divided into hydraulic stability results and stress related results. The hydraulic stability results will be discussed at first to make sure that the hydraulic stability of the model breakwater agree with earlier investigations

Before, examining the actual stress signals, a description of the signals will be presented in relation to the phenomena of interest. Subsequently, the analysis itself will be described. Finally, the conclusions and recommendations are presented.

2 Developments in breakwater design

2.1 General

Although much research has been done on breakwater design only a few of the developments are mentioned here, i.e. only those that are most relevant to this investigation. First of all the Hudson formula will be discussed, whereafter the stability formulae of Van der Meer will be explained.

Next, the internal stresses will be examined more closely on basis of the CUR C70 investigation. Finally the investigations performed at AUC in association with CERC (Aalborg University Center and Coastal Engineering Research Centre respectively) will conclude this short overall view on the developments in breakwater design.

2.2 Hudson

One of the best known stability formula is the formula of Hudson (1959). Hudson developed an empirical formula, based on a series of experiments, for the calculation of the weight of armour units on a rubble mound breakwater :

$$W = \frac{\rho_a g H^3}{K_D \Delta^3 \cot \alpha} \quad \text{or} \quad \frac{H}{\Delta D} = \sqrt[3]{K_D \cot \alpha} \quad (2.1)$$

where W = weight of armour units [N]
 ρ_a = mass density of armour units [kg/m³]
g = acceleration of gravity [m/s²]
H = wave height [m]
 K_D = damage coefficient [-]
 Δ = relative density of armour units [-]
D = diameter of the armour units [m]
 $\cot \alpha$ = slope angle [-]

Firstly, only a rock slope has been used in the tests. Nevertheless, using a different value for K_D (Shore Protection Manual, 1984) it was possible to calculate a minimum weight for other types of armour units. For the various types of armour units, the corresponding values of K_D have been determined from additional model tests performed by CERC.

Secondly, the Hudson formula was derived on the basis of tests using monochromatic waves. It does not take into account the wave period, the breaking of waves and the storm duration.

2.3 Van der Meer

New stability formulae have been developed by Van der Meer (1988). An extensive series of hydraulic model tests has been performed, testing different kinds of armour units, which did include the above mentioned parameters. His empirical formula for tetrapods on a rubble mound breakwater yields (Van der Meer, 1987; valid only for $\cot \alpha = 1.5$):

$$\frac{H_s}{\Delta D_n} = (3.75 \frac{N_{od}^{0.5}}{N^{0.25}} + 0.85) s_m^{-0.2} \quad (2.2)$$

where H_s = significant wave height [m]
 Δ = relative mass density of armour units [-]
 D_n = nominal diameter of armour units [m]
 N_{od} = damage level [-]
 N = number of waves [-]
 s_m = wave steepness [-]

For different storm durations, i.e. number of waves, the stability number ($H_s/\Delta D_n$) vs. the wave steepness, s_m , yields typical plots like Figure 2.1, indicating that the storm duration, the wave steepness and the level of damage are important as well for the calculation of the hydraulic stability of a breakwater. The damage level varies from $N_{od} = 0$ (start of damage) up to $N_{od} = 1.5$ (severe damage)

Although the Van der Meer formula takes into account more parameters than the Hudson formula, the strength of concrete armour units has not been included.

2.4 CUR C70 research

2.4.1 General

A group of research institutes, consultants and contractors in The Netherlands initiated a program to investigate systematically the problem of structural behaviour and to develop procedures for design and construction of larger size concrete armour units which do include these structural parameters. This work was carried out under the auspices of CUR. (Centre for civil engineering research and codes)

This CUR C70 investigation defined two areas on which improved knowledge was most urgently required:

- the relation of hydraulic and geometrical conditions with rocking behaviour of armour units
- the load-time relation of colliding units, i.e. the development of the stress in time as a result of a concrete to concrete collision.

In addition, the possibilities to optimize the block strength, like adding steel bars or fibres, has been considered further. The first two items will be discussed below briefly.

2.4.2 Loads due to rocking

If the Van der Meer stability formula (equation. 2.2) is modified using the number of moving units, N_{omov} , instead of the number of displaced units, N_{od} , the following equation evolves: (CUR C70, 1989)

$$\frac{H_s}{\Delta D_n} = (3.75 \frac{N_{omov}^{0.5}}{N^{0.25}} + 0.85) s_m^{-0.2} - 0.5 \quad (2.3)$$

The number of moving units was recorded by single frame technique. The single frame technique employs a wave gauge mounted along a breakwater slope, which sends a pulse to a camera each time the water level passes downward a selected level. (close to the rundown point) By projecting these single frames one after another, the movements and the frequency of movement can be observed.

An extensive analysis showed that about 40% of the rocking units moved only once. The other 60% moved about 4 times (on average) which gives that the number of impacts is about three times the number of moving (rocking) units. If it is assumed that units displaced out of the layer cause also about three impacts, the total number of impacts will then be about three times the number of moved units: (Van der Meer and Heydra, 1990)

$$N_{o,imp} = 3 N_{omov} \quad (2.4)$$

where $N_{o,imp}$ = number of impacts
 N_{omov} = number of moved units

Although most of the *rocking* was concentrated around the SWL, the distribution of the *impacts* was considered to be more or less uniform from SWL to the toe of the structure. Furthermore, it was concluded that the upper level of movement was $1 \cdot H_s$ above SWL and thought to be linearly decreasing from SWL to this $1 \cdot H_s$ above SWL. (Figure 2.2)

The impact momentum originating from collisions of rocking units can be described using the following parameters :

- acceleration	a	[m/s ²]
- duration of impact	Δt	[s]
- development of acceleration in time	ψ	[-] (shape factor)
- mass	M	[kg]

It is possible to determine this momentum, $M \cdot \int a \, dt$, in a hydraulic scale model. A few of the small scale armour units were instrumented with an accelerometer placed in the centre of the unit. The accelerations of the armour units, due to wave action, were determined showing typical plots like Figure 2.3.

The measured acceleration signal itself could not be scaled correctly because of the inability to scale the material characteristics correctly. In this way an elasto-plastic collision in prototype will be observed as an elastic collision in the scale model. (CUR C70, 1989) However, the *integrated* signal of the accelerations, i.e. the impact *velocity*,

could be scaled to prototype, using Froude, i.e., $\lambda_v = \lambda_L^{0.5}$. (CUR C70, 1990) Figure 2.4 gives a graphical presentation of the distribution of these impacts velocities along the breakwater slope.

2.4.3 The load-time relation

An impact between two neighbouring concrete bodies may occur as a result of rocking which is in its turn caused by the wave action in front of the breakwater. Van Mier and Lenos (1991) have investigated a way to describe the load-time history caused by concrete to concrete collisions. It was found that these collisions could be described using an elasto-plastic model. Each collision can be divided into three different stages : (see Figure 2.5)

- a) stage one : has got a constant loading rate, c_1 .
- b) stage two : shows a constant load or slightly increasing or decreasing load, c_2 . (the plastic behaviour)
- c) stage three : the last stage is characterized by a constant unloading or restitution rate, c_3 .

The value of the loading rate, c_1 , depends on the elastic properties of the concrete used as well as on the geometry of the contact surface. When the cracking and crushing of the concrete becomes dominant, a plastic response can be observed. The loading rate, c_2 , depends on the material behaviour :

- $c_2 = 0$ when a stable elastic/plastic stress distribution is maintained
- $c_2 > 0$ for a hardening material
- $c_2 < 0$ for a softening material

In the third and last stage, elastic restitution occurs. This value c_3 is not necessarily equal to c_1 .

2.4.4 Result of CUR C70

In the final phase of the research practical applications on the behaviour of concrete armour units have been developed. The design procedure, which resulted from the integration of the study results, i.e., :

- displacements, movements and impacts of armour units.
- impact velocities
- impact behaviour
- strength model

has been incorporated in the computer program, "ROCKING". For the probabilistic part of the calculation a Monte Carlo simulation has been used. The program calculates the number of broken units for a given combination of environmental conditions, armour unit characteristics and material properties. (CUR C70, 1990)

Furthermore, the program "ROCKING" has been applied to evaluate various measures to reduce the chance of breakage :

- the use of reinforced concrete
- the reduction of residual stresses
- modification of the impact surface to reduce impact forces

2.4.5 Comments on CUR C70

The accelerations are measured in the centre of the unit. It is difficult to find out if the accelerations are caused by translations, rotations or a combination.

It is not possible to determine static stresses (stresses caused by the weight of the armour units) and quasi-static stresses (stresses caused by the motion of the water assumed as a additional static stress) The assumption was that impact stresses would be the main cause for breakage of armour units.

2.5 Research at Aalborg University Center

2.5.1 General

Whereas the CUR C70 group concentrates on impacts due to rocking, AUC investigated the strength of concrete armour units by directly measuring stresses in small scale models, not only measuring the loads or stresses caused by impacts but also the static and quasi-static stresses as well.

A way of determining these total tensile stresses of an armour unit is to use strain gauges mounted directly on the concrete surface or on steel bars cast in the concrete close to the surface where the largest stress occur, i.e., in the critical sections.

2.5.2 Description of method

AUC has inserted a load cell in one of the two shank-fluke sections of a number of dolosse to investigate the stress distribution of dolosse. The dolos has been chosen because it was one of the types of armour units commonly used on rubble mound breakwaters.

Due to its slender form the dolos is vulnerable to breakage because the limited cross section can cause relatively large tensile stresses. Of all the component forces and moments, the two orthogonal bending moments, M_y and M_z , and torque, T , around the axial axis appear to be dominant. (Burcharth, 1991)

Beam theory has been used to calculate the maximum principal tensile stress at the surface, σ_T , using the cross sectional components moments as follows : (Figure 2.6)

$$\sigma_T = \frac{\sigma}{2} + \sqrt{\left(\frac{\sigma}{2}\right)^2 + \tau^2} \quad (2.5)$$

where σ_T = maximum principal tensile stress [N/mm²]
 σ = normal stress [N/mm²]
 τ = shear stress [N/mm²]

$$\sigma = \frac{\sqrt{M_y^2 + M_z^2}}{W_b} \quad \tau = \frac{T}{W_b/2} \quad (2.6)$$

where M_y = orthogonal bending moment [Nmm]
 M_z = orthogonal bending moment [Nmm]
 T = torque [Nmm]
 W_b = modulus of strain gauged cross section [mm³]

Failure is taken as the appearance of the first crack at the surface, i.e.

$$\sigma_T \geq S_T \quad (2.7)$$

where S_T = maximum tensile strength [N/mm²]

2.5.3 Scaling of stresses

The converted tensile stresses were separated into two components, i.e., the static and quasi-static stresses on one side and the dynamic stresses (impact stresses) on the other. (Figure 2.7)

The static and quasi-static stress contributions were converted into a range of prototype dolos sizes using the linear scaling law, given in equation 2.8. (Burcharth, 1992)

$$\lambda_{\sigma_{static}} = \lambda_{\sigma_{quasi-static}} = \lambda_{\rho_w} \cdot \lambda_L \quad (2.8)$$

where $\lambda_{\sigma_{static}}$ = scaling factor for the static stress [-]
 $\lambda_{\sigma_{quasi-static}}$ = scaling factor for the quasi-static stress [-]
 λ_{ρ_w} = scaling factor for mass density of water [-]
 λ_L = scaling factor for length [-]

The impact stress contributions were converted into the same prototype ranges using a non-linear scaling law for colliding solid bodies instead, see equation 2.9. (Burcharth, 1992) (linear with the square root of the length scale)

$$\lambda_{\sigma_{impact}} = (\lambda_{\rho_a} \cdot \lambda_{E_a} \cdot \lambda_L)^{0.5} \quad (2.9)$$

where $\lambda_{\sigma_{impact}}$ = scaling factor for the impact stress [-]
 λ_{E_a} = scaling factor for elasticity [-]
 λ_{ρ_a} = scaling factor for mass density armour units [-]

The relative importance of static, pulsating and impact stresses depends on the type and size of the units, the slope angle, the position on the slope and the wave characteristics. The dolos stresses were treated as an extreme value problem. No distinction with respect to the dolos position on the slope was made because in practice the same type of units will be used over the whole height of the slope.

2.5.4 Comments on direct method

The calculations of the maximum principle tensile stresses from component forces in the instrumented section are based on the assumption of linear stress distributions. However, it is known that the distributions generally are non-linear. Moreover, the assumption of linear stress distribution related to bending moments and torque is not valid because concrete is an elasto-plastic material. (Burcharth et al, 1991)

The load-time relation of the colliding concrete bodies (Figure 2.3) has not been taken into account. Due to the plastic behaviour of the prototype concrete armour units, the acting forces, caused by the collisions, will be smaller when compared to an elastic collision.

One of the problems to overcome when applying the load cell technique is that impact response is not reproduced to scale because the presence of the load cell makes the dynamic material properties, e.g. the modulus of elasticity, different from those of the monolithic prototype unit.

3 Model test set-up

3.1 General

In this chapter the set-up of the model tests will be described. Firstly, the structural parameters are given in section 3.2, followed by the environmental parameters in section 3.3. Only the main structural parameters will be discussed below. Next, in section 3.4 the parameters related to the stress measuring part of the model tests will be regarded.

In section 3.5 an overview of all the governing parameters is given with their values and ranges that will be used in the test series. Finally, in section 3.6 the test program is described, making a distinction between the performed investigations :

- The stress measuring part in line with the AUC investigation
- The additional tests purely to look at the stability of the breakwater
- The CUR C70 related investigation.

3.2 Structural parameters

Armour units

The mass of armour units used in the tests was 0.290 kg. These monolithic tetrapods were readily available at Delft Hydraulics painted in different colours. Six different colours were used in order to identify the number of displaced units more easily. The tetrapods were placed in colour straps with a height of two tetrapods each.

Mass density

The material used for these tetrapods is mortar with a specific density of $\rho_a = 2307 \text{ kg/m}^3$.

Slope angle

Most of the prototype structures are made using a slope of 1:1.5. Therefore, this slope angle was chosen in these hydraulic model tests as well.

Foreshore

The foreshore used was 1:50.

A graphical presentation of the model breakwater is given in Figure 3.1. The tests have been conducted in the 1.0 m. wide, 1.2 m. deep and 50 m. long Scheldt flume at Delft Hydraulics. An overview of the set up is given in Figure 3.2.

3.3 Environmental parameters

Wave height

At this stage a *test series* was defined. A test series consisted of 4 *test runs* where each subsequent *test run* has a larger value for the significant wave height, H_s . These values, at the wave board, were 0.10, 0.15, 0.20 and 0.25 m. respectively. The water elevation was measured using resistance type wave gauges near the wave board. (Figure 3.2) The significant wave height, H_s , was defined as the average wave height of the highest 1/3 of

the waves.

The reflected waves travelling back from the breakwater were determined using two wave gauges. In this way the incident and reflected spectrum were determined. From the incident wave spectrum the incoming significant wave height, H_s , was determined.

Before starting with the actual tests, a relation between H_s at the wave board and H_s at the location of toe of the structure, without the structure present in the flume, was derived. (Figure 3.3) During the tests the wave height was only measured near the wave board. The actual significant wave height, H_s , at the toe of the breakwater was determined afterwards using the above mentioned relation.

Wave steepness

For the wave steepness at the wave board, two fixed values are chosen, i.e. $s_{op} = 0.02$ en 0.04 . Using the above stated values for the significant wave height, H_s , it is possible to calculate the accompanying peak period, T_p of the wave spectrum. When the energy density spectrum is given as well it is possible to calculate the average wave period, T_m , from the known peak period, T_p ,

Number of waves

The number of waves within each test run is set at 200. Although 200 is a rather small number it is chosen because the largest movement is generally thought to take place during the attack of the first 100-200 waves, which also creates the largest stresses. (Burcharth, 1993) Furthermore, the storage capacity has its limits on the length of an individual test run as will be explained further on.

Water depth

The last environmental parameter is the water depth in front of the structure. Two water depths were used, i.e. $h_{toe} = 0.30$ m and $h_{toe} = 0.50$ m. The first water depth represents a depth limited situation, causing waves to break on the sloping foreshore. (Table A.1, Appendix A) With the second water depth the waves became somewhat higher due to shoaling. (Table A.2, Appendix A)

3.4 Stresses inside tetrapods

3.4.1 Parameters

Next to the structural and environmental parameters, the location of a tetrapod along the slope was introduced as a governing parameter. In Figure 3.4 the specific locations that were investigated are shown.

Furthermore, at each location two different orientations of the instrumented leg were tested as well. In Figure 3.5 the two possible orientations of the armour units in both the top layer and bottom layer are shown. In the possibilities a and c the instrumented leg points in a downward direction along the slope of the breakwater and in the possibilities b and d the instrumented leg points in a direction perpendicular to the slope of the breakwater.

3.4.2 Phenomena of interest

Based on research and experience the following characteristics should be identified within the signal :

- a static component due to its own weight
- a quasi-static component due to drag and lift and inertia of the moving water
- a dynamic component as a result of concrete to concrete collisions

Static and quasi-static component

The quasi-static component has more or less the same period compared to the average wave period of the wave field acting on the breakwater. As the scaling laws for the static stress are valid for the quasi-static stresses as well, they will be treated in the same way.

Dynamic component

The dynamic component of the stress signal has a period that is much smaller compared to the period of the static and quasi-static stress fluctuations, implying different scaling laws, see section 2.5. Moreover, the magnitude of these impacts is of a different order than the quasi-static fluctuations. Calibrations tests have shown that the natural frequency of the impacts applied on the instrumented leg of a tetrapod was approximately 800 Hz. The calibration of the instrumented tetrapods has been performed at AUC. (MAST II, 1994)

3.4.3 Measurement of moments

The same small strain gauged load cells as used at AUC (see chapter 2) were used in this research. However, as torque is believed not to play any significant role in the stress distribution inside a tetrapod, only the two bending moments have been used. The load cell was placed in one of the four critical sections of a tetrapod, Figure 3.6. In total, five of these instrumented tetrapods were used.

From these two bending moments the maximal principal tensile stress can be calculated. In line with the AUC research, there is chosen to use beam theory for calculation of this maximum principal stress, σ_T , in a tetrapod leg.

$$\sigma_T = \frac{\sqrt{M_y^2 + M_z^2}}{W_b} \quad (3.1)$$

However, axial forces and shear forces play an important role as well. The error that originates from neglecting these forces has to be accounted for afterwards. From this point on, beam theory is used for further calculations.

3.4.4 Frequency

In general 20 points per period should be enough to adequately describe a periodical phenomenon. In this case, this would lead to a sample frequency of $800 \cdot 20 = 16000$ Hz per channel. With five instrumented tetrapods on the breakwater slope and bearing in mind the limited storage capacity of a computer this would lead to a very short test duration.

Setting the sample frequency per channel at 6000 Hz the data generation is reduced to such

a degree that it is possible to perform a test run with a length of 200 waves. The error originating from this reduction in sample frequency from 16000 to 6000 is approximately 4% and has to be accounted for afterwards by separating the impact portion from the stress signal and multiplying this impact portion with a factor of 1.04.

3.5 Overview

An overview of the governing parameters and their ranges or values used in the small scale model tests is given below in table 3.1.

variable	notation		range/value	remarks
mass	M	[kg]	0.290	tetrapods
mass density	ρ_a	[kg/m ³]	2307	
slope angle	$\cot \alpha$	[-]	1.5	
water depth	h_{toe}	[m]	0.30 and 0.50	at toe
wave height	H_s	[m]	0.10 - 0.25	irregular, JONSWAP
wave period	T_p	[s]	1.3 - 2.8	peak period
wave steepness	s_p	[-]	0.02 and 0.04	near the wave board
number of waves	N	[-]	200	
location of tetrapod		[-]	5	
orientation of leg		[-]	2	

Table 3.1 Different parameters and their values or ranges

Four test series were identified using the above stated variables : (see also Table A.1 and A.2)

test series	s_p [-]	h_{toe} [m]
JS01-JS04	0.02	0.30
JS05-JS08	0.04	0.30
JS11-JS14	0.02	0.50
JS15-JS18	0.04	0.50

Table 3.2 Four test series

Furthermore, a combination of a significant wave height, H_s , a wave steepness, s_p , and a water depth, h_{toe} , is referred to as a sea state. In this way a total of 16 sea states can be identified.

3.6 Test program

As already stated before, a *test series* in its turn consists of 4 parts or *test runs*, with successive values for the significant wave height, H_s , of 0.10, 0.15, 0.20 and 0.25 m. respectively. The wave steepness and water depth were kept constant throughout a test series. Before each new series the armour layer will be completely removed and rebuilt.

Furthermore, three parts can be identified within the investigation. Each part had its own specific objective. The parts were referred to as :

- "main program"
- "additional stability tests"
- "CUR related program"

Below, the three programs of this investigation are described.

3.6.1 "Main program"

This part of the research is performed in line with the investigation performed at AUC. Therefore the instrumented tetrapods were placed in the same locations that were investigated at AUC (Figure 3.4; locations 1 to 5). The four test series of table 3.2 were all repeated five times. By doing so, the total number of stress signals that has been obtained amounts up to 400.

$$H_s \cdot h_{toe} \cdot s_p \cdot \text{location} \cdot \text{repetitions} = 4 \cdot 2 \cdot 2 \cdot 5 \cdot 5 = 400 \text{ signals.}$$

At this stage a *category* of stress signals is defined. A *category* is a unique combination of H_s , h_{toe} , s_{op} , location and orientation. Hence, the 400 signals obtained in this "main program" can be divided into 160 categories.

$$H_s \cdot h_{toe} \cdot s_p \cdot \text{location} \cdot \text{orientation} = 4 \cdot 2 \cdot 2 \cdot 5 \cdot 2 = 160 \text{ categories.}$$

Next to the measuring of the stresses, the number of displaced units was determined as well. In Table B.1 governing parameters of each of the test series are enumerated.

3.6.2 "Additional stability tests"

Next, each of the above mentioned test series, i.e., JS01-JS04, JS05-JS08 etc, were performed once again, this time according to Table B.2 (Appendix B). These tests were done purely to look at the hydraulic stability of the armour layer.

3.6.3 "CUR C70 related program"

The two lines of research mentioned in chapter 2, i.e. the CUR C70 research and the AUC research, used different approaches to derive a description of the stresses in armour units exposed to wave action. Therefore, a third part in this investigation was defined. The objective of this part was to verify the results of the CUR C70 research using the approach of the other line of research, i.e. the direct method of measuring stresses as used at AUC.

To compare the results more easily the instrumented tetrapods were placed not in the positions 1 to 5 as used in the main program, but in the same positions that have been investigated by the CUR C70 research. These locations are also given in Figure 3.4, and referred to as locations *a to e*.

For this part the same sea states were used as used in test series JS05-JS08, referred to as JS05C-JS08C to make a better distinction. In the "CUR related program" another 100 stress signals were obtained which could be divided into 40 categories.

Again, the number of displaced units was determined as well.

4 Hydraulic stability of tetrapods

4.1 General

As explained in chapter 3 the hydraulic damage has been observed and recorded next to the stress measurements in tetrapods, to see whether the performed tests, from the hydraulic stability point of view, would agree with the existing stability formulae for tetrapods on a breakwater slope.

Firstly, in section 4.2, the additional performed stability test series for comparison with the hydraulic stability formula will be analyzed. Secondly, the stability comparison of the two other parts of the investigation, i.e. the main program together with the CUR C70 related program, are presented in a similar way. This comparison will be made in section 4.3.

The hydraulic stability of the "main program" and the "CUR related program" of this investigation will be presented together as the difference between the two partial programs lies only in the stress measuring part which will be dealt with later on in this report. At this stage only the hydraulic stability is considered.

Finally, in section 4.4 some conclusions will be drawn.

4.2 "Additional stability tests"

4.2.1 General

These additional stability tests have only been performed once for each test series. The recorded damage of these tests can be compared with the damage calculated from the stability formula for tetrapods, equation 4.1, based on a large number of experiments. (Van der Meer, 1988)

$$\frac{H_s}{\Delta D_n} = b \cdot \left(3.75 \frac{N_{od}^{0.5}}{N^{0.25}} + 0.85 \right) s_{om}^{-0.2}$$

or

$$N_{od} = \left(\frac{\frac{H_s}{\Delta D_n}}{\frac{b \cdot s_{om}^{-0.2}}{3.75}} - 0.85 \right)^2 N^{0.5} \quad (4.1)$$

where H_s = significant wave height [m]
 Δ = relative mass density of armour units [-]
 D_n = diameter of armour units [m]
 N_{od} = number of displaced units [-]
 N = number of waves [-]
 s_{om} = wave steepness [-]
 b = 1 for the average curve

For each first test run of a test series, i.e. JS01, JS05, JS11 and JS15, the observed damage, if any, and the calculated average damage can easily be compared. But for the other test runs within a series the preceding runs caused settlement and maybe even some damage that has to be regarded as well. In other words, when calculating the damage with the above stated formulae, the history of damage of the breakwater slope has to be taken into account.

4.2.2 Cumulative damage

In Figure 4.1 and 4.2 a first comparison between the recorded and the calculated average damage is made. The dotted line represents the average damage calculated with the above stated formula.

The reliability of equation 4.1 can be described by considering b as a stochastic variable with a normal distribution. (Van der Meer and Heydra, 1991) When using, $b = 0.863$ and $b = 1.164$, for instance, both the 90% confidence levels are found. In the presented figures, the confidence interval is represented by the vertical solid lines

From these figures 4.1 and 4.2 it can be seen that the recorded damage shows a tendency to be smaller than the calculated average damage with the stability formula. Firstly, the small number of waves used in the first test runs of a series may have its effect. This effect will also be present in the calculated damage of the subsequent test runs.

Secondly, the stability formula is based on test results obtained from a large number of experiments with deep water situations in front of the structure. However, in this investigation also shallow water situations have occurred, resulting in a different $H_{2\%}/H_s$ ratio.

A third reason may be the rather large number of tetrapods per m^2 which causes the breakwater to be more stable. Research to investigate the influence of the number of tetrapods per m^2 in more detail was performed just after this investigation. The results are not yet known by the author. Therefore, only the first two reasons will be treated below.

4.2.3 Number of waves

Because the stability formula has been curve fitted on results of tests using a 1000 waves or more, the calculated damage caused by 200 waves, as used in the first one or two test runs of a test series, have been based on an interpolation.

When looking at the stability of rock on slopes for instance, it is concluded by Van der Meer that for the first 1000 waves a linear relation between N_{od} and N gives better agreement with reality compared to the relation in which N_{od} is proportional to $N^{0.5}$. (Van

der Meer, 1988)

Assuming that this may be valid for tetrapods as well, a graph like Figure 4.3 can be derived. It can be seen that, for the linear relation, the damage will grow less rapidly in the first waves than with the relation where N_{od} is proportional to $N^{0.5}$.

Comparison between the recorded damage and the calculated average damage, for the situation where $h_{toe} = 0.30$ m, is shown in Figure 4.4. The 90% confidence interval is shown as well. For the specific test runs that consisted of less than 1000 waves (Table B.2; Appendix B) a linear interpolation has been used. Still, the comparison is not satisfactory.

4.2.4 Using $H_{2\%}$ instead of H_s

On deep water the ratio $H_{2\%}/H_s$ has a constant value of 1.4. In Figure 4.5 it can be seen that this is not the case in front of the structure as used this investigation. A trend is shown where the ratio $H_{2\%}/1.4*H_s$ decreases with decreasing relative water depth. Since damage is probably inflicted by the highest waves, a reduction of the value of $H_{2\%}$ due to the limited water depth will lead to smaller damage. Basing the stability formula on $H_{2\%}$ instead of H_s will therefore give better results.

In the Figures 4.6 and 4.7 the recorded damage is again plotted together with the calculated average damage together with the corresponding 90% confidence interval. In this case $H_{2\%}$ was used instead of H_s to characterize the wave field. It can be seen from these Figures that a better agreement is obtained.

4.3 "Main program" and "CUR C70 related program"

As stated already in section 4.1, the stability comparison of the "main program" and the "CUR related program" are discussed collectively as the difference between these two programs lies only in the stress measuring part of the investigation.

In the Figures 4.8 and 4.9 a first comparison between the calculated average damage and the recorded damage is shown without any correction. The solid lines are the development of the damage within a test series. The various repetitions of the test series can be identified. The dotted line represents the calculated average damage. Again, it can be seen the recorded damage is much smaller compared to the calculated average damage, i.e., the dotted line. The various errors will be accounted for, similar as in section 4.2.

Because all the test runs, of the two programs discussed here, consisted of less than 1000 waves, even after taking into account the cumulative effect of the preceding test runs, the calculated damage curves are based on a linear interpolation between $N = 0$ and $N = 1000$ waves. The procedure was similar as the one described in section 4.2.1

In the Figures 4.10 and 4.11 the calculated average damage is corrected for, using H_s and a linear interpolation between $N=0$ and $N=1000$ waves. In Figure 4.12 and 4.13 this correction is extended, using $H_{2\%}$ instead of H_s .

For the depth limited test series, i.e., JS01-JS04 and JS05-JS08 the recorded damage is still on the low side when compared to the calculated average damage but in most of the cases the recorded damage lies within the 90% confidence interval. For the situation which is not influenced by the water depth, i.e., $h_{toe} = 0.50$ m, the recorded damage, on average, agrees rather well with the calculated damage.

4.4 Conclusion

Although this investigation involved a rather small number of tests, it appears that for the shallow water situation other phenomena, besides the number of waves and the $H_{2\%}/H_s$ ratio, play a role as well.

Speaking in terms of hydraulic stability of the armour layer of this particular breakwater, it can be seen from Figure 4.12 and 4.13 that the performed test series of the "main program" and the "CUR related program" agree with the existing hydraulic stability formula well enough to safely continue the investigation on stresses in tetrapods.

5 Analysis of the stress signal

5.1 General

In this chapter the preliminary analysis of the stress signal will be treated. In section 3.4 it has been shown that the appropriate frequency of 16000 Hz had to be reduced to keep the produced amount of data manageable. Therefore, the actual sample frequency has been set at 6000 Hz. Even when doing so, a test series of 800 waves, i.e., 4 test runs of approximately 200 waves each, produced around 250 Mb, which could just be stored on the hard disk of the PC available.

The consequence was that before a new test series could be performed, the signal was treated in a preliminary analysis reducing the amount of data by picking out only the values of interest. In this way the data was reduced to such a volume that it could be stored on a diskette. In section 5.2 the preliminary analysis, conform the one used at AUC will be explained.

However, the AUC approach did not work in the present research in a satisfactory manner, caused by, firstly, a change in the static stress, σ_{static} , and, secondly, the sensitivity of the AUC equipment to the electricity cables. The two mentioned phenomena will be treated in section 5.3.

In section 5.4 and section 5.5 a new (DUT) preliminary analysis will be presented for both the static and quasi static component of the stress signal as well the dynamic component of the signal. In section 5.6, finally, the consequences of an inaccurate AUC analysis will be discussed.

5.2 AUC analysis of stress signal

An average stress over the entire length of the test run, $\sigma_{overall}$, is defined. $\sigma_{overall}$ is determined by resampling the signal with a frequency of 20 Hz over the entire length of the signal.

Firstly, this $\sigma_{overall}$ is used to track the maxima of the quasi-static part, $\sigma_{quasi,max}$, defined as the maximum of the stress signal between two up crossings of the original signal with $\sigma_{overall}$. This method is similar to the treatment of a simple surface wave signal.

Secondly, the $\sigma_{overall}$ is used to find the impacts that may be present in the signal. When the stress signal reaches a value of C_1 times $\sigma_{overall}$ a more closer look is taken at that particular part of the signal as it might contain an impact. The second criterion an impact has to meet is that the steepness of the signal has to exceed a certain preset value, C_2 . Both the constants C_1 and C_2 are based on extensive research and testing at AUC, giving satisfactory results there.

The stress analysis was performed right after the test series had been completed. In

practice this means that the static, the quasi static maxima and the impacts were determined and stored. After this analysis, the hard disk was cleared to perform a next test series. Only for a limited number of test series, the raw data was stored in its entirety.

Looking at pictures of the stress signal obtained at DH (Figure 5.1) and at AUC (Figure 5.2) two major differences can be noticed. Firstly, the width of the DH signal compared to the AUC signal. Secondly, the change of the static stress, σ_{static} , in some of the DH signals. Both differences cause the first analysis to become inaccurate. A new analysis had to be developed which did account for the noise and the change of static stress.

5.3 Differences between DH and AUC signal

5.3.1 Noise

The DH signal is apparently disturbed by noise when compared to the AUC signal, even though at Delft Hydraulics the same equipment has been used and the test set up was very much the same. Therefore, the nature of these differences can not be explained based on physical differences in test set up or circumstances.

It appeared that the equipment, which was made available by AUC, was very sensitive to the electricity cables running through the laboratory. Figure 5.3 shows an enlarged part of one of the signals of Figure 5.1. Whereas a more or less straight line is expected the fluctuations of the signal as a consequence of noise can be seen. Spectrum analysis showed that the frequency of the noise was 50 Hz on which higher order fluctuations of this 50 Hz were superimposed.

5.3.2 Change of σ_{static}

Due to the nature of a slope of tetrapods, it is possible that units will move and be displaced under the influence of wave action, resulting in a more stable armour layer. During the movements or displacements the static stresses of one of the legs of the tetrapod may change as well.

This is clearly a physical phenomena which will surely not only occur in model tests and therefore has to be taken into account in the stress analysis. Figure 5.4 shows such a sudden decline of the signal most likely caused by the movement of the armour units. Other examples have been found as well, where relocation leads to a higher average stress.

5.3.3 Conclusions

The preliminary analysis, conform the analysis developed at AUC, made use of an average stress, $\sigma_{overall}$, based on an entire test run. This method gives good results assuming that the $\sigma_{overall}$ is representative for the entire test run, i.e. no sudden changes (declines or increases) of the static stress, σ_{static} , appear. *But what to do if those sudden changes do appear?*

Furthermore, did these changes in σ_{static} , due to the movement and/or displacements, only appear in this investigation or did they appear in the AUC investigation on dolosse as well? And, if these changes did occur, what were the magnitudes of these changes?

The consequence of a sudden change in the stress level is the fact that for an extended period no (up)crossings are found so that no separate waves can be distinguished.

On the other hand, the case that σ_{overall} does describe the signal in an adequate way and the quasi-static fluctuations are small, i.e., a ratio $\sigma/\sigma_{\text{overall}}$ of approximately 1, caused problems as well. Around the zero crossings of the main wave, additional zero crossings are generated by the noise. This leads to a large number of "waves" amongst which a high percentage in the lower amplitude range. Hence, a large number of values of $\sigma_{\text{quasi,max}}$ is found as well.

The above mentioned facts plead for a different approach in which no longer the overall static stress is used but a static stress or moving average should be used representing the local signal in a more accurate way.

5.4 Development of second analysis for $\sigma_{\text{quasi,max}}$

In order to acquire these quasi-static maxima a new preliminary analysis has been derived, mainly consisting of *two low-pass filters* after one another. A low-pass filter basically consists of a procedure that allows low frequency components to pass undisturbed, and that filters out the higher frequency components.

By applying this filter techniques, the impacts are filtered or removed as well. However, at this stage, i.e., the determination of the quasi-static maxima, these impacts are not important yet.

5.4.1 Removal of noise

As already stated, the frequencies of the phenomena of interest are clearly different from the frequencies of the noise problem. Hence, it is possible to use a low-pass filter technique to remove all noise related frequencies.

Three cut-off frequencies have been tested for this first low-pass filter, i.e., 20, 10 and 5 Hz respectively. An example of such low-pass filter application is shown in Figure 5.5. The low-pass frequency of 10 Hz is chosen, staying well above the highest frequency of the phenomena of interest ($1/T_m$) and removing the noise without distorting the signal.

5.4.2 Reduction of number of points

The large number of points of 6000 per second is no longer necessary for the analysis of the quasi-static part of the signal. The above mentioned filtered stress signal can be still be adequately described using a significantly smaller number of points. As a rule of thumb a signal can still be adequately described using a minimum of 20 points per period.

The average wave period, T_m , of the wave field used in the hydraulic model tests lies between 1 and 3 seconds. By reducing the number of points of the signal from 6000 to 40 points per second, a good picture of the signal can still be preserved. (Figure 5.6)

This reduced signal forms the input of the second low-pass filter of the new preliminary analysis for the $\sigma_{\text{quasi,max}}$.

5.4.3 The problem of variation of σ_{local}

The problem of a sudden change of the stress level can be solved by determining a moving average of the signal. This moving average can be determined using a low-pass filter technique again with an even lower cut off frequency than the one described in section 5.4.1.

This second low-pass filter removes the quasi-static component to define σ_{local} . Again, three cut-off frequencies have been compared, i.e. a filter with the size of 3, 5 and 10 times the average wave period of the test run, to calculate this σ_{local} :

$$f_{\text{lowpass}} = \frac{1}{C_3 \cdot T_m} \quad (5.1)$$

where $f_{\text{low-pass}}$ = low-pass frequency [Hz]
 C_3 = number of average wave periods (3,5 or 10)
 T_m = average wave period [s]

Figure 5.7 shows an example of the application of such a low-pass filter. The upper right corner gives the moving average calculated using $3 \cdot T_m$. The lower left figure holds for $5 \cdot T_m$, whereas the lower right figure uses $10 \cdot T_m$ as window size. The upper left figure shows the 10 Hz low-pass filtered signal, i.e. the input signal of this second filter. It was decided to continue with $C_3 = 3$.

5.5 Development of second analysis for σ_{impact}

For the determination of the impacts the original signal has to be taken. In this case a low-pass filter of 50 Hz was used on the original signal to determine a local average based on 1/50 of a second following the signal, with the superimposed noise, very closely. (Figure 5.8)

At this stage an impact is defined as a stress value larger than C_4 times the stress value of the low-pass values in that specific point taking into account the absolute value of the noise. Because the low-pass signal follows the original signal to a high degree the value C_4 can be much smaller compared to C_1 causing more impact to be found. C_4 is set at 1.5.

5.6 Consequences

This new method of analyzing can only be applied to the test series that have been stored on tape or other devices which amounts to 24% of the performed tests. (6 out of 25)

Fortunately, this does not mean that the other 76% of the performed test series have been a waist of time. Plotting the obtained quasi-static maxima of the preliminary AUC analysis in a time-stress figure, i.e., plotting the distribution of the found $\sigma_{\text{quasi,max}}$ of a test run as a function of the time t , may help.

If, for instance, the stress signal shows a sudden drop of static stress (Figure 5.4), which means that the overall static stress, σ_{overall} is an inaccurate description of the local stress signal, no quasi-static maxima will be found for an extended period.

Figure 5.9 shows the distribution of determined $\sigma_{\text{quasi,max}}$ values of the same test run of which Figure 5.4 is a small part. It can clearly be seen that due to such changes the preliminary AUC analysis did not work correctly.

In addition, Figure 5.10 shows the distribution of determined $\sigma_{\text{quasi,max}}$ values of the same stress signal using the new preliminary analysis. A more evenly distributed picture is obtained.

If such distributions, as shown in Figure 5.10, could be found amongst the test results that have been acquired using the preliminary AUC analysis, this would mean that the results of these particular test runs can be taken along in further analyses.

Figure 5.11 shows such a distribution amongst the results of the preliminary AUC results which is taken along. In this way 160 test runs were considered fit for further analysis, increasing the percentage of useful tests from 24% to 56%.

6 Analysis of quasi-static maxima

6.1 General

The parameters discussed in chapter 3 are used as such within this investigation as each of them is thought to have its own specific effect on the tensile stresses inside tetrapods. The 16 different sea states that can be identified using the *hydraulic* parameters, .i.e., H_s , h_{toe} and s_{op} , together with the *location* and *orientation* of an instrumented tetrapod with the accompanying stress results, form the basis of this statistical investigation.

In section 6.2 the limited number of repetitions and the influence on the statistical analysis will be regarded. The necessity of reduction of variables as a result of this limited number of repetitions will be discussed in section 6.3. In section 6.4 the various remaining combinations of parameters will be regarded, whereafter, some remarks will be placed concerning the obtained results. Finally, in section 6.6 some conclusions will be drawn.

6.2 Repetitions

With the parameters mentioned (H_s , h_{toe} , s_{op} , location and orientation) a total of 200 different categories of signals were produced, 160 resulting from the "main program" of the investigation and another 40 from the "CUR C70 related program" of the investigation. (see section 3.6)

To be able to describe the occurring stresses in each of the 200 categories of signals adequately, each category (or combination of parameters) has been tested a number of times to eliminate random effects. In total 500 stress signals were obtained. (see section 3.6)

However, due to the data reduction, as described in the previous chapter, caused by the unforeseen problems, the number of repetitions has been reduced, sometimes even leading to the loss of all results for some of the categories.

The categories of the "main program" are presented in probability plots shown in Figure C.1 to C.32. The categories of the "CUR related program" are shown in Figure C.33 to C.40. (Appendix C) Each Figure shows the probability curves of 5 of the categories, i.e. the particular categories that are obtained by changing only the location of the instrumented tetrapod.

From these figures it can be concluded that the intention of finding descriptions or trends in which all the parameters discussed above are presented is not realistic. In some of the cases, the differences in magnitude within a combination itself are larger than the differences between the combinations individually.

However, a few remarks concerning these figures can be made. First of all, the maxima of the stresses have a tendency to increase somewhat with increasing wave height. Secondly, the stress in the situation in which $h_{toe} = 0.50$ m. are bigger compared to the stresses measured in the situation of $h_{toe} = 0.30$ m.

6.3 Reduction of combinations

Parameters that have, *in this investigation*, negligible or no recognizable effects on the tensile stress distributions can be eliminated. Looking at the probability plots, no clear trend can be found between the magnitude of the stresses and, firstly, the orientation of the instrumented leg and, secondly, the location of the tetrapod.

A reason that no trend could be found between the stresses and the orientation may be caused by random placement. In both orientations a tetrapod has neighbouring tetrapods resting on its instrumented leg. As far as the location is concerned it is possible that the investigated area is too small (only one tetrapod height above and below SWL) to recognize a trend as such.

Looking at the wave steepness of the sea states in combination with the slope angle of the breakwater one can tell something about the breaker type of the waves. Both wave steepness situations, i.e. $s_{op} = 0.02$ and 0.04 at the wave board, caused the waves to be of the surging type ($\xi > 5$). Hence, the wave steepness did not have much influence on the stress distributions.

However, a different wave steepness causes the waves to react differently to the sloping foreshore, resulting in a different H_s at the toe of the breakwater. Therefore, the wave steepness will be kept as a governing parameter in the further analysis.

This leaves only the wave height, water depth and the wave steepness as important parameters. In other words, only the sea states seem to have their effect on the tensile stress distributions inside tetrapods for the area of one tetrapod height below and one tetrapod height above SWL.

6.4 Deriving trends between sea states and stresses

In table 6.1 the number of repetitions for each of the sea states is given.

H_s [m]	0.10	0.15	0.20	0.25
h_{toe} [m], s_{op} [-]				
0.30, 0.02	16	16	19	22
0.30, 0.04	21	26	32	35
0.50, 0.02	11	12	13	14
0.50, 0.04	10	10	13	10

Table 6.1 Number of repetitions of the sea states.

When the accompanying stress results of these sea states are presented in histograms, Figures like D.1 to D.4 (Appendix D) are obtained. The lines represent a Log Normal fit of the form :

$$f\left(\ln \frac{\sigma_{quasi,max}}{\rho_a g D_n}\right) = \frac{1}{\sqrt{2\pi} \sigma_{LN}} \cdot e^{-\frac{1}{2} \left[\frac{\ln\left(\frac{\sigma_{quasi,max}}{\rho_a g D_n}\right) - \mu_{LN}}{\sigma_{LN}} \right]^2} \quad (6.1)$$

where $\sigma_{quasi,max}$ = absolute value of the maximum within a stress wave [Mpa]
 ρ_a = mass density of the armour units [kg/m³]
 g = gravitational acceleration [m/s²]
 D_n = nominal diameter of tetrapod [m]
 μ_{LN} = average of LN distribution [-]
 σ_{LN} = standard deviation of LN distribution [-]

For some of the presented combinations the fit is rather poor. However, the fit gets better as the number of points increases. As a result of this tendency, together with the knowledge obtained from the dolosse investigation at AUC (Burcharth, 1991), the assumption is made that the stress distributions follows the Log Normal distribution. However, more tests need to be done to substantiate this assumption.

In Figures 6.1 and 6.2 the average, μ_{LN} , and the standard deviation, σ_{LN} , of the LN fits of the 16 different sea states are plotted against the significant wave height, respectively. Looking at Figure 6.1 two groups of points can be identified, i.e., one group of points for each water depth. For both groups an increase of the average values of the LN distribution with increasing water depth can be seen. At this stage a linear relation, according to equation 6.2, is assumed.

$$h_{toe} = 0.30 \text{ m.} \quad \mu_{LN} = 0.8 + 0.1 \frac{H_s}{D_n} \quad (6.2a)$$

$$h_{toe} = 0.50 \text{ m.} \quad \mu_{LN} = 1.6 + 0.02 \frac{H_s}{D_n} \quad (6.2b)$$

where μ_{LN} = average of the LN fit [-]
 H_s = significant wave height at toe of structure [m]
 D_n = nominal diameter [m]

The standard deviation decreases with increasing wave height. Again, a linear relation is assumed, equation 6.3.

$$\sigma_{LN} = 0.6 - 0.02 \frac{H_s}{D_n} \quad (6.3)$$

where σ_{LN} = standard deviation of the LN fit [-]

The water depth has significant influence on the average of the LN distribution. The standard deviation of the LN distribution is not influenced by the water depth. Also, it can be seen that the influence of the different wave steepnesses is indeed small.

With the trends for these distribution parameters, i.e., equation 6.1 and 6.2, it is possible to calculate a μ_{LN} and a σ_{LN} for a given ratio H_s/D_n . With these values for the two parameters of the LN distribution, the maximum value of the combined static and quasi-static stress in *one leg of the tetrapod*, corresponding to a certain exceedance probability, can be calculated.

When using different exceedance probabilities, for example 50%, 90%, 95%, 98% and 99%, preliminary design charts for the stresses inside one leg of a tetrapod were obtained. (Figure 6.3) The upper part of Figure 6.3 represents the depth limited situation, $h_{toe} = 0.30 \text{ m.}$ The lower part represents the situation which was not influenced by the water depth, $h_{toe} = 0.50 \text{ m.}$

The thick horizontal lines give the dimensionless tensile strength for various prototype tetrapods for both water depths, assuming $\sigma_T = 2.0 \text{ MPa.}$

6.5 Remarks

The formulae for the average stress of the LN distribution, i.e. equations 6.2a and 6.2b, display an unexpected picture. For the smaller values of H_s , the average stresses of the stress distributions were expected to be more or less the same for both water depths. Large differences were only expected in the test runs with the higher significant wave heights caused by breaking of the waves in the case of the depth limited situation ($h_{toe} = 0.30 \text{ m.}$)

Instead, Figure 6.1 shows the largest differences in the region of the smaller values for the significant wave height. Obviously, not only the significant wave height influences the stress distributions but there have to be other aspects as well.

One of these reasons could be that the size of the breakwater. In the case of $h_{toe} = 0.50$ m., the breakwater is much larger compared to the breakwater in the case of $h_{toe} = 0.30$ m., causing more settlement and, hence, larger static stresses. Therefore, attention should be paid not only to the combined static and quasi-static stresses but to the static stresses alone as well.

Furthermore, there is only paid attention to the general form of the LN fit, Appendix D, analogous to the procedure followed at AUC. As the interest lies in the area of exceedance percentages of 5%, 1% or even smaller, a more closer look should be taken at the right tail of the distribution instead of looking at the overall picture.

6.6 Continuation of the present investigation

The next step to come to design criteria for breakwater slopes armoured with monolithic concrete tetrapods is to establish such a design diagram which takes into account all four legs of a tetrapod instead of one.

Furthermore, the dynamic part of the signal, i.e., the stresses caused by impacts are not implemented yet. This will be of great importance for the overall description of stresses inside tetrapods. The author will continue this investigation trying to implement this dynamic part.

Finally, if a description for the impact stresses is found, a comparison with the work of the CUR C70 investigation can be made. In this way it might be possible to link the CUR C70 investigation on one side with the AUC investigation and this present investigation on the other side.

In other words, a great effort has to be made to try and link the method of calculating stresses based on the accelerations of the units and the method of directly measuring the stresses in the units itself.

6.7 Conclusions

Although a substantial part of the data has been lost, a number of conclusions can be drawn:

- Because of the large differences between subsequent test runs under identical conditions, the randomness of the construction method must have large influence on the variation in stress level. This means that reliable conclusions, concerning the influence of the individual parameters H_s , h_{toe} , s_{op} , location and orientation, can only be made after performing large number of tests.

- In this investigation, the influence of the wave steepness on the stress distribution is small.
- In this investigation, also the orientation of the instrumented leg of the tetrapod has negligible influence on the stress distribution.
- For the locations of the tetrapod, within a tetrapod height above and below SWL, also a negligible influence on the stress distributions has been found.
- With increasing number of values for $\sigma_{\text{quasi,max}}$, the Log Normal fit gets better. Whether or not the stress distributions can indeed be described using a Log Normal distribution has to be investigated further.
- From Figure 6.3 it can be concluded that, in this investigation, the static and quasi-static stresses inside the tetrapods armour units do not exceed the tensile strength of the various prototype tetrapods, in fact there is still quite a margin. However, there is only paid attention to the failure of one tetrapod leg. Furthermore, the impacts have to be implemented.

7 Conclusions and Recommendations

7.1 Conclusions

Below, the most important conclusions, mentioned throughout this report, are summarized once more :

- The stress signal has such a random nature that the preliminary analysis analogous to the preliminary AUC analysis, which was based on a long term average, could not be applied in this case. Instead, a preliminary analysis based on a moving average was used.
- Because of the large differences between subsequent test runs under identical conditions, the randomness of the construction method must have large influence on the variation in stress level. This means that reliable conclusions, concerning the influence of the individual parameters, can only be made after performing large number of tests.
Because of this fact, research concentrating on a more overall description of the stability and strength of a breakwater slope might be preferable to one that concentrates on a description based on individual armour units.
- In this investigation, the orientation of the instrumented leg of the tetrapod has negligible influence on the stress distribution.
- For the locations of the tetrapod, within a tetrapod height above and below SWL, also a negligible influence on the stress distributions has been found.
- The influence of the wave steepness on the stress distribution is small.
- With increasing number of values for $\sigma_{\text{quasi,max}}$, the Log Normal fit gets better. Whether or not the distributions of the quasi static maxima can indeed be described using a Log Normal distribution has to be investigated in more detail.
- Linear tendencies are assumed between both of the parameters of a Log Normal distribution and the wave height. The average of the LN distributions increases with increasing H_s . The standard deviation decreases with increasing H_s .
- Assuming the Log Normal distribution is valid, the combined static and quasi-static stresses inside the tetrapods armour units do not exceed the tensile strength of the various prototype tetrapods. In fact there is still quite a margin. However, there is only paid attention to the failure mechanism of one tetrapod leg. Furthermore the impacts have to be implemented.

7.2 Recommendations

A number of recommendations or suggestions are enumerated below :

- In this investigation, it was tried to establish a relation between the waves acting in front of a breakwater and the occurring stresses inside tetrapods, as a result of this wave action. In analogy to the research at AUC, there is assumed that the quasi-static maxima of the stress signal follow the Log Normal distribution. More tests need to be done to be able to judge if this assumption is correct.
- Moreover, the stress levels that are exceeded only by a few percent of the occurring stresses, i.e., 5%, 1% or an even smaller percentage, are of interest when looking at the strength of concrete armour units. Therefore, a more closer look should be taken at the right tail of the distribution instead of looking at the overall fit of the LN distribution.
- The influence of the wave steepness on the distribution of the combined quasi-static maxima was found to be negligible. Also the location and orientation of the instrumented tetrapod appeared to have no influence. Again, more tests should be performed to substantiate this.
- For the calculation of the stresses in a tetrapod beam theory was used. Hence shear forces have been neglected. A tetrapod however, looks much more like a console in which shear forces have to be considered as well. The error originating from this assumption has still to be quantified.
- Attention should also be paid to the static stress as well instead of taking the static and quasi static stresses together, as the settlement of a breakwater may have influence on the static stress level. More settlement will take place in a larger breakwater.
- Also, a more closer look has to be taken at the right tail of the distributions as the right tail, i.e. stress values that are only exceeded by a few percent, will be of most interest when looking at the breakage of concrete armour units.

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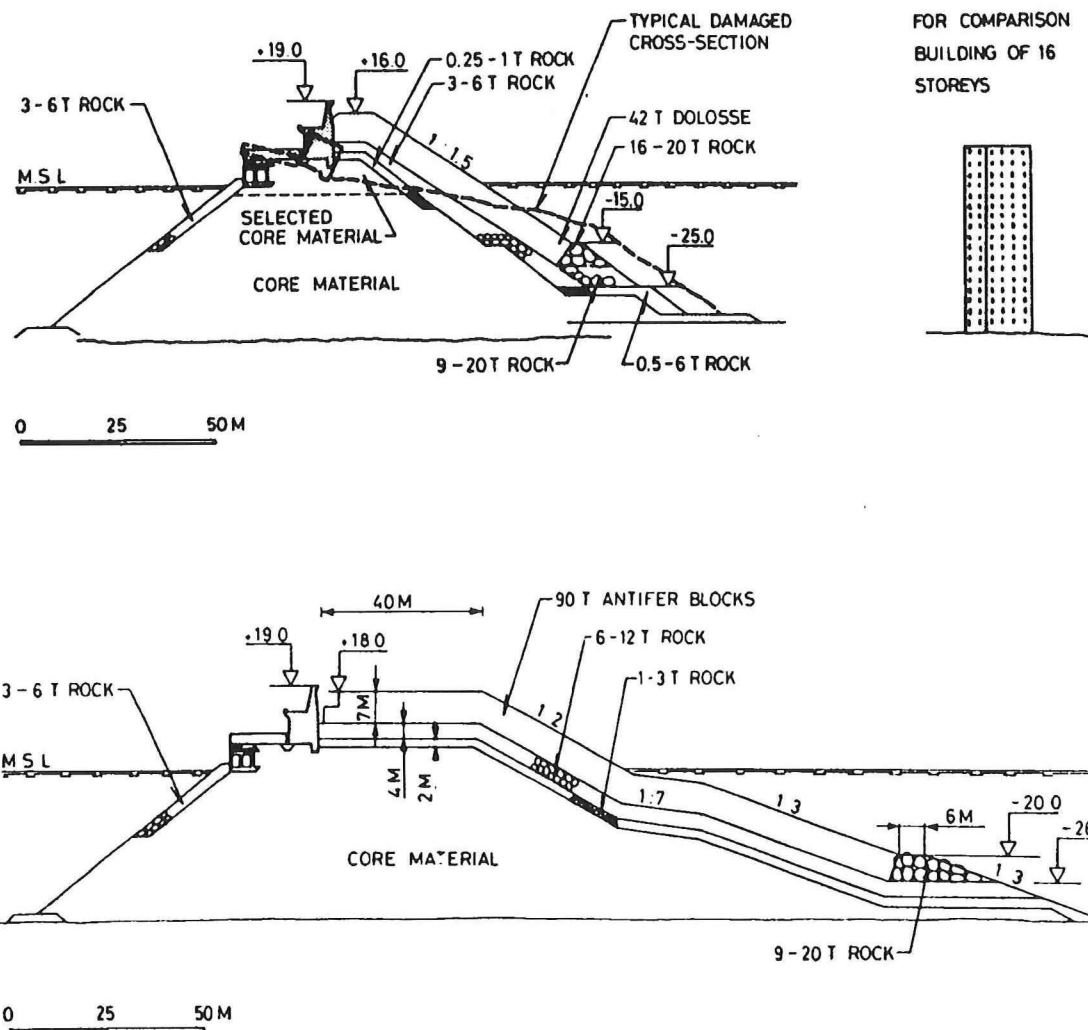


Figure 1.1 Typical cross section of Sines breakwater, Portugal, before and after repair.

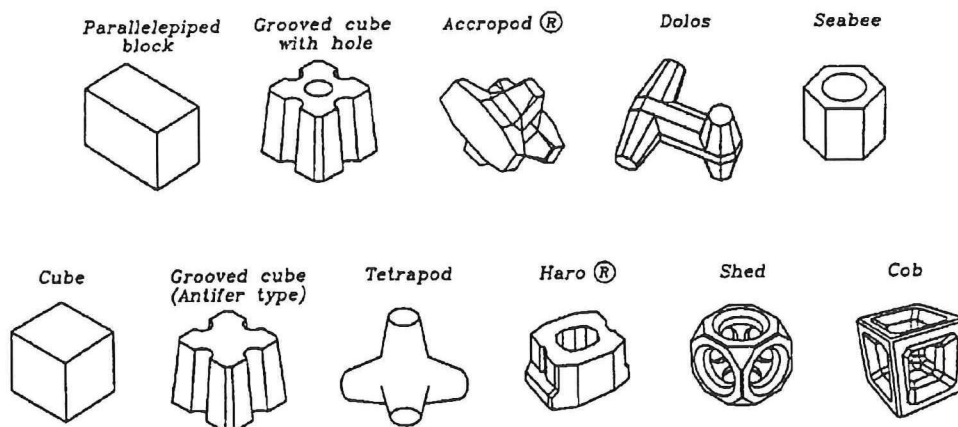


Figure 1.2 Different types of concrete armour units (Burcharth, 1993)

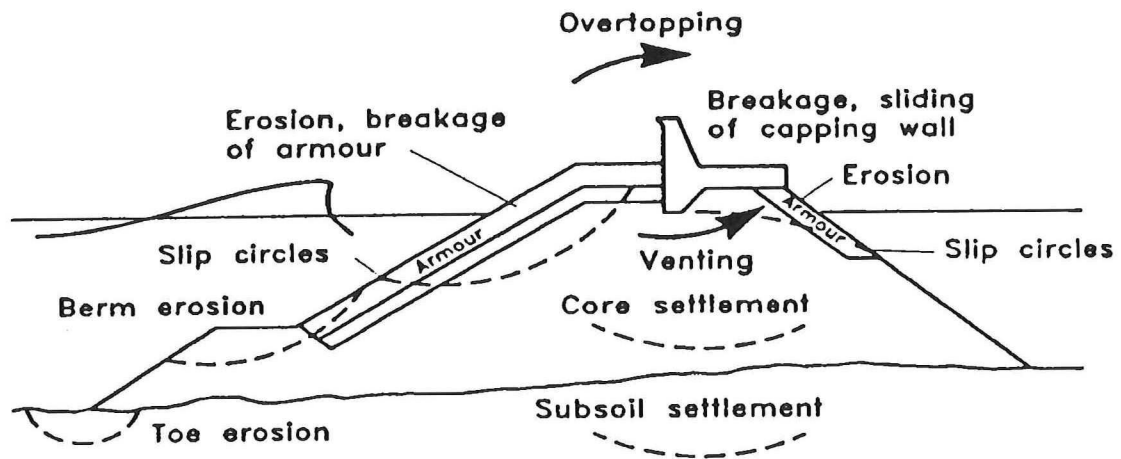


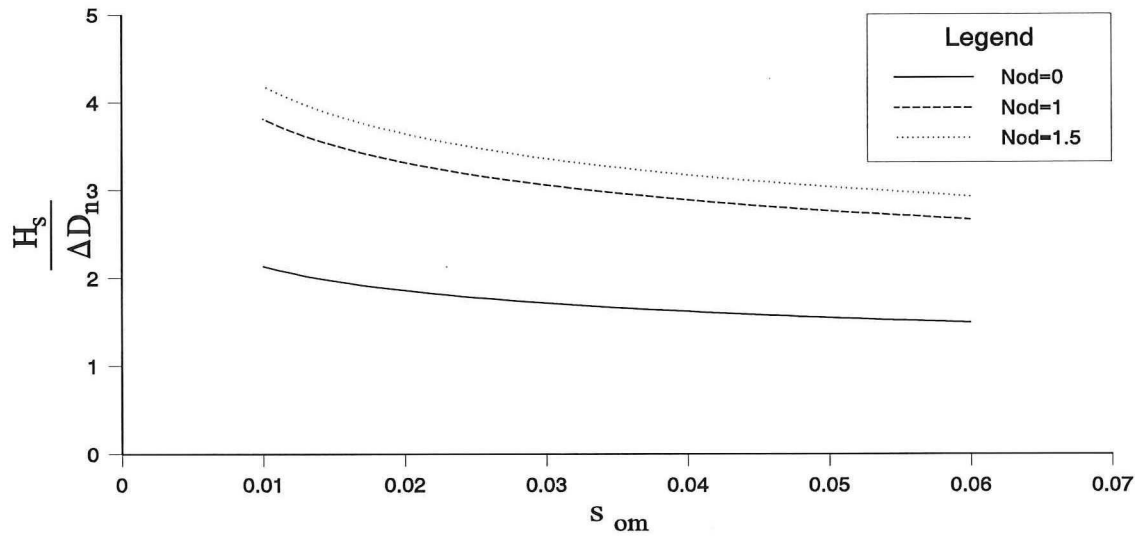
Figure 1.3 Different failure mechanisms for rubble mound breakwaters



Figure 1.4 Definition of stress transfer function (Burcharth, 1991)

Stability of tetrapods

Number of waves N=1000



Stability of tetrapods

Number of waves N=3000

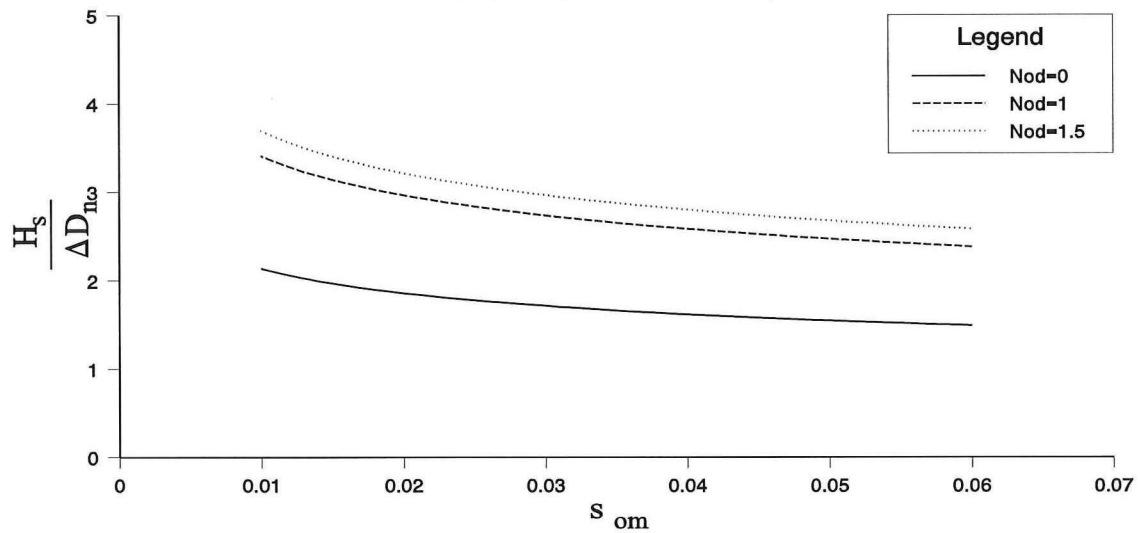


Figure 2.1 Stability number, $H_s/\Delta D_n$, vs. wave steepness, s_{om} for $N=1000$ (top) and $N=3000$ (bottom). (equation 2.2)

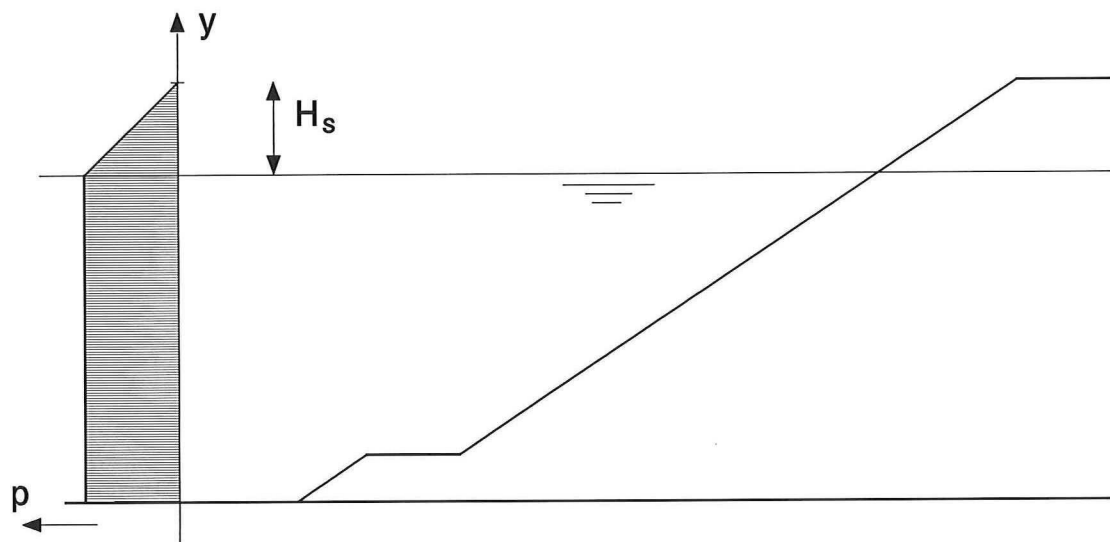
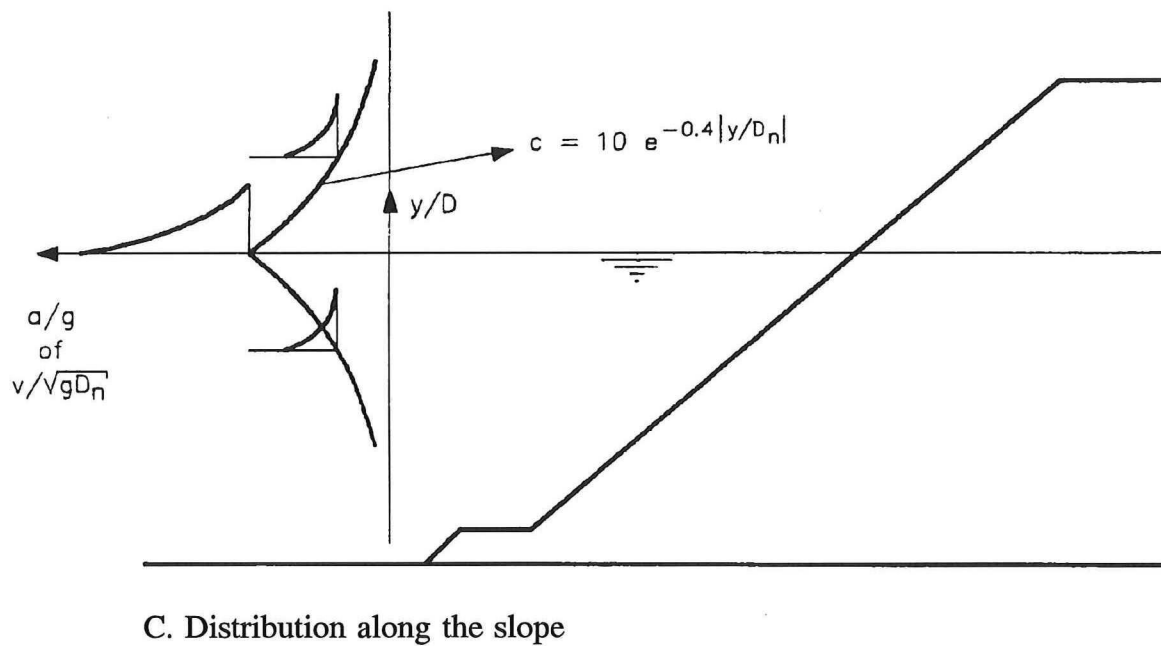
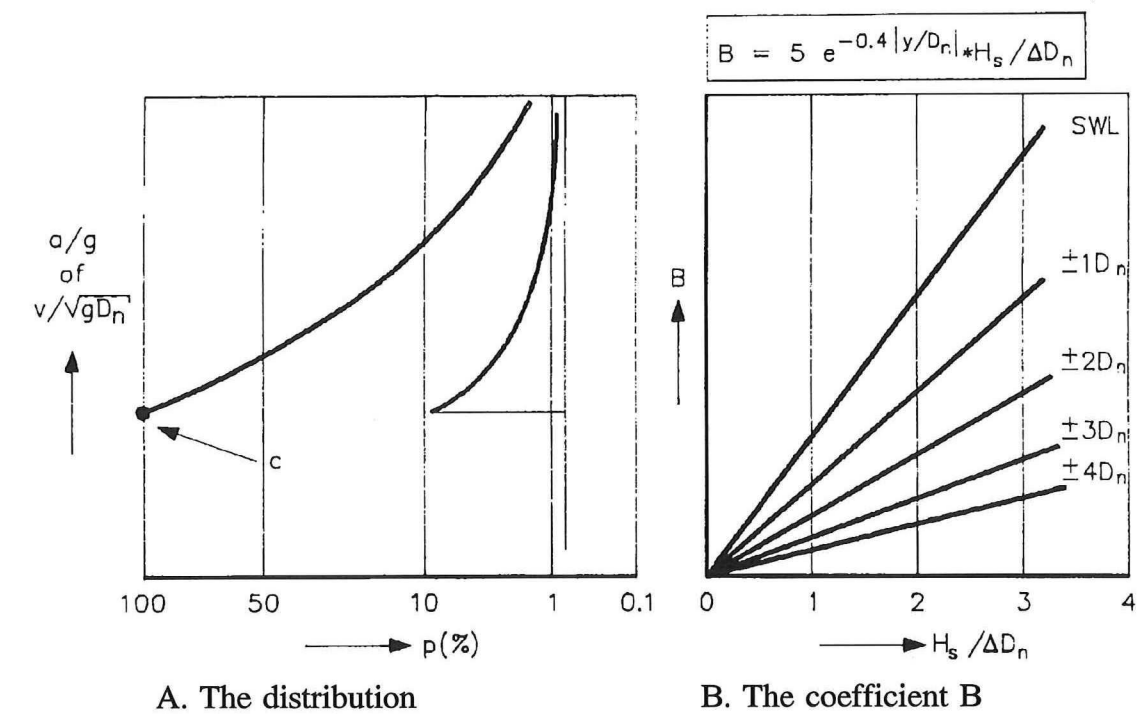


Figure 2.2 Distribution of impacts along slope (Van der Meer and Heydra, 1991)



$$p(a/g) = e^{-(a/g-c)/B}$$

Figure 2.3 Graphical presentation of the distribution of impact velocities (CUR C70)

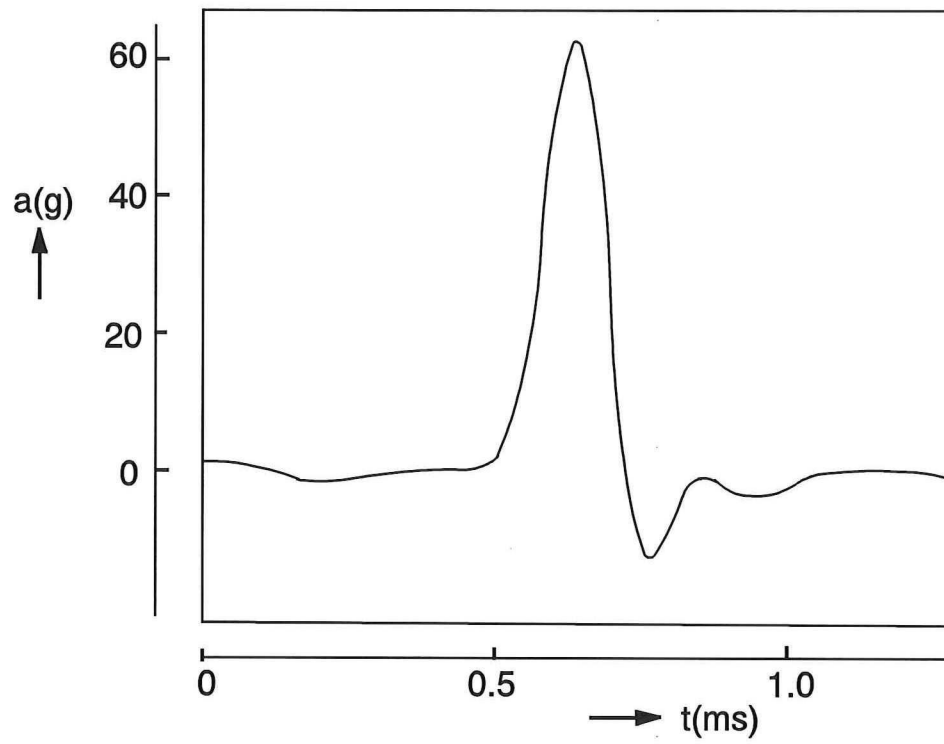


Figure 2.4 Accelerations measured due to rocking (CUR C70, 1990)

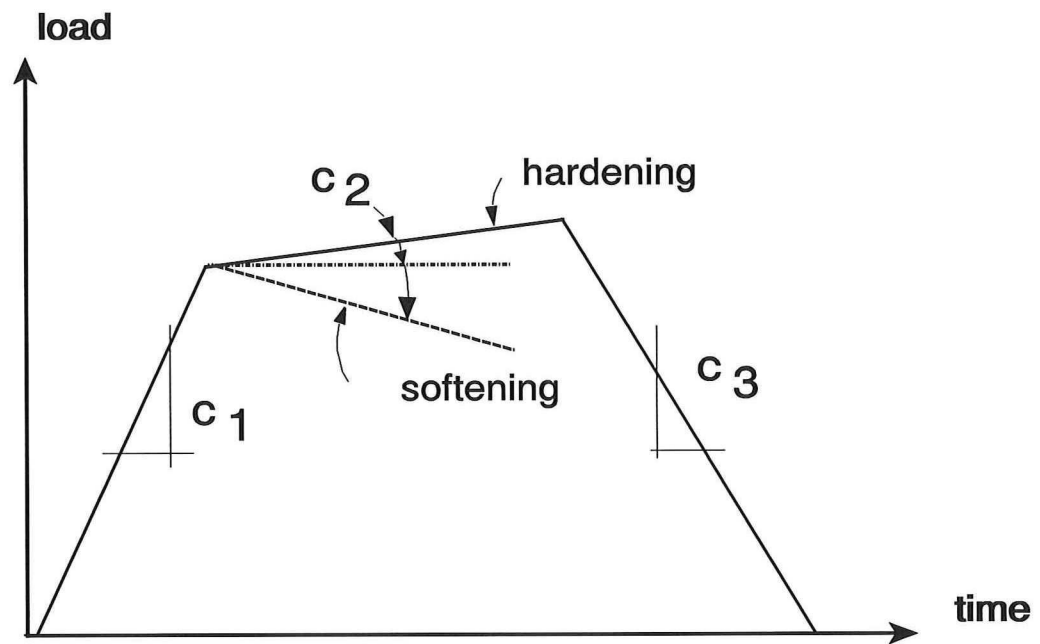


Figure 2.5 Elasto-plastic model (Van Mier and Lenos, 1991)

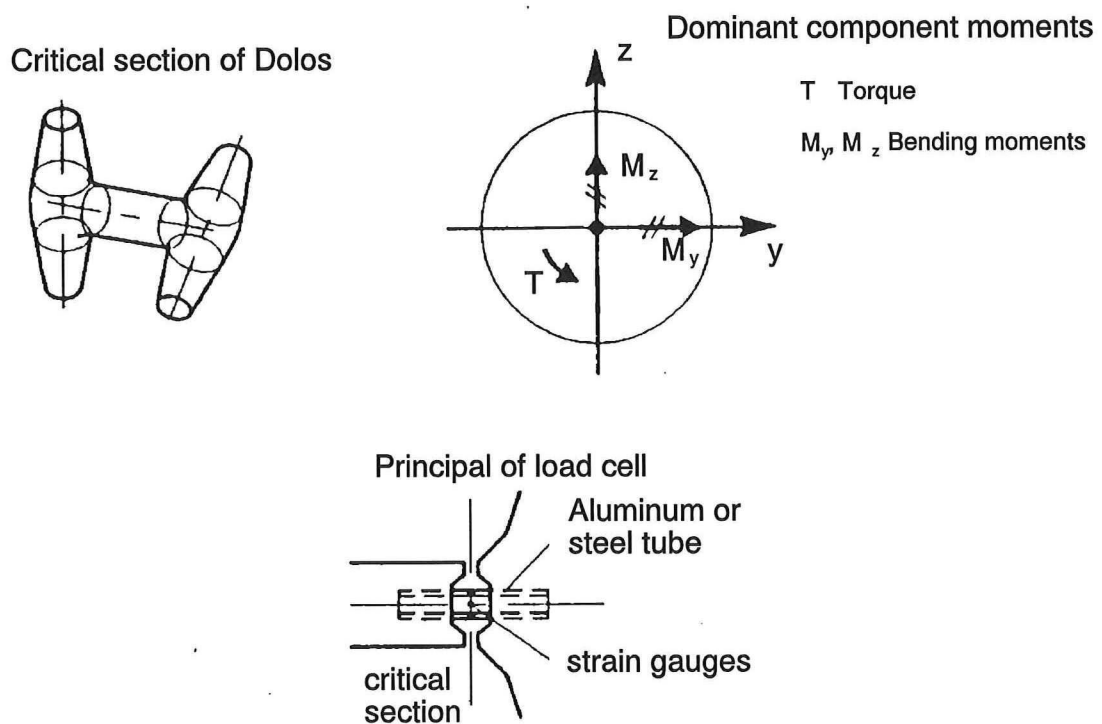


Figure 2.6 Illustration of critical sections and related component forces and moments (Burcharth, 1991)

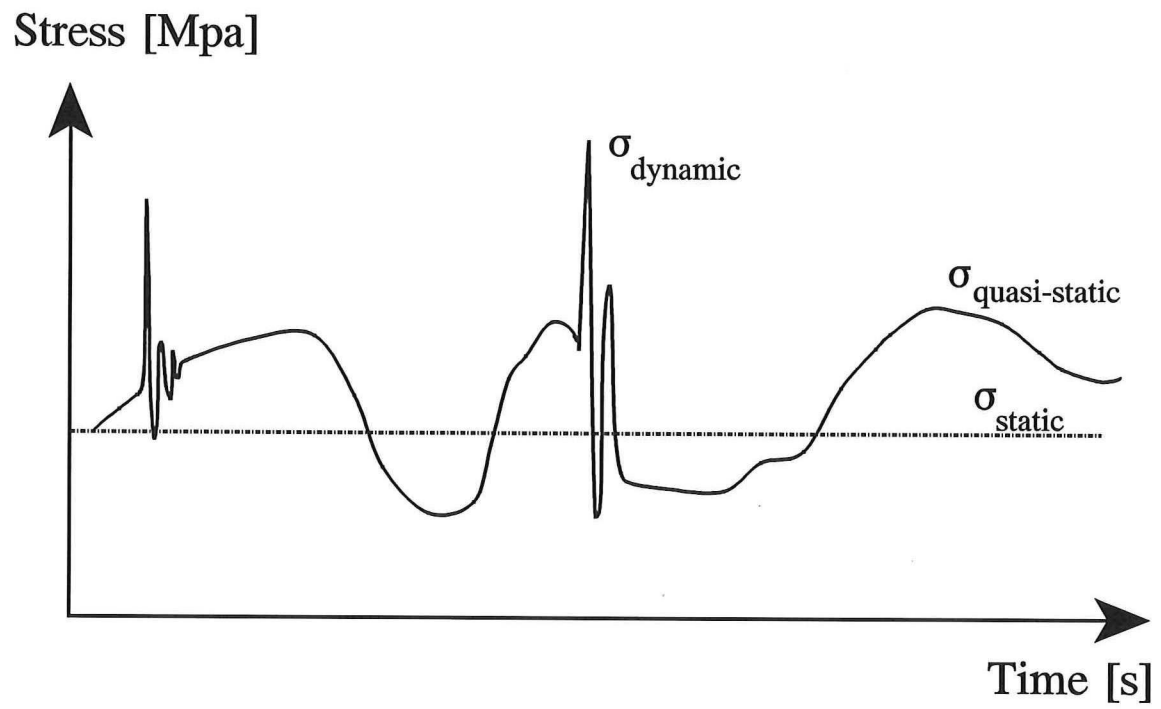
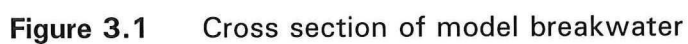
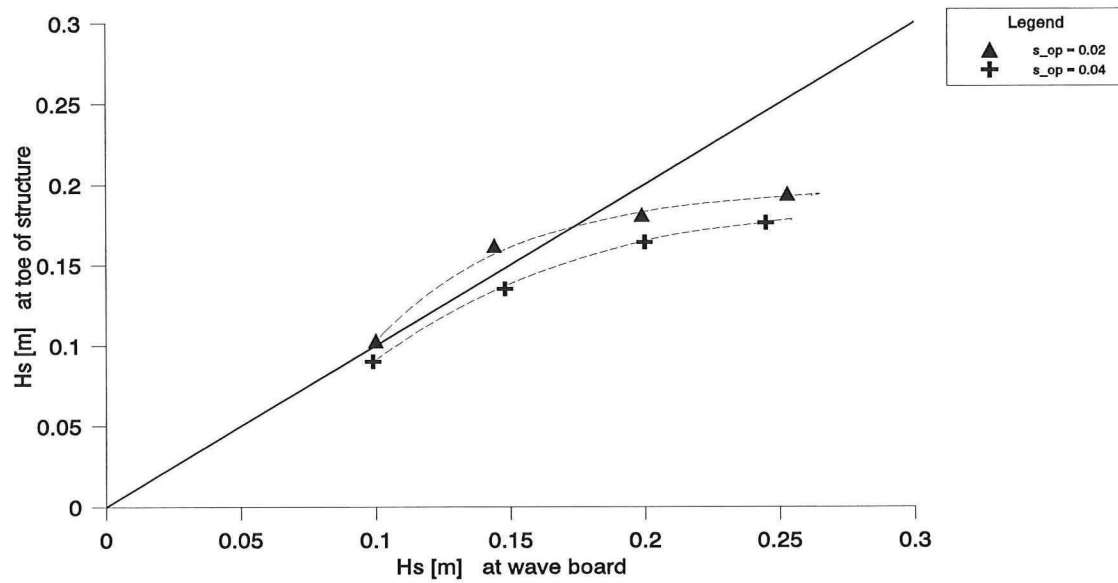


Figure 2.7 The three components of the stress signal



Influence of Foreshore



Influence of Foreshore

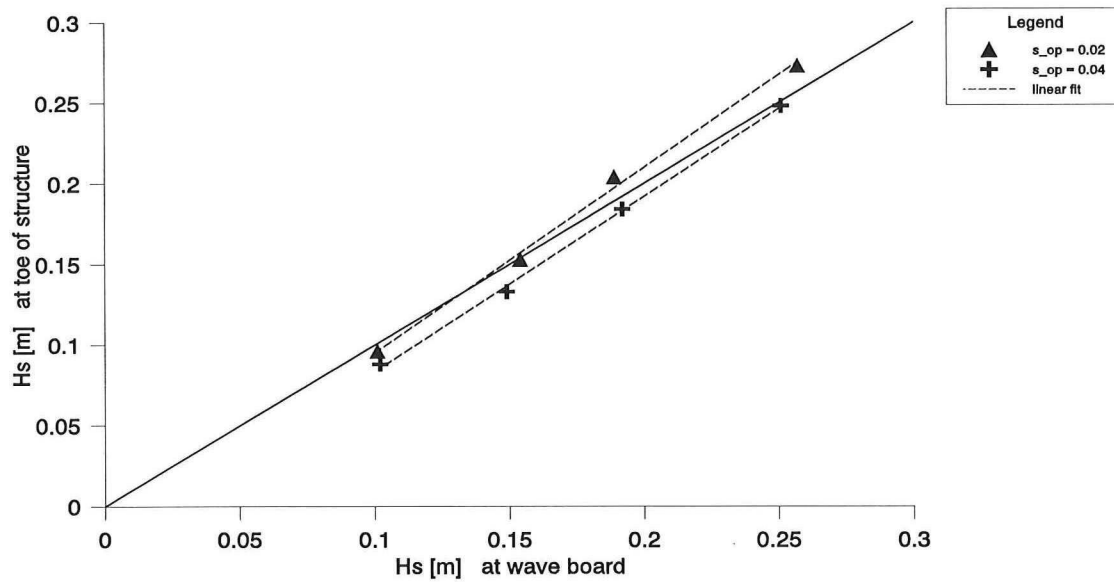


Figure 3.3 Influence of foreshore on significant wave height. $h_{toe} = 0.30$ m. (top) and $h_{toe} = 0.50$ m. (bottom)

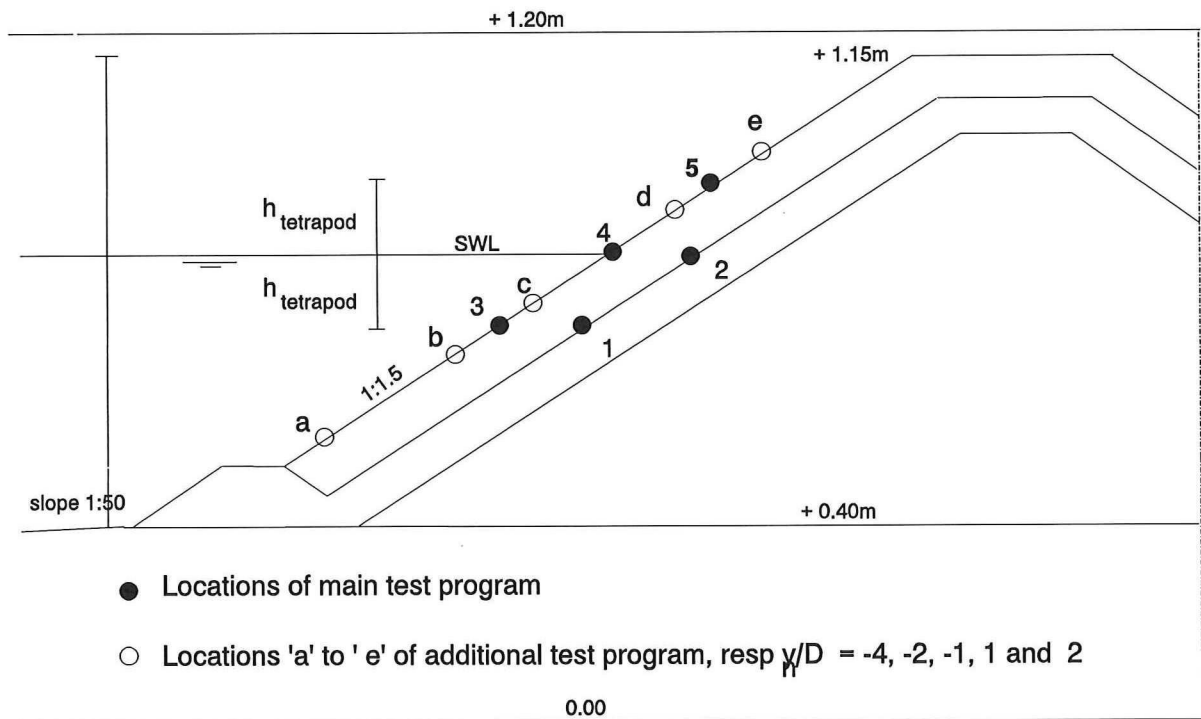


Figure 3.4 Position of instrumented tetrapods.

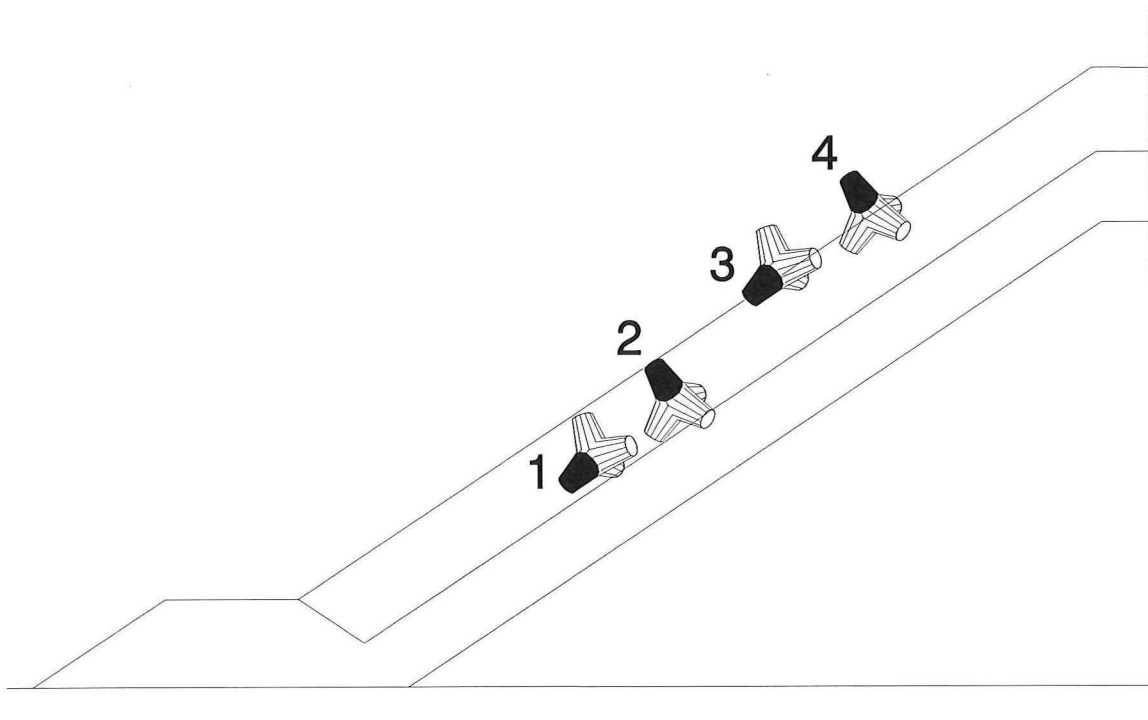


Figure 3.5 Orientation of instrumented leg in bottom and toplayer of armour layer

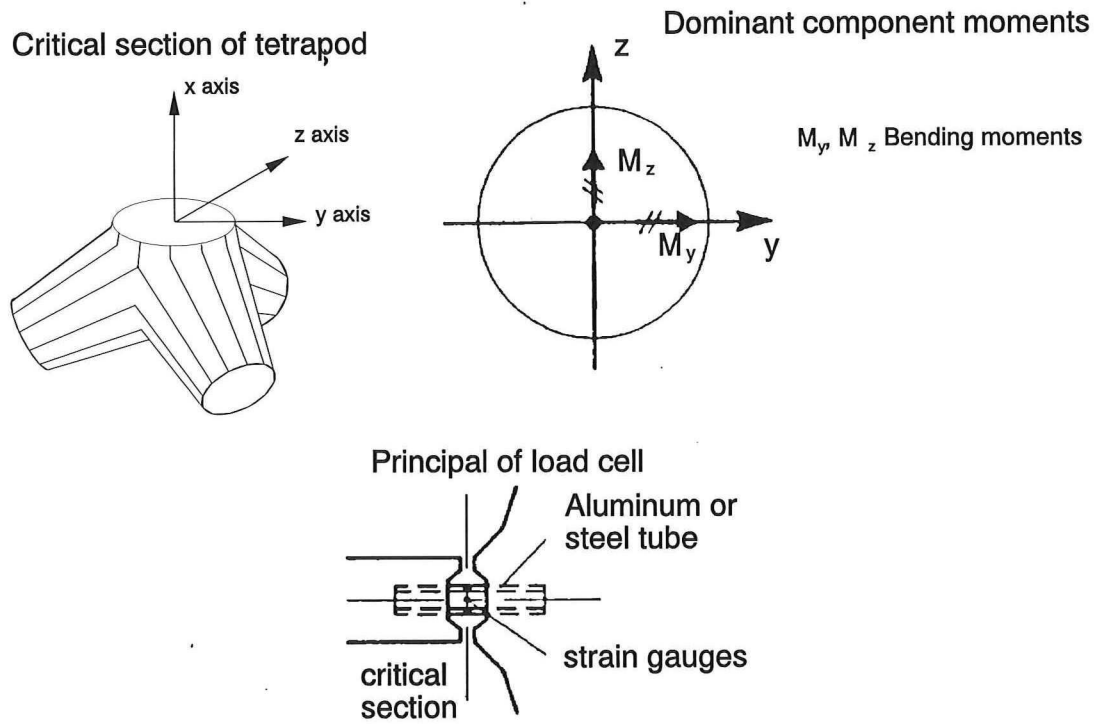
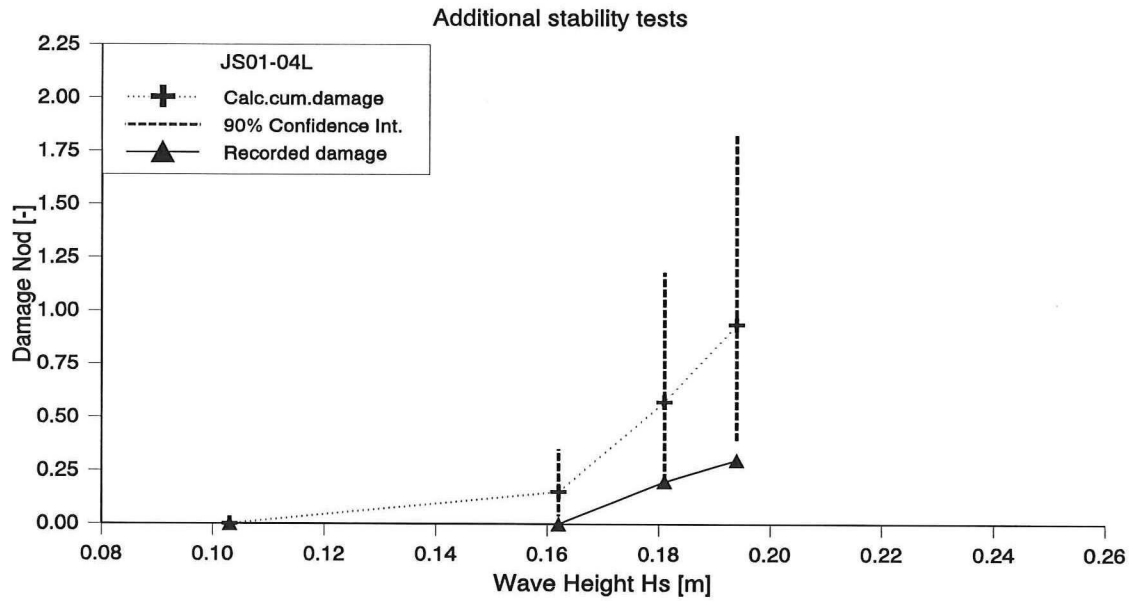


Figure 3.6 Illustration of critical section and related component moments of a tetrapod.

Stability of Tetrapods



Stability of Tetrapods

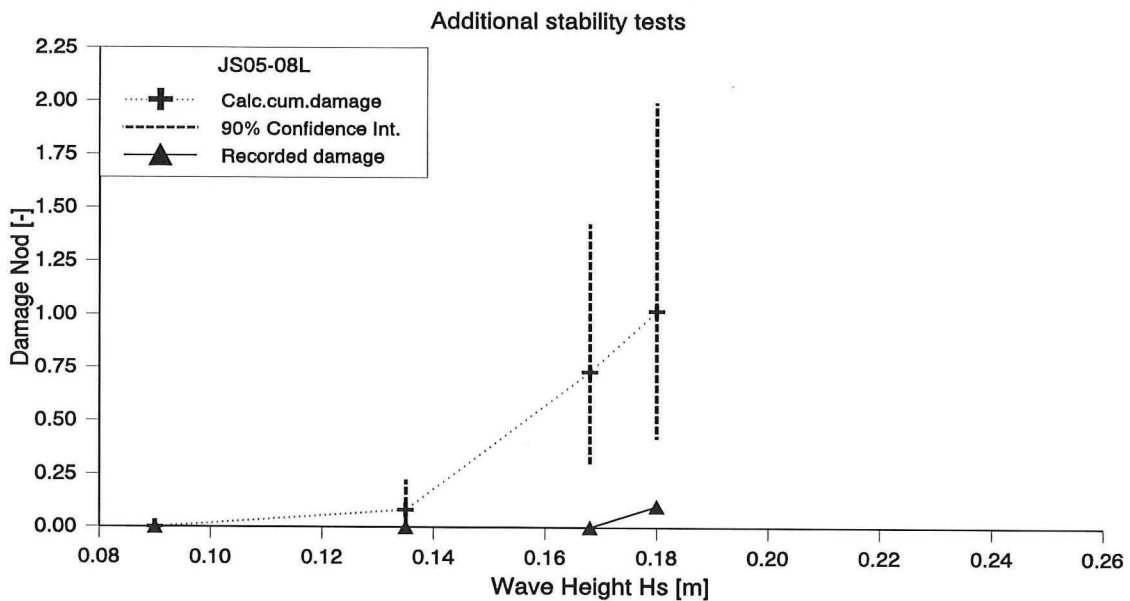
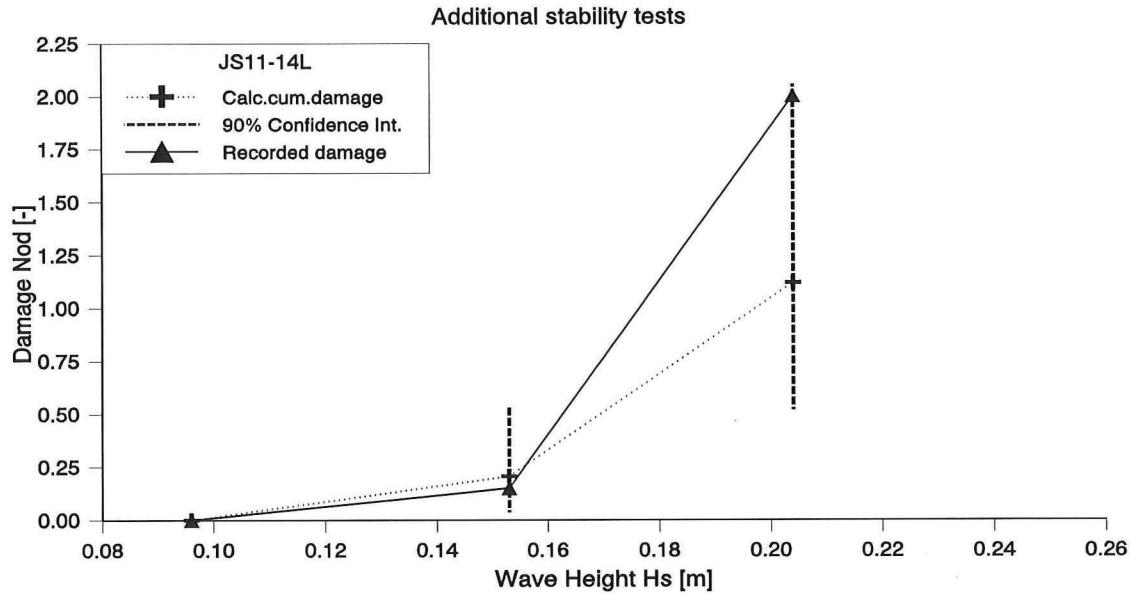


Figure 4.1 Additional stability test series JS01-JS04L and JS05-JS08L
 First comparison between calculated average damage and the recorded damage, for $h_{toe} = 0.30$ m, using H_s .
 With 90% confidence intervals on the calculated average damage.

Stability of Tetrapods



Stability of Tetrapods

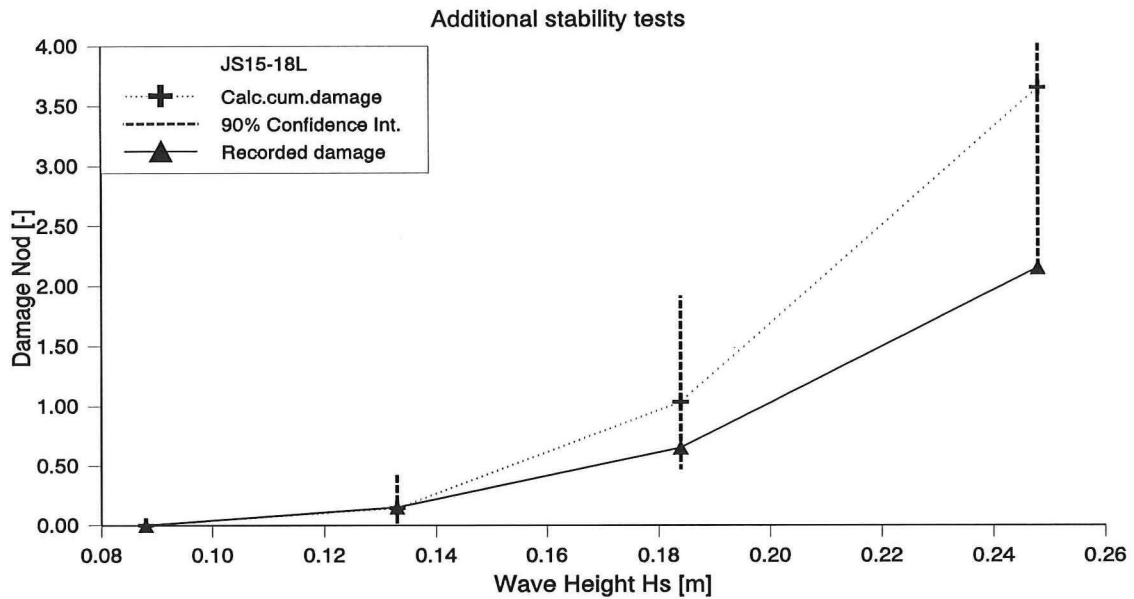


Figure 4.2 Additional stability test series JS11-JS14L and JS15-JS18L
First comparison between calculated average damage and the recorded damage, for $h_{toe} = 0.50$ m, using H_s .
With 90% confidence intervals on the calculated average damage

Stability of Tetrapod

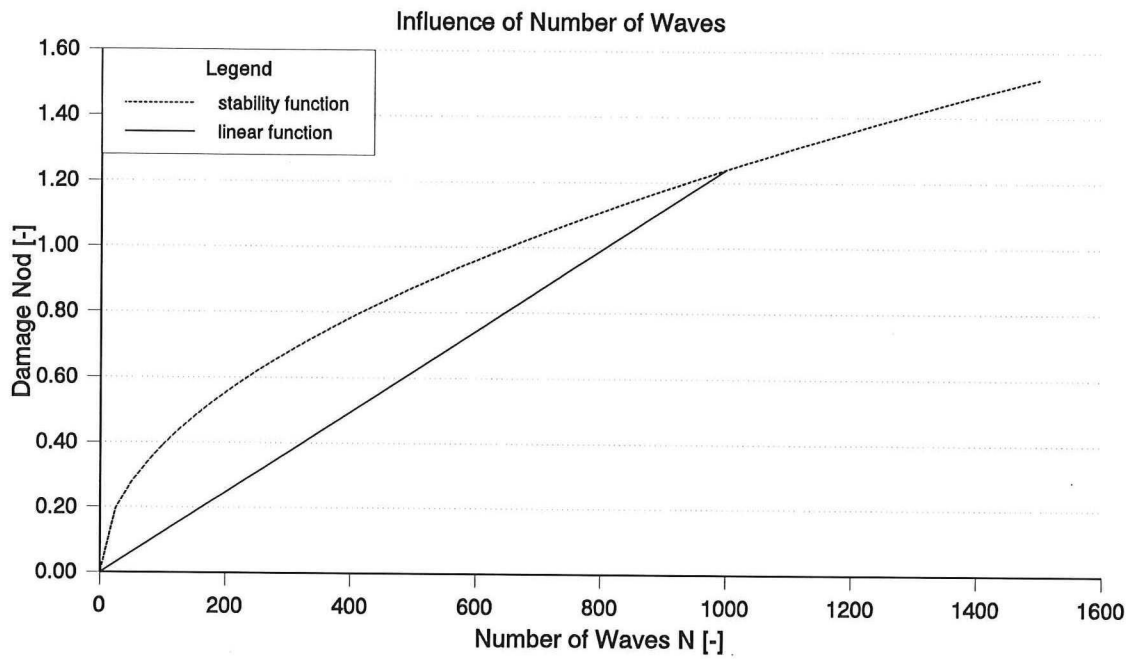
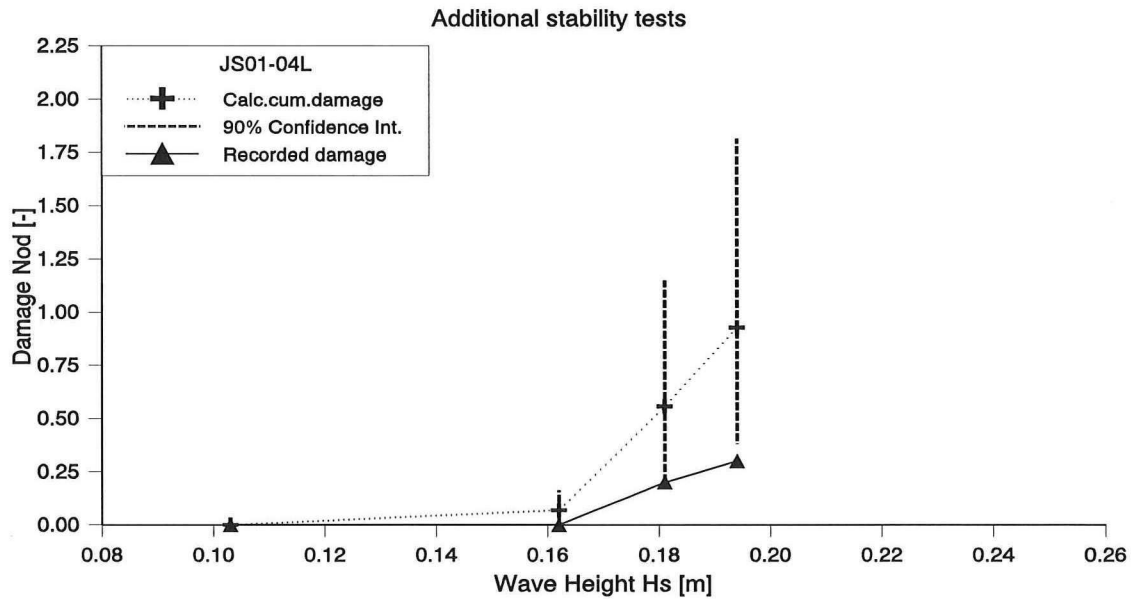


Figure 4.3 Stability formula (Eq. 3.1) and linear part between $N = 0$ and $N = 1000$ waves

Stability of Tetrapods



Stability of Tetrapods

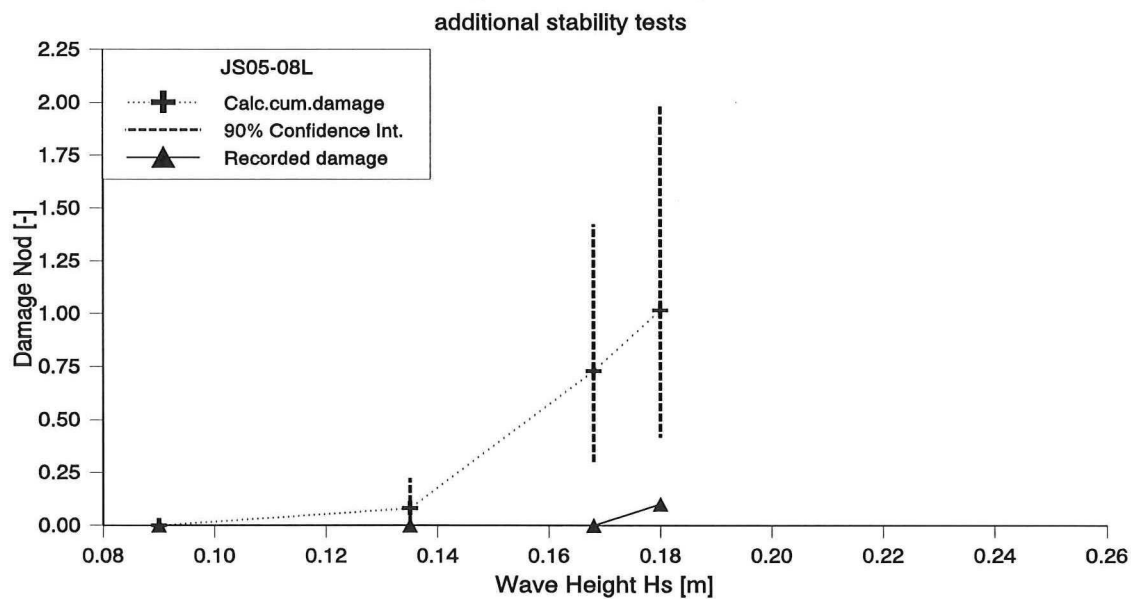


Figure 4.4 Additional stability test series JS01-JS04L and JS05-JS08L
Influence of linear interpolation, when $N < 1000$, for $h_{toe} = 0.30$ m.

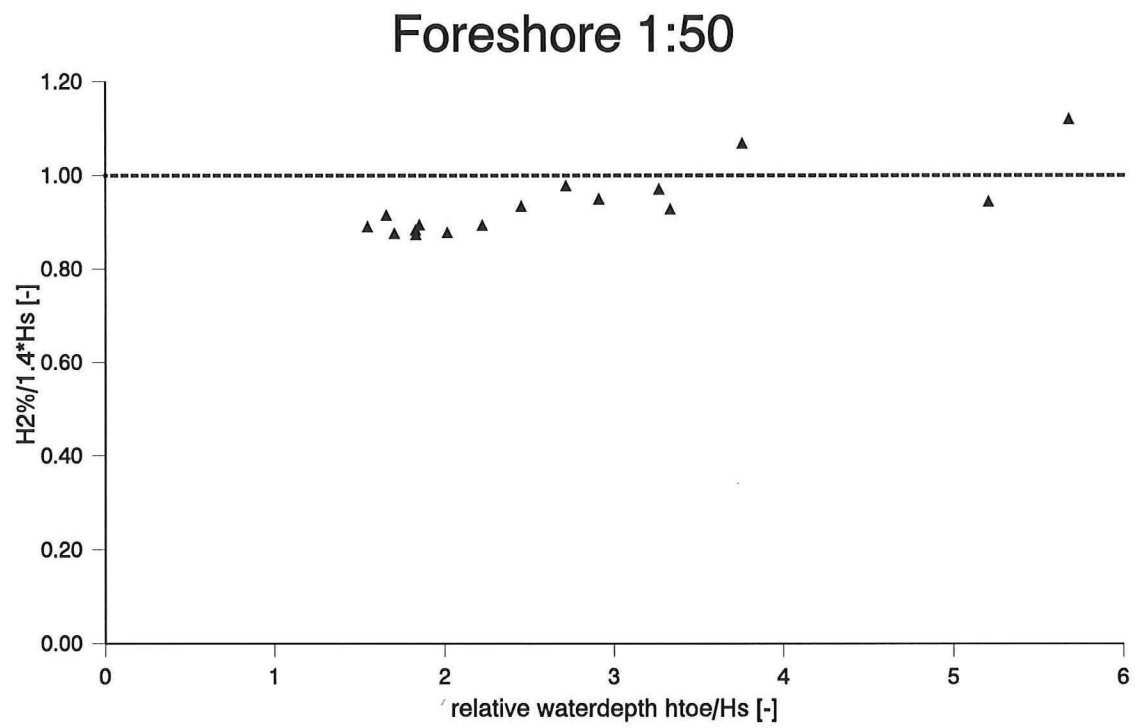
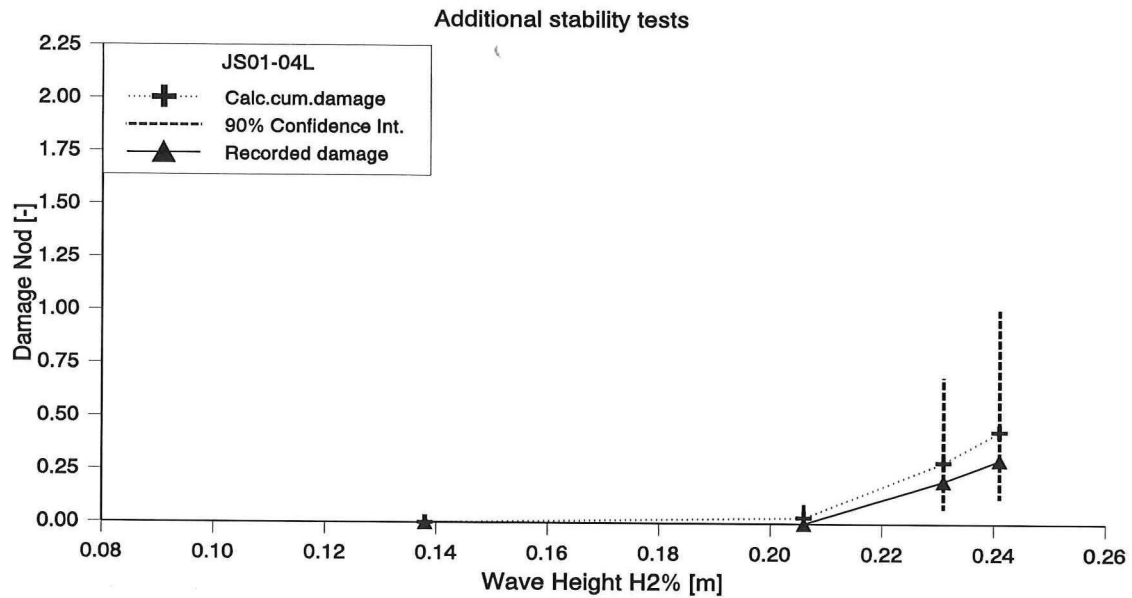


Figure 4.5 Influence of the ratio h_{toe}/H_s on the ratio $H_{2\%}/1.4 \cdot H_s$.

Stability of Tetrapods



Stability of Tetrapods

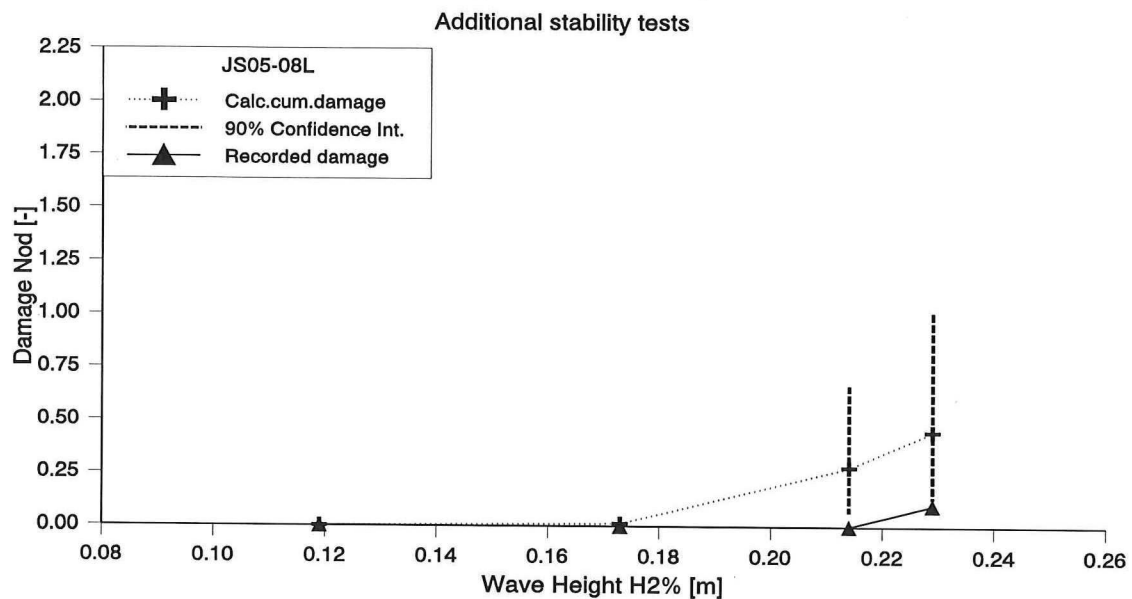
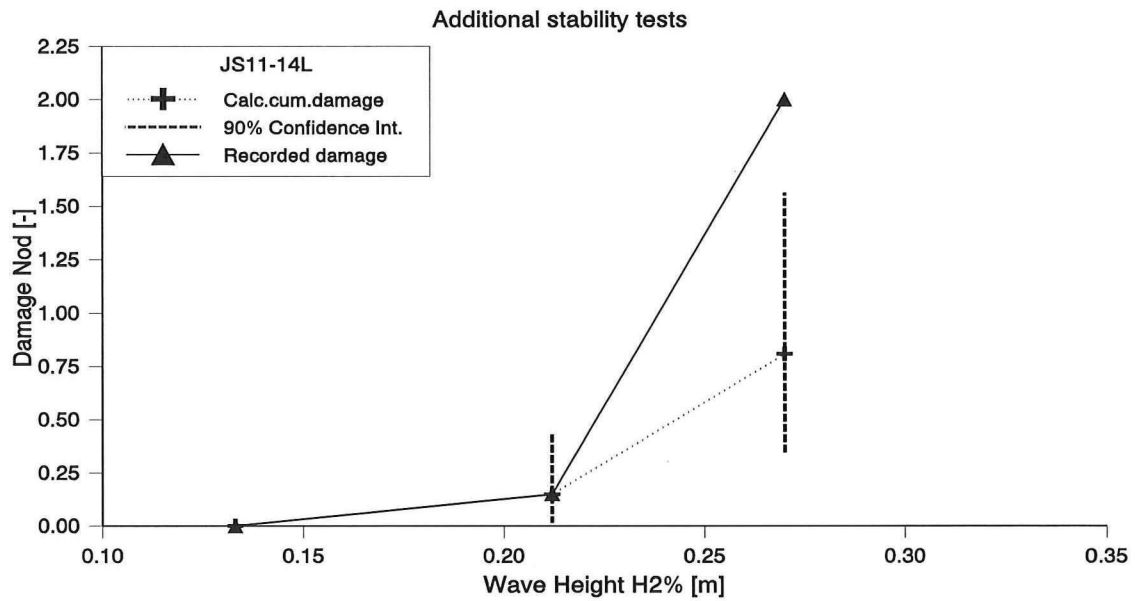


Figure 4.6 Additional stability test series JS01-JS04L and JS05-JS08L
Comparison between calculated average damage and the recorded damage,
for $h_{toe} = 0.30$ m, using $H_{2\%}$.

Stability of Tetrapods



Stability of Tetrapods

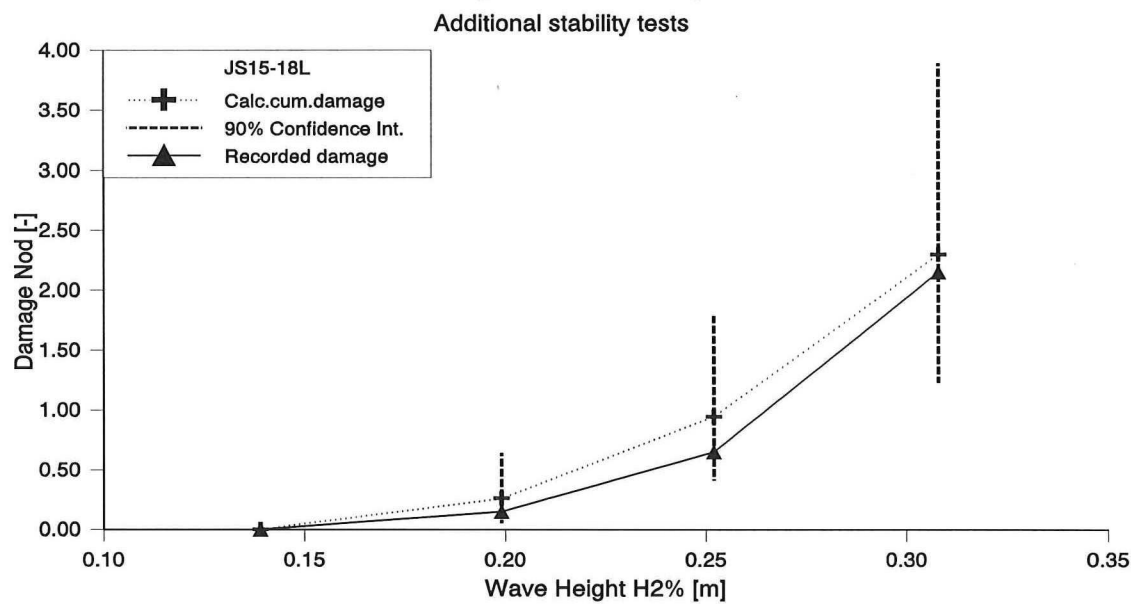
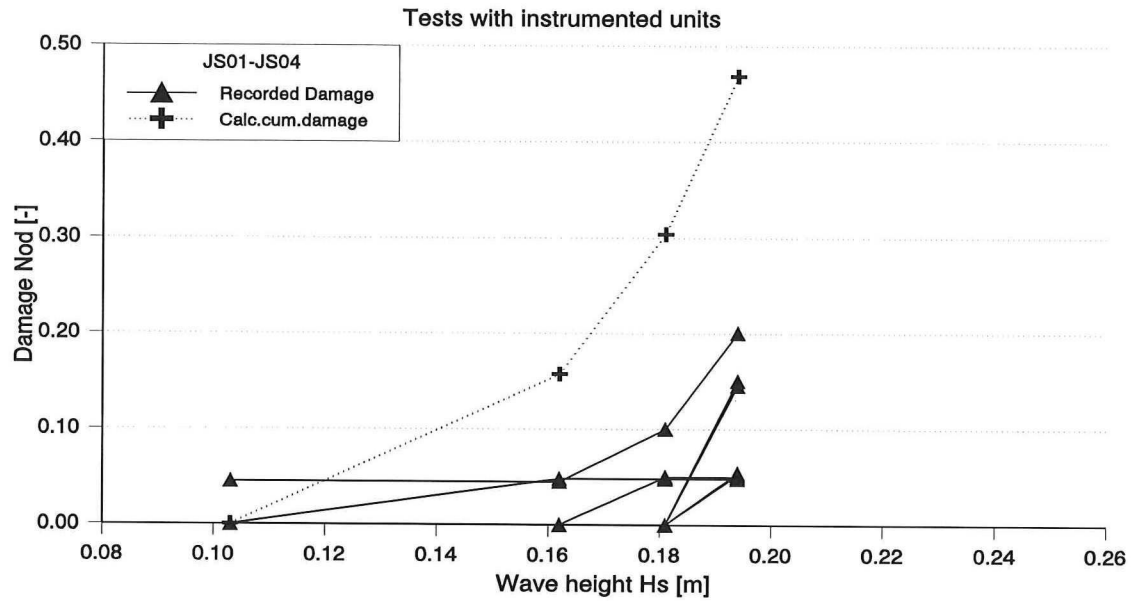


Figure 4.7 Additional stability test series JS11-JS14L and JS15-JS18L
Comparison between calculated average damage and the recorded damage, for $h_{toe} = 0.50$ m, using $H_{2\%}$.

Stability of Tetrapods



Stability of Tetrapods

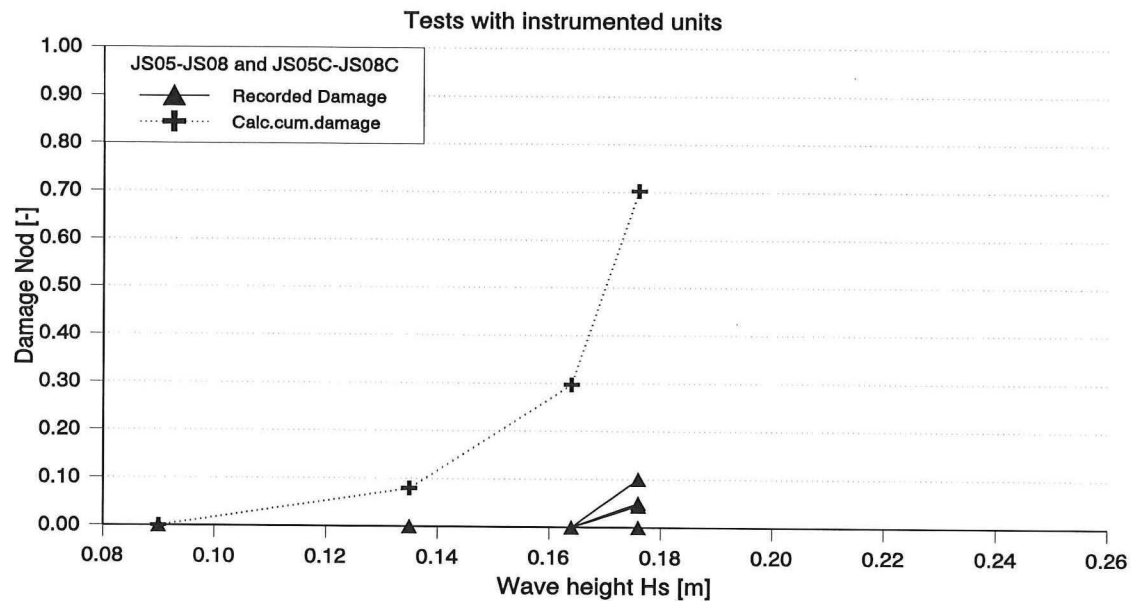
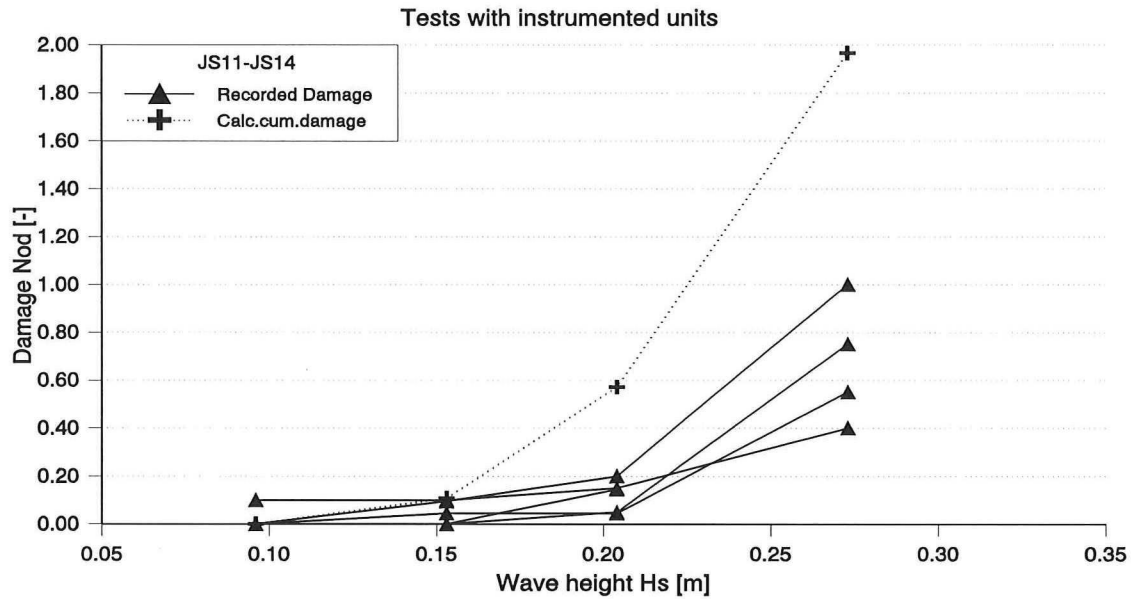


Figure 4.8 Hydraulic stability of the stress measuring tests. All repetitions are shown. Comparison of the damage recordings of the stress measuring tests with the calculated average damage for $h_{toe} = 0.30$ m. (Number of waves of each test run approx. 200)

Stability of Tetrapods



Stability of Tetrapods

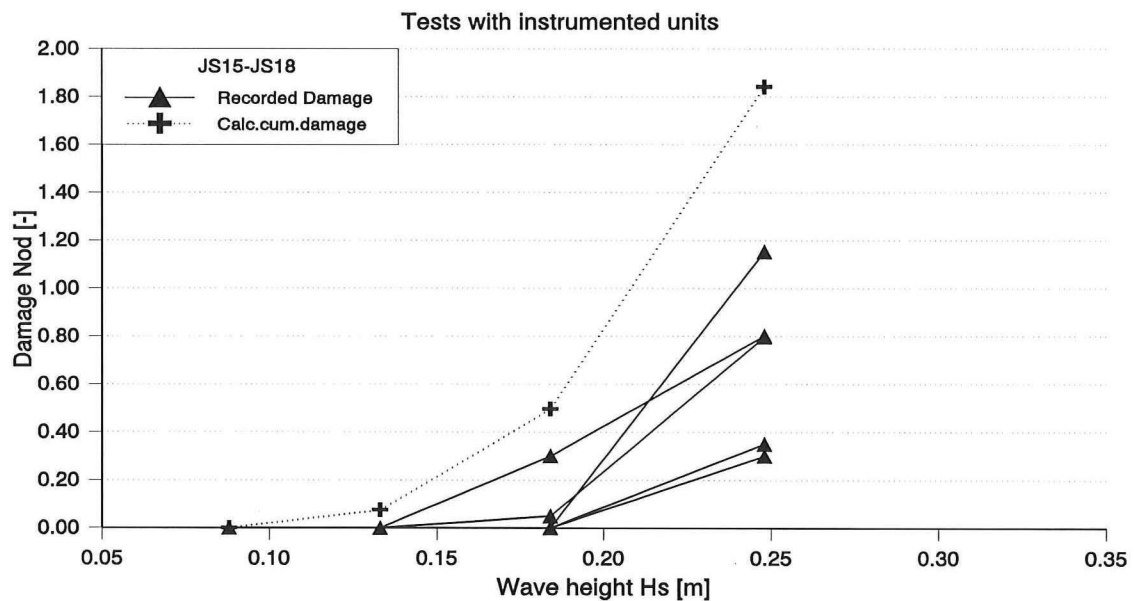
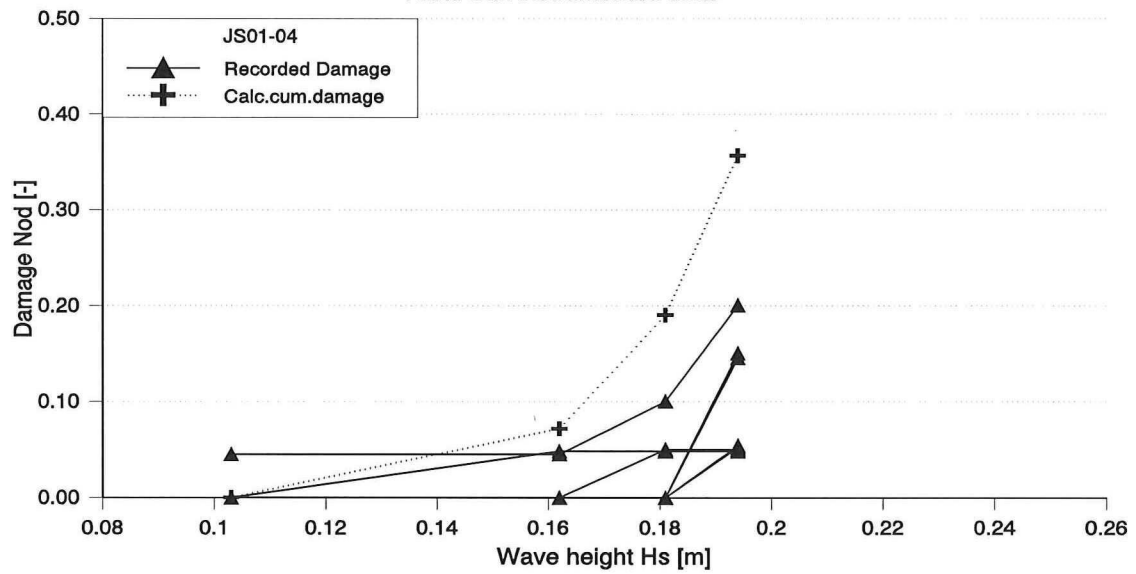


Figure 4.9 Hydraulic stability of the stress measuring tests. All repetitions are shown. Comparison of the damage recordings of the stress measuring tests with the calculated damage for $h_{toe} = 0.50$ m. (Number of waves of each test run approx. 200)

Stability of Tetrapods

Tests with instrumented units



Stability of Tetrapods

Tests with instrumented units

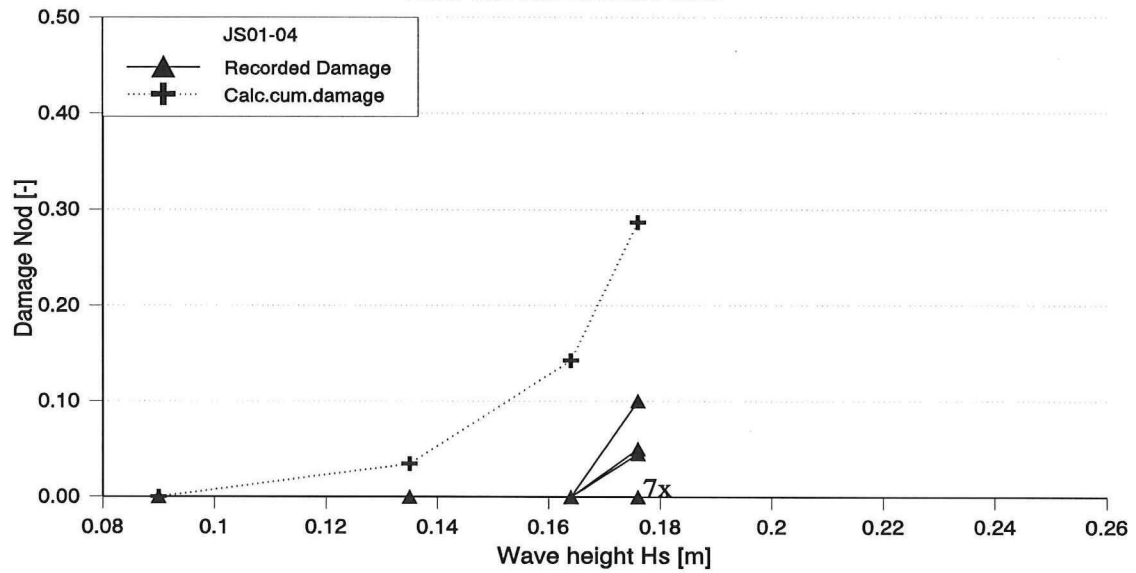
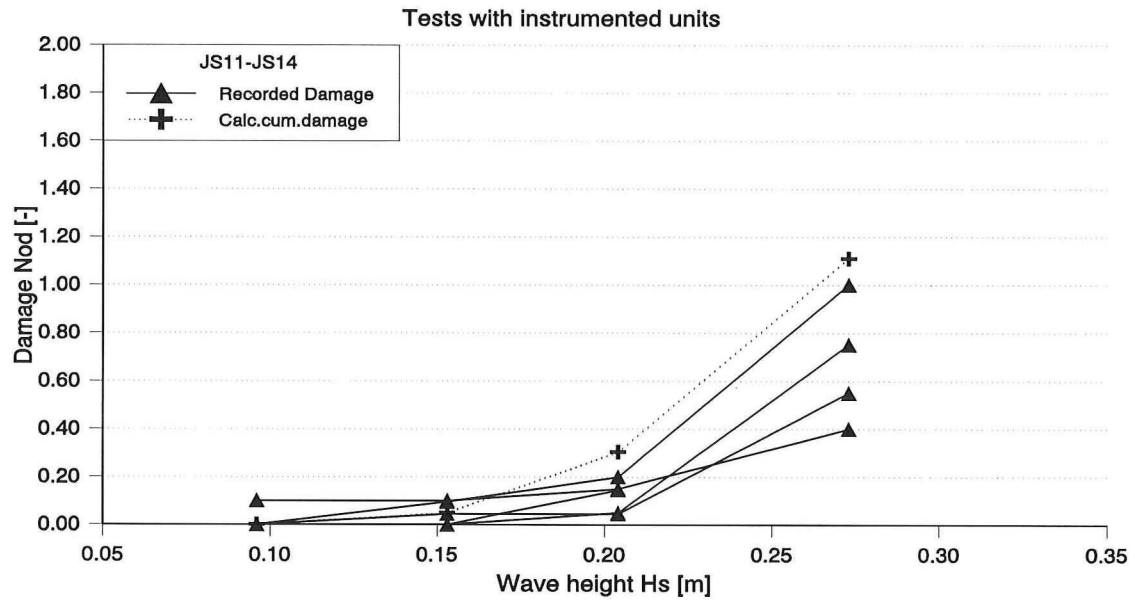


Figure 4.10 Calculated damage, based on H_s and linear interpolated, and recorded damage for $h_{toe} = 0.30$ m.

Stability of Tetrapods



Stability of Tetrapods

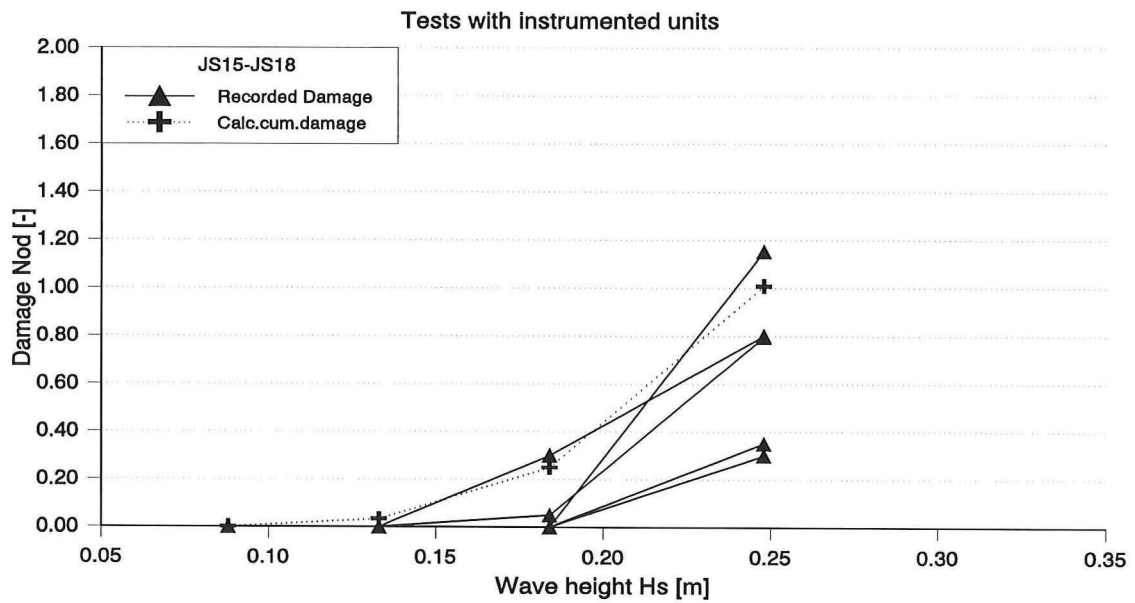
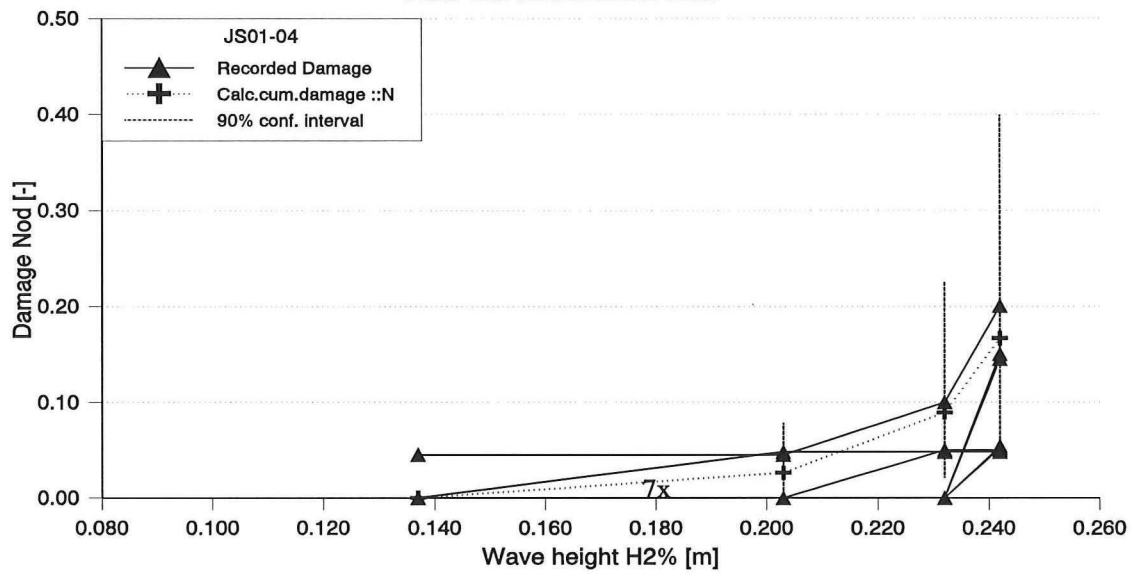


Figure 4.11 Calculated damage, based on H_s and linear interpolated, and recorded damage for $h_{toe} = 0.50$ m.

Stability of Tetrapods

Tests with instrumented units



Stability of Tetrapods

Tests with instrumented units

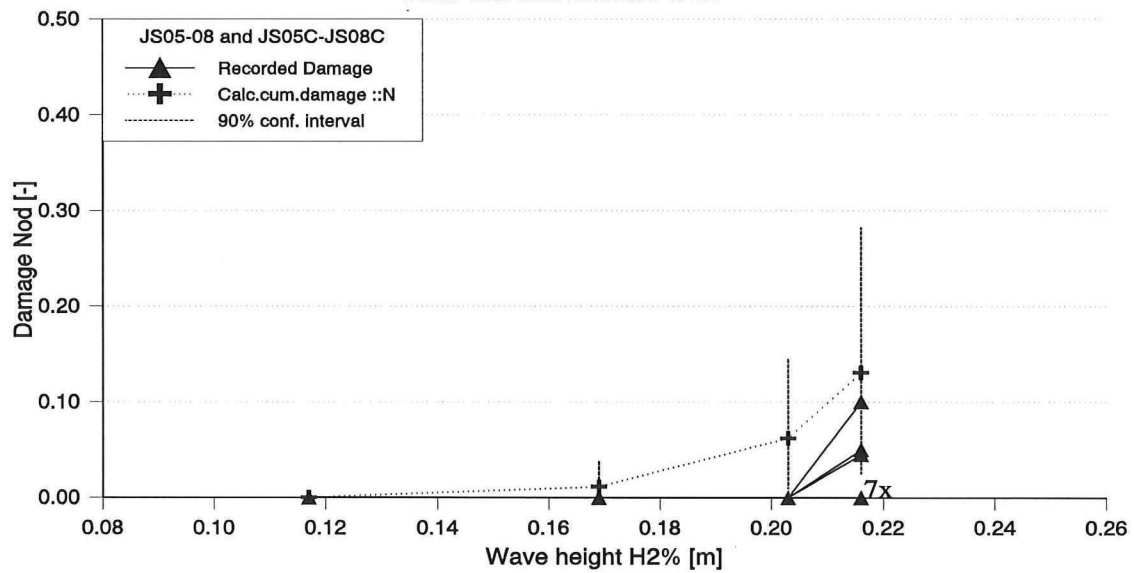
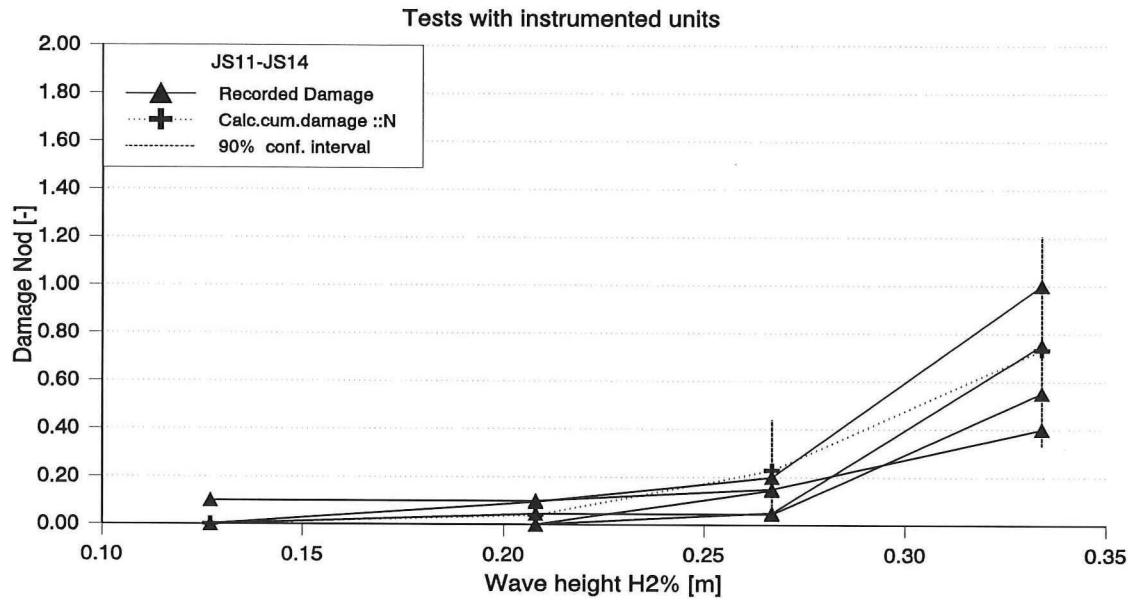


Figure 4.12 Comparison between calculated and recorded damage, for $h_{toe} = 0.30$ m, using $H_{2\%}$. (Number of waves of each test run approx. 200)

Stability of Tetrapods



Stability of Tetrapods

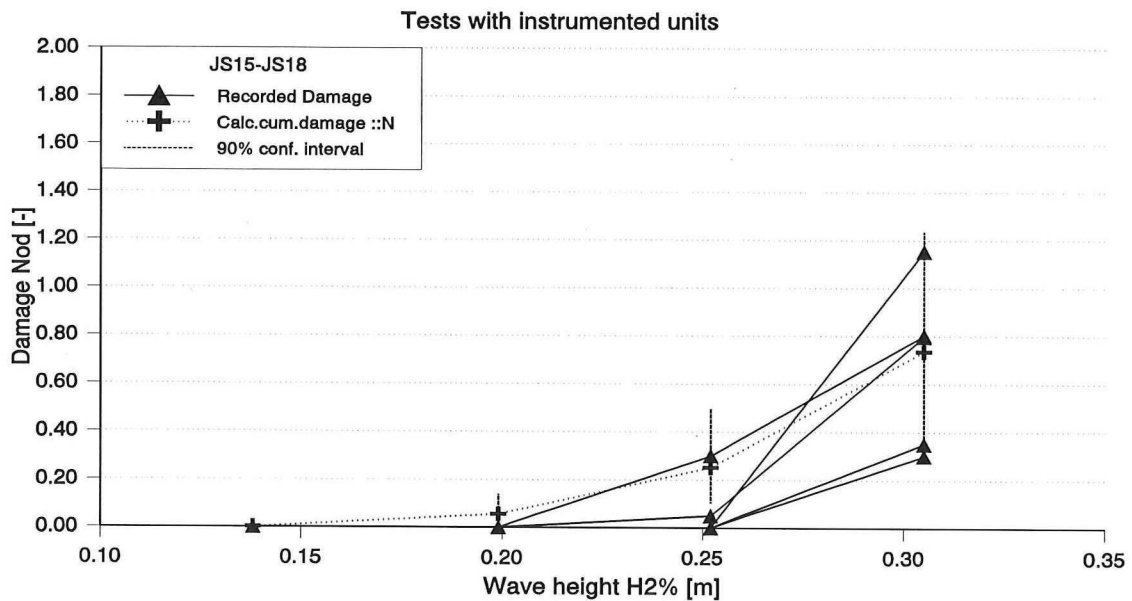


Figure 4.13 Comparison between calculated and recorded damage, for $h_{toe} = 0.50$ m, using $H_{2\%}$. (Number of waves of each test run approx. 200)

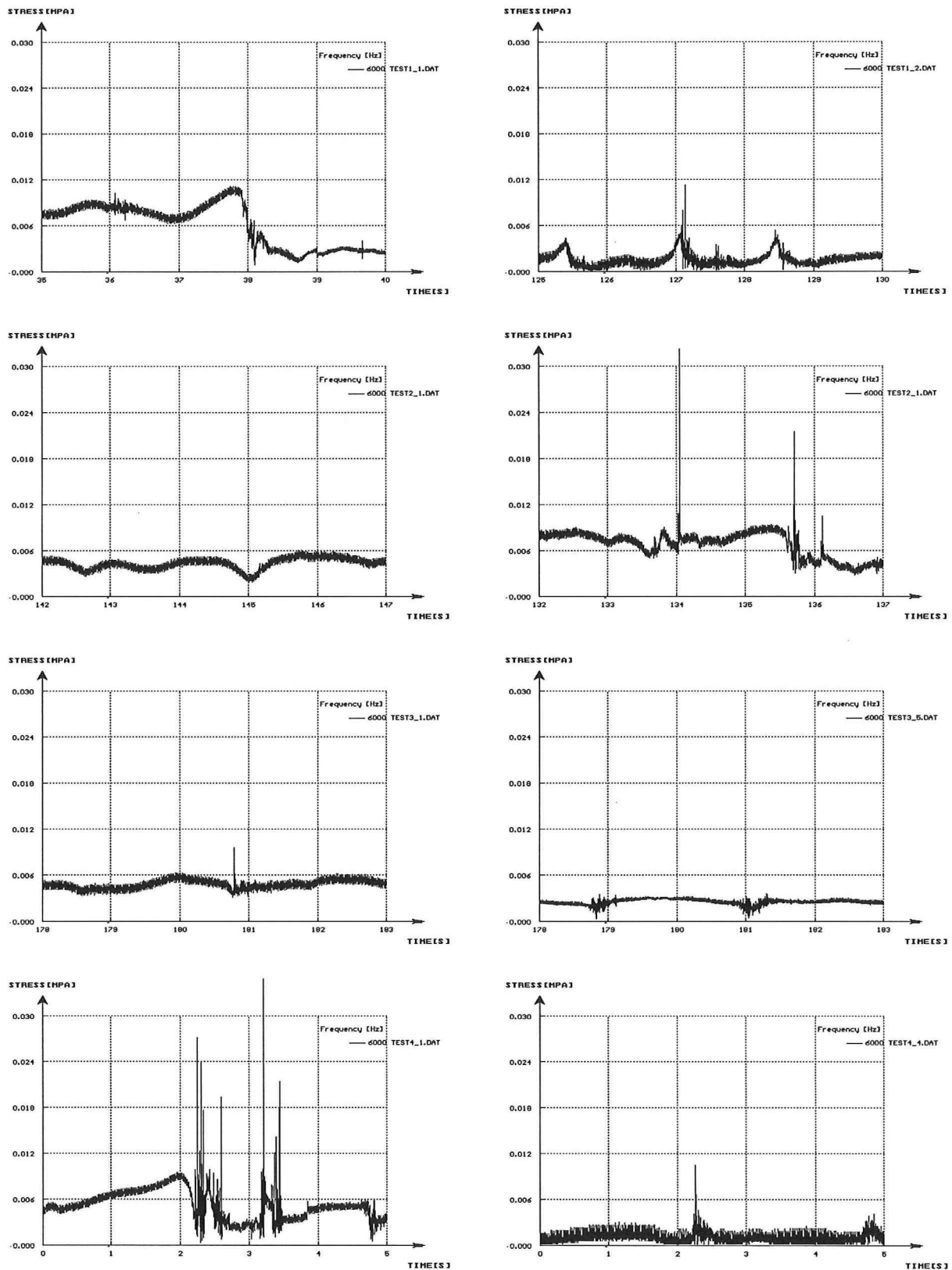


Figure 5.1 Examples of stress signals.

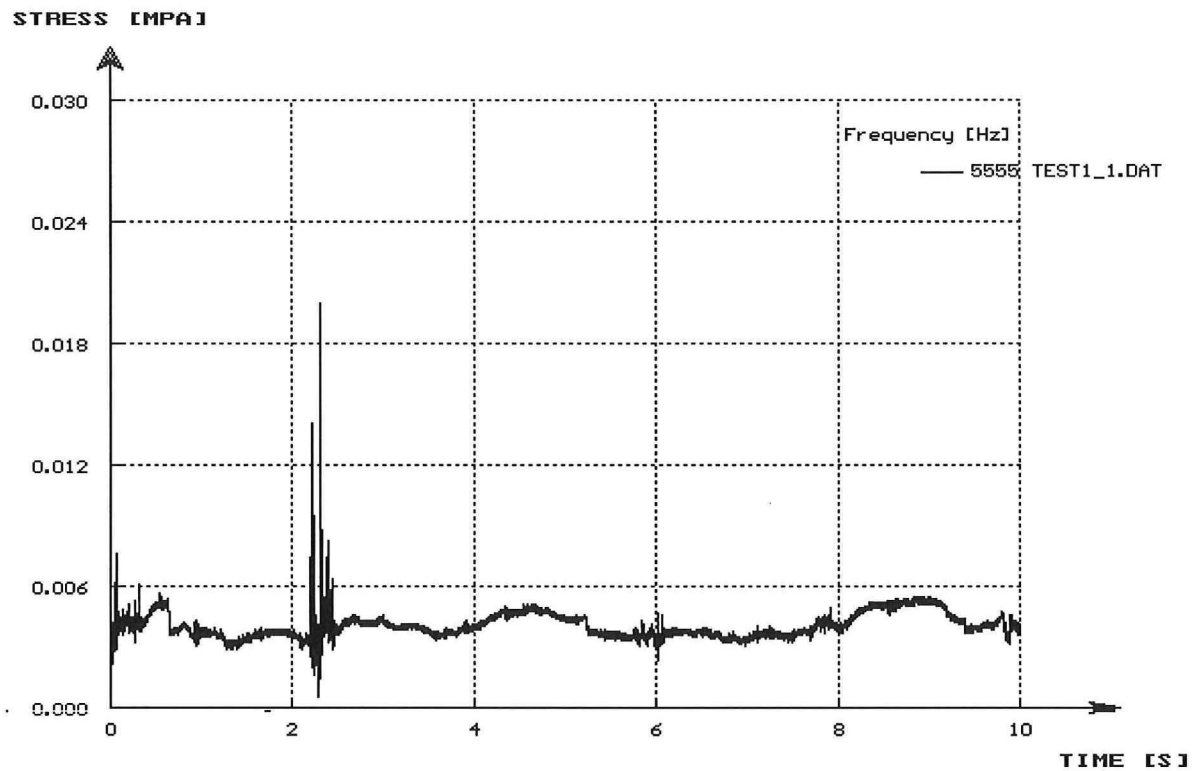


Figure 5.2 Example of AUC stress signal.

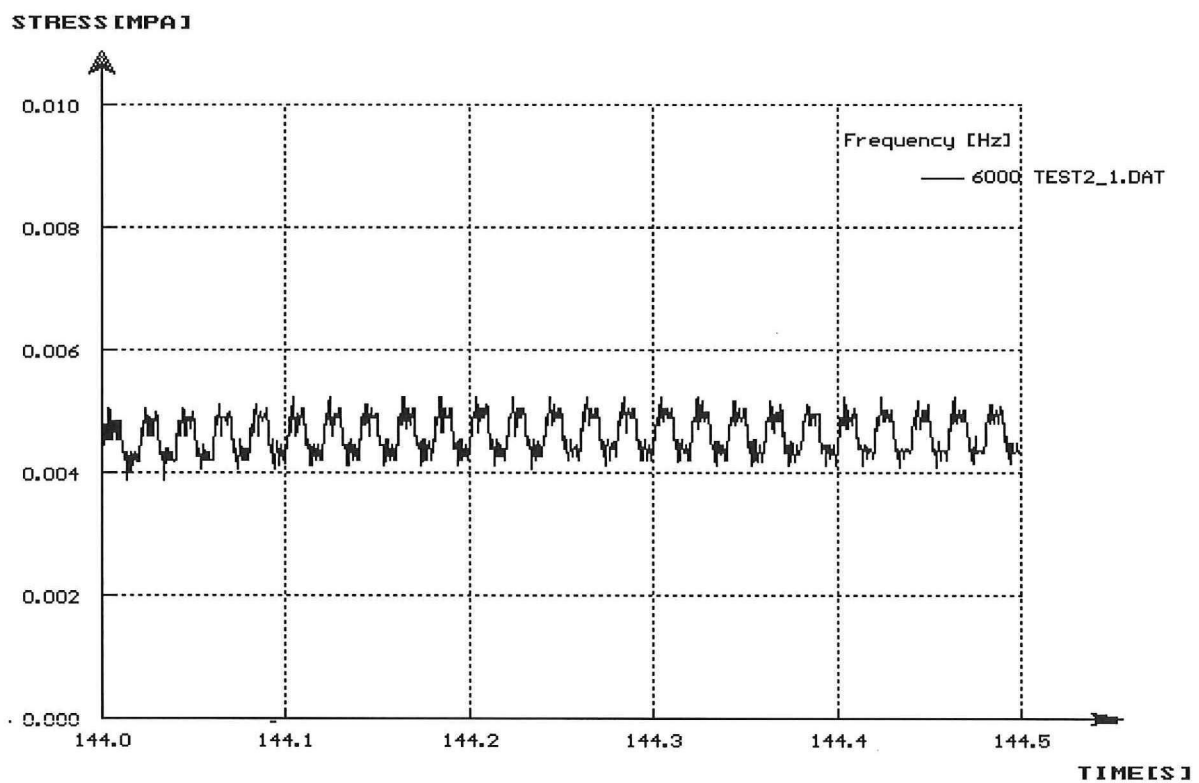


Figure 5.3 Enlargement of part of a DH stress signal

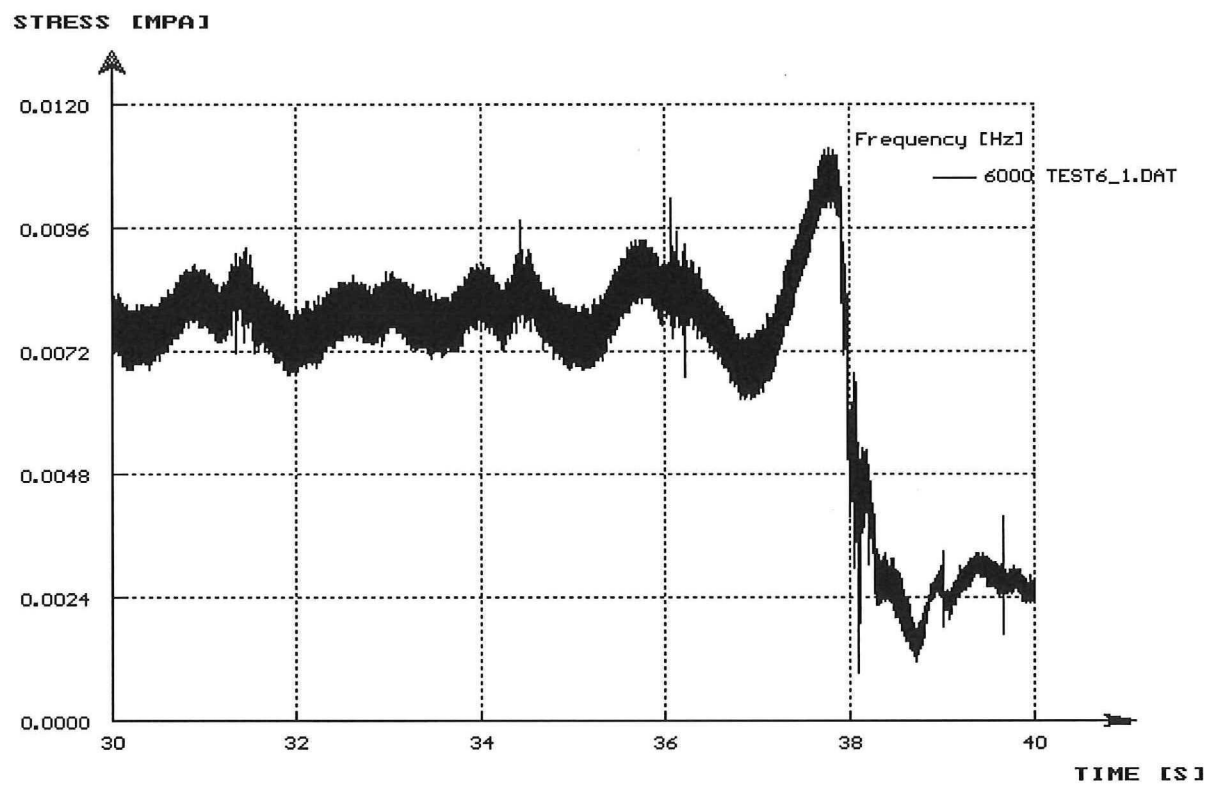


Figure 5.4 Example of sudden decline in DH stress signal

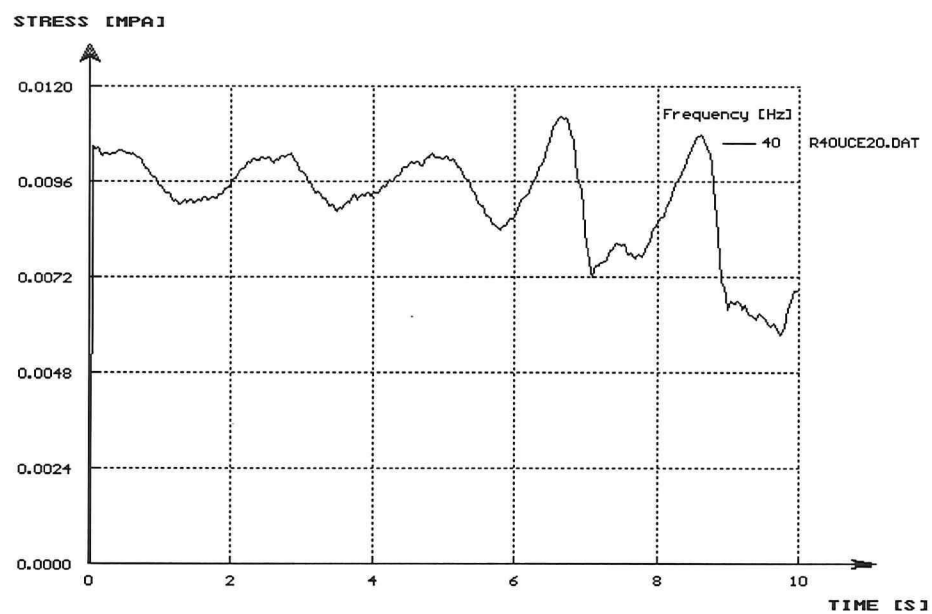
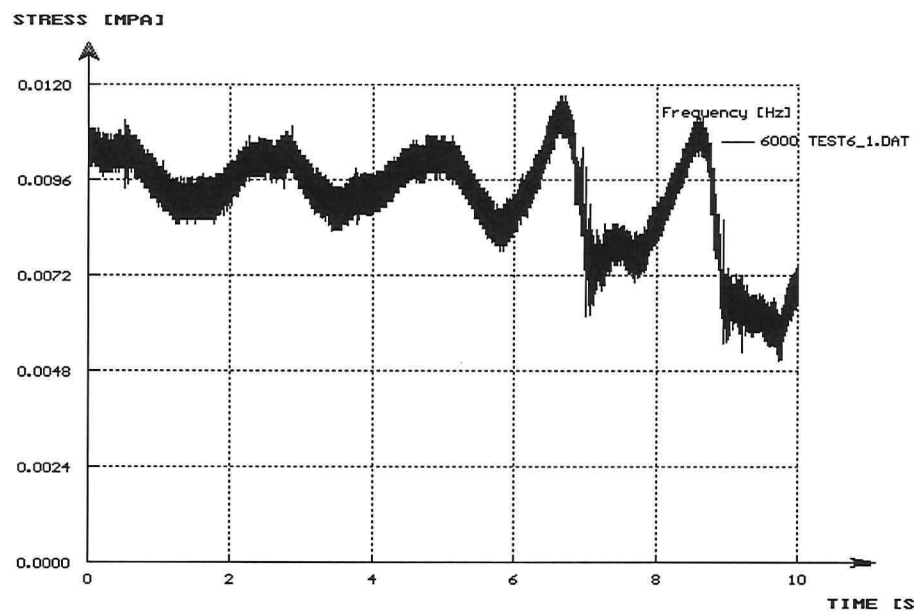
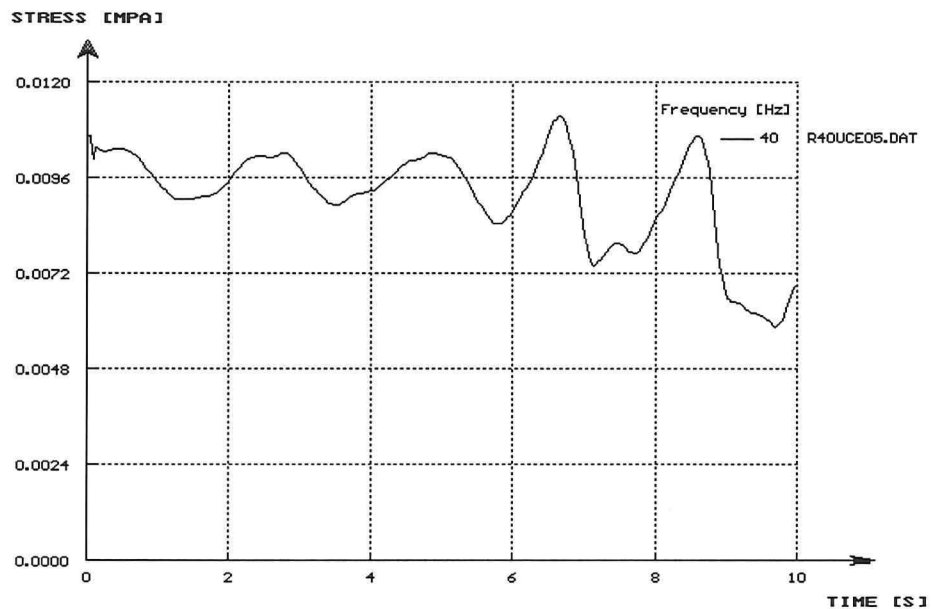
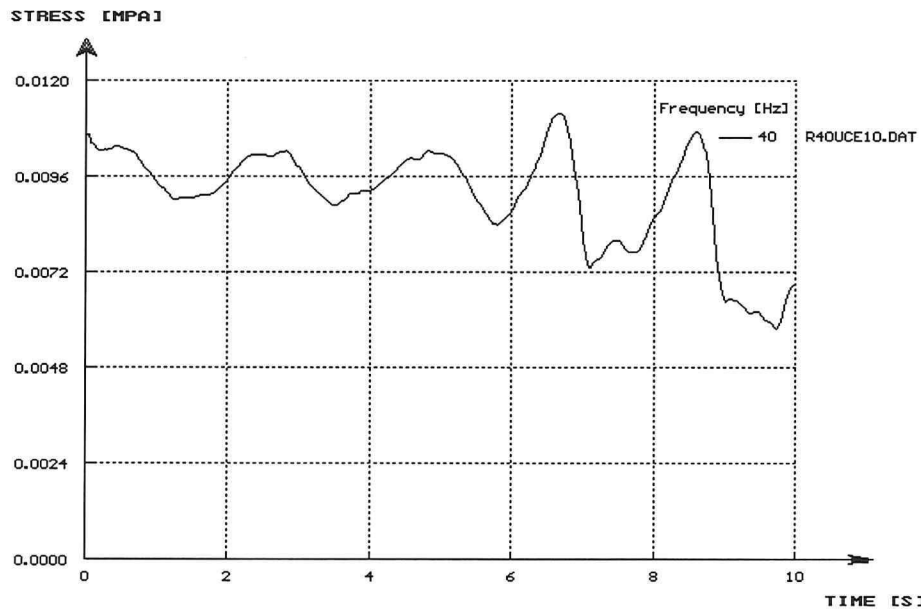


Figure 5.5 (a) part of a original signal. (b-d) Signal filtered at 20, 10 and 5 Hz resp.



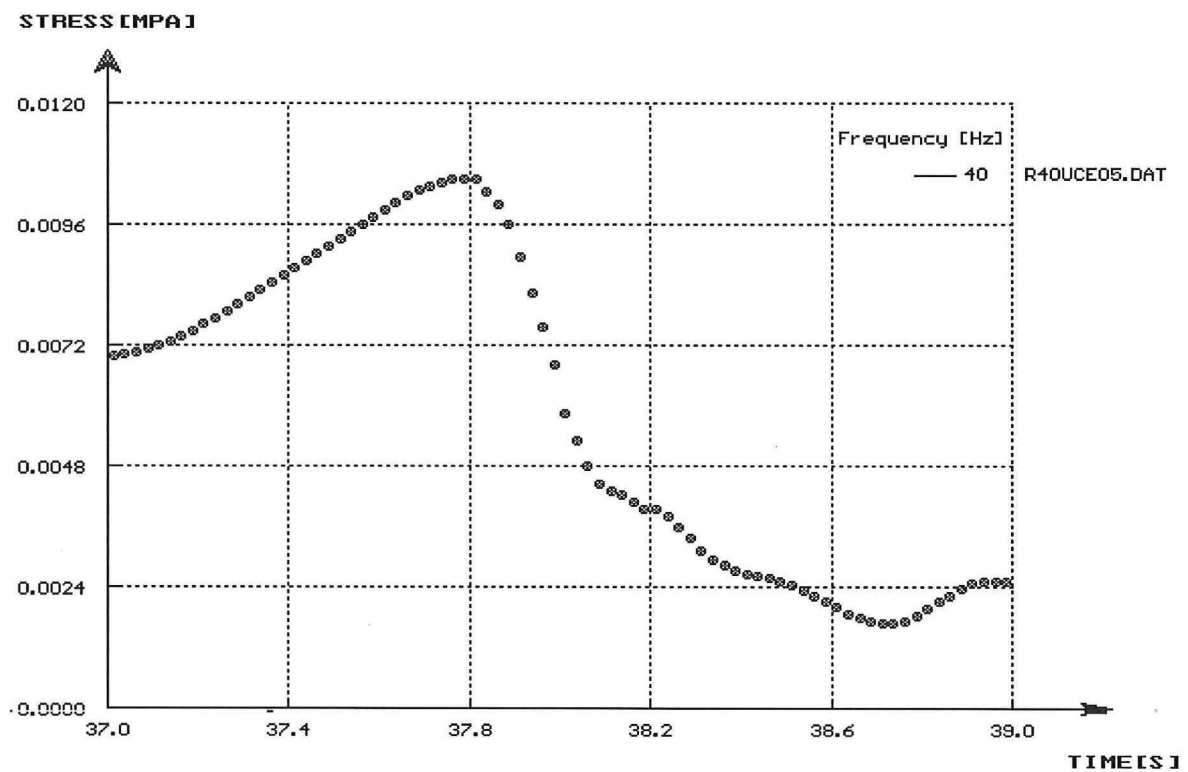
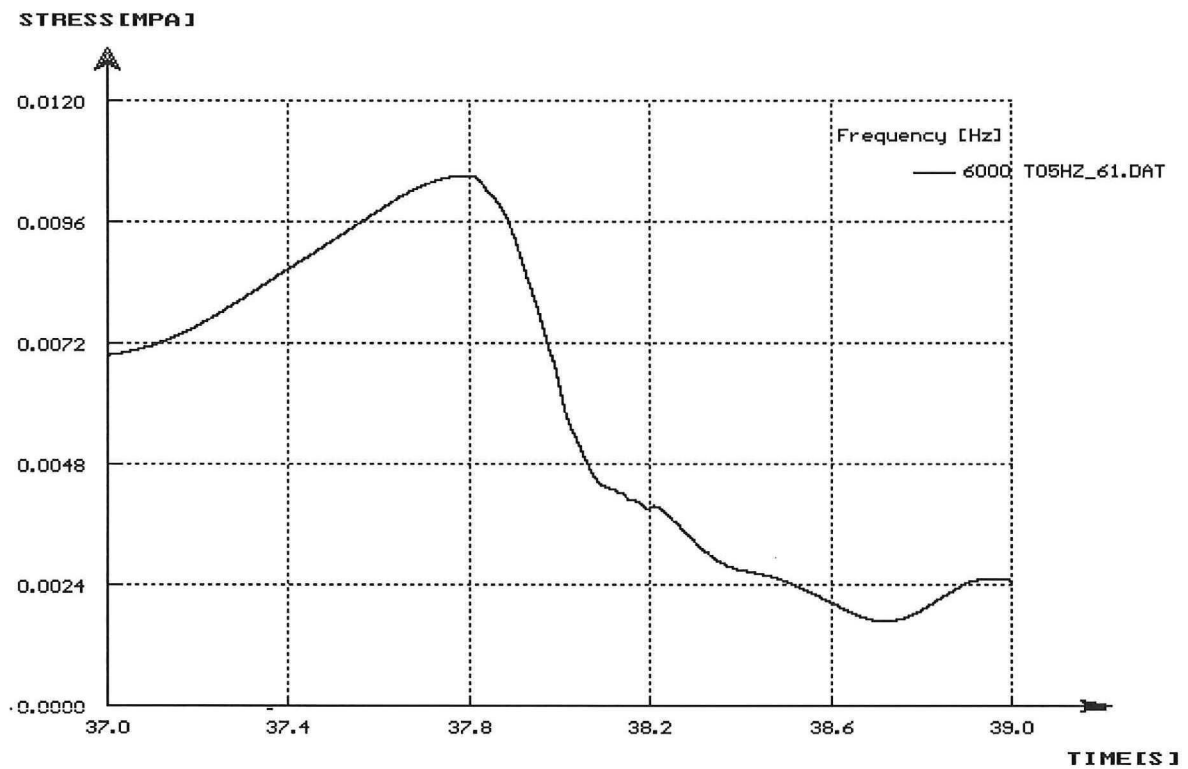


Figure 5.6 Reduction of the number of points from 6000 (top) to 40 (bottom) points per second

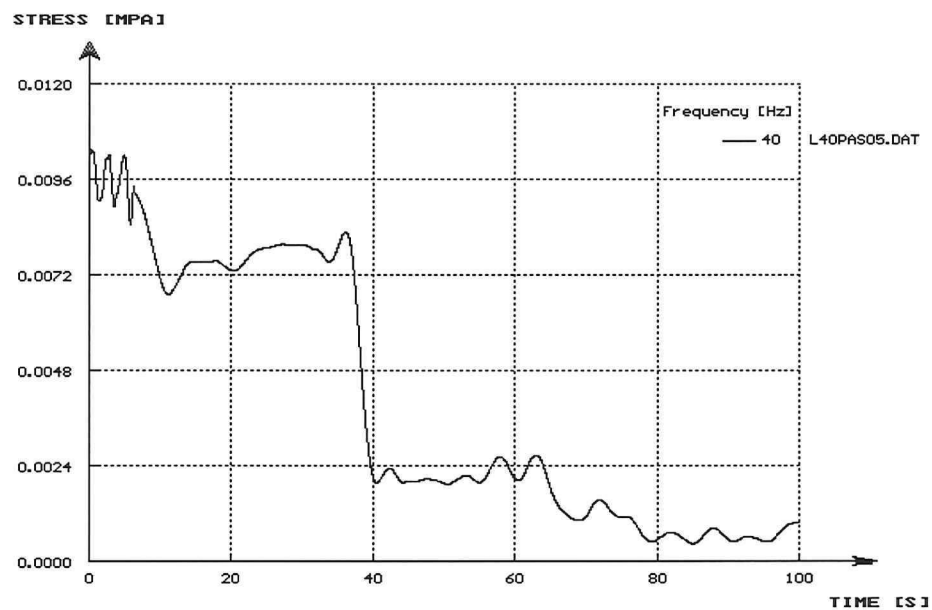
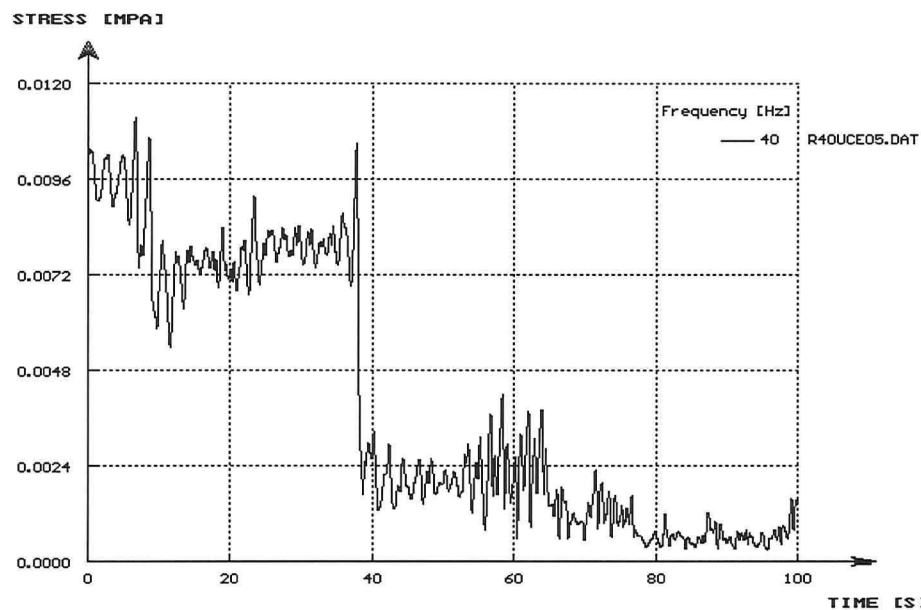
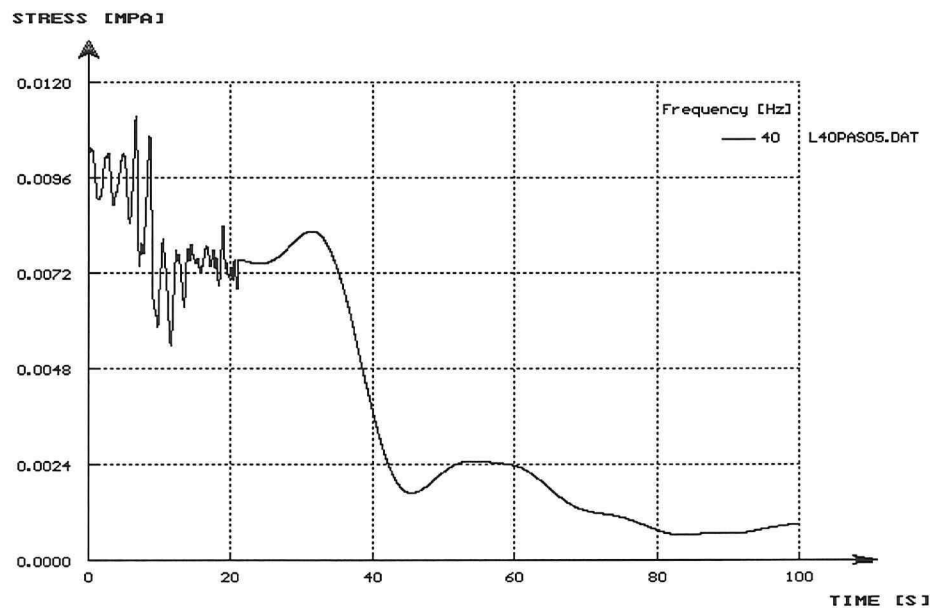
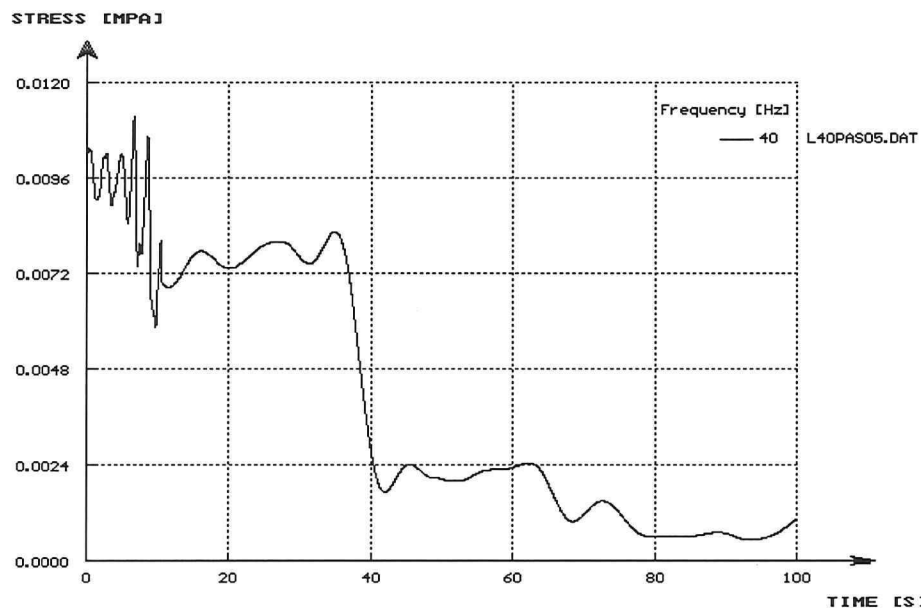


Figure 5.7 (a) Example of smoothed signal. (b-d) Moving average determined with a window size of 3, 5 and 10 times T_m resp.



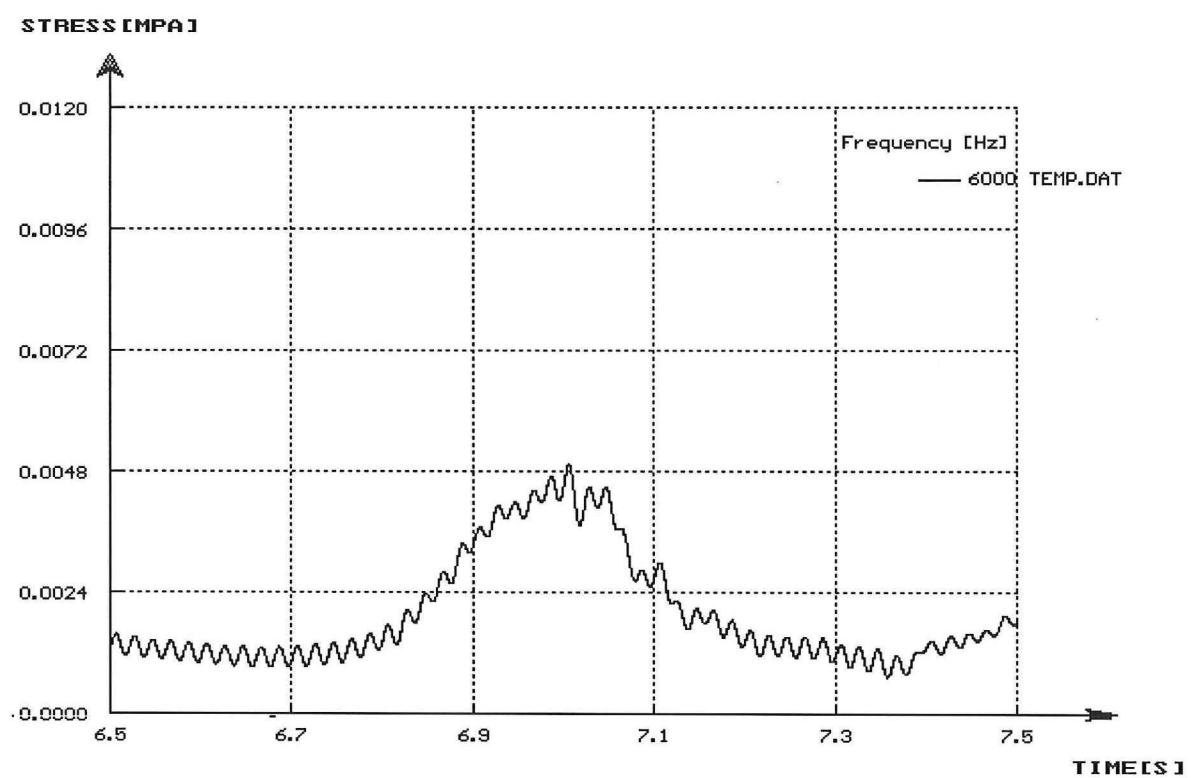
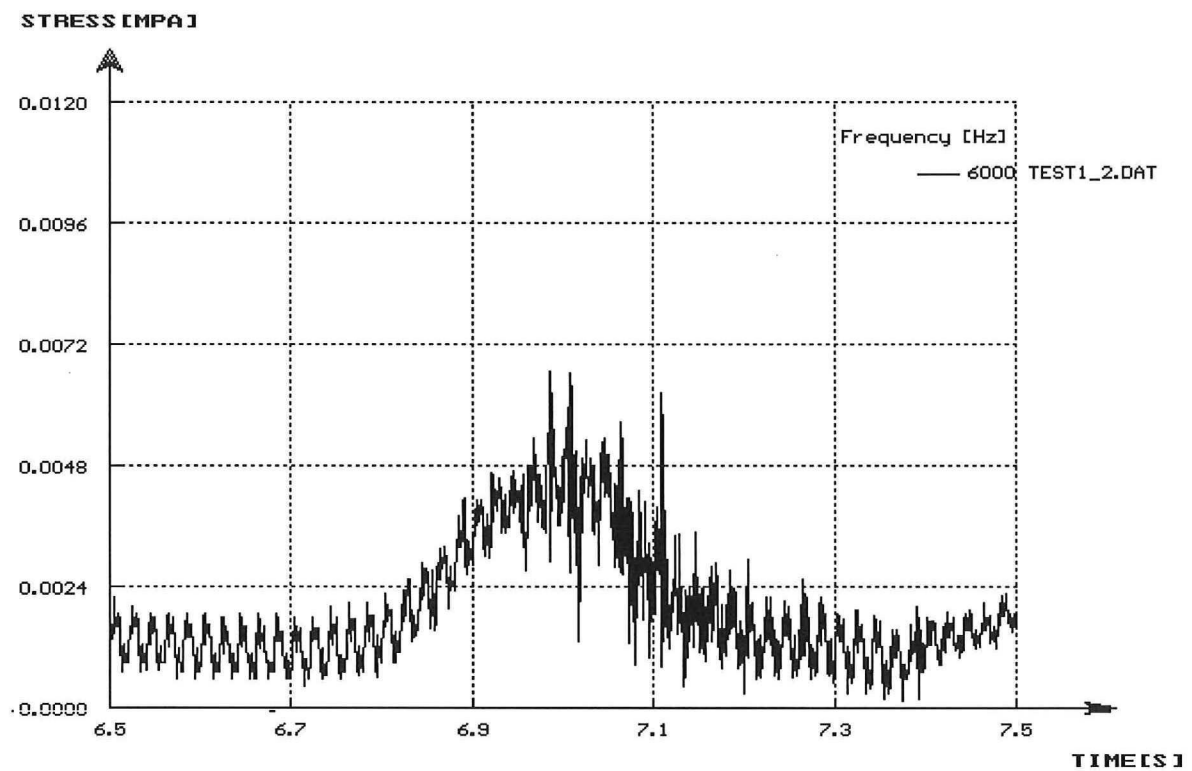


Figure 5.8 Application of a 50 Hz low-pass filter

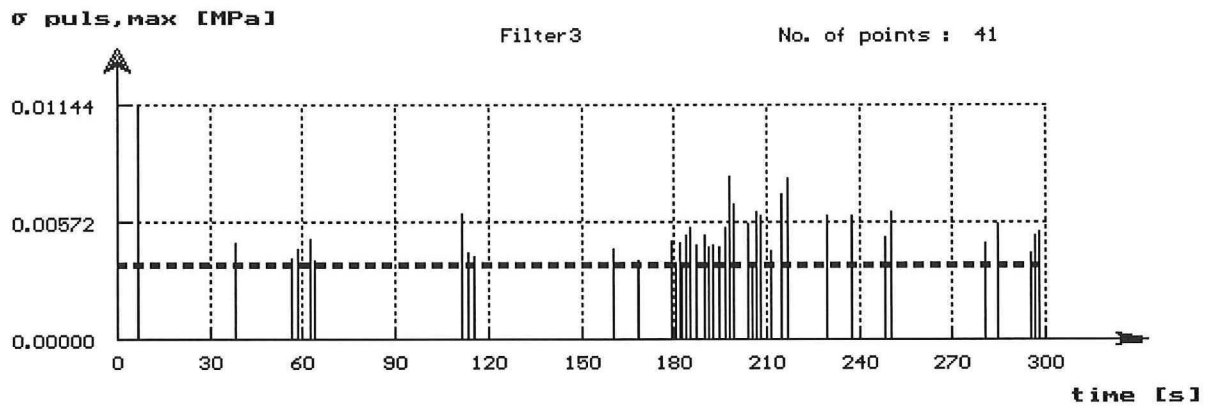


Figure 5.9 Distribution of quasi static maxima in time, found using first method of analyzing (Tetrapod 1; hydraulic test setup described in chapter 3.1)

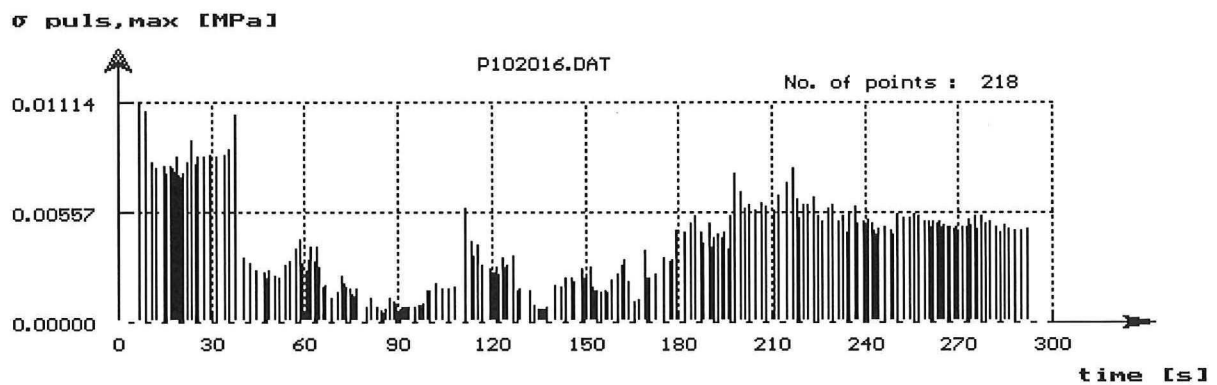


Figure 5.10 Distribution of quasi static maxima in time using new method of analyzing. (Tetrapod 1; hydraulic set up described in chapter 3.1)

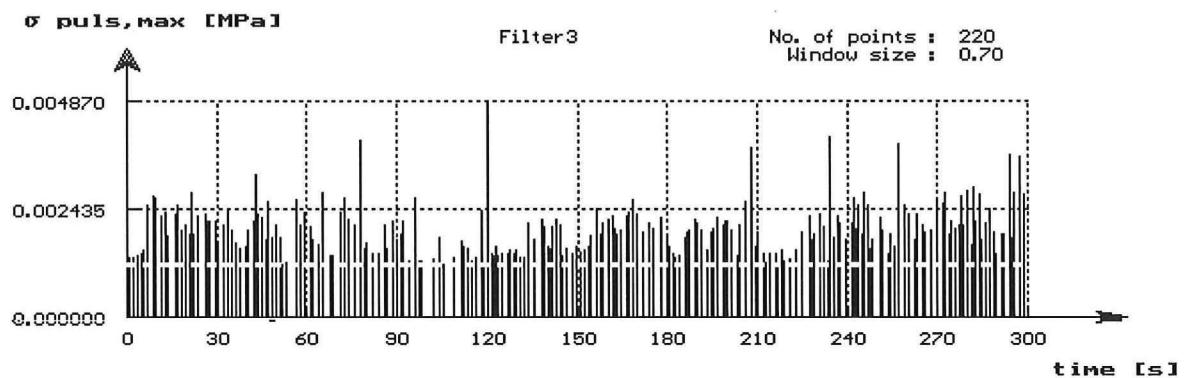


Figure 5.11 Example of an even distribution of quasi static maxima in time found using the first method of analyzing.

Parameters of fitted LN distribution

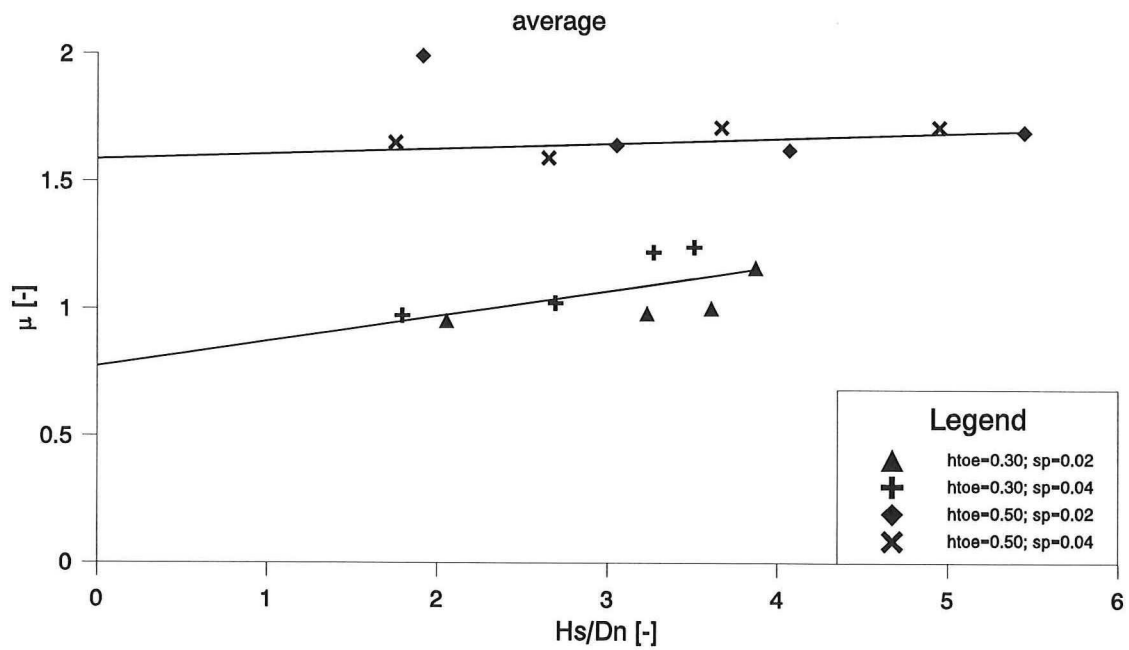


Figure 6.1 Average of LN distribution as a function of H_s/D_n

Parameters of fitted LN distribution

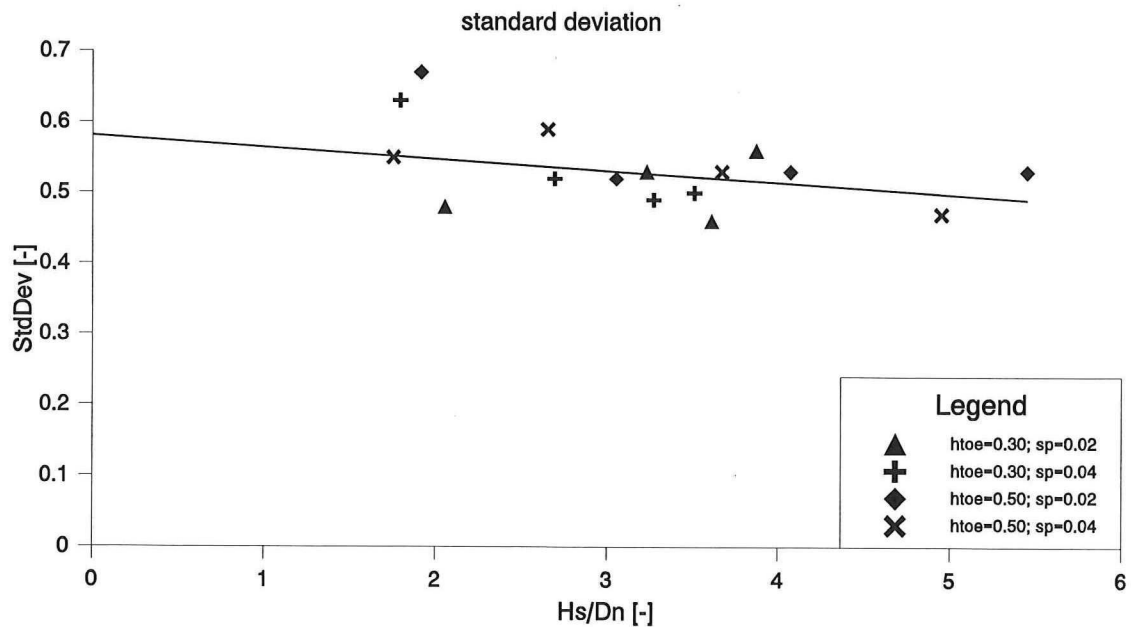


Figure 6.2 Standard Deviation of LN distribution as a function of H_s/D_n

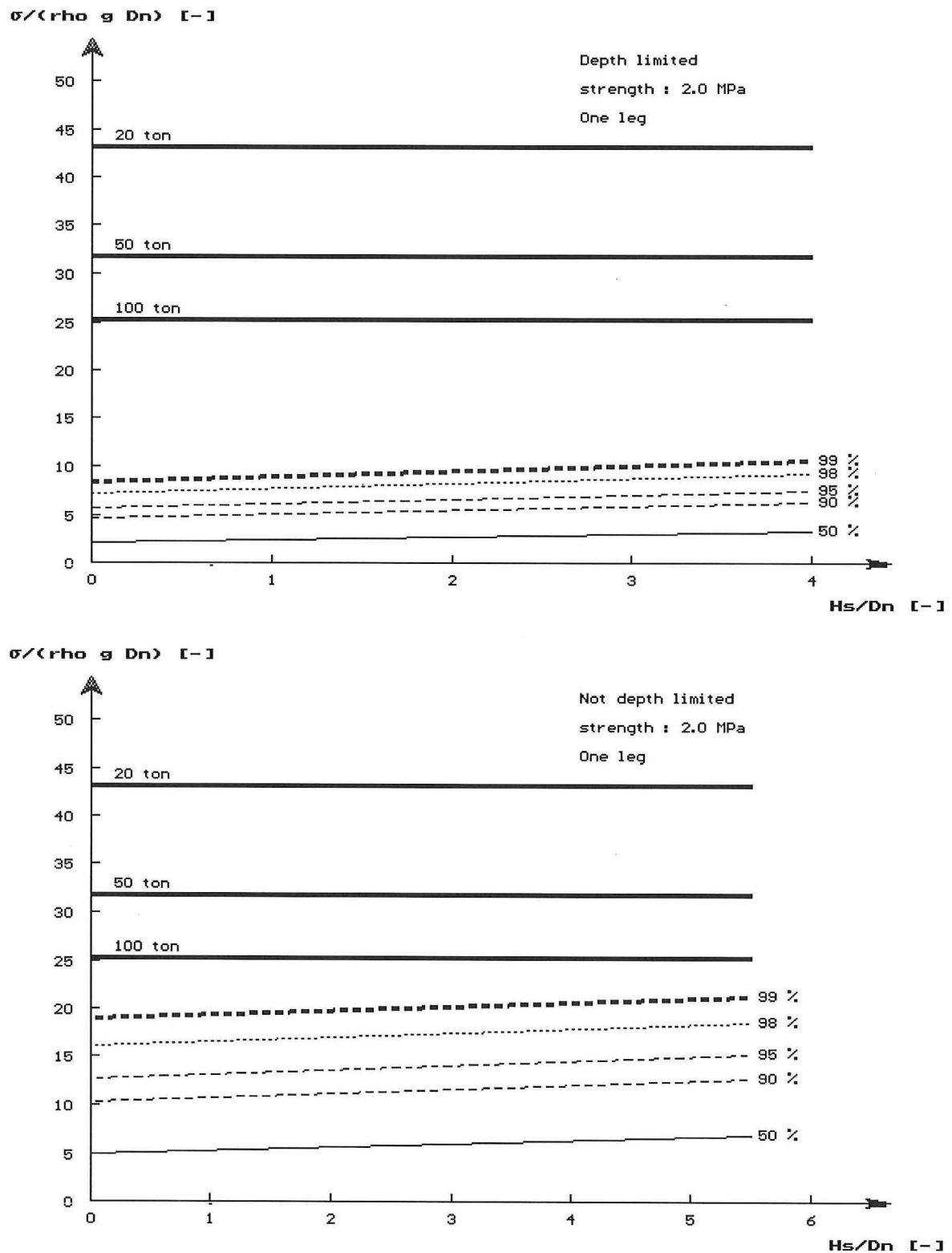


Figure 6.3 Preliminary design chart based on one leg of a tetrapod for both depth limited situations (top) and situations not influenced by the water depth (bottom).
The horizontal lines represent the tensile strength of various prototypes.

Appendices

Table A.1 : $h_{\text{toe}} = 0.30 \text{ m}$

Near the wave board		(water depth = 0.70 m)			
		JS01	JS02	JS03	JS04
H_s	[m]	0.10	0.15	0.20	0.25
T_p	[s]	1.8	2.2	2.5	2.8
s_p	[-]	0.020	0.020	0.020	0.020
T_m	[s]	1.4	1.7	2.0	2.2
s_m	[-]	0.032	0.032	0.033	0.033
At the toe of the structure		(waterdepth = 0.30 m)			
		JS01	JS02	JS03	JS04
H_s	[m]	0.103	0.162	0.181	0.194
$H_s/\gamma D_n$	[-]	1.6	2.5	2.8	3.0
s_{om}	[-]	0.033	0.034	0.030	0.025
Near the wave board		(water depth = 0.70 m)			
		JS05	JS06	JS07	JS08
H_s	[m]	0.10	0.15	0.20	0.25
T_p	[s]	1.3	1.5	1.8	2.0
s_p	[-]	0.038	0.043	0.040	0.040
T_m	[s]	1.0	1.2	1.4	1.6
s_m	[-]	0.061	0.068	0.063	0.064
At the toe of the structure		(waterdepth = 0.30 m)			
		JS05	JS06	JS07	JS08
H_s	[m]	0.090	0.135	0.164	0.176
$H_s/\gamma D_n$	[-]	1.4	2.1	2.5	2.7
s_{om}	[-]	0.055	0.062	0.052	0.045

Table A.2 : $h_{\text{toe}} = 0.50 \text{ m}$

Near the wave board		(water depth = 0.90 m)			
		JS11	JS12	JS13	JS14
H_s	[m]	0.10	0.15	0.20	0.25
T_p	[s]	1.8	2.2	2.5	2.8
s_p	[-]	0.020	0.020	0.020	0.020
T_m	[s]	1.4	1.7	2.0	2.2
s_m	[-]	0.032	0.032	0.033	0.033
At the toe of the structure		(waterdepth = 0.50 m)			
		JS11	JS12	JS13	JS14
H_s	[m]	0.096	0.153	0.204	0.273
$H_s/\gamma D_n$	[-]	1.5	2.3	3.1	4.2
s_{om}	[-]	0.030	0.032	0.034	0.036
Near the wave board		(water depth = 0.90 m)			
		JS15	JS16	JS17	JS18
H_s	[m]	0.10	0.15	0.20	0.25
T_p	[s]	1.3	1.5	1.8	2.0
s_p	[-]	0.038	0.043	0.040	0.040
T_m	[s]	1.0	1.2	1.4	1.6
s_m	[-]	0.061	0.068	0.063	0.064
At the toe of the structure		(waterdepth = 0.50 m)			
		JS15	JS16	JS17	JS18
H_s	[m]	0.088	0.133	0.184	0.248
$H_s/\gamma D_n$	[-]	1.3	2.0	2.8	3.8
s_{om}	[-]	0.053	0.061	0.058	0.064

Table B.1 : Main Program

Water depth $h_{toe} = 0.30$ m Number of units : 329				Water depth $h_{toe} = 0.50$ m Number of units : 432			
test run code	H_s [m]	N [-]	s_p^* [-]	test run code	H_s [m]	N [-]	s_p^* [-]
JS01	0.10	200	0.02	JS11	0.10	200	0.02
JS02	0.15	200		JS12	0.15	200	
JS03	0.20	200		JS13	0.20	200	
JS04	0.25	200		JS14	0.25	200	
JS05	0.10	200	0.04	JS15	0.10	200	0.04
JS06	0.15	200		JS16	0.15	200	
JS07	0.20	200		JS17	0.20	200	
JS08	0.25	200		JS18	0.25	200	

* Wave steepness near the wave board

Test series consisting of test runs JS05-JS08 has been used in the CUR C70 related part of the investigation, referred to as JS05C-JS08C.

Table B2 : Additional stability tests

Water depth $h_{toe} = 0.30$ m Number of units : 329				Water depth $h_{toe} = 0.50$ m Number of units : 432			
test run code	H_s [m]	N [-]	s_p^* [-]	test run code	H_s [m]	N [-]	s_p^* [-]
JS01	0.10	200 [#]	0.02	JS11	0.10	200 [#]	0.02
JS02	0.15	200 [#]		JS12L	0.15	1000	
JS03L	0.20	1000		JS13L	0.20	1000	
JS04L	0.25	1000		JS14L	0.25	1000	
JS05	0.10	200 [#]	0.04	JS15	0.10	200 [#]	0.04
JS06	0.15	200 [#]		JS16L	0.15	1000	
JS07L	0.20	1000		JS17L	0.20	1000	
JS08L	0.25	1000		JS18L	0.25	1000	

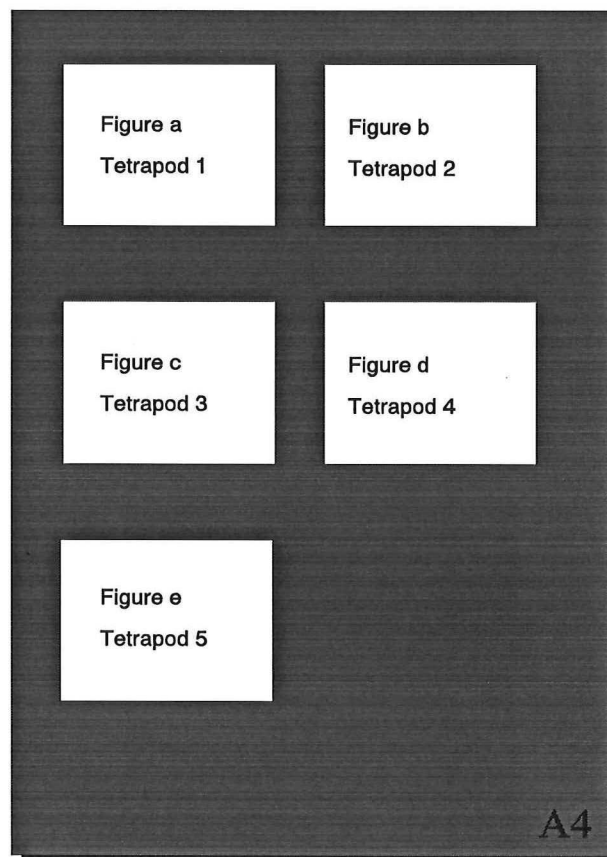
* Wave steepness near the wave board

Only the larger values for the significant wave height are of interest for the stability of the armour layer. Therefore, the first one or two test runs (the ones with smaller value for the significant wave height) consist of 200 waves per test run, instead of 1000.

Appendix C

In this appendix probability plots are shown of all the 200 combinations that could be made with the parameters of this investigation. Repetitions, if any, are taken together and shown in one plot.

Each Figure contains 5 probability plots, one for each tetrapod, according to the below shown Figure. Due to the data reduction as a result of the inadequate first analysis there are some combinations for which there is no data anymore. Consequently, there will be no probability plot for that particular tetrapod.



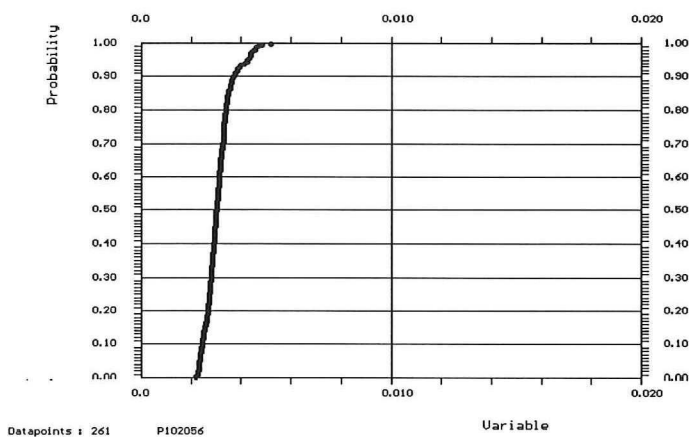
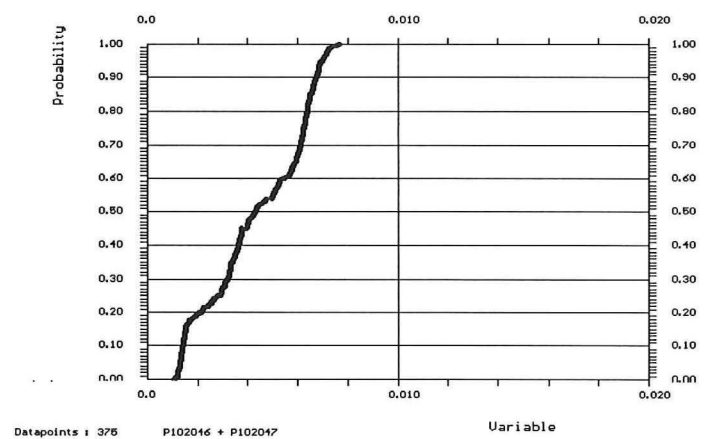
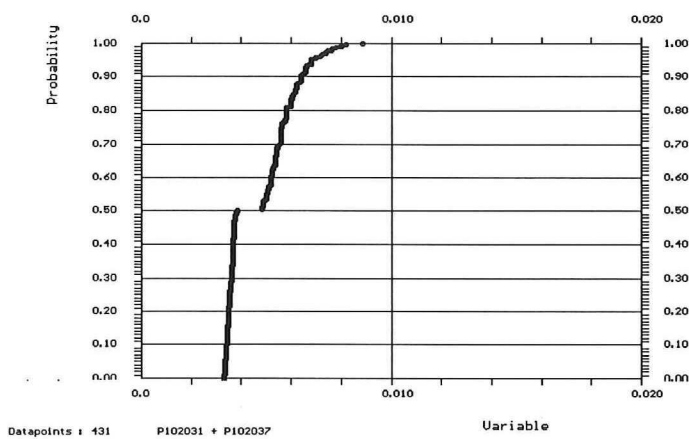
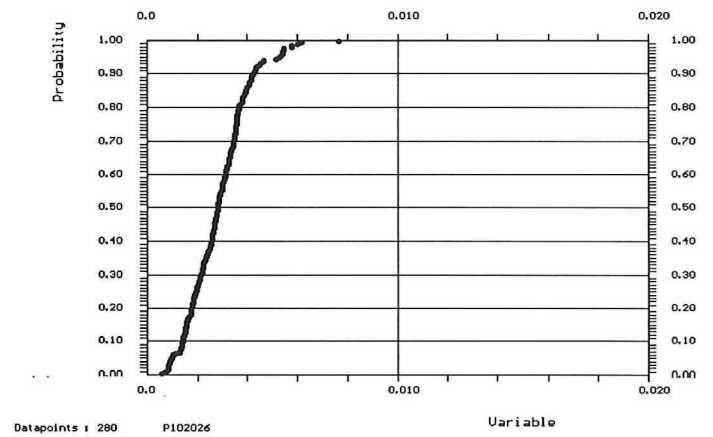
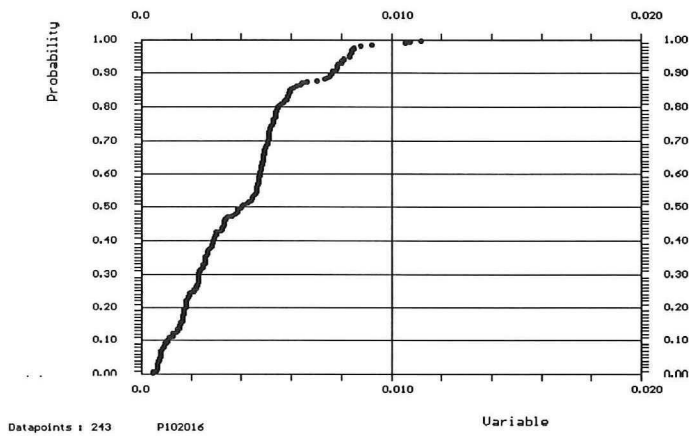


Figure C.1 Probability curves of quasi static maxima of tetrapods 1 to 5.
 $H_s = 0.10$ m. $s_{op} = 0.02$. $h_{toe} = 0.30$ m.
 Orientation of instrumented leg : downward along slope.

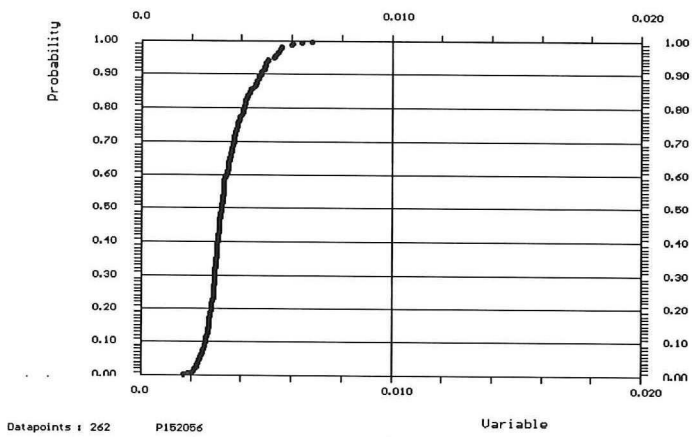
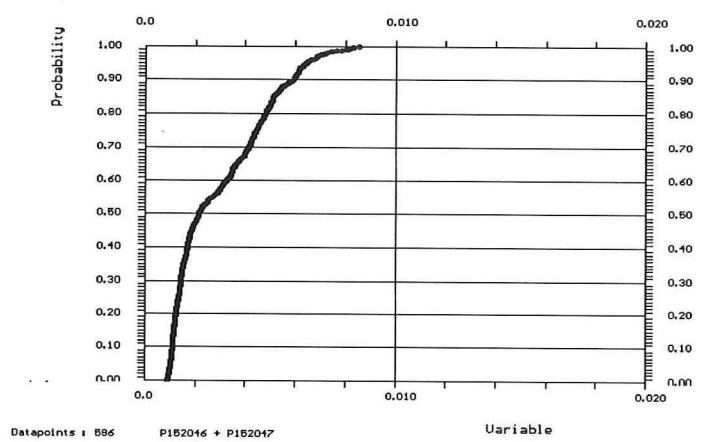
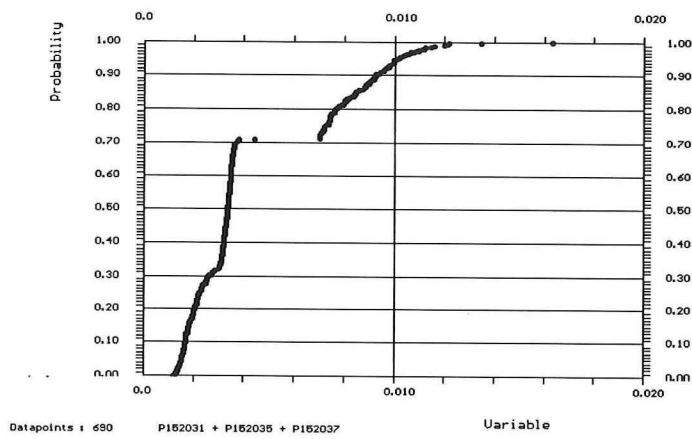
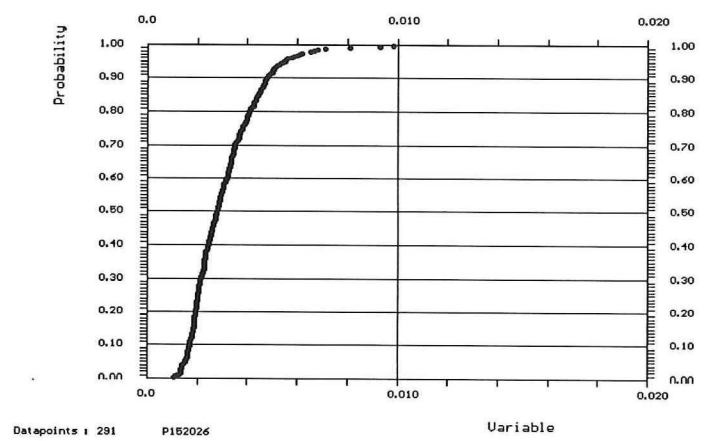
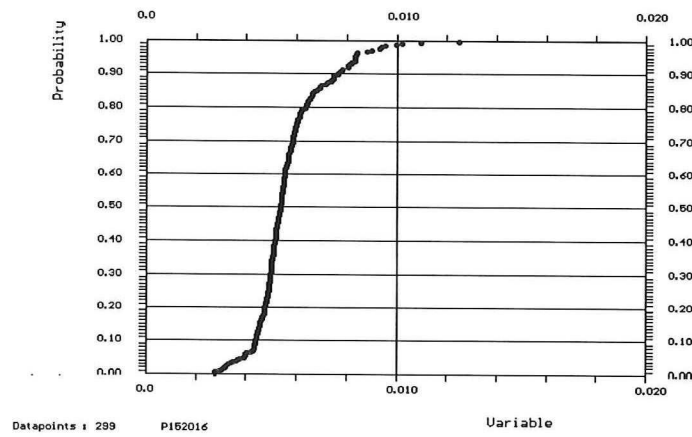


Figure C.2 Probability curves of quasi static maxima of tetrapods 1 to 5.
 $H_s = 0.15$ m. $s_{op} = 0.02$. $h_{toe} = 0.30$ m.
 Orientation of instrumented leg : downward along slope.

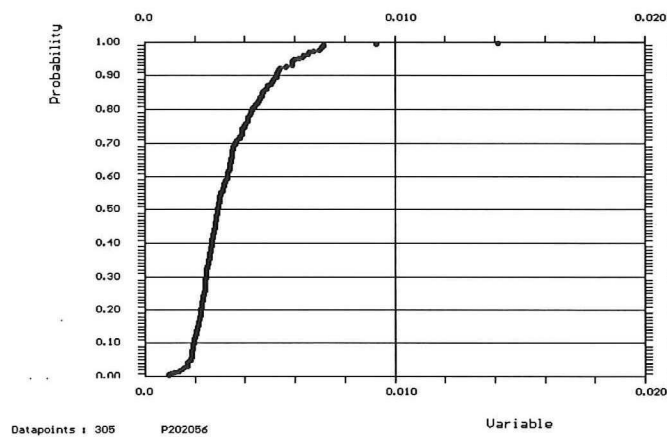
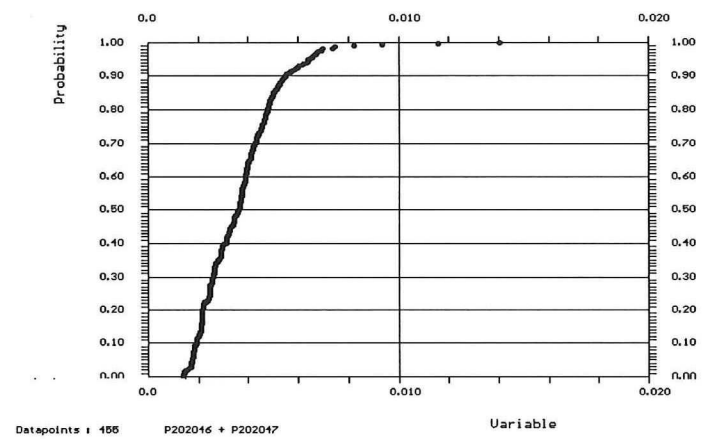
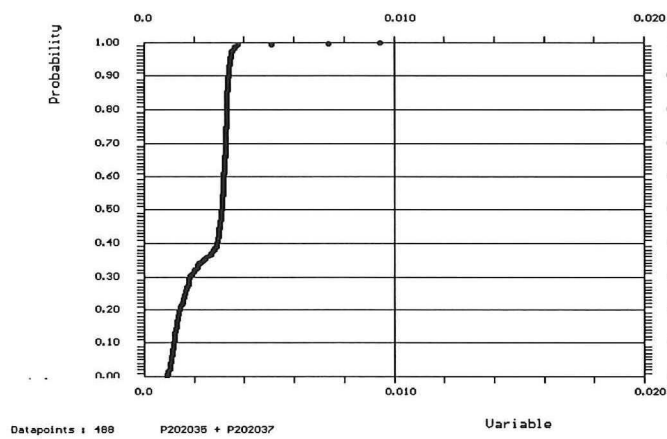
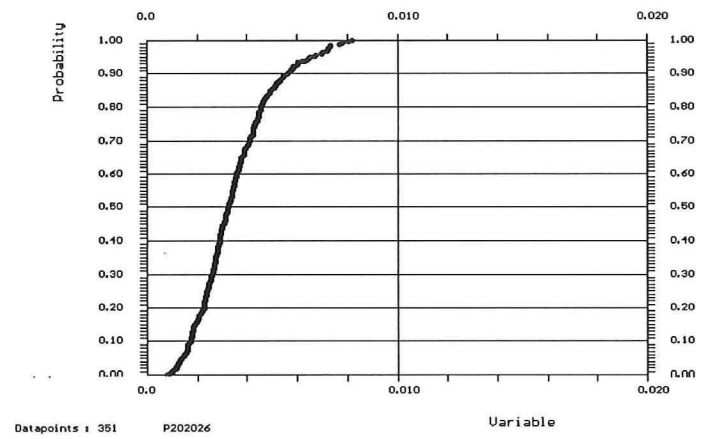
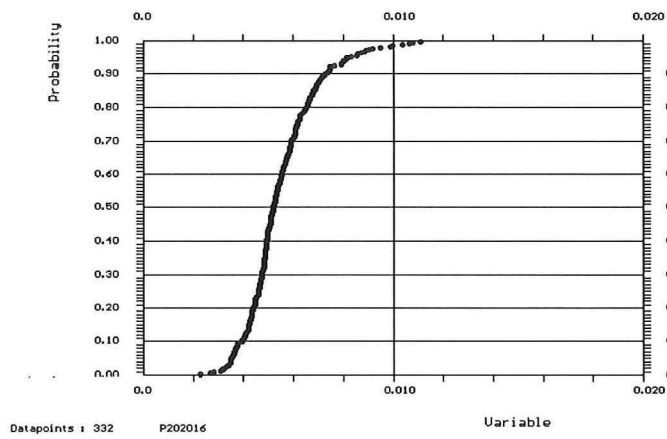


Figure C.3 Probability curves of quasi static maxima of tetrapods 1 to 5.
 $H_s = 0.20$ m. $s_{op} = 0.02$. $h_{toe} = 0.30$ m.
 Orientation of instrumented leg : downward along slope.

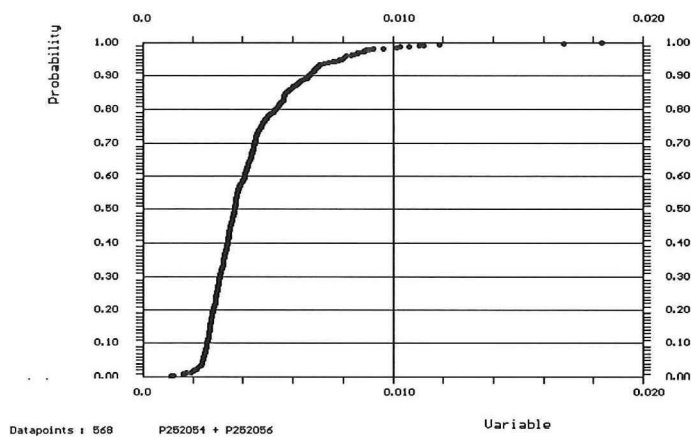
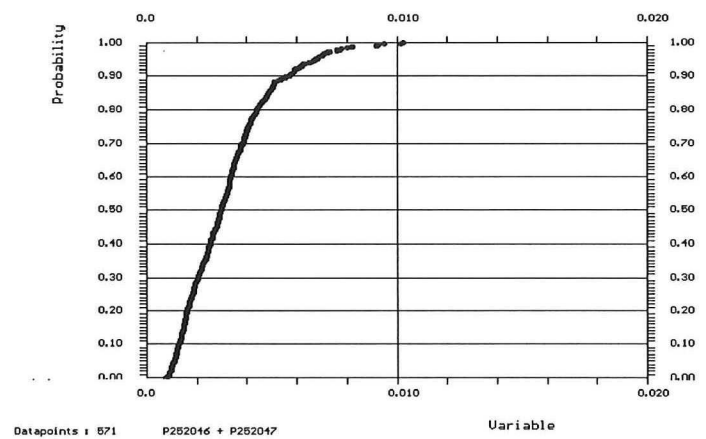
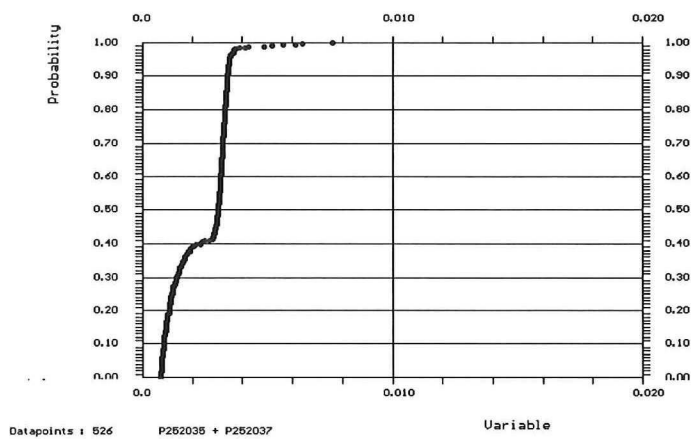
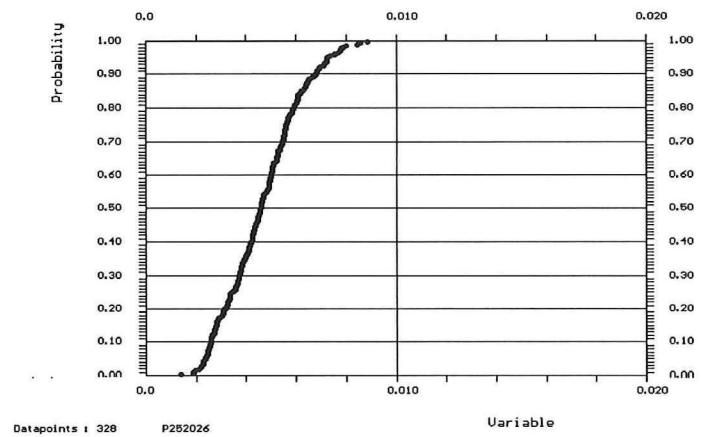
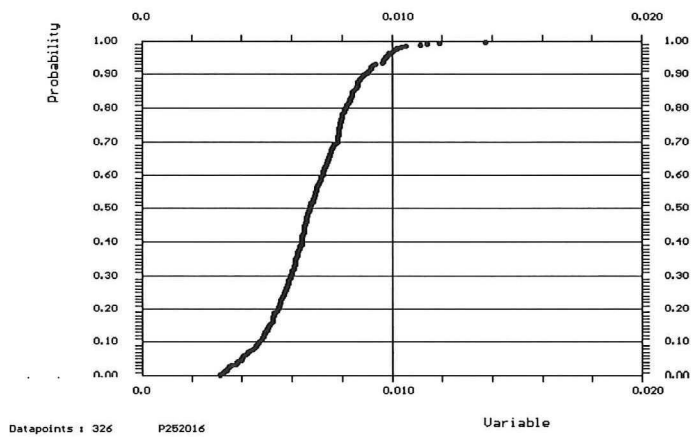


Figure C.4 Probability curves of quasi static maxima of tetrapods 1 to 5.
 $H_s = 0.25$ m. $s_{op} = 0.02$. $h_{toe} = 0.30$ m.
 Orientation of instrumented leg : downward along slope.

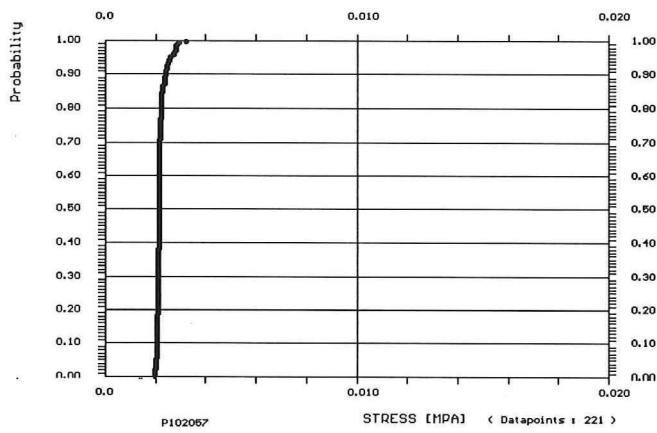
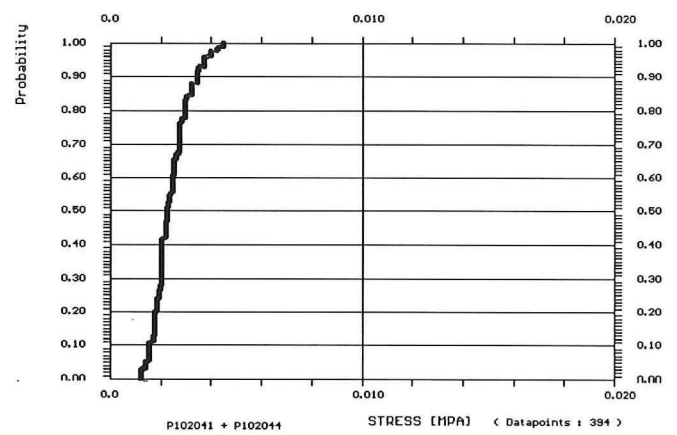
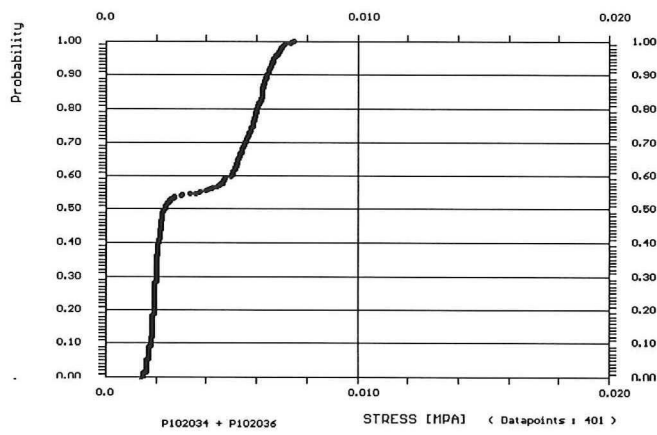
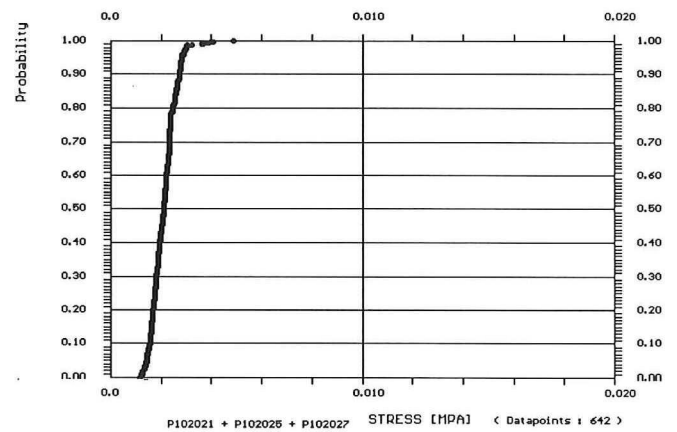
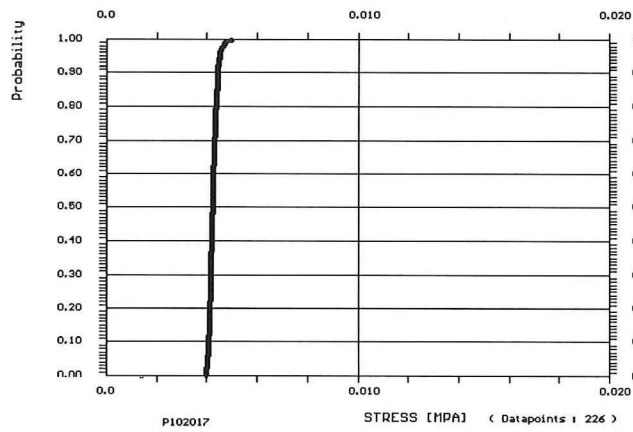


Figure C.5 Probability curves of quasi static maxima of tetrapods 1 to 5.
 $H_s = 0.10$ m. $s_{op} = 0.02$. $h_{toe} = 0.30$ m
 Orientation of instrumented leg : perpendicular to slope.

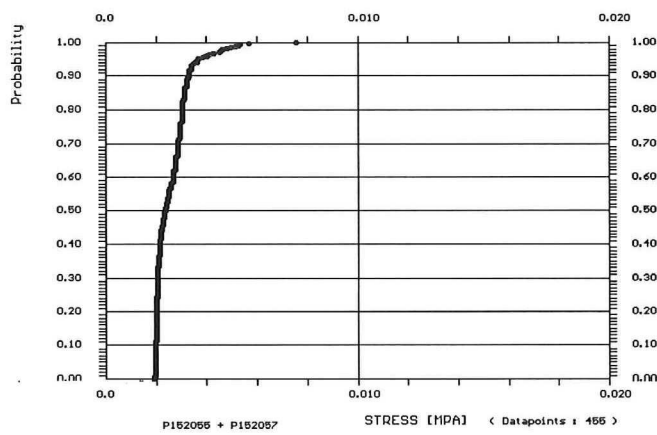
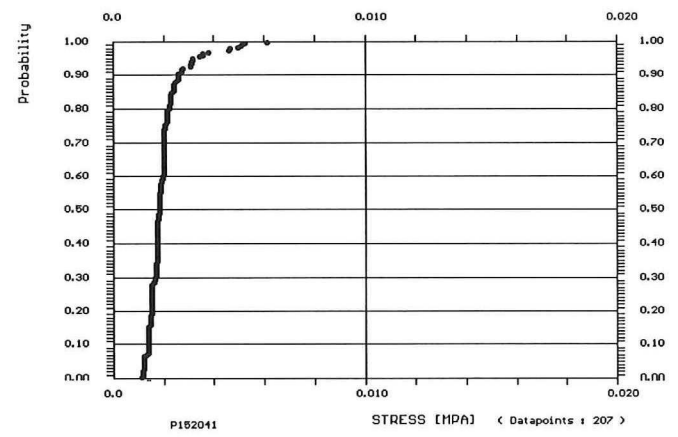
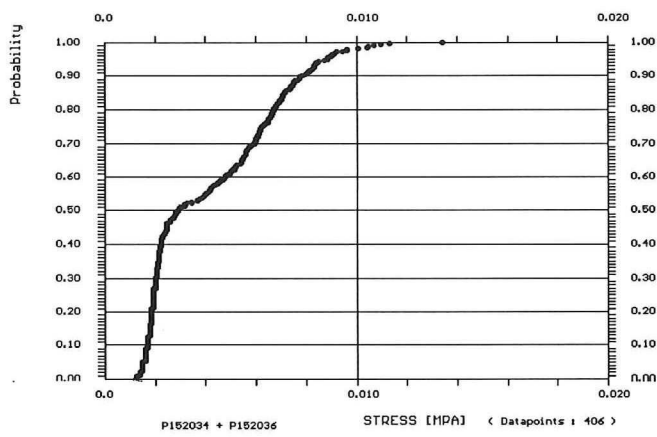
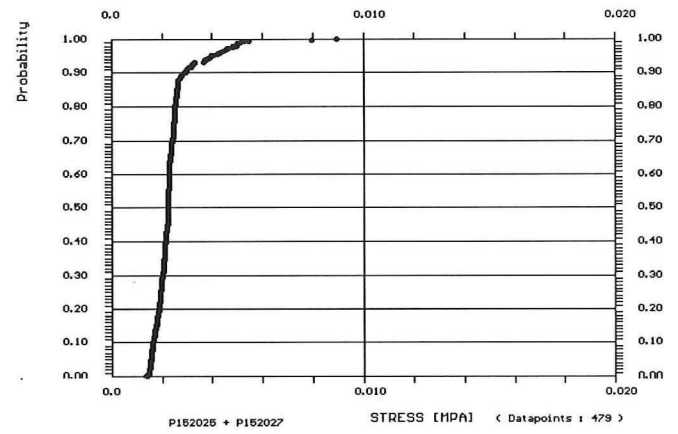
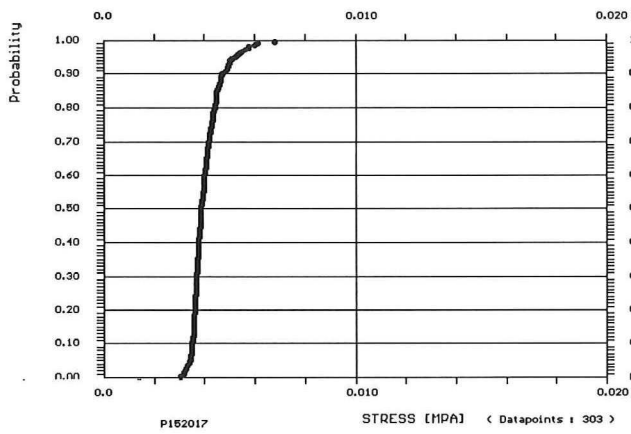


Figure C.6 Probability curves of quasi static maxima of tetrapods 1 to 5.
 $H_s = 0.15$ m. $s_{op} = 0.02$. $h_{toe} = 0.30$ m
 Orientation of instrumented leg : perpendicular to slope.

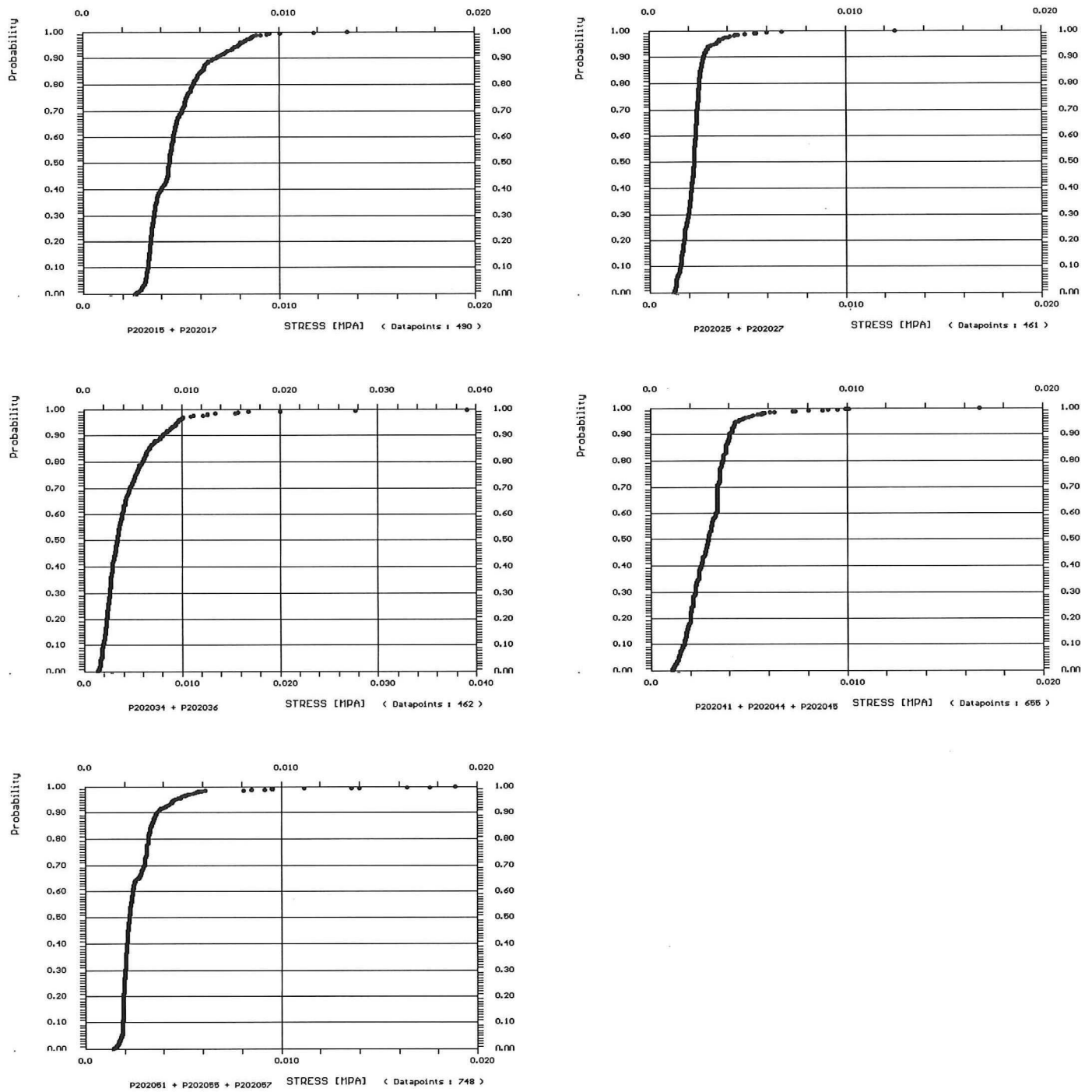


Figure C.7 Probability curves of quasi static maxima of tetrapods 1 to 5.
 $H_s = 0.20$ m. $s_{op} = 0.02$. $h_{toe} = 0.30$ m
 Orientation of instrumented leg : perpendicular to slope.

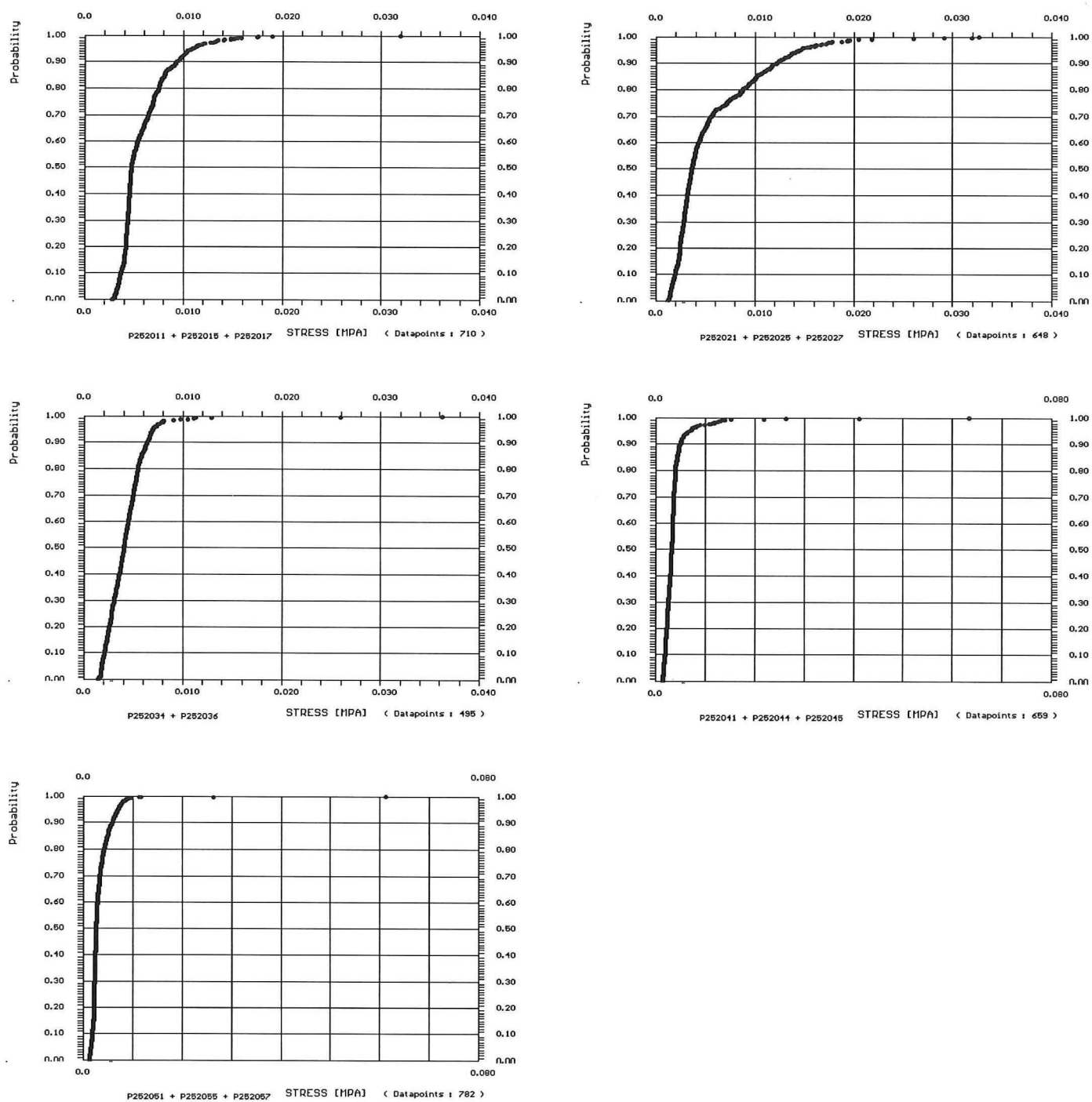


Figure C.8 Probability curves of quasi static maxima of tetrapods 1 to 5.
 $H_s = 0.25$ m. $s_{op} = 0.02$. $h_{toe} = 0.30$ m
 Orientation of instrumented leg : perpendicular to slope.

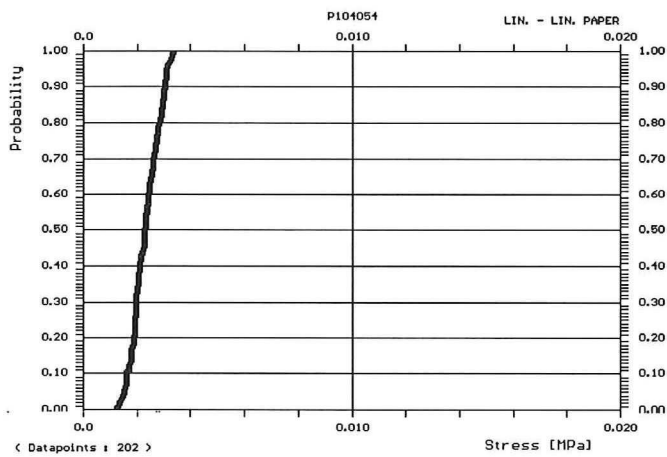
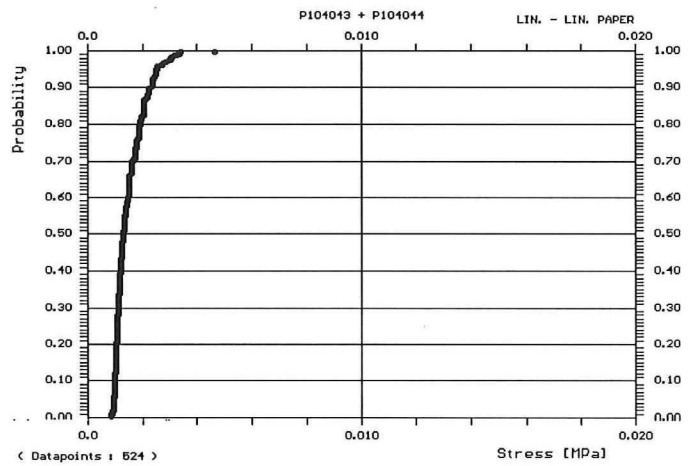
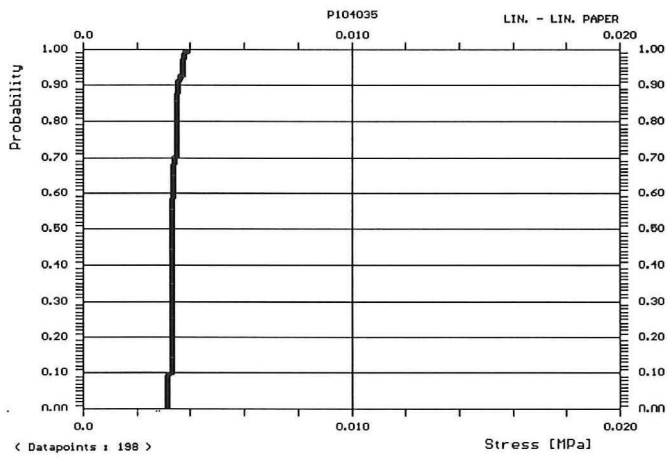
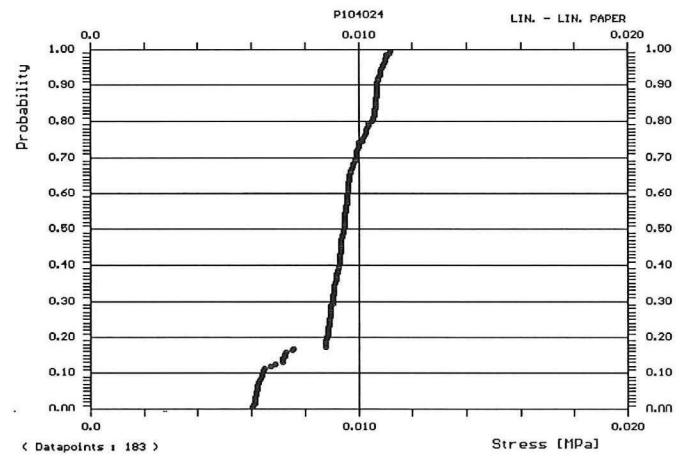


Figure C.9 Probability curves of quasi static maxima of tetrapods 1 to 5.
 $H_s = 0.10$ m. $s_{op} = 0.04$. $h_{toe} = 0.30$ m
 Orientation of instrumented leg : downward along slope.

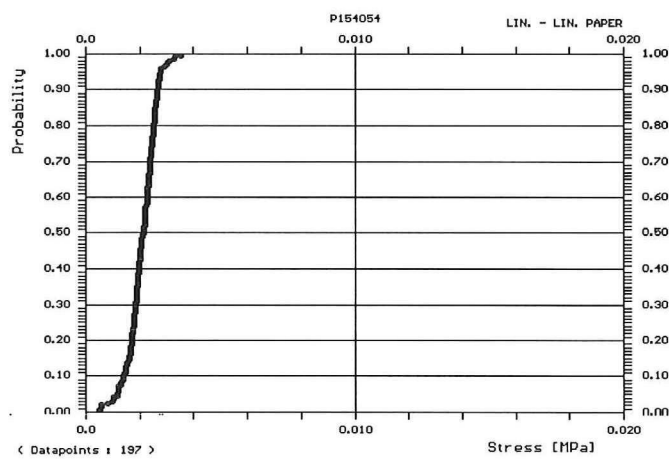
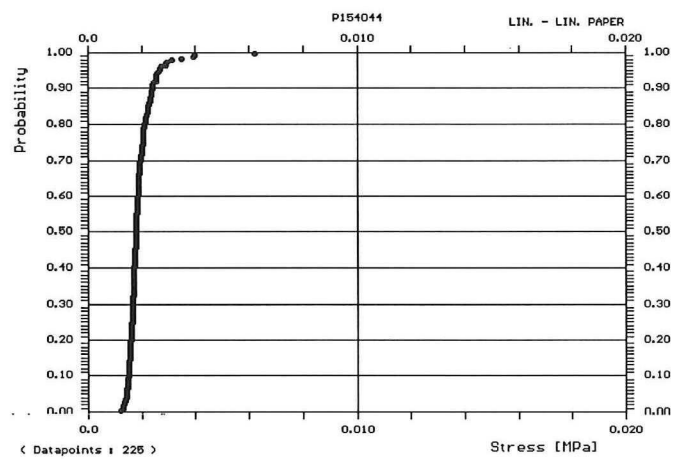
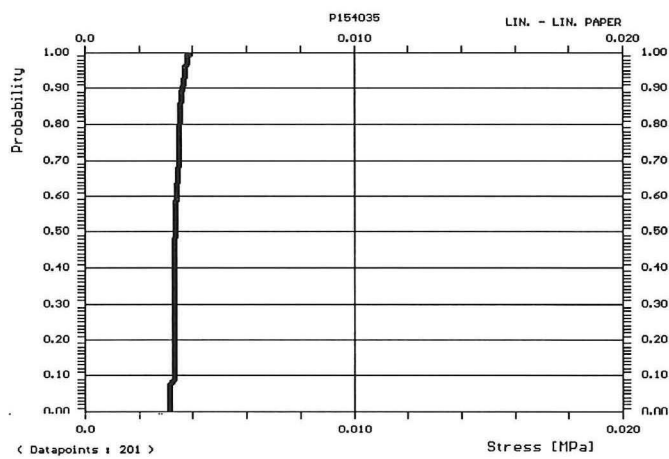
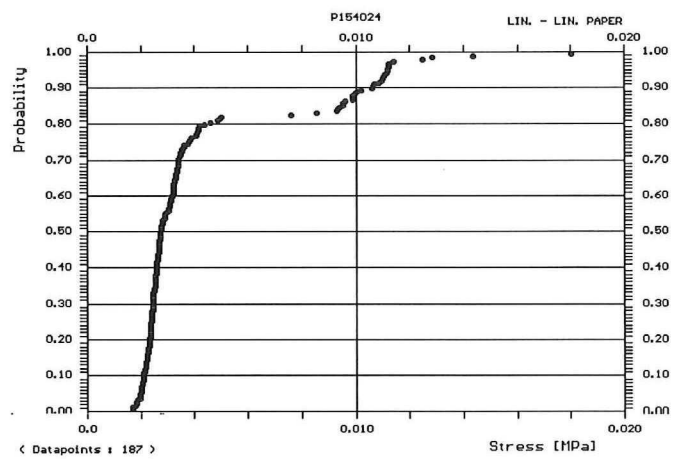
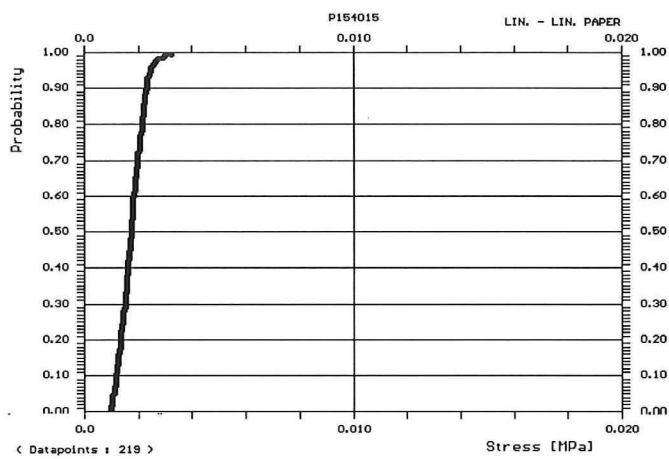


Figure C.10 Probability curves of quasi static maxima of tetrapods 1 to 5.
 $H_s = 0.15$ m. $s_{op} = 0.04$. $h_{toe} = 0.30$ m
 Orientation of instrumented leg : downward along slope.

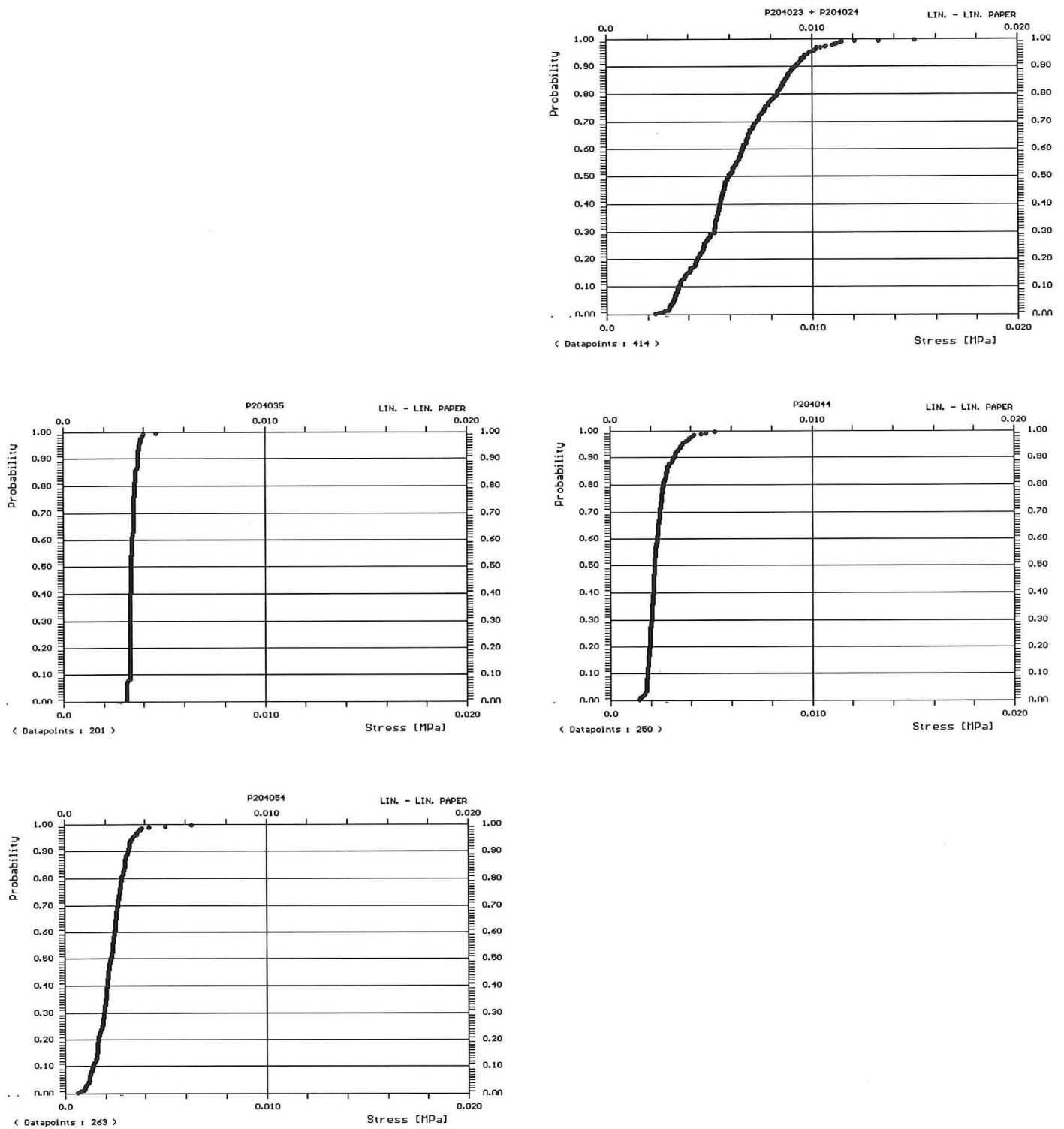


Figure C.11 Probability curves of quasi static maxima of tetrapods 1 to 5.
 $H_s = 0.20$ m. $s_{op} = 0.04$. $h_{toe} = 0.30$ m.
 Orientation of instrumented leg : downward along slope.

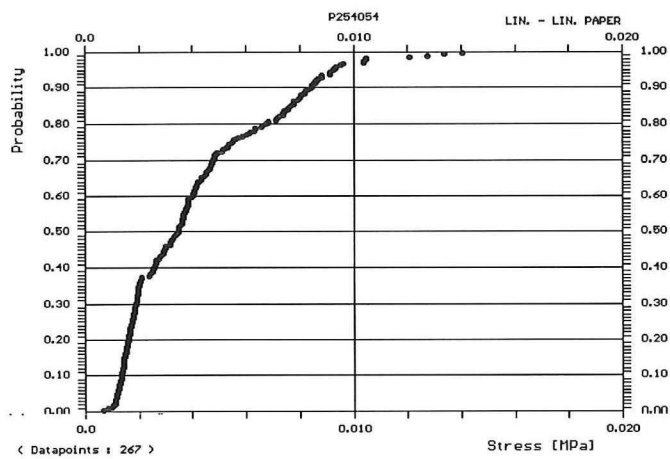
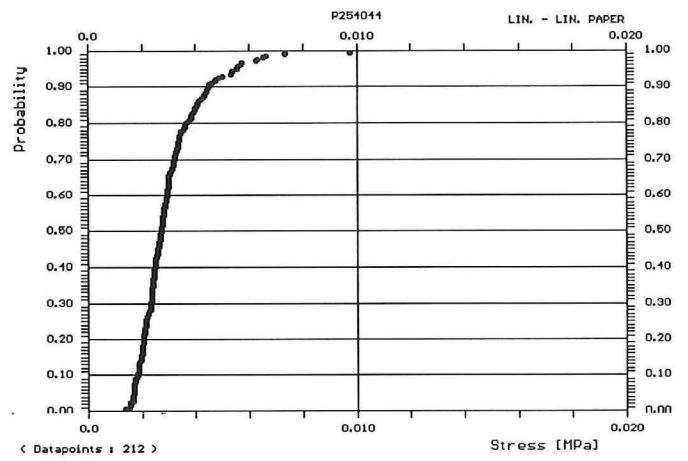
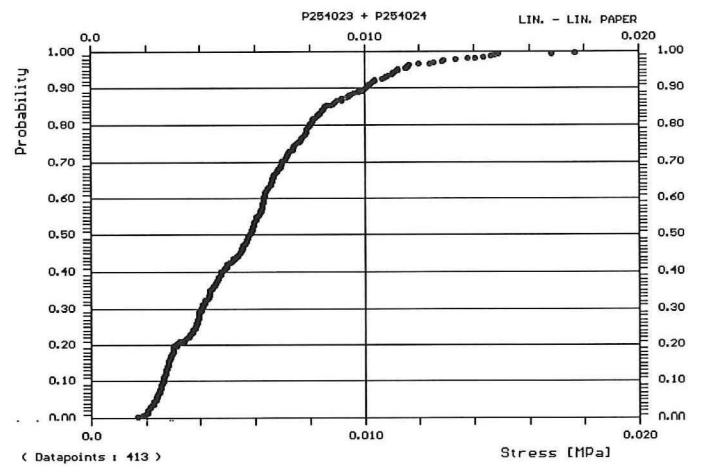


Figure C.12 Probability curves of quasi static maxima of tetrapods 1 to 5.
 $H_s = 0.25$ m. $s_{op} = 0.04$. $h_{toe} = 0.30$ m.
 Orientation of instrumented leg : downward along slope.

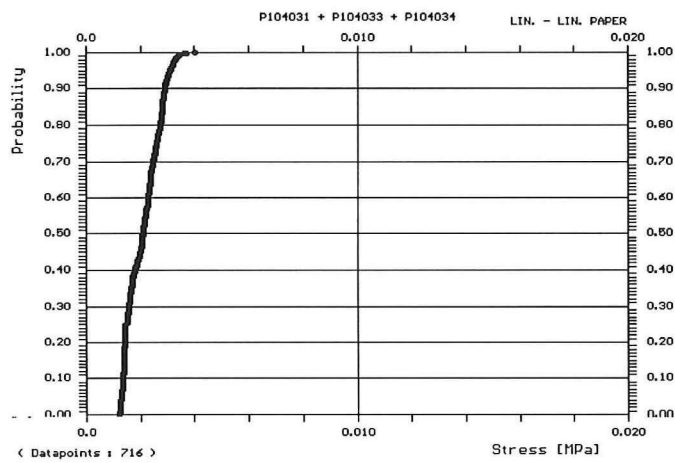
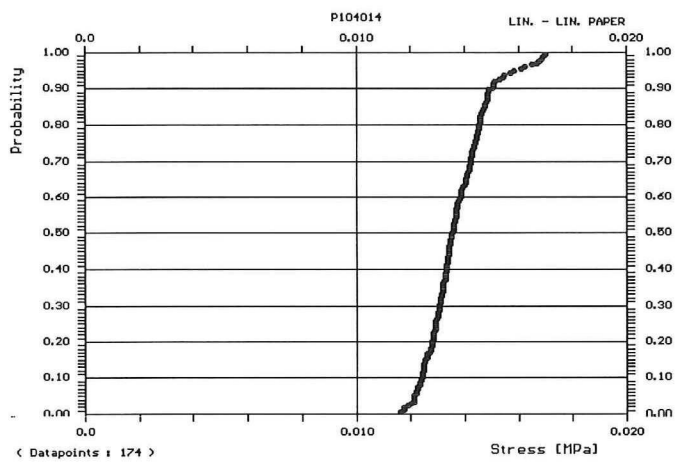


Figure C.13 Probability curves of quasi static maxima of tetrapods 1 to 5.
 $H_s = 0.10$ m. $s_{op} = 0.04$. $h_{toe} = 0.30$ m.
 Orientation of instrumented leg : perpendicular to slope.

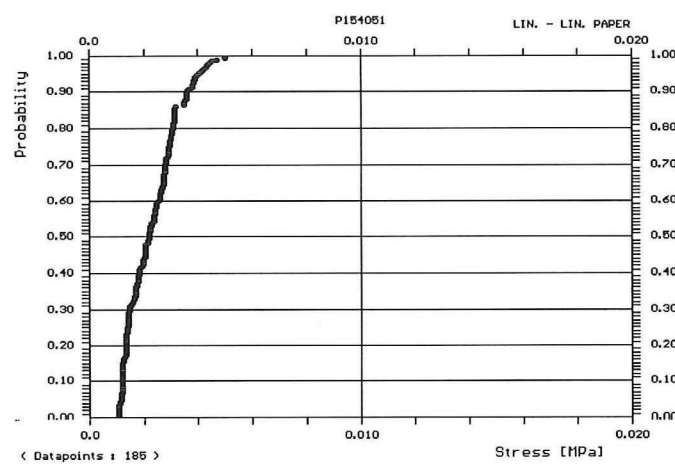
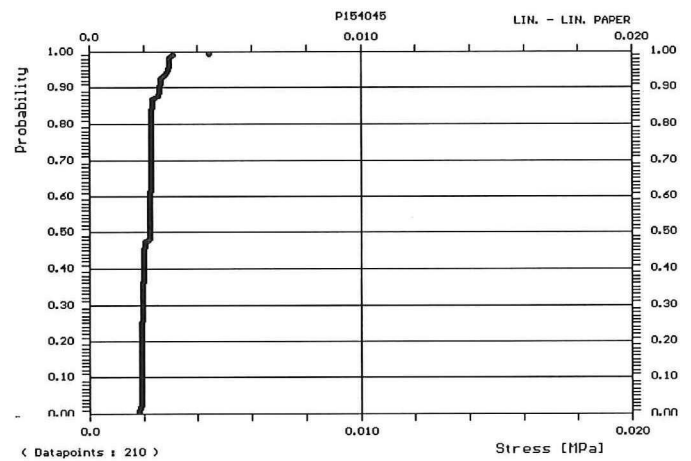
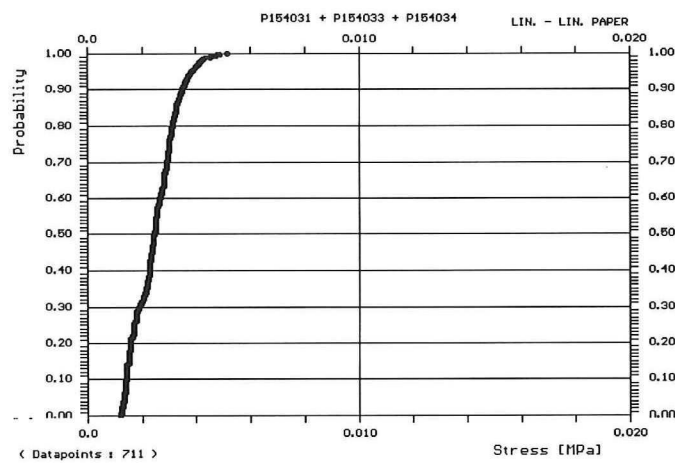
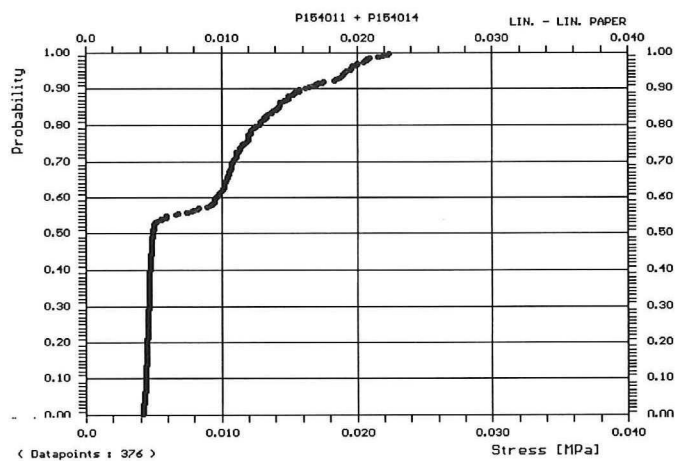


Figure C.14 Probability curves of quasi static maxima of tetrapods 1 to 5.
 $H_s = 0.15$ m. $s_{op} = 0.04$. $h_{toe} = 0.30$ m.
 Orientation of instrumented leg : perpendicular to slope.

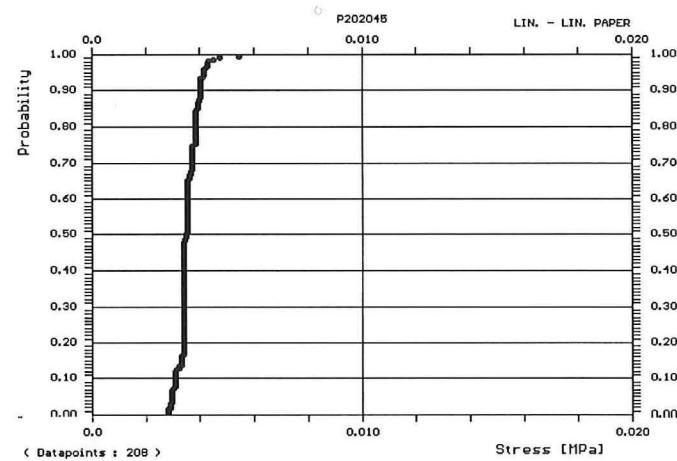
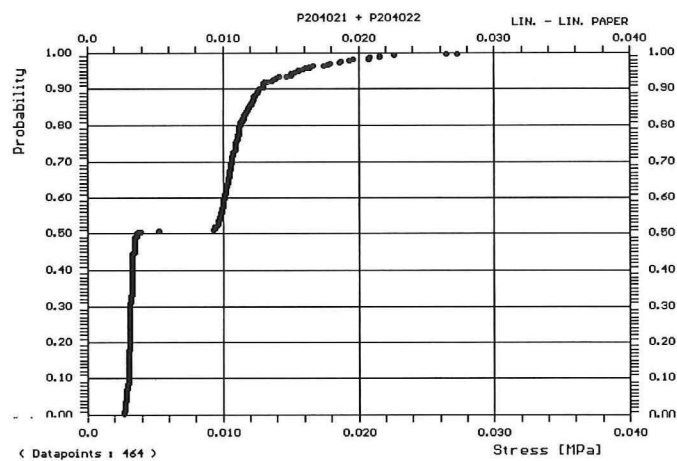
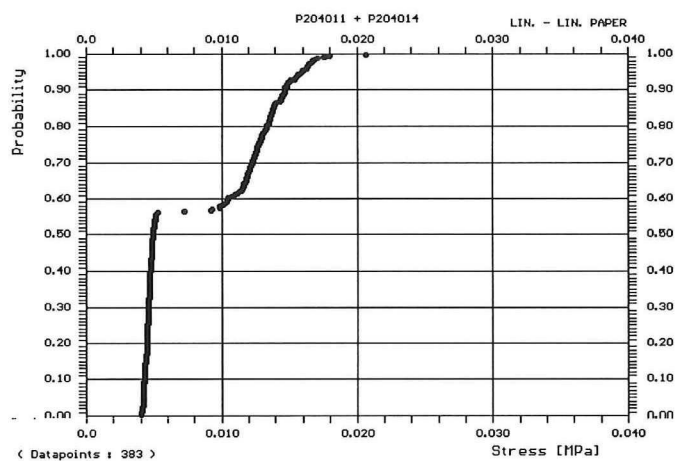


Figure C.15 Probability curves of quasi static maxima of tetrapods 1 to 5.
 $H_s = 0.20$ m. $s_{op} = 0.04$. $h_{toe} = 0.30$ m.
 Orientation of instrumented leg : perpendicular to slope.

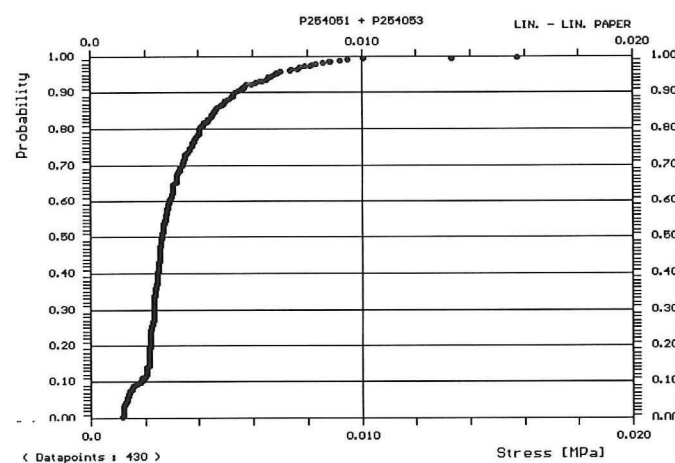
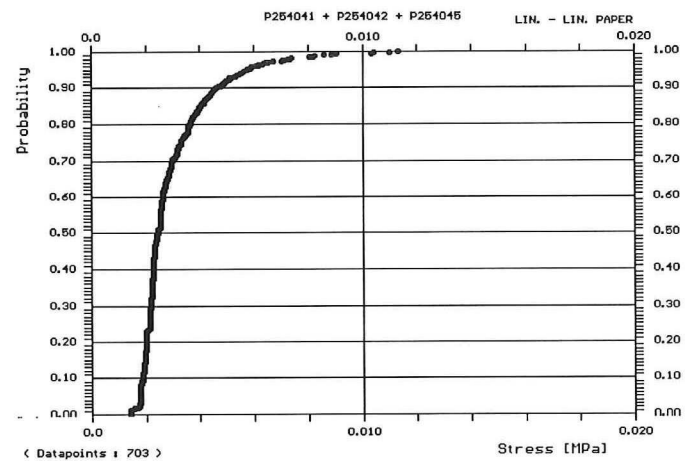
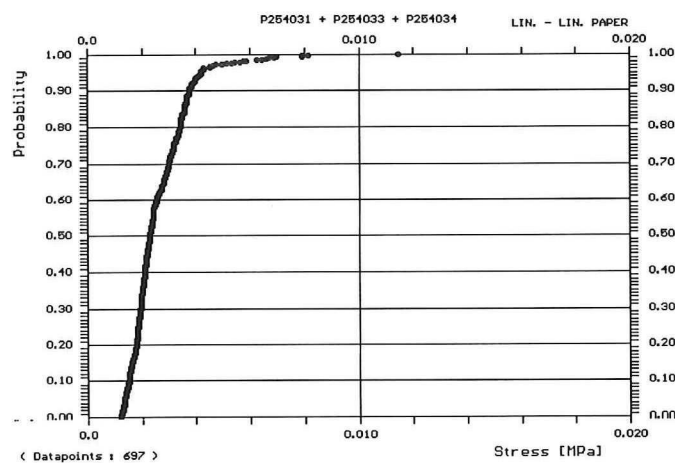
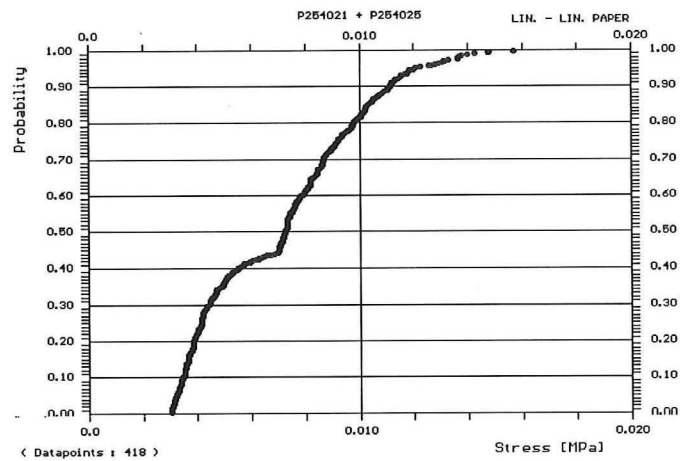
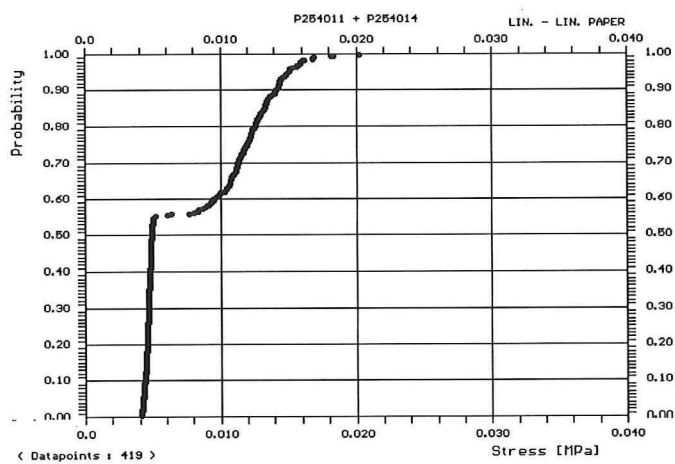


Figure C.16 Probability curves of quasi static maxima of tetrapods 1 to 5.
 $H_s = 0.25$ m. $s_{op} = 0.04$. $h_{toe} = 0.30$ m.
 Orientation of instrumented leg : perpendicular to slope.

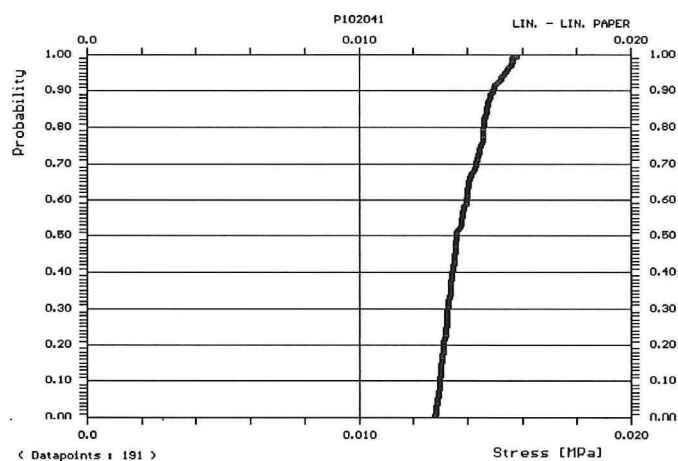
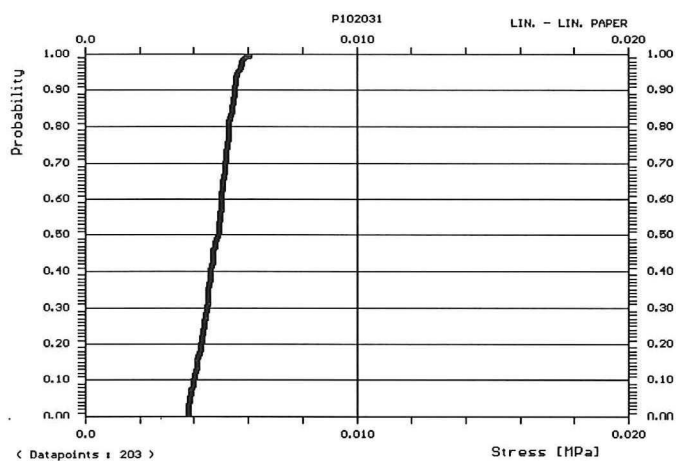
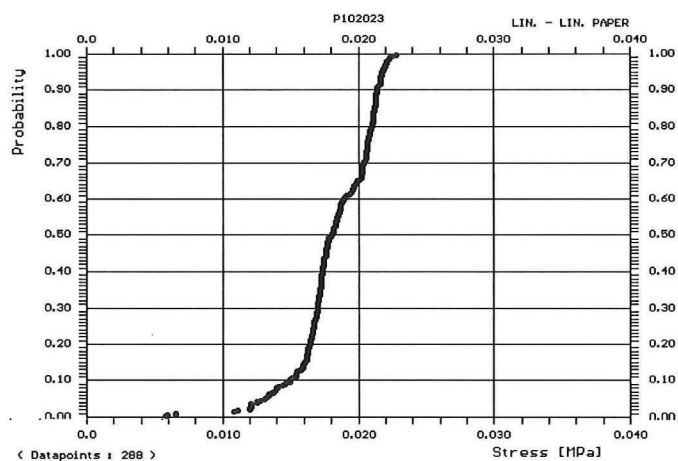
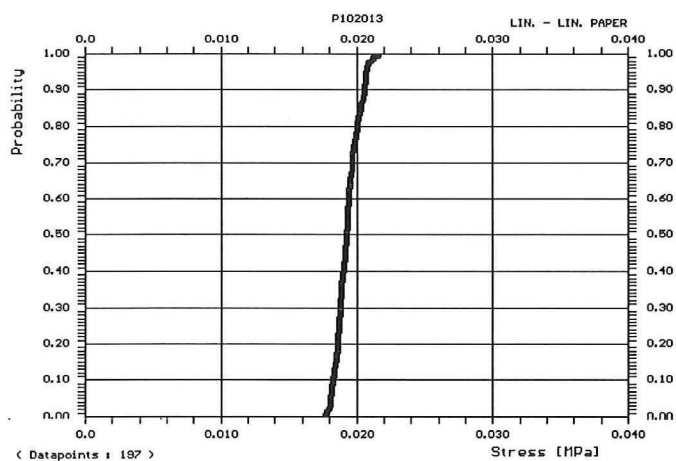


Figure C.17 Probability curves of quasi static maxima of tetrapods 1 to 5.
 $H_s = 0.10$ m. $s_{op} = 0.02$. $h_{toe} = 0.50$ m.
 Orientation of instrumented leg : downward along slope.

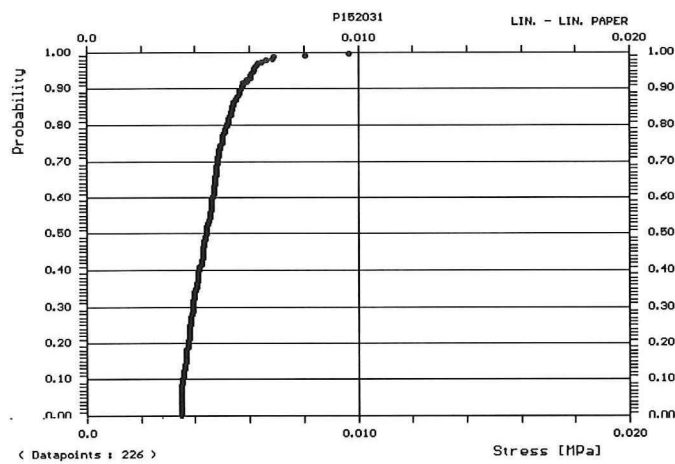
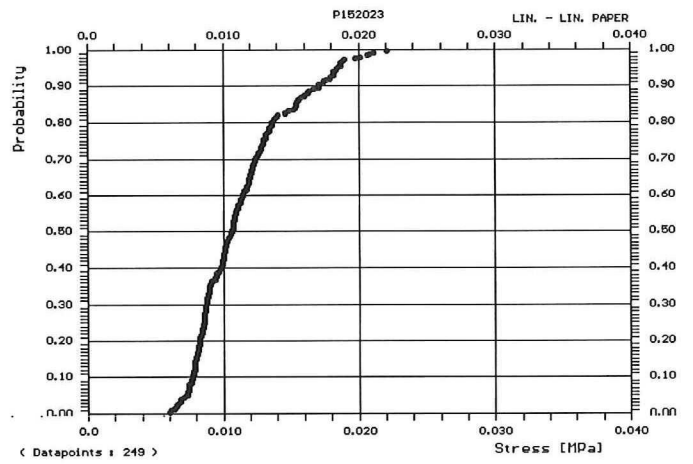
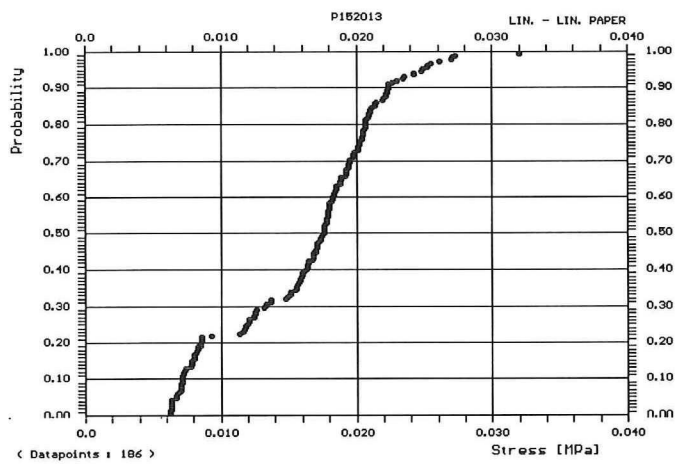


Figure C.18 Probability curves of quasi static maxima of tetrapods 1 to 5.
 $H_s = 0.15$ m. $s_{op} = 0.02$. $h_{toe} = 0.50$ m.
 Orientation of instrumented leg : downward along slope.

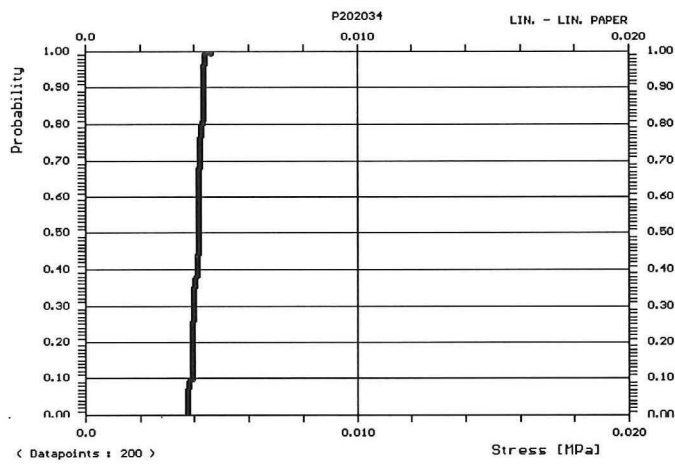
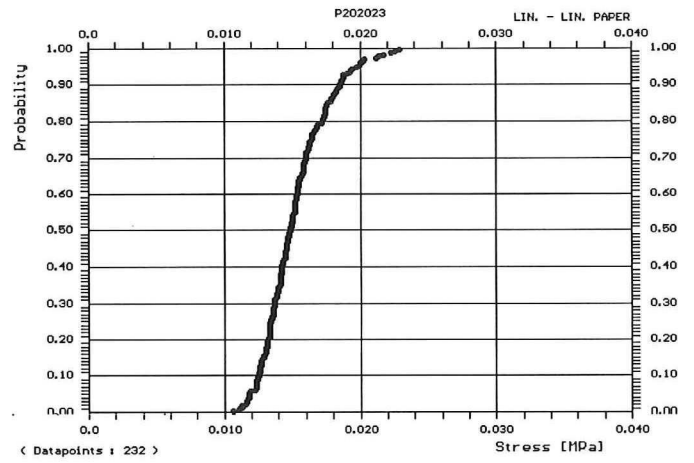
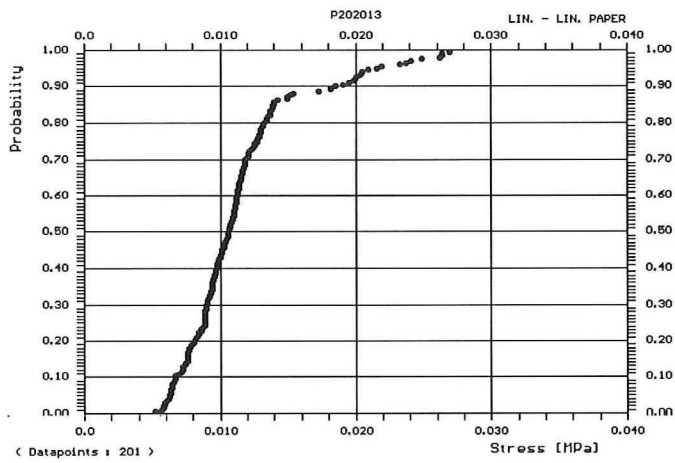


Figure C.19 Probability curves of quasi static maxima of tetrapods 1 to 5.
 $H_s = 0.20$ m. $s_{op} = 0.02$. $h_{toe} = 0.50$ m.
 Orientation of instrumented leg : downward along slope.

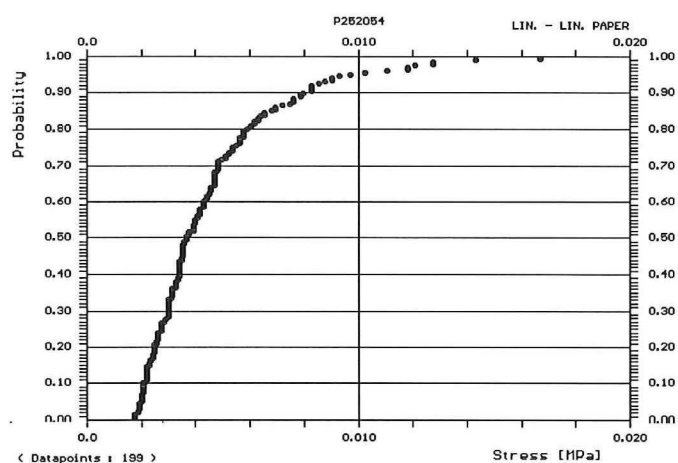
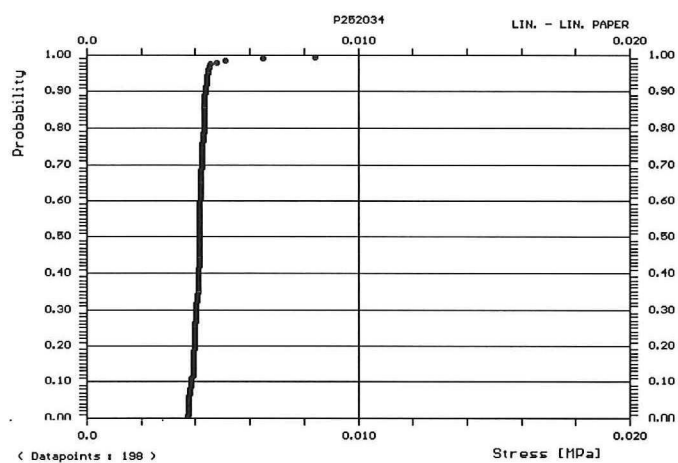
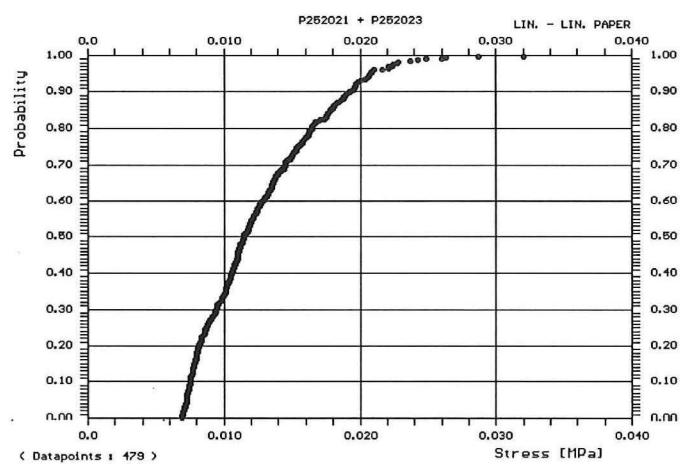
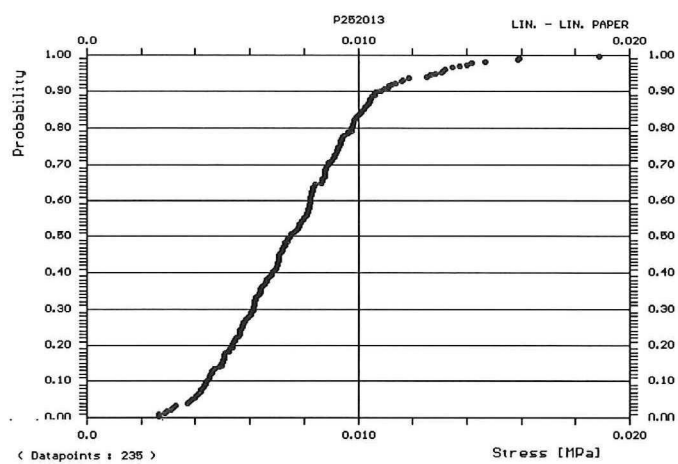


Figure C.20 Probability curves of quasi static maxima of tetrapods 1 to 5.
 $H_s = 0.25$ m. $s_{op} = 0.02$. $h_{toe} = 0.50$ m.
 Orientation of instrumented leg : downward along slope.

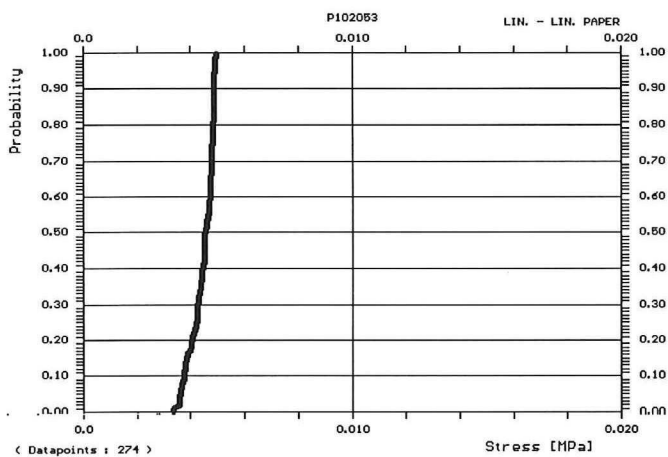
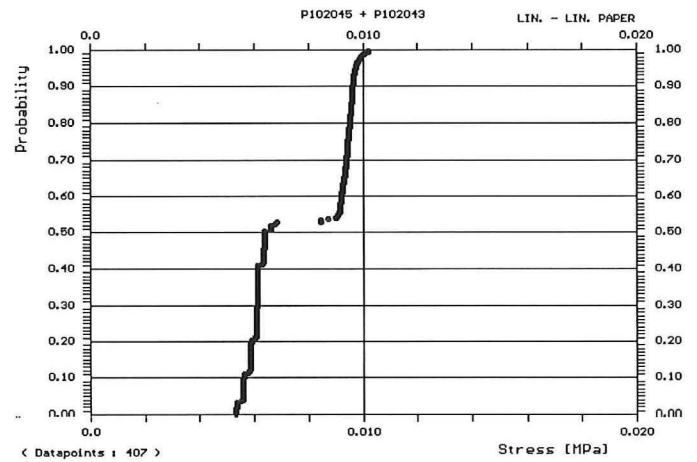
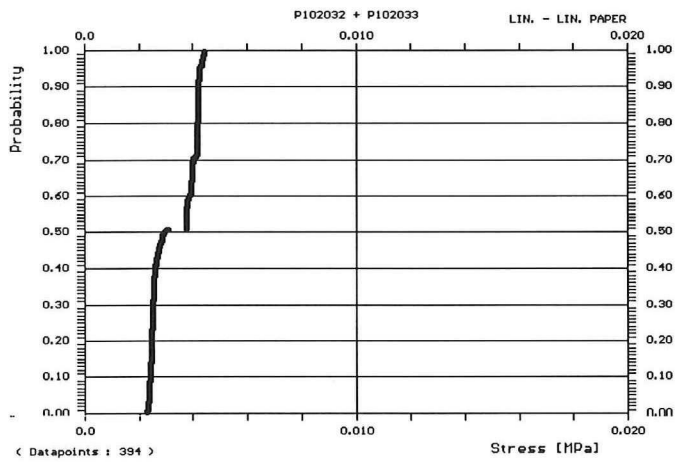
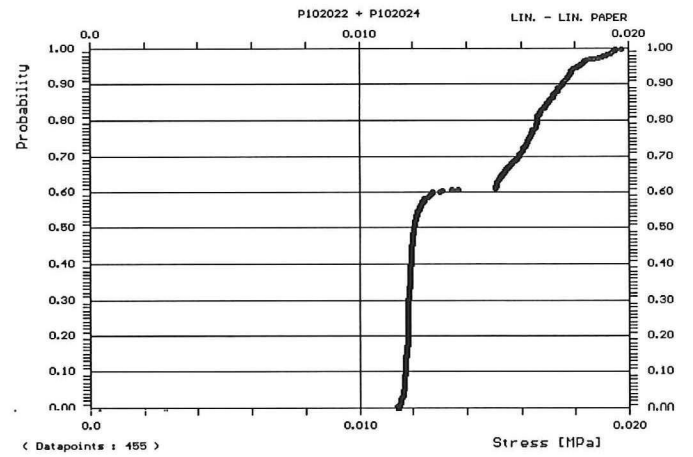


Figure C.21 Probability curves of quasi static maxima of tetrapods 1 to 5.
 $H_s = 0.10$ m. $s_{op} = 0.02$. $h_{toe} = 0.50$ m.
 Orientation of instrumented leg : perpendicular to slope.

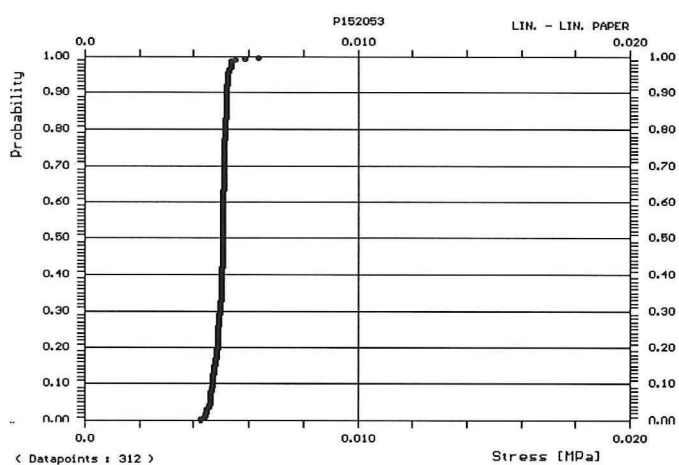
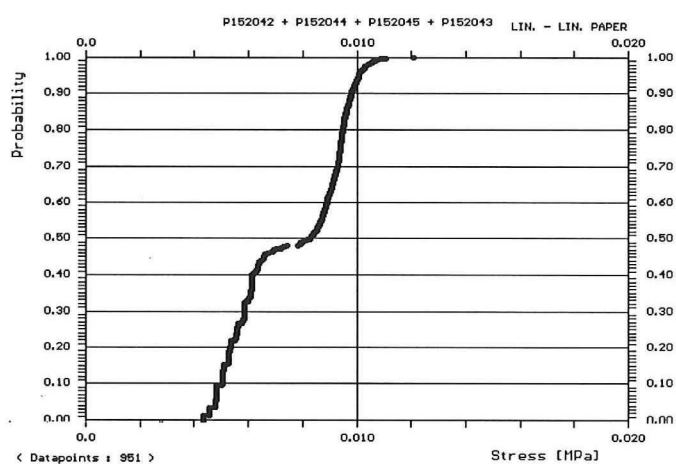
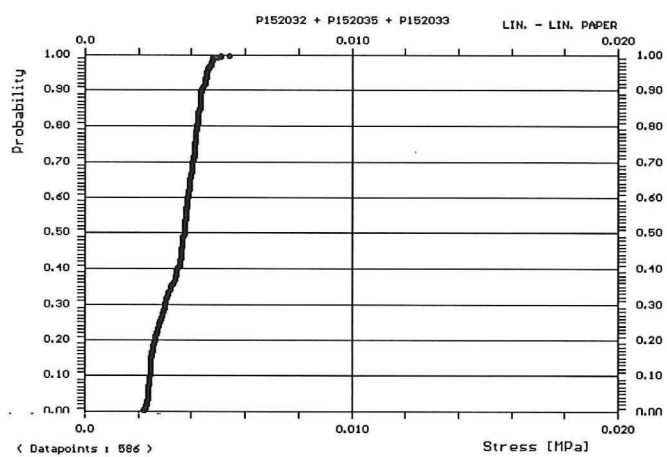
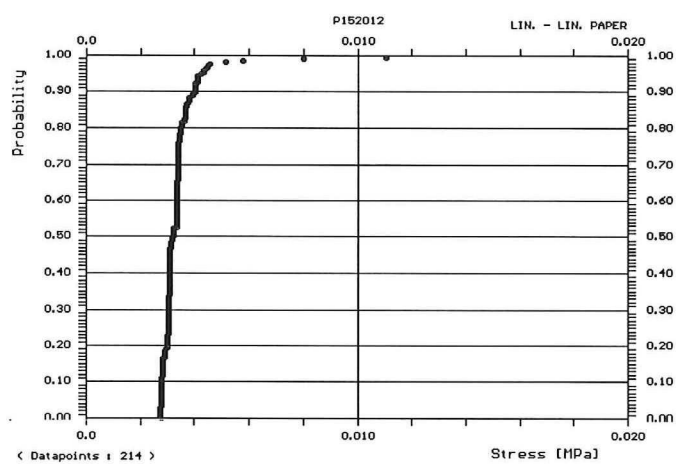


Figure C.22 Probability curves of quasi static maxima of tetrapods 1 to 5.
 $H_s = 0.15$ m. $s_{op} = 0.02$. $h_{toe} = 0.50$ m.
 Orientation of instrumented leg : perpendicular to slope.

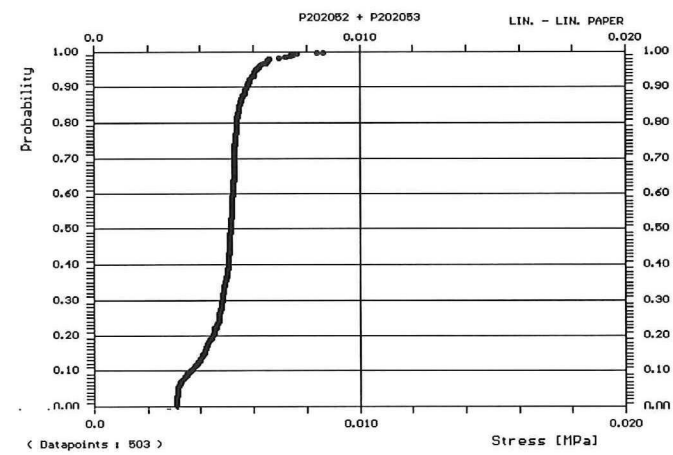
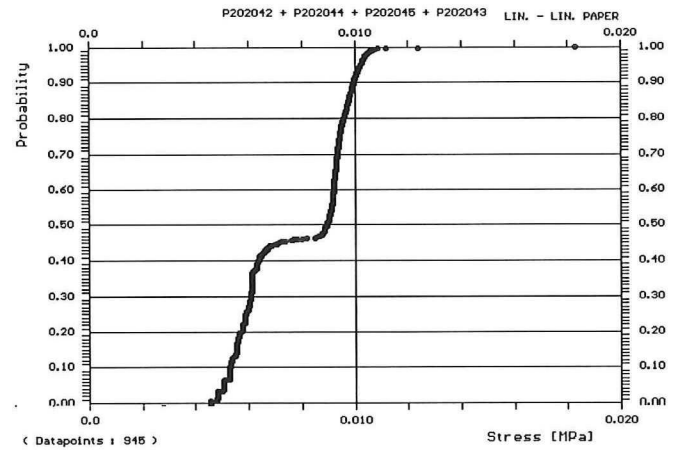
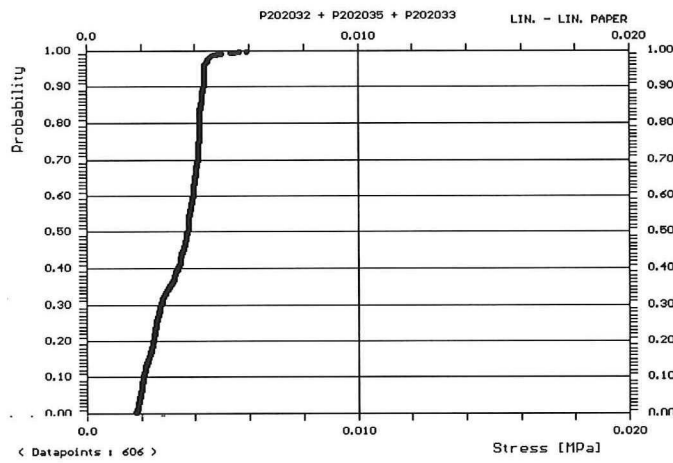
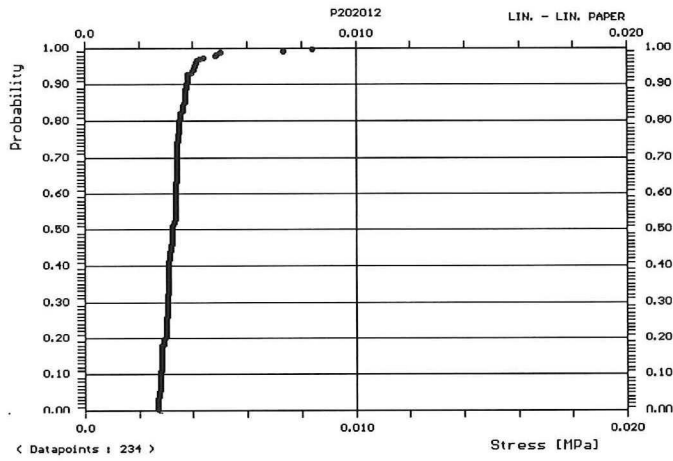


Figure C.23 Probability curves of quasi static maxima of tetrapods 1 to 5.
 $H_s = 0.20$ m. $s_{op} = 0.02$. $h_{toe} = 0.50$ m.
 Orientation of instrumented leg : perpendicular to slope.

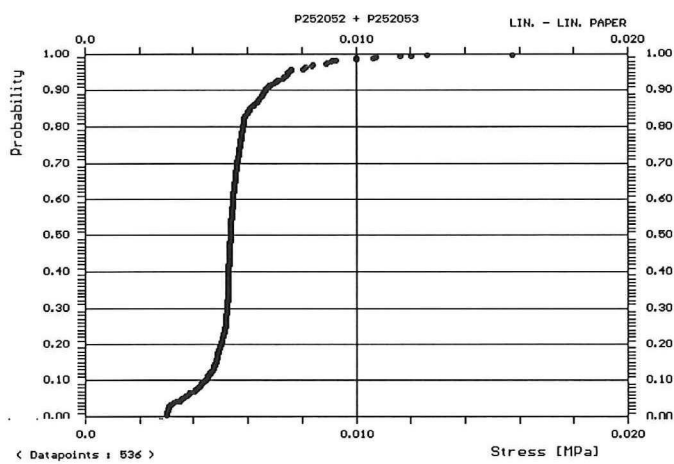
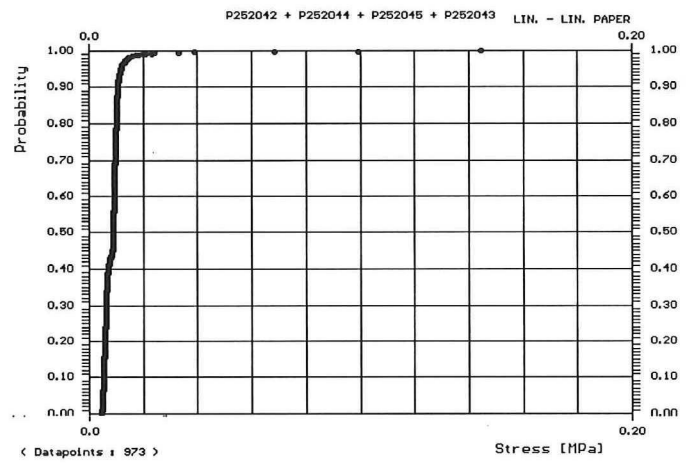
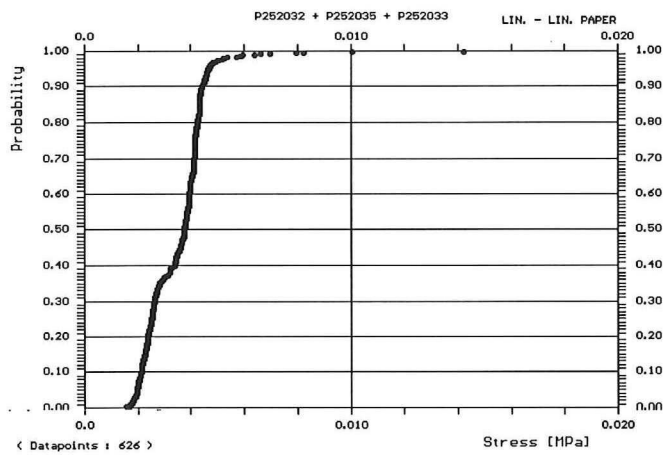


Figure C.24 Probability curves of quasi static maxima of tetrapods 1 to 5.
 $H_s = 0.25$ m. $s_{op} = 0.02$. $h_{toe} = 0.50$ m.
 Orientation of instrumented leg : perpendicular to slope.

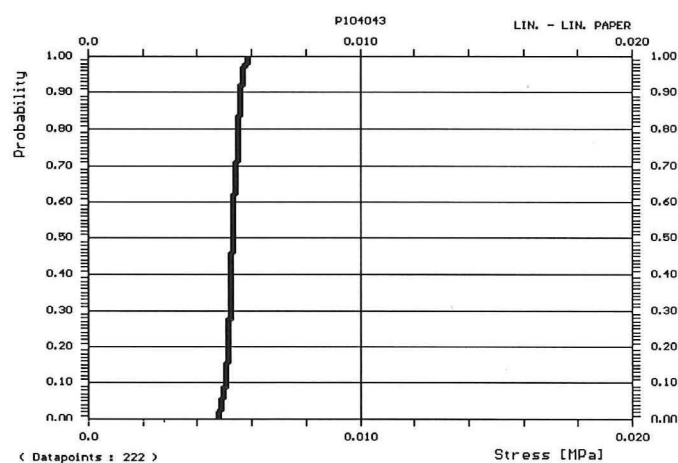
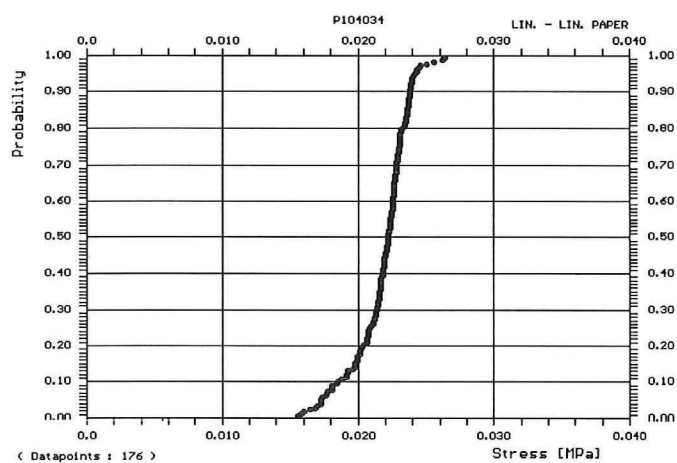
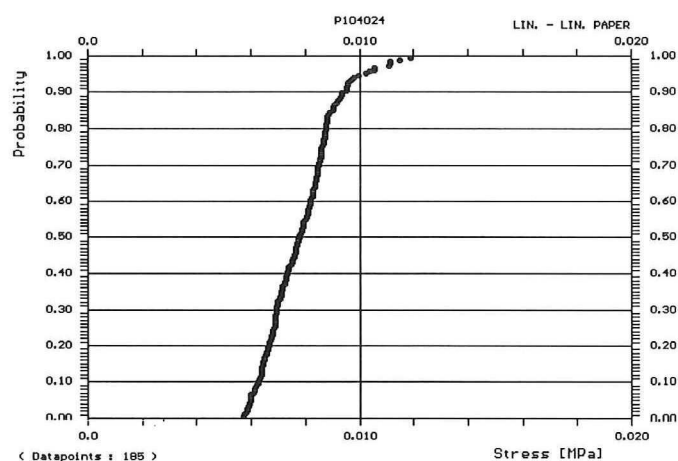
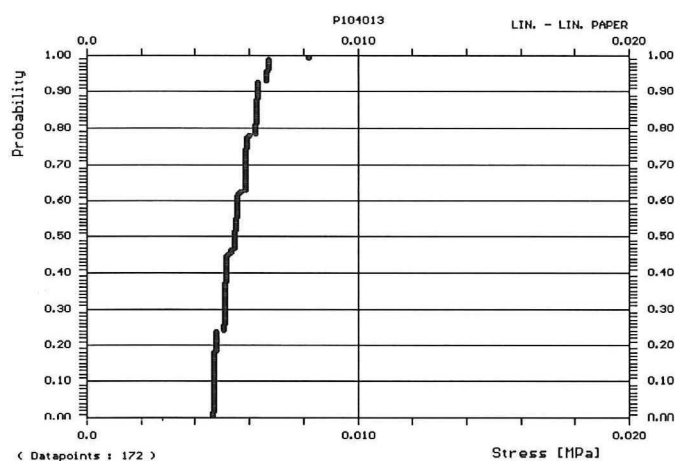


Figure C.25 Probability curves of quasi static maxima of tetrapods 1 to 5.
 $H_s = 0.10$ m. $s_{op} = 0.04$. $h_{toe} = 0.50$ m.
 Orientation of instrumented leg : downward along slope.

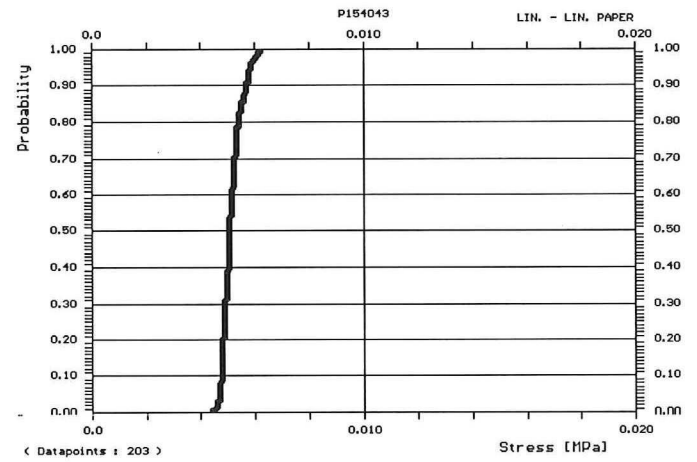
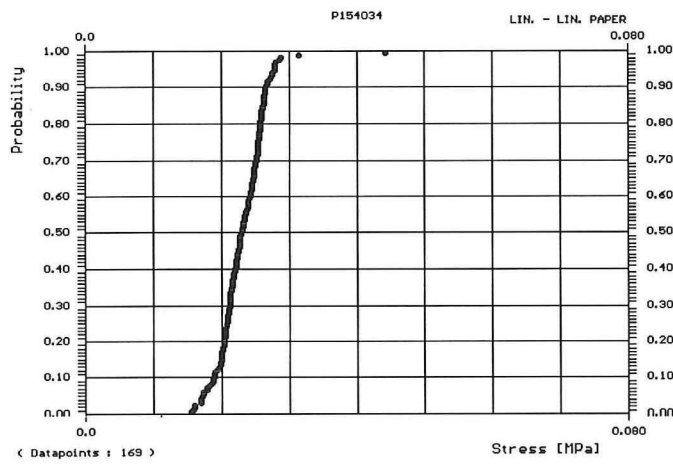
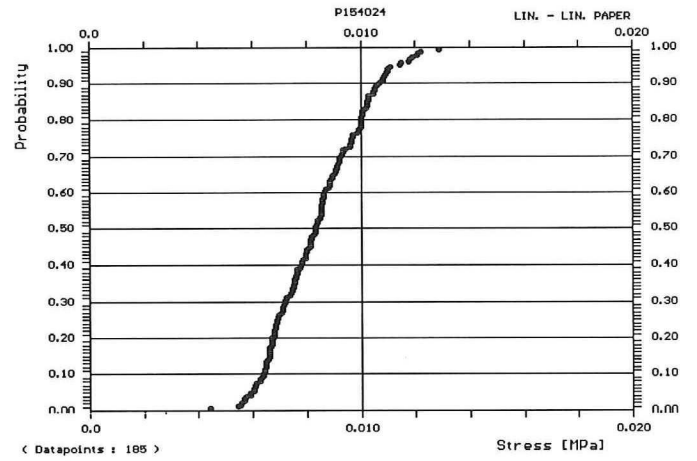
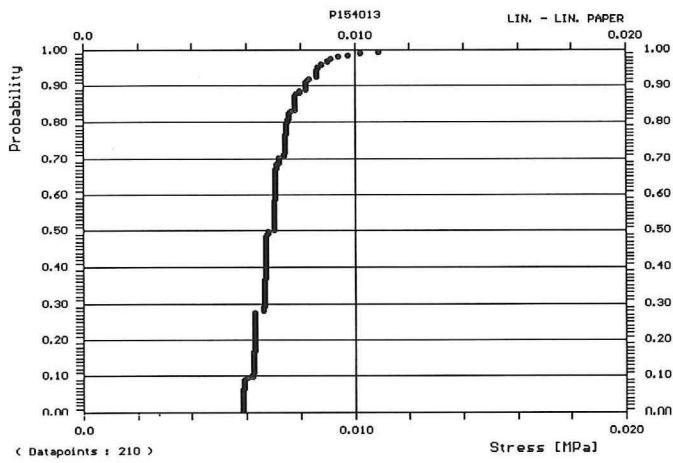


Figure C.26 Probability curves of quasi static maxima of tetrapods 1 to 5.
 $H_s = 0.15$ m. $s_{op} = 0.04$. $h_{toe} = 0.50$ m.
 Orientation of instrumented leg : downward along slope.

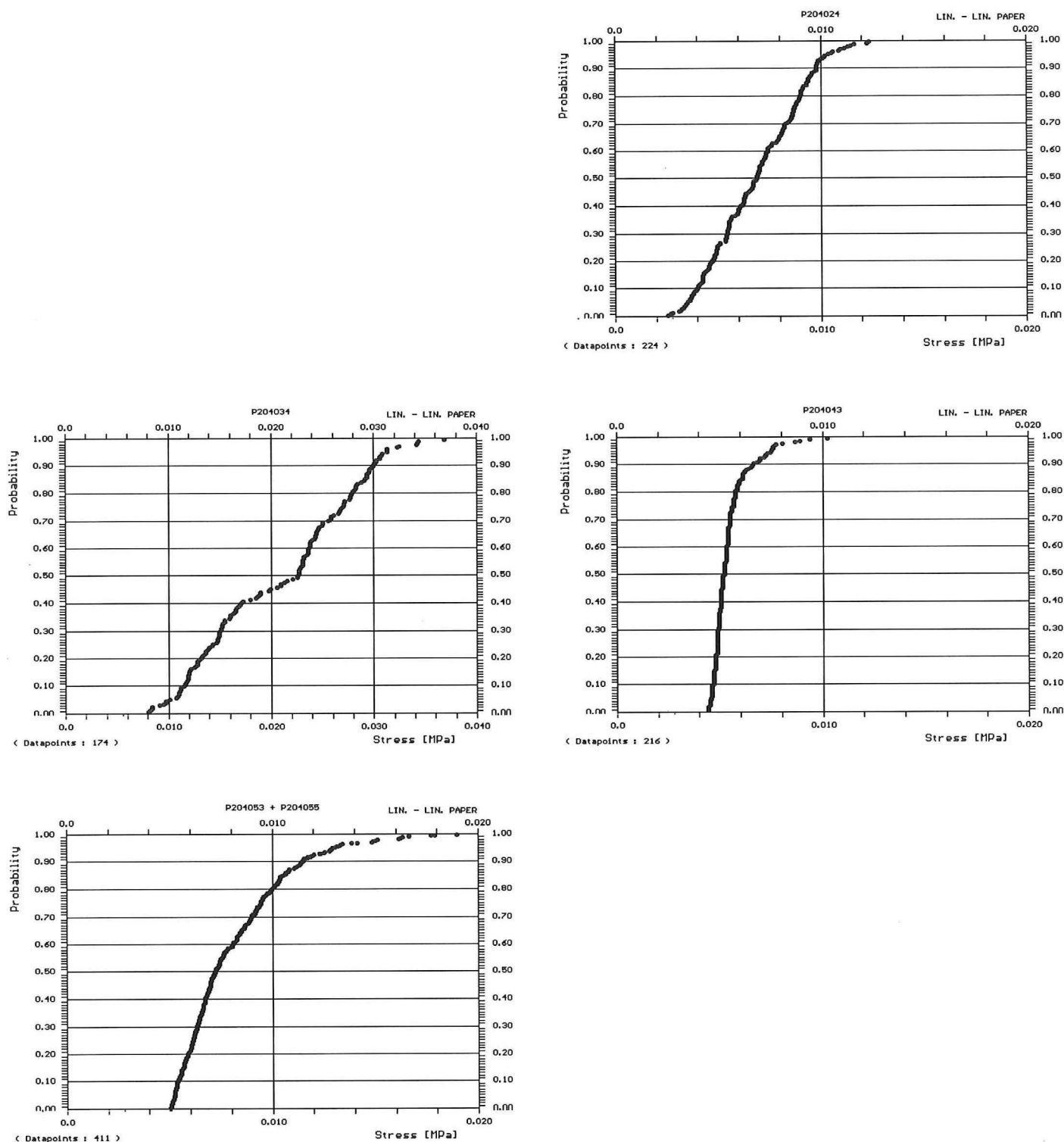


Figure C.27 Probability curves of quasi static maxima of tetrapods 1 to 5.
 $H_s = 0.20$ m. $s_{op} = 0.04$. $h_{toe} = 0.50$ m.
 Orientation of instrumented leg : downward along slope.

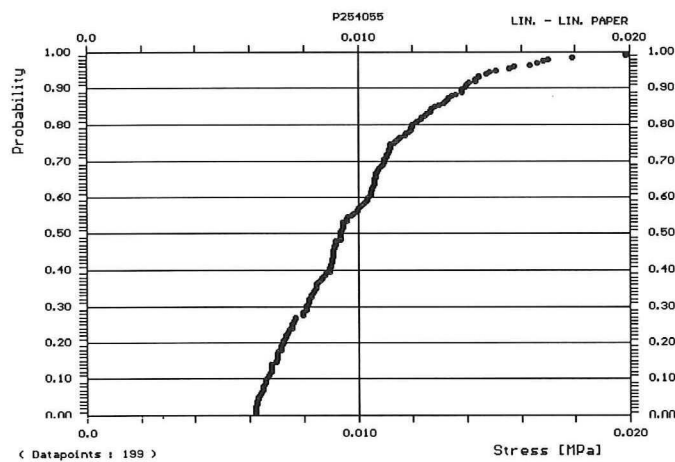
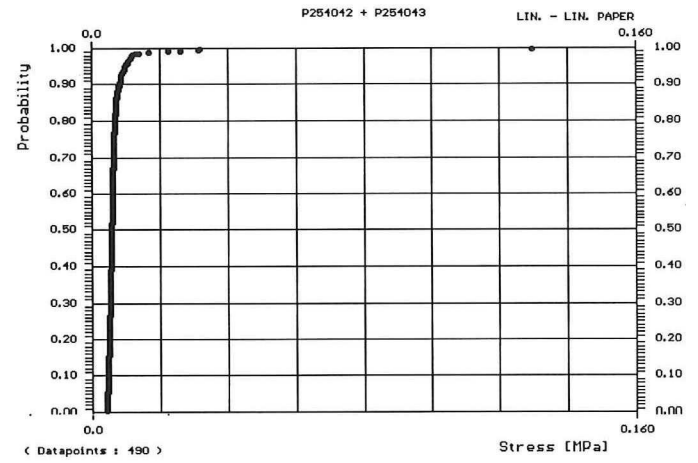
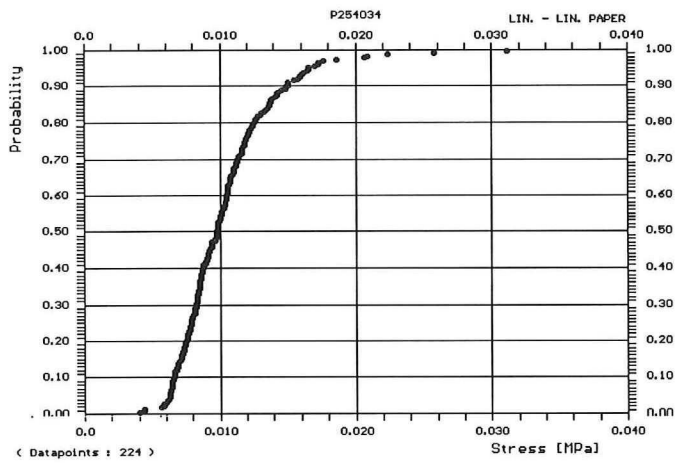
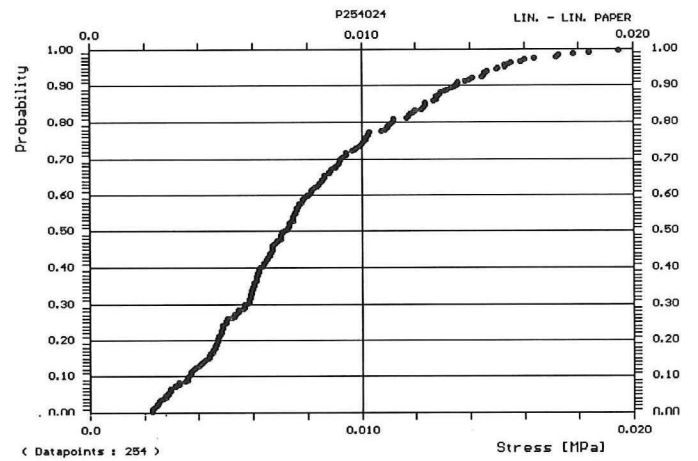


Figure C.28 Probability curves of quasi static maxima of tetrapods 1 to 5.
 $H_s = 0.25$ m. $s_{op} = 0.04$. $h_{toe} = 0.50$ m.
 Orientation of instrumented leg : downward along slope.

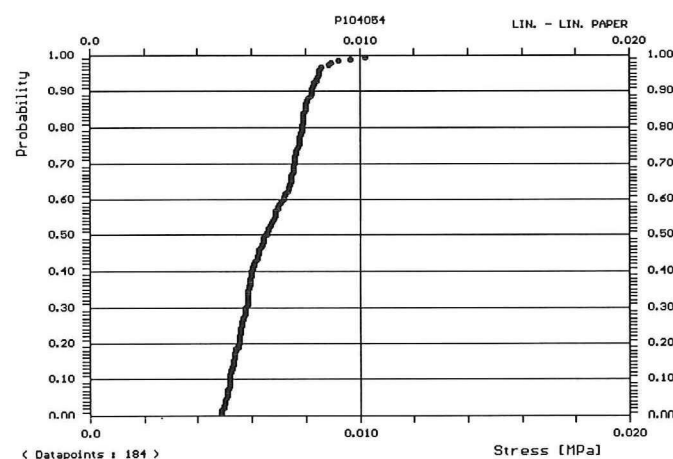
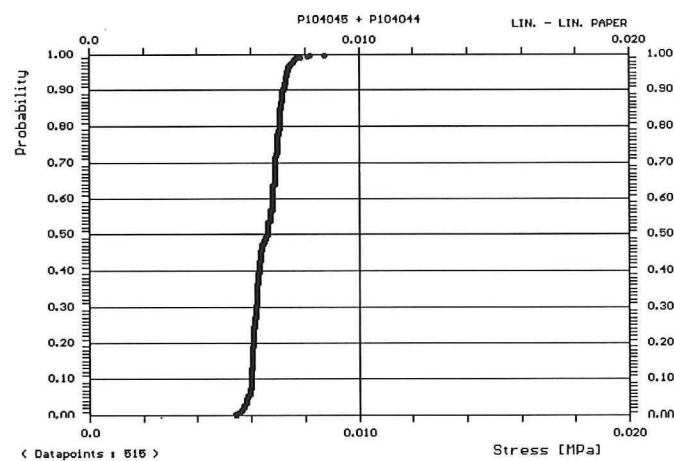
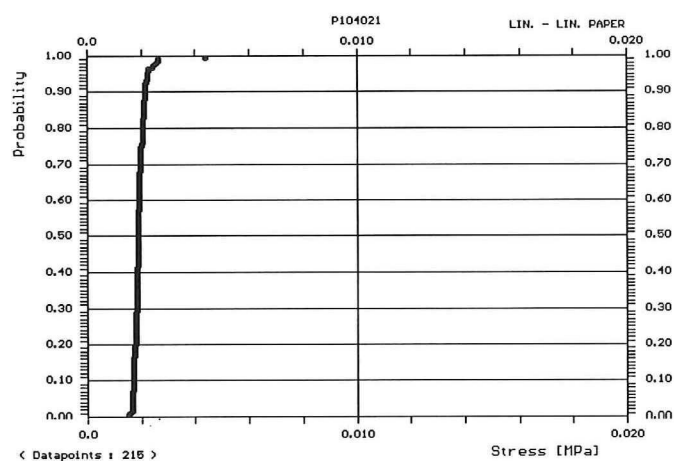
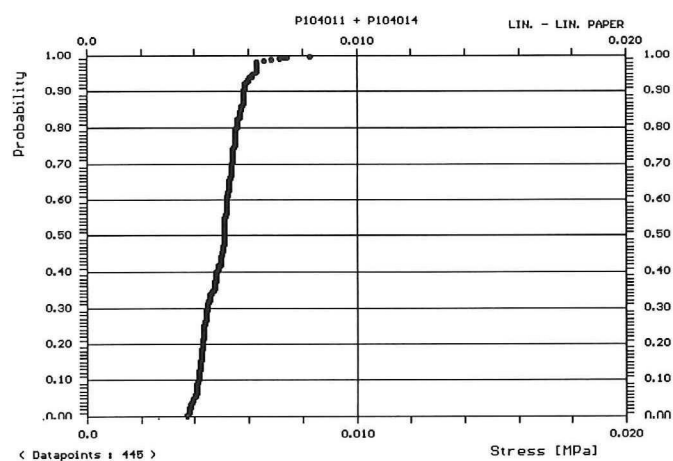


Figure C.29 Probability curves of quasi static maxima of tetrapods 1 to 5.
 $H_s = 0.10$ m. $s_{op} = 0.04$. $h_{toe} = 0.50$ m.
 Orientation of instrumented leg : perpendicular to slope.

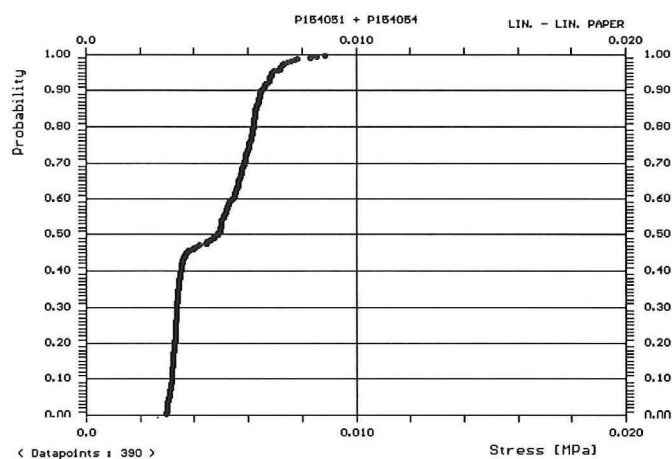
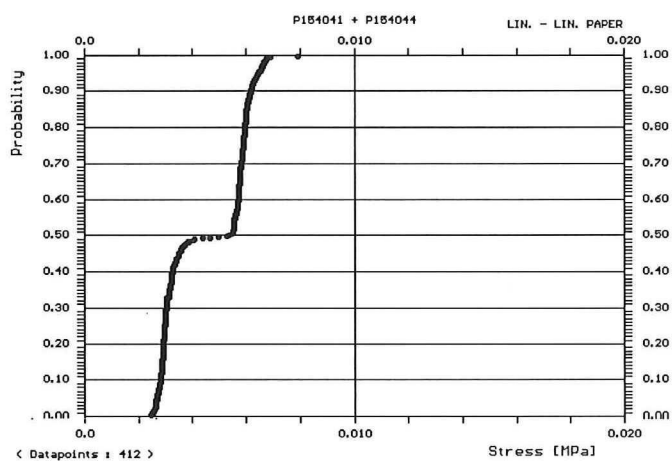
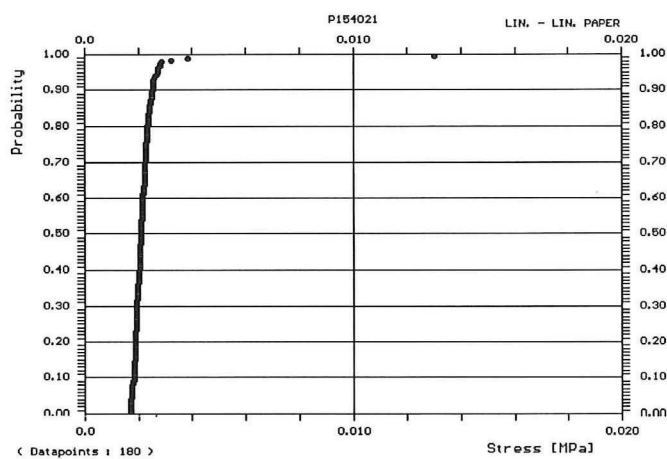
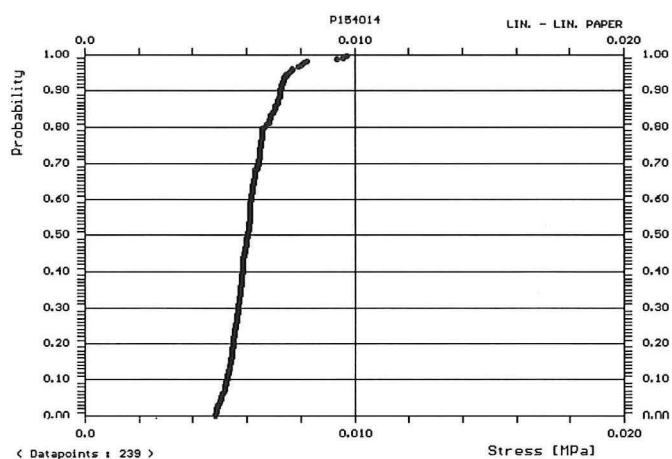


Figure C.30 Probability curves of quasi static maxima of tetrapods 1 to 5.
 $H_s = 0.15$ m. $s_{op} = 0.04$. $h_{toe} = 0.50$ m.
 Orientation of instrumented leg : perpendicular to slope.

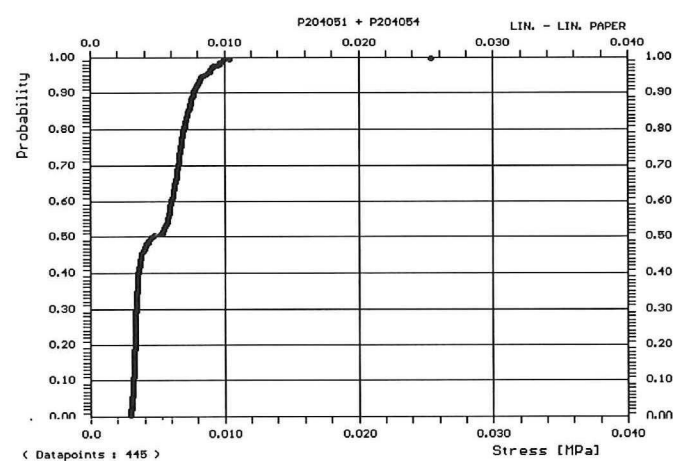
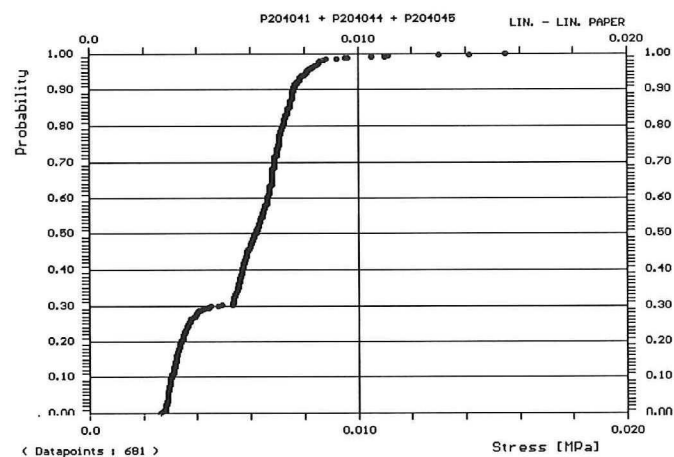
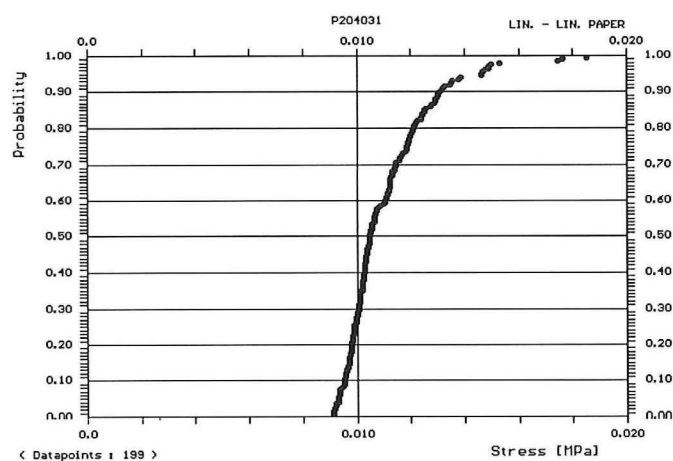
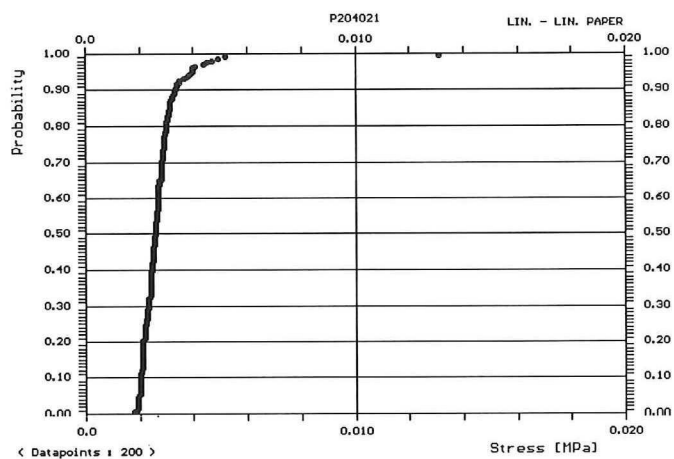
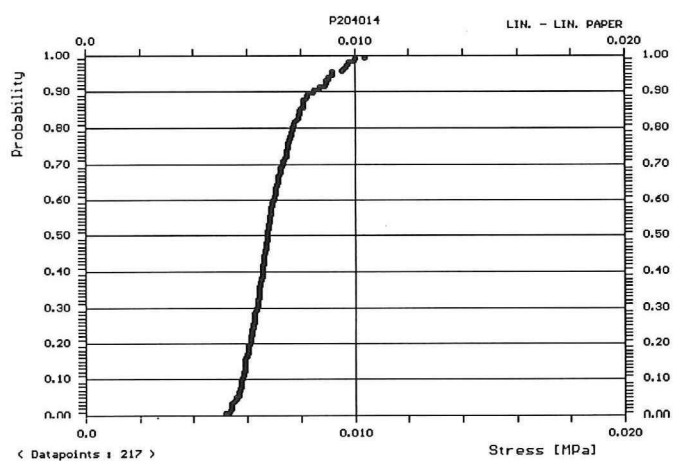


Figure C.31 Probability curves of quasi static maxima of tetrapods 1 to 5.
 $H_s = 0.20$ m. $s_{op} = 0.04$. $h_{toe} = 0.50$ m.
 Orientation of instrumented leg : perpendicular to slope.

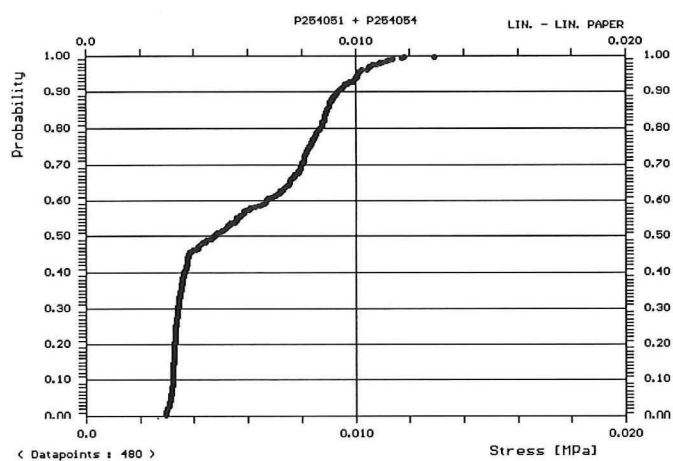
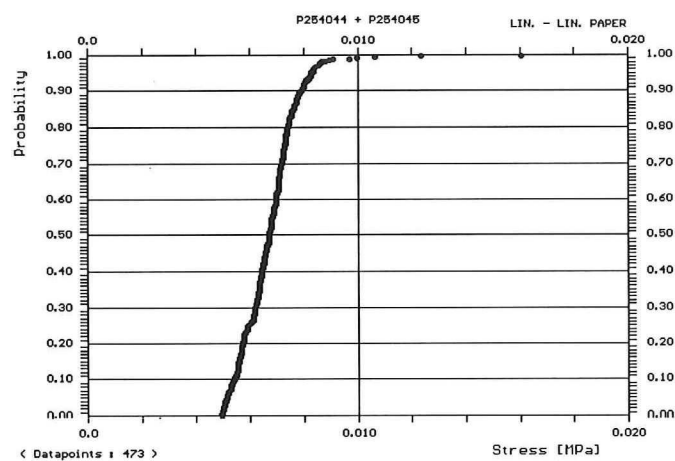
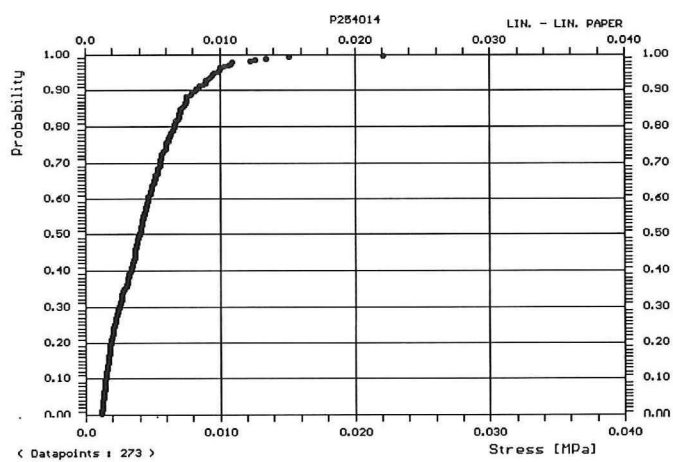


Figure C.32 Probability curves of quasi static maxima of tetrapods 1 to 5.
 $H_s = 0.25$ m. $s_{op} = 0.04$. $h_{toe} = 0.50$ m.
 Orientation of instrumented leg : perpendicular to slope.

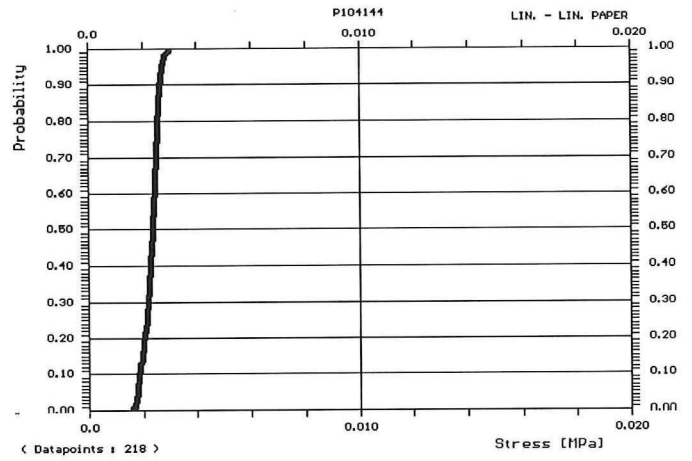
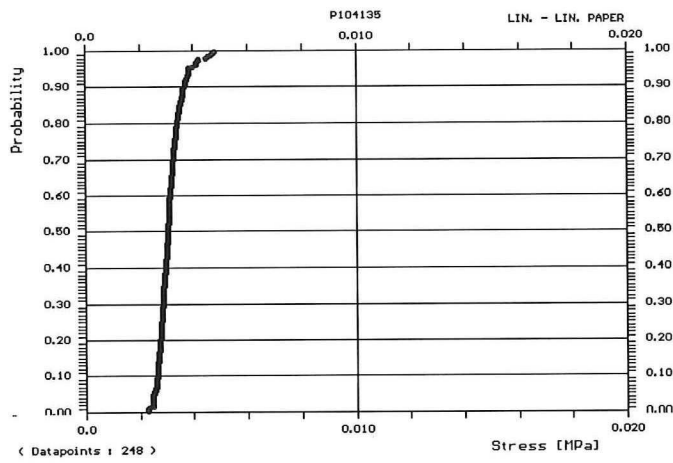
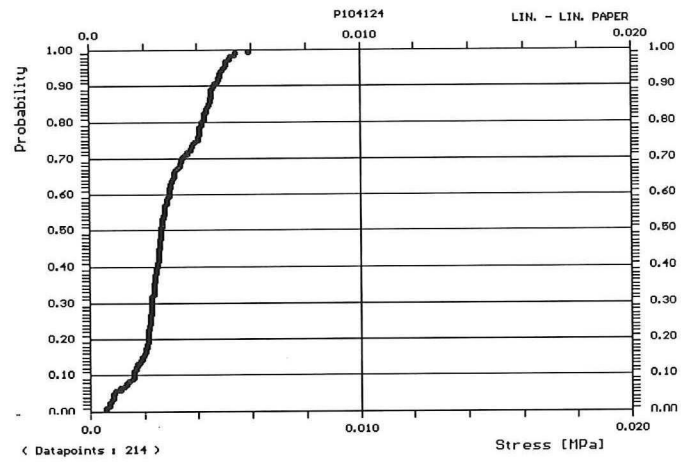
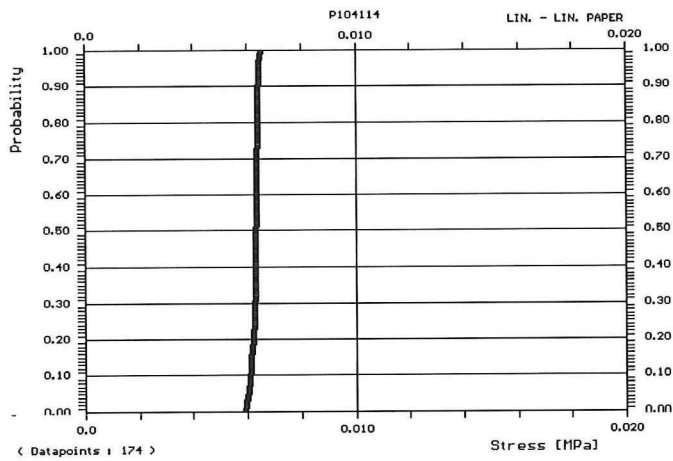


Figure C.33 Probability curves of quasi static maxima of tetrapods 1 to 5.

$H_s = 0.10$ m. $s_{op} = 0.04$. $h_{toe} = 0.30$ m.

Orientation of instrumented leg : downward along slope.

All tetrapods are situated in the toplayer of the armour layer. Locations are -4, -2, -1, 1, 2 y/D_n m relative to the water level for tetrapod 1 to 5 respectively.

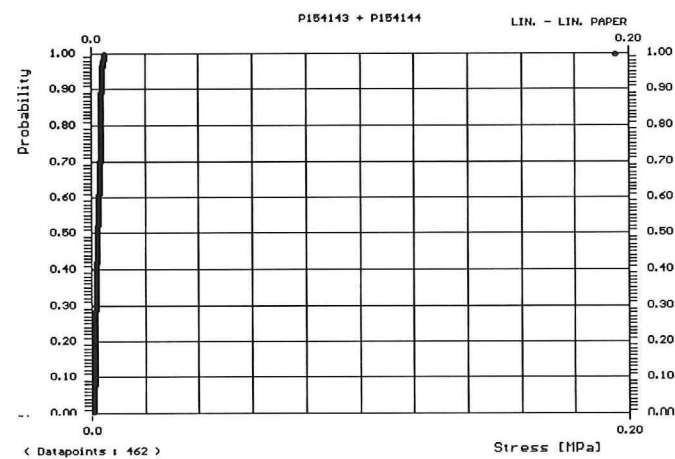
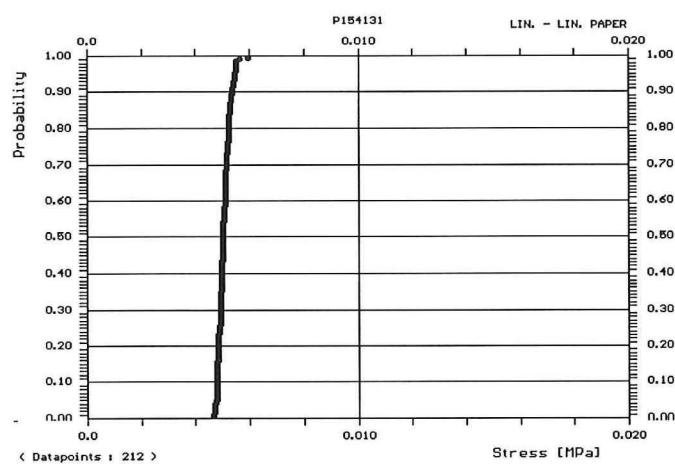
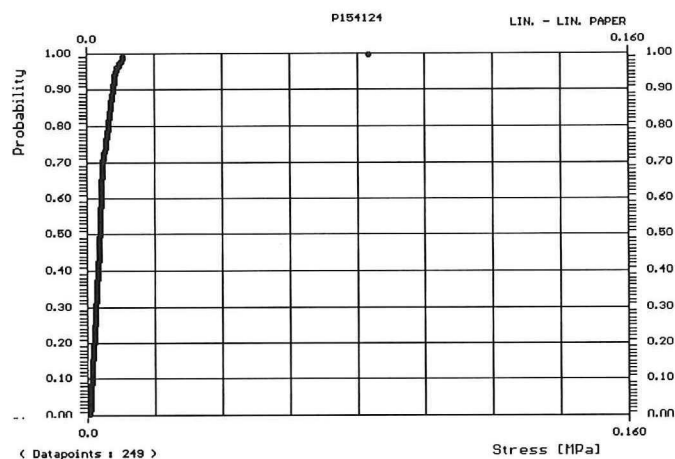
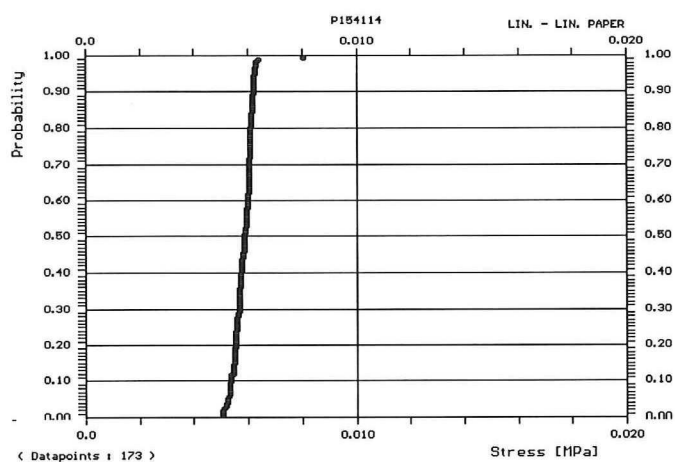


Figure C.34 Probability curves of quasi static maxima of tetrapods 1 to 5.

$H_s = 0.15$ m. $s_{op} = 0.04$. $h_{toe} = 0.30$ m.

Orientation of instrumented leg : downward along slope.

All tetrapods are situated in the toplayer of the armour layer. Locations are -4, -2, -1, 1, 2 y/D_n m relative to the water level for tetrapod 1 to 5 respectively.

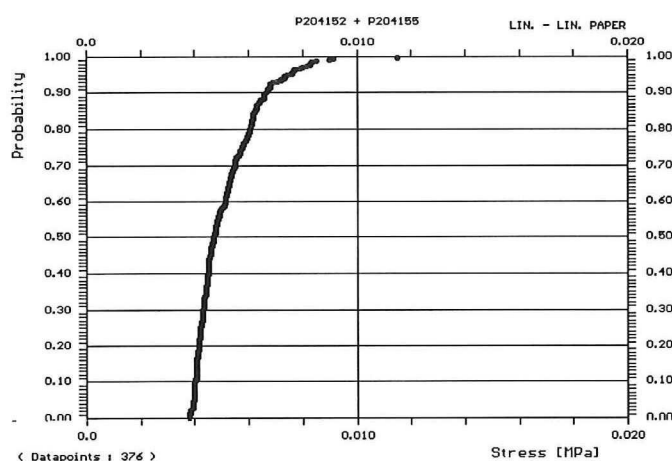
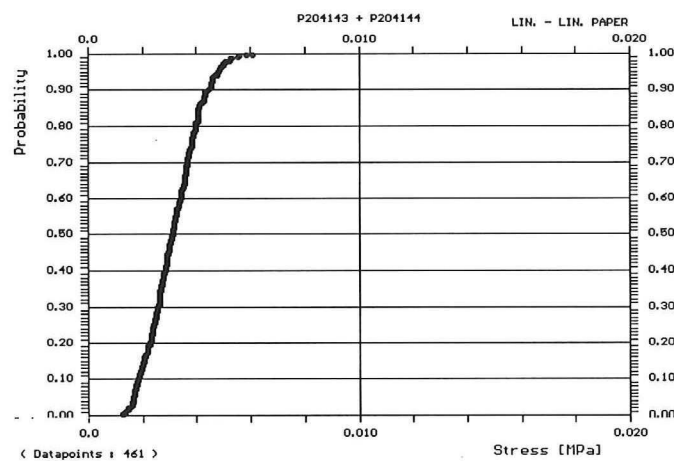
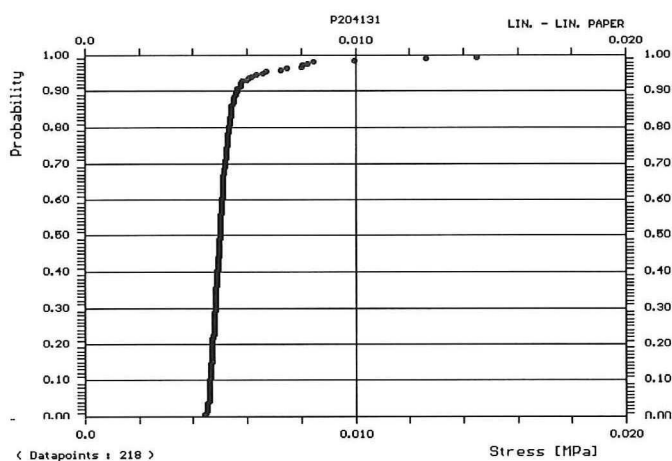
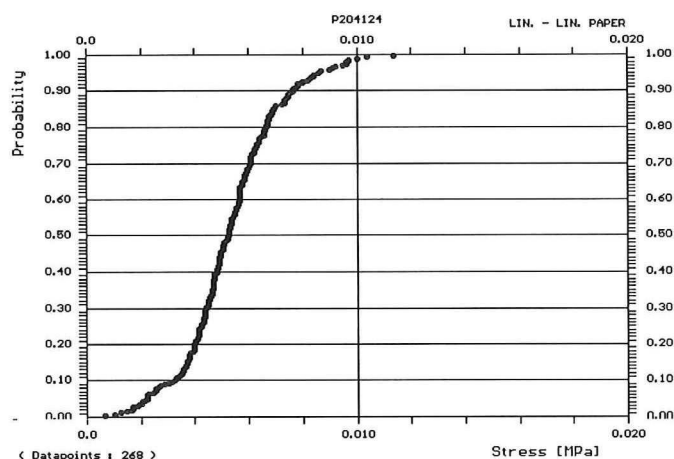
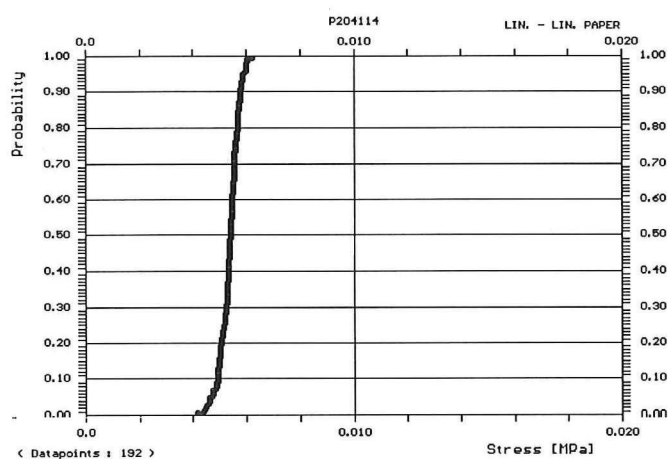


Figure C.35 Probability curves of quasi static maxima of tetrapods 1 to 5.

$H_s = 0.20$ m. $s_{op} = 0.04$. $h_{toe} = 0.30$ m.

Orientation of instrumented leg : downward along slope.

All tetrapods are situated in the toplayer of the armour layer. Locations are -4, -2, -1, 1, 2 y/D_n m relative to the water level for tetrapod 1 to 5 respectively.

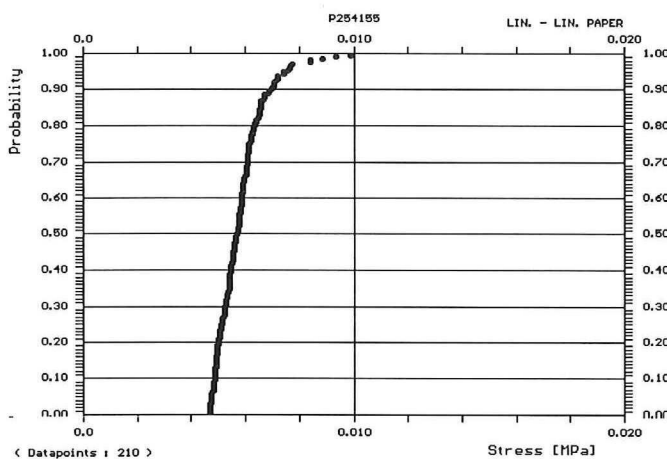
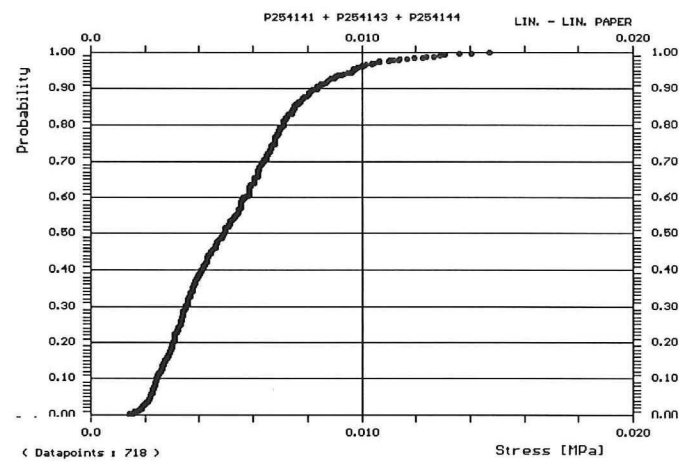
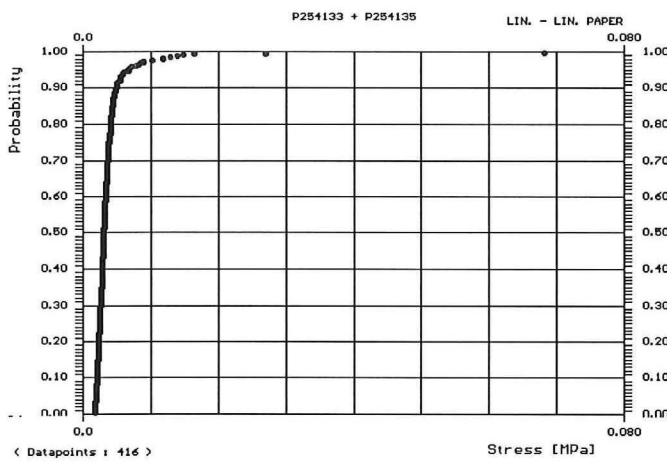
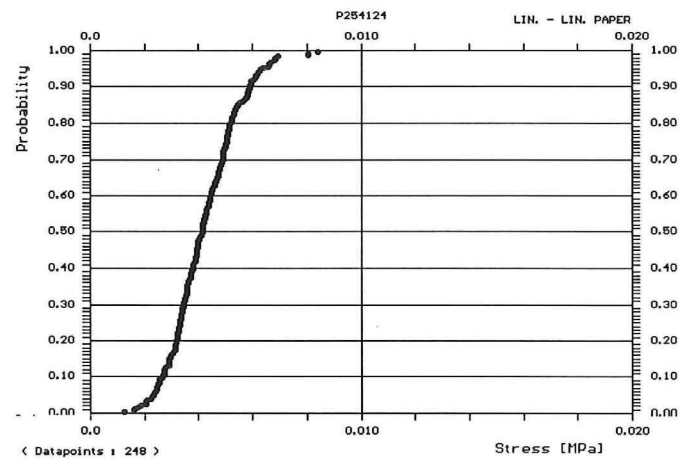
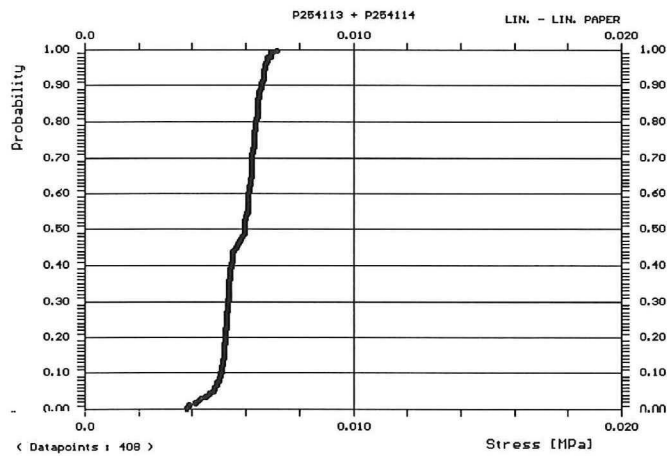


Figure C.36 Probability curves of quasi static maxima of tetrapods 1 to 5.

$H_s = 0.25$ m. $s_{op} = 0.04$. $h_{toe} = 0.30$ m.

Orientation of instrumented leg : downward along slope.

All tetrapods are situated in the toplayer of the armour layer. Locations are -4, -2, -1, 1, 2 y/D_n m relative to the water level for tetrapod 1 to 5 respectively.

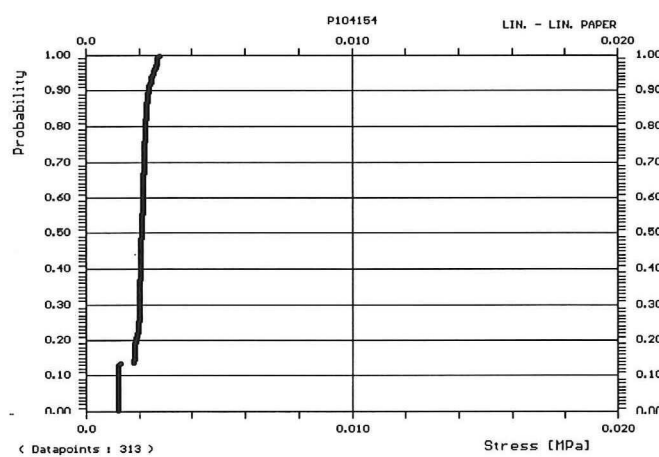
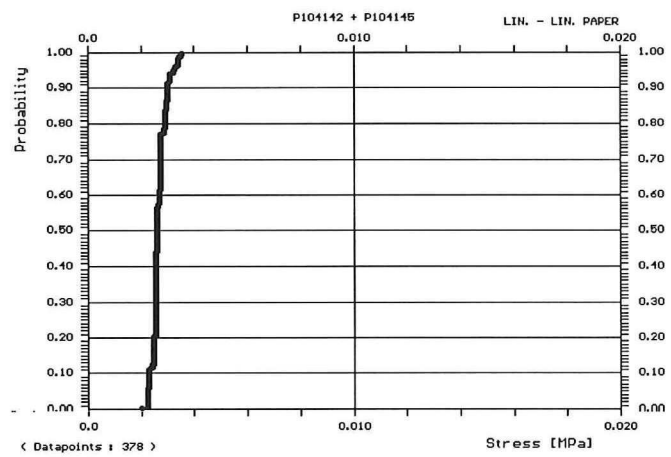
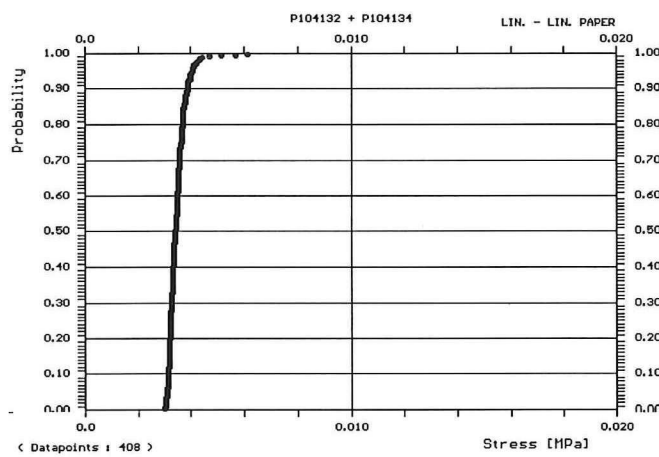
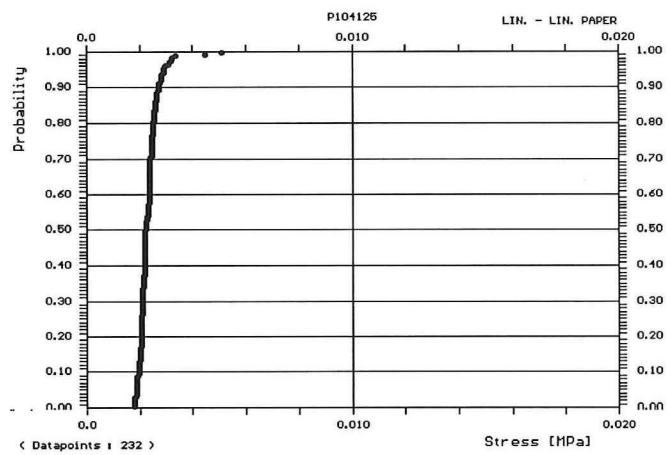
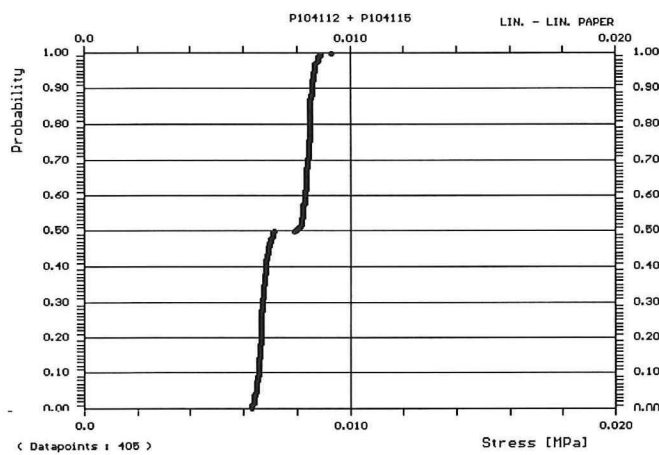


Figure C.37 Probability curves of quasi static maxima of tetrapods 1 to 5.

$H_s = 0.10$ m. $s_{op} = 0.04$. $h_{toe} = 0.30$ m.

Orientation of instrumented leg : perpendicular to slope.

All tetrapods are situated in the toplayer of the armour layer. Locations are -4, -2, -1, 1, 2 y/D_n m relative to the water level for tetrapods 1 to 5 respectively.

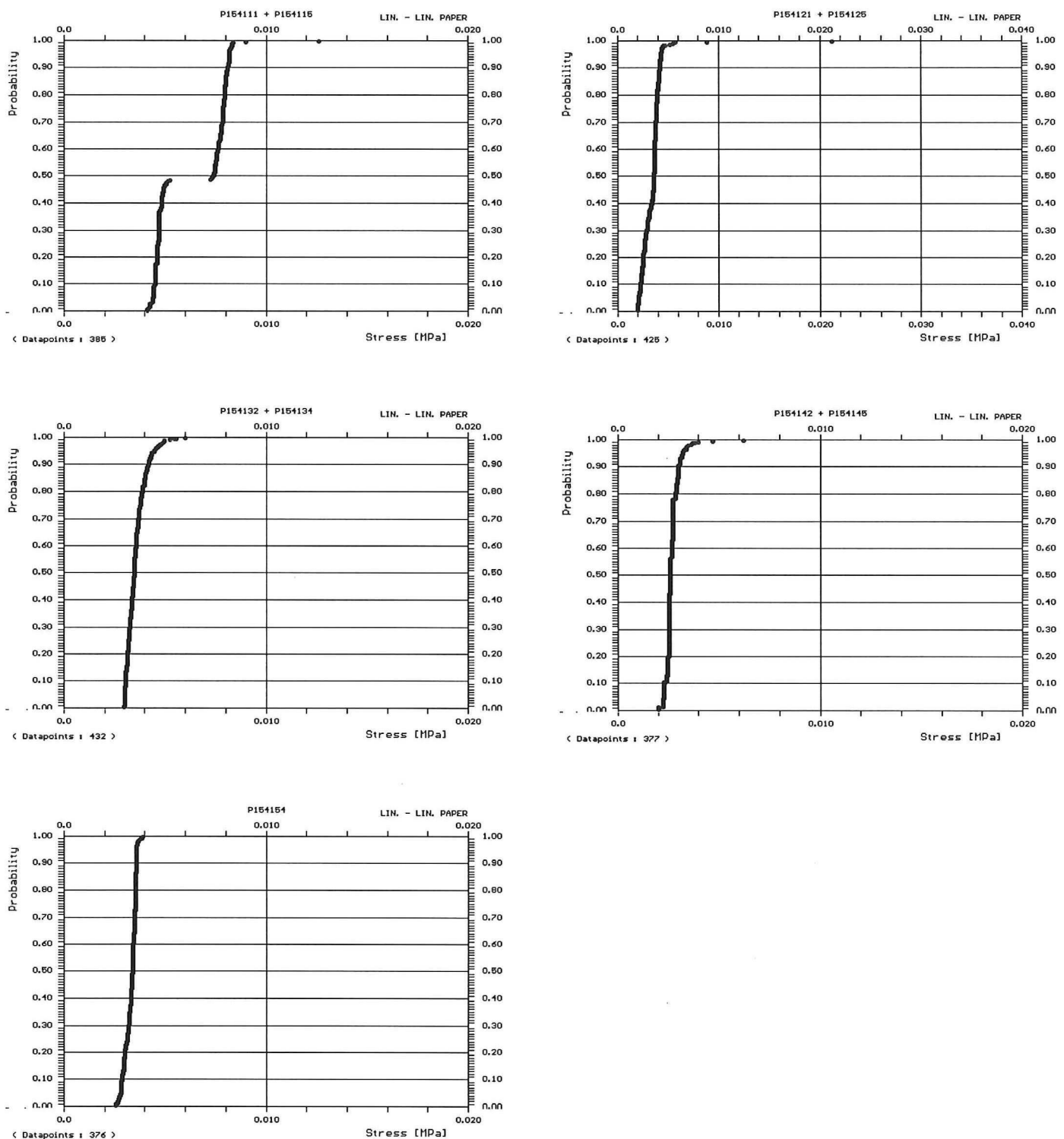


Figure C.38 Probability curves of quasi static maxima of tetrapods 1 to 5.

$H_s = 0.15$ m. $s_{op} = 0.04$. $h_{toe} = 0.30$ m.

Orientation of instrumented leg : perpendicular to slope.

All tetrapods are situated in the toplayer of the armour layer. Locations are -4, -2, -1, 1, 2 y/D_n m relative to the water level for tetrapods 1 to 5 respectively.

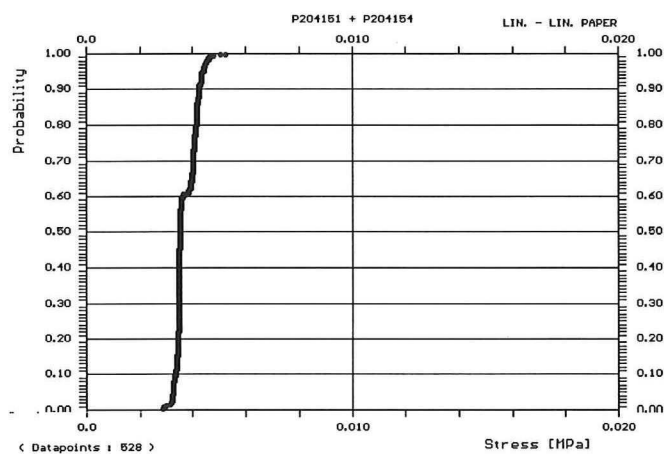
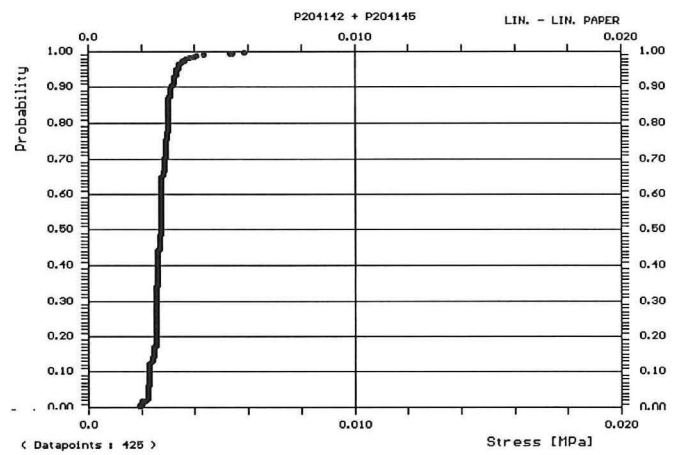
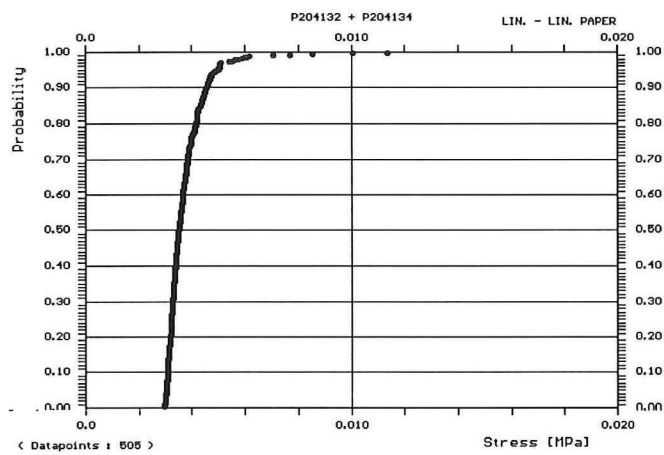
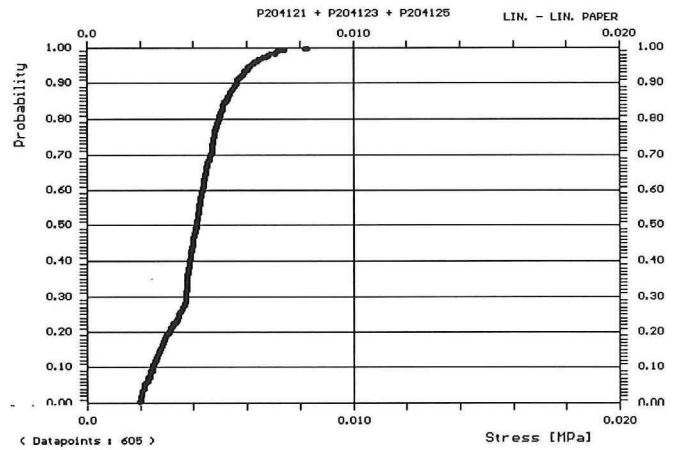
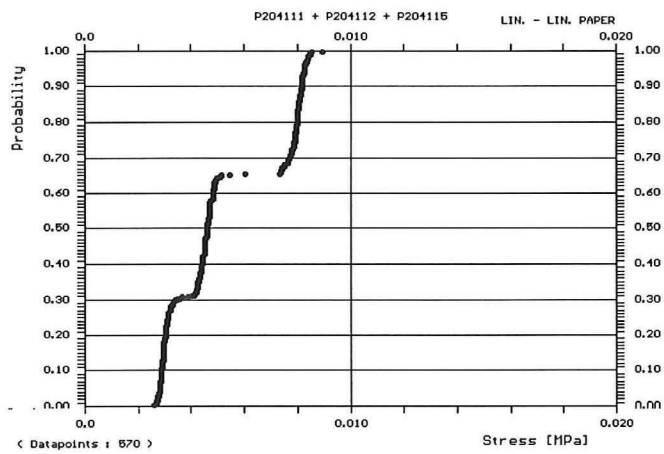


Figure C.39 Probability curves of quasi static maxima of tetrapods 1 to 5.

$H_s = 0.20$ m. $s_{op} = 0.04$. $h_{toe} = 0.30$ m.

Orientation of instrumented leg : perpendicular to slope.

All tetrapods are situated in the toplayer of the armour layer. Locations are -4, -2, -1, 1, 2 y/D_n m relative to the water level for tetrapod 1 to 5 respectively.

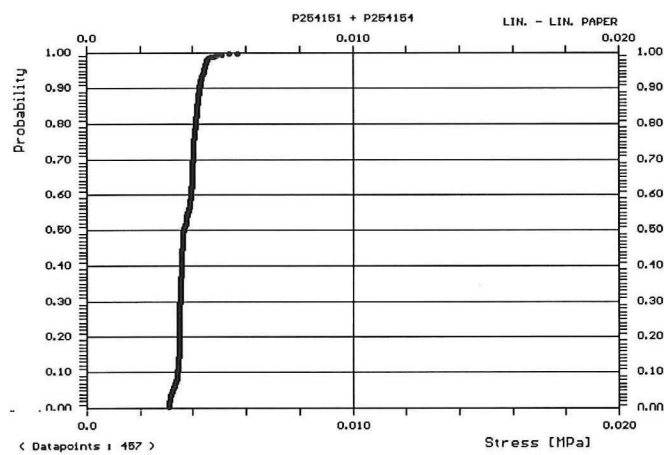
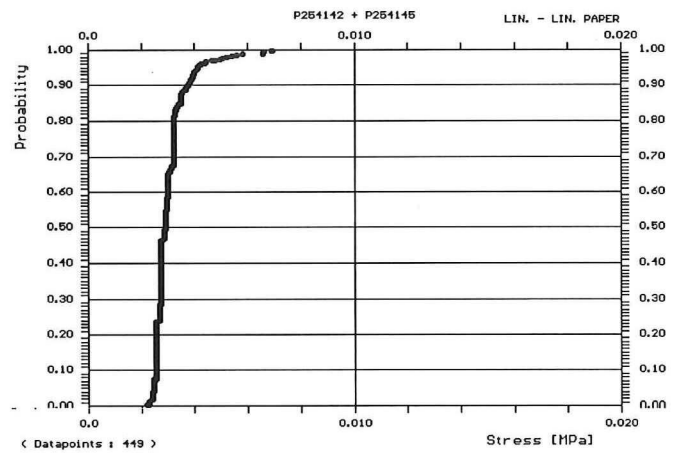
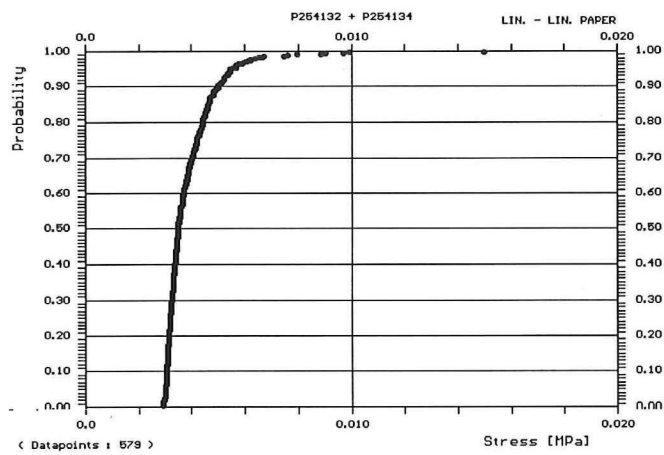
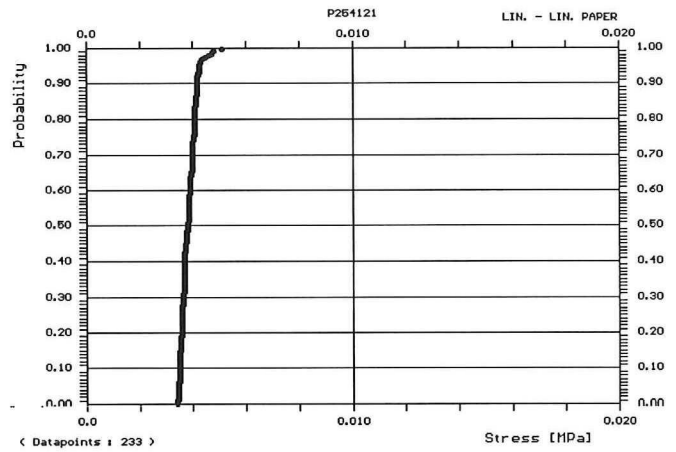
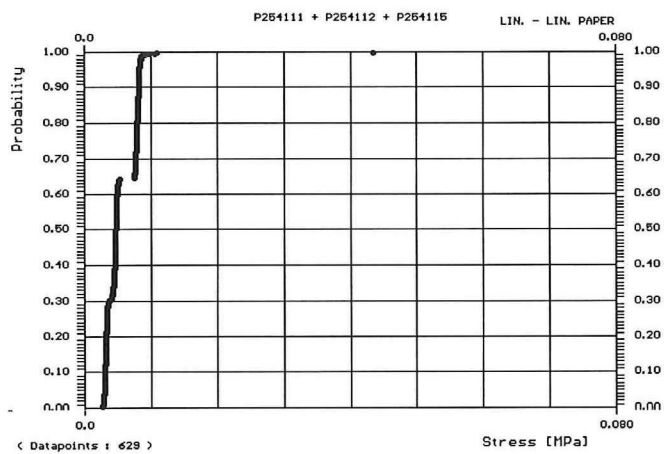


Figure C.40 Probability curves of quasi static maxima of tetrapods 1 to 5.

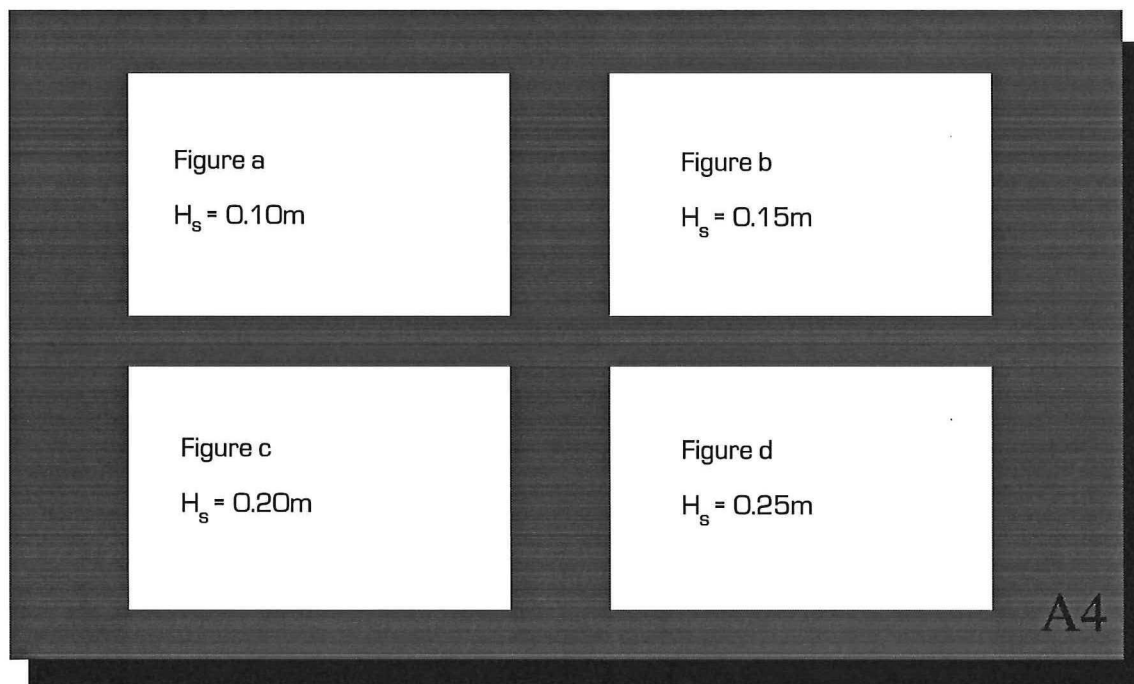
$H_s = 0.25$ m. $s_{op} = 0.04$. $h_{toe} = 0.30$ m.

Orientation of instrumented leg : perpendicular to slope.

All tetrapods are situated in the toplayer of the armour layer. Locations are -4, -2, -1, 1, 2 y/D_n m relative to the water level for tetrapods 1 to 5 respectively.

Appendix D

Each Figure contains 4 histograms. Between these four histograms only the significant wave height, H_s , changes according to the below presented scheme :



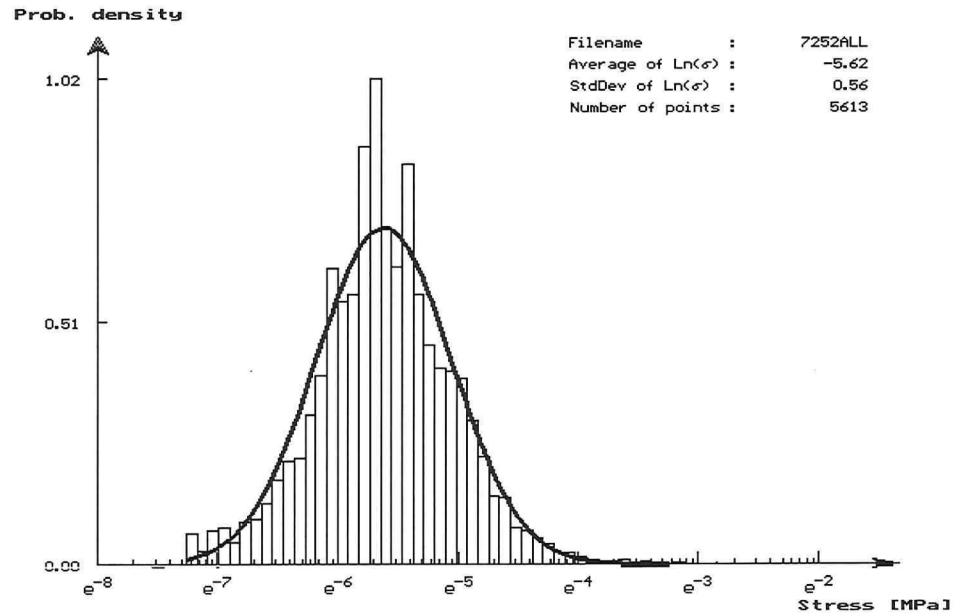
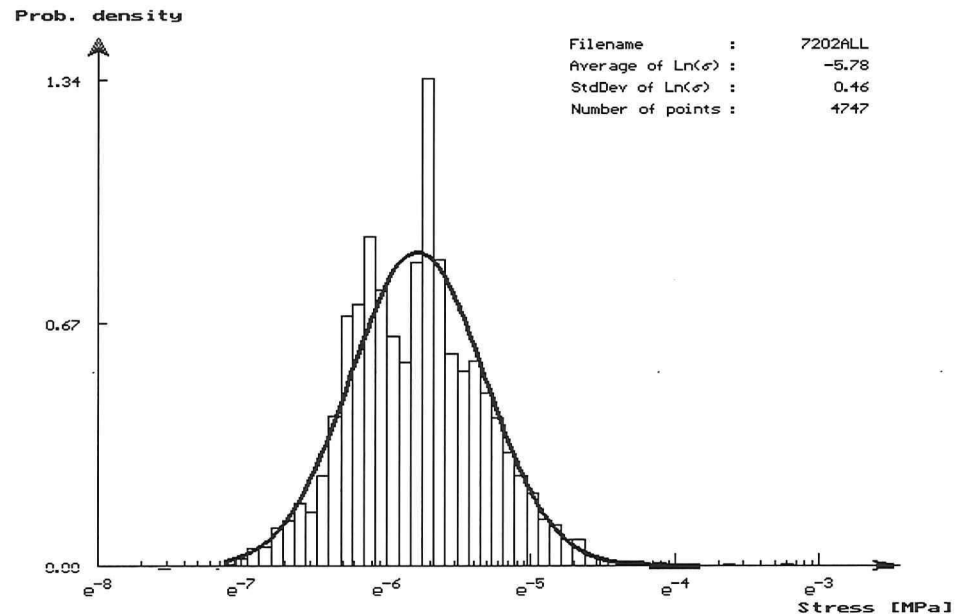
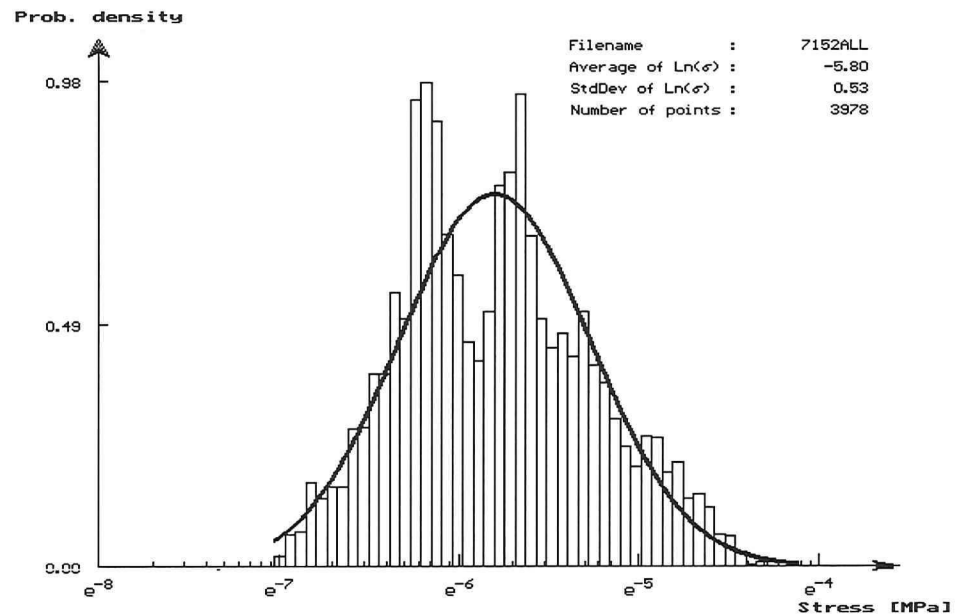
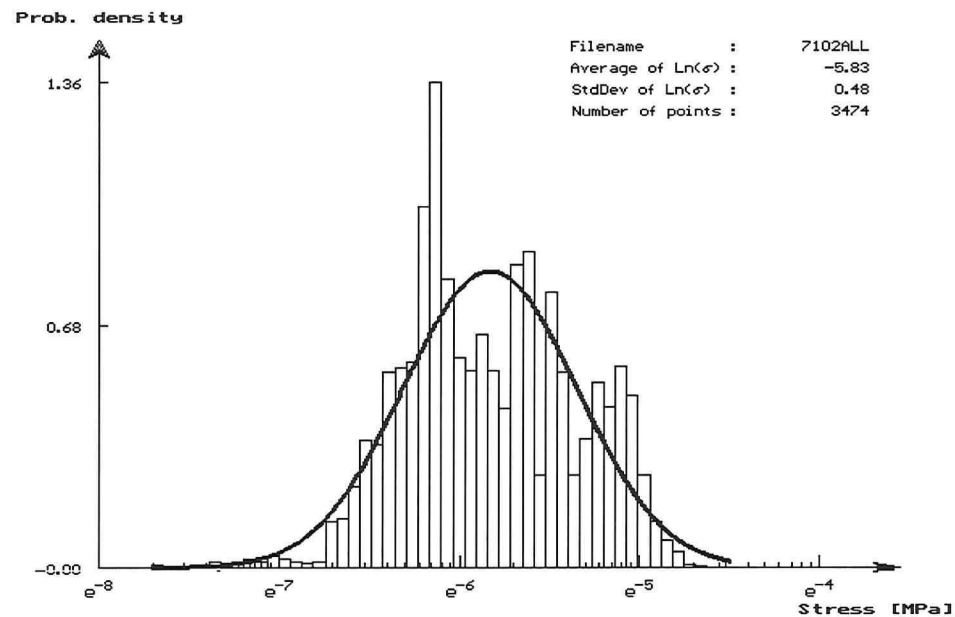


Figure D.1 Histograms of pulsating maxima. $h_{\text{toe}} = 0.30$ m. and $s_{\text{op}} = 0.02$

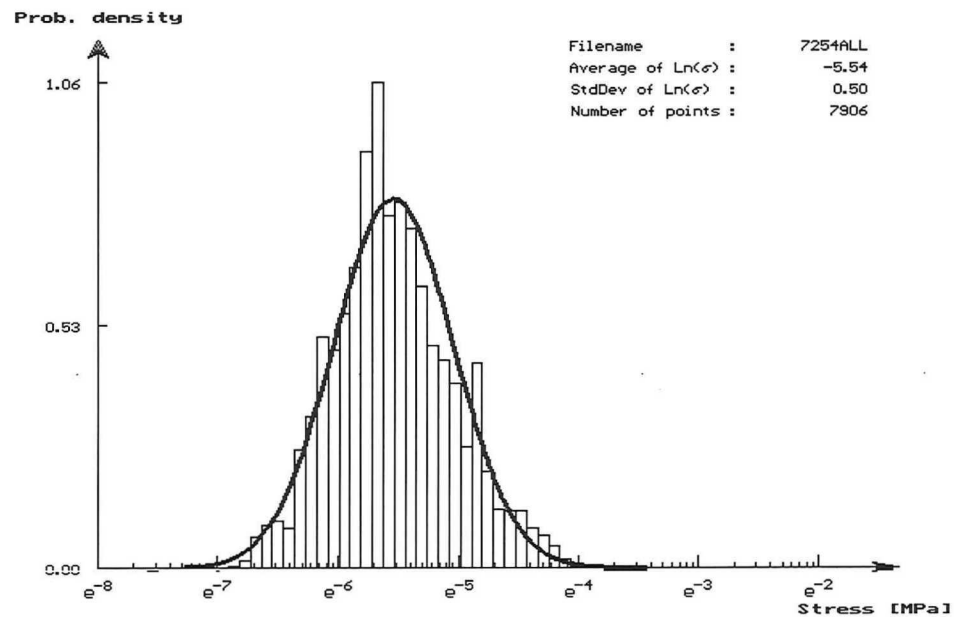
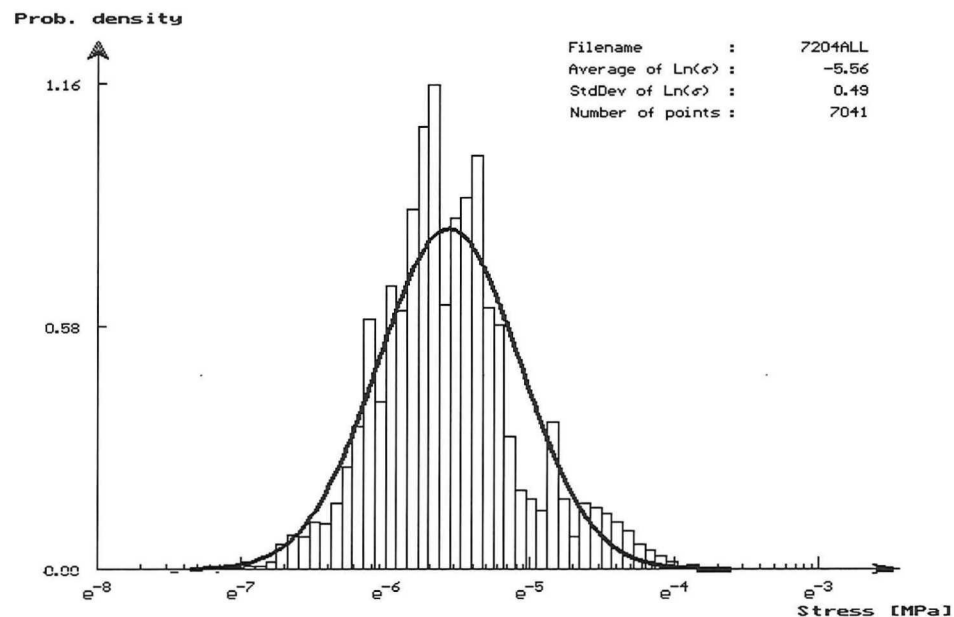
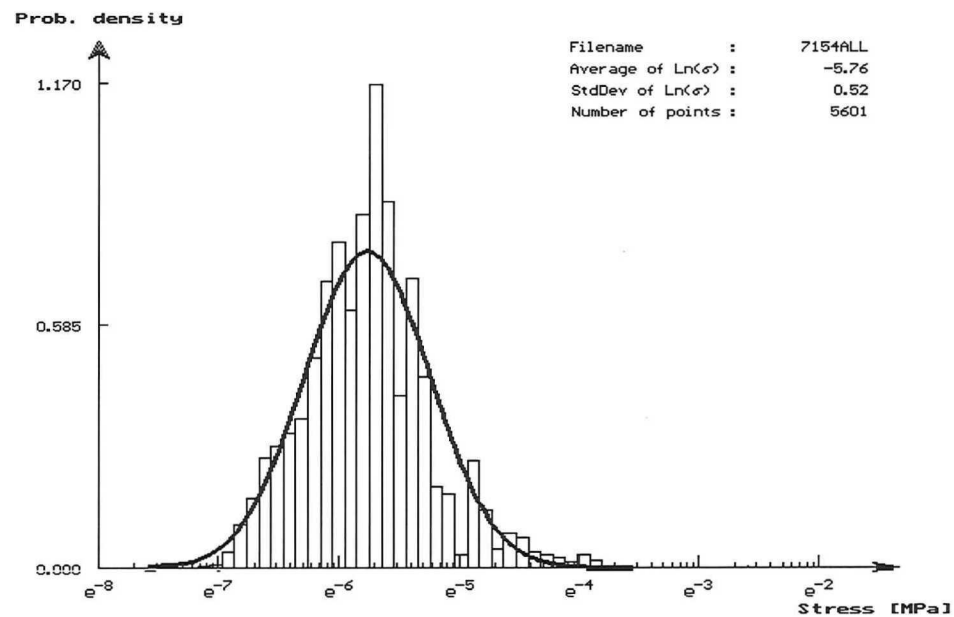
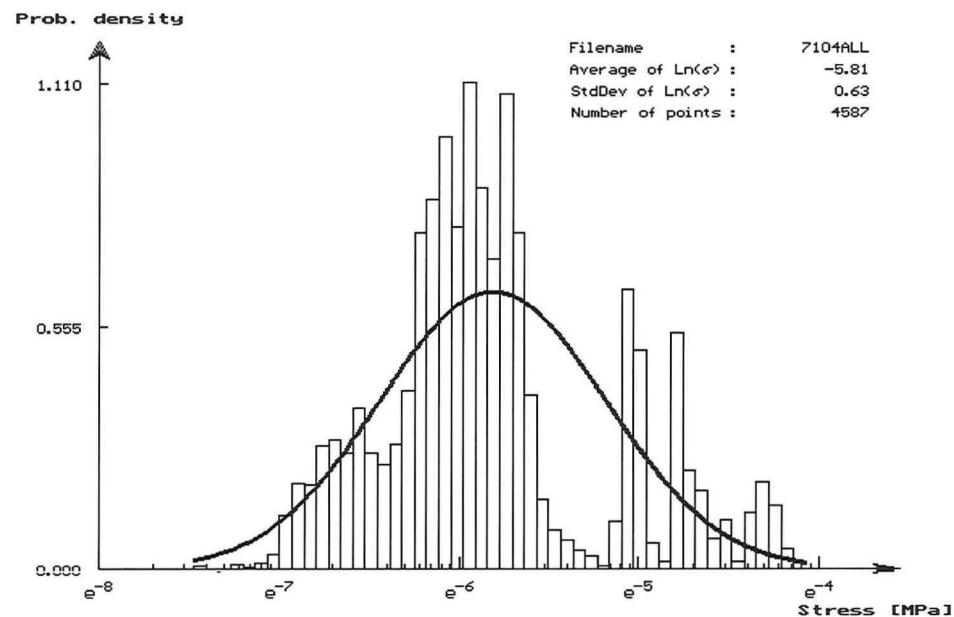


Figure D.2 Histograms of pulsating maxima. $h_{toe} = 0.50$ m. and $s_{op} = 0.04$

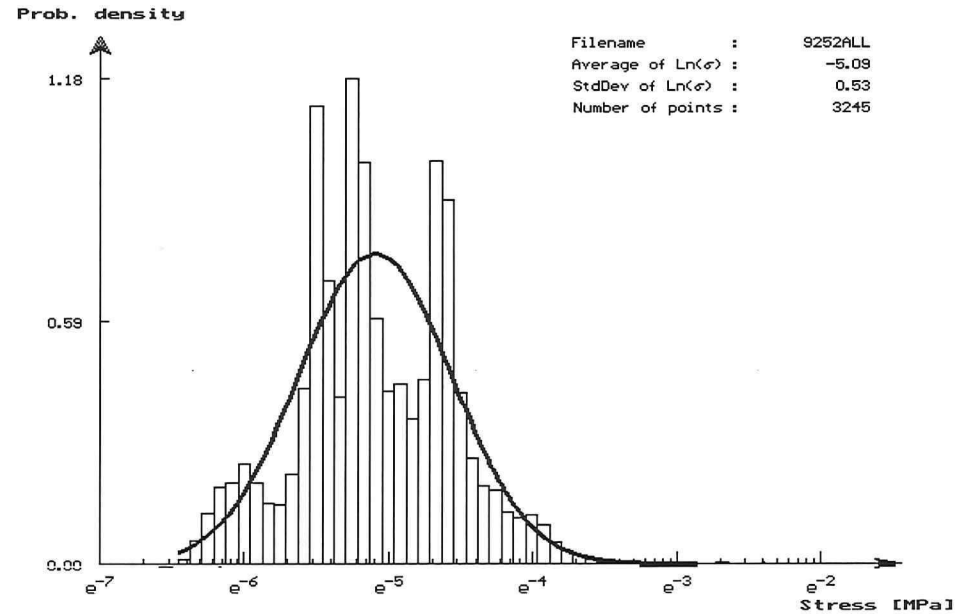
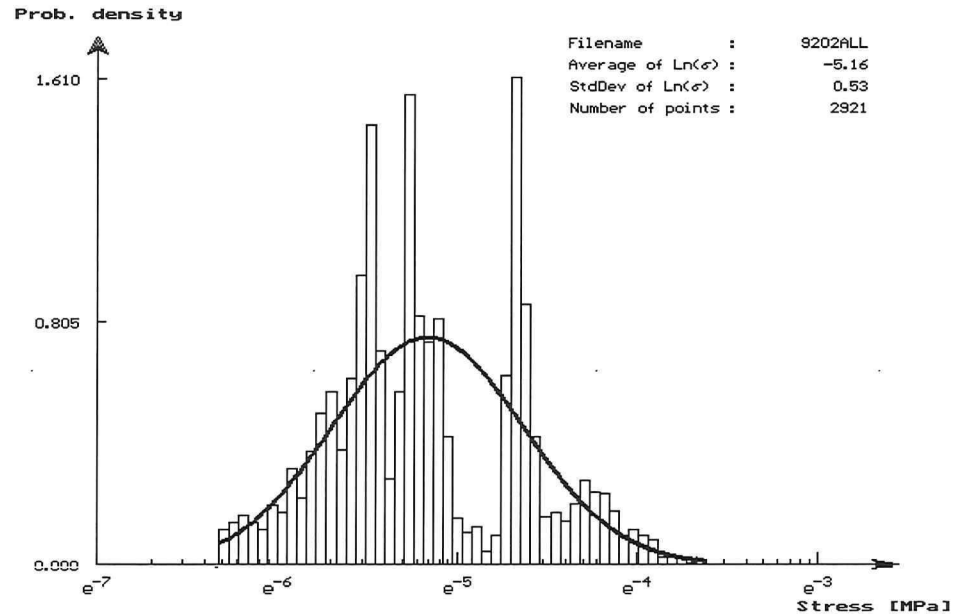
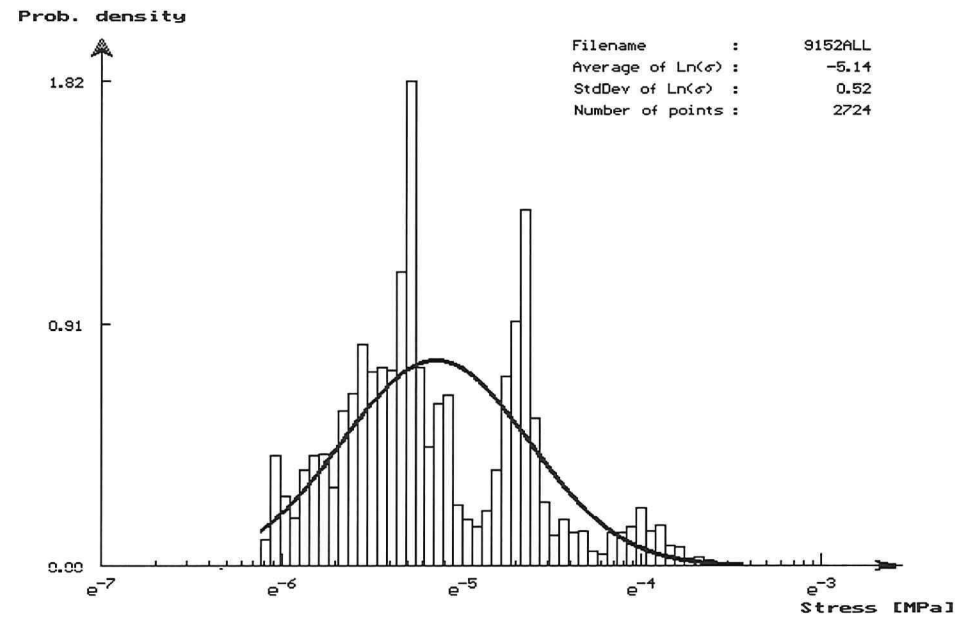
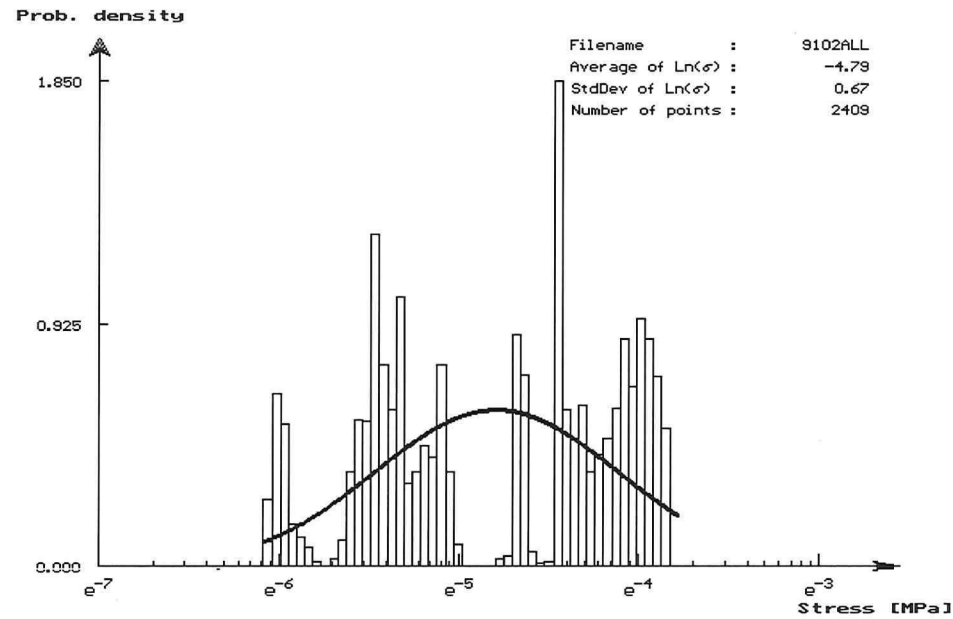


Figure D.3 Histograms of pulsating maxima. $h_{\text{toe}} = 0.50$ m. and $s_{\text{op}} = 0.02$

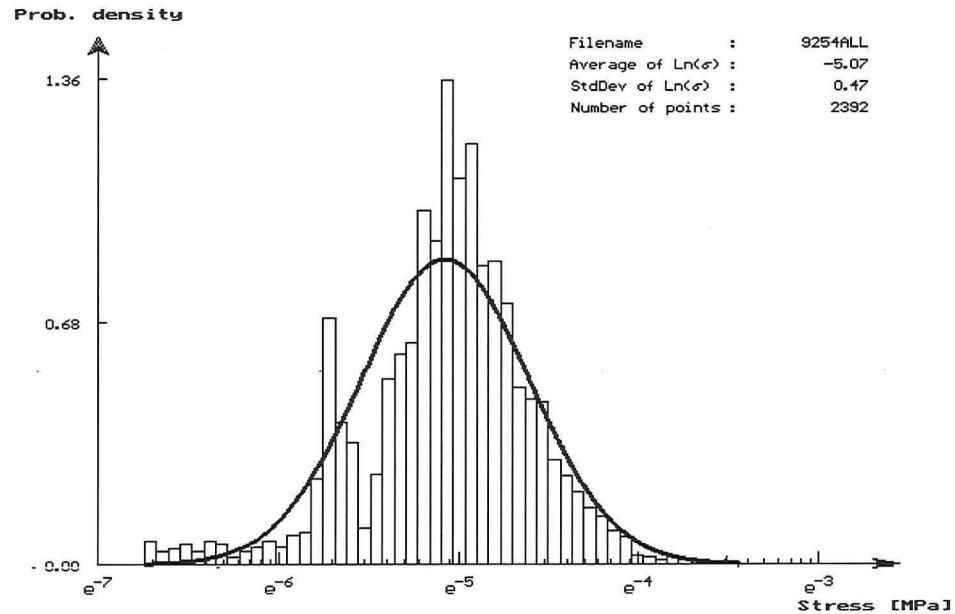
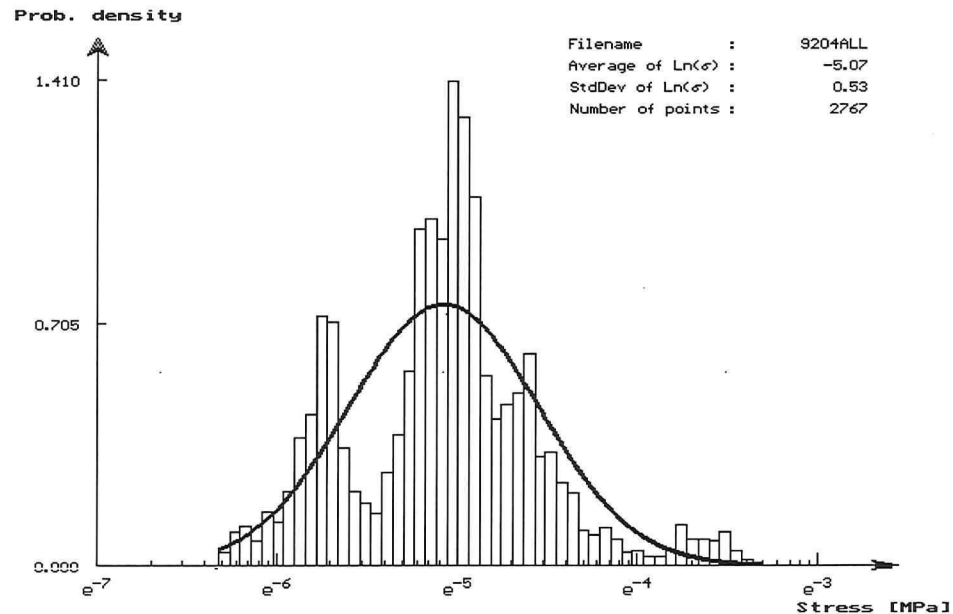
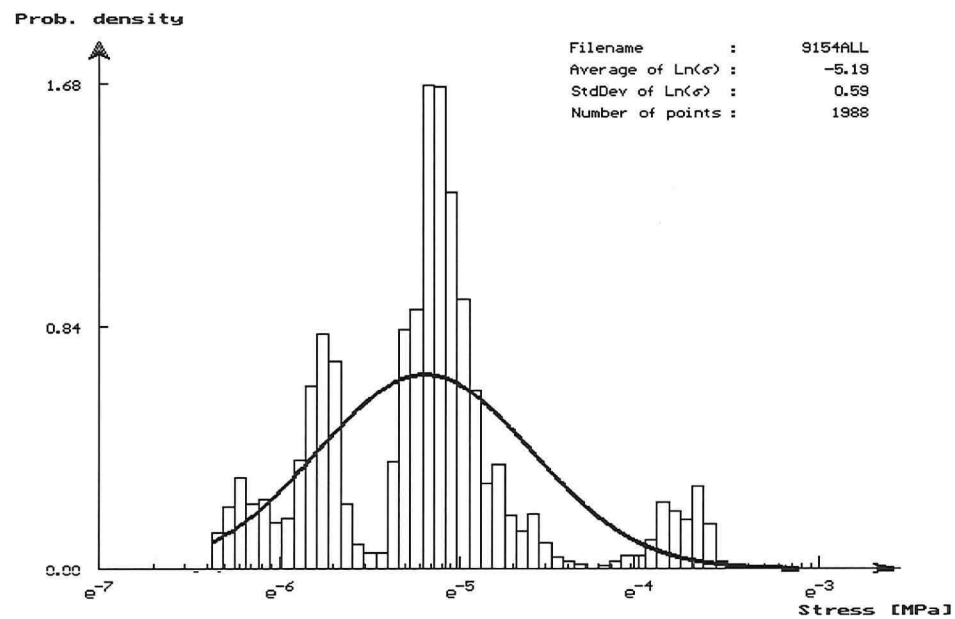
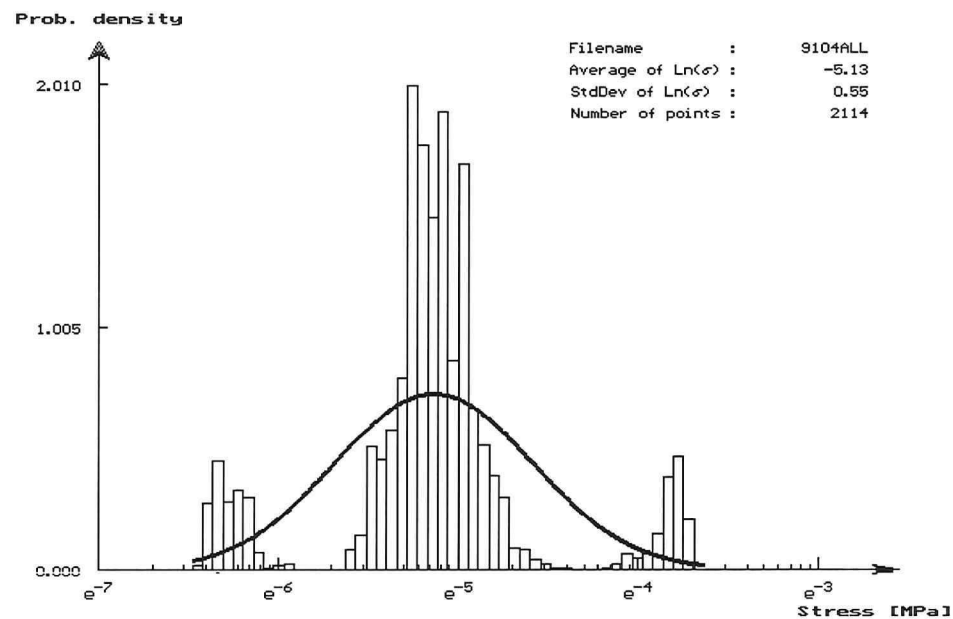
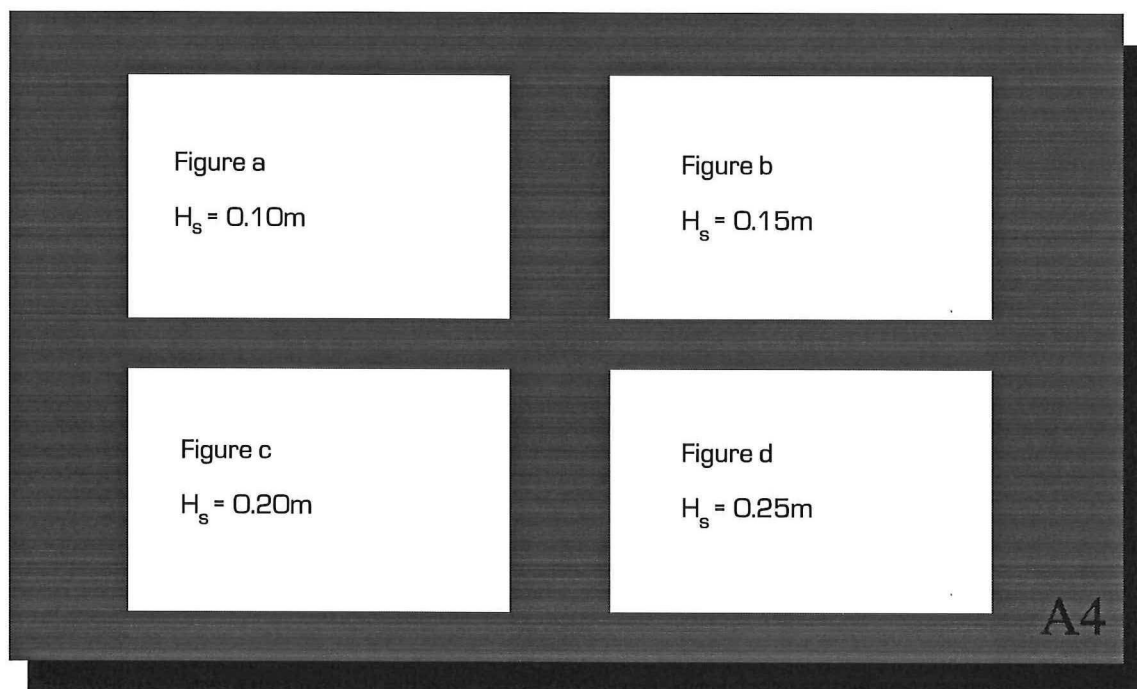


Figure D.4 Histograms of pulsating maxima: $h_{loc} = 0.50$ m. and $s_{op} = 0.04$

Appendix E

Each Figure contains 4 plots on Log Normal probability paper. Between these four probability plots only the significant wave height, H_s , changes according to the below presented scheme :



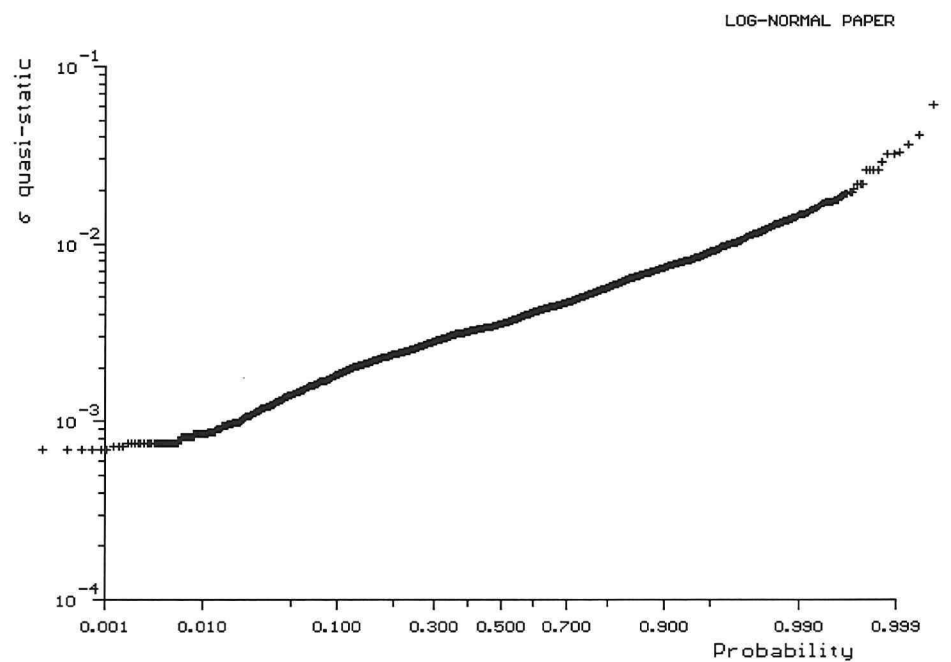
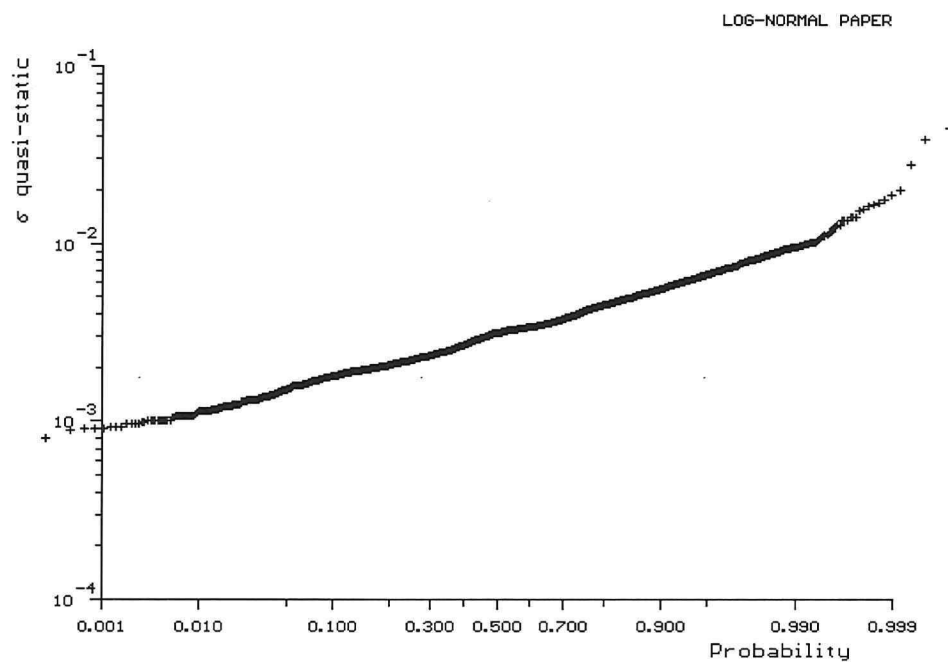
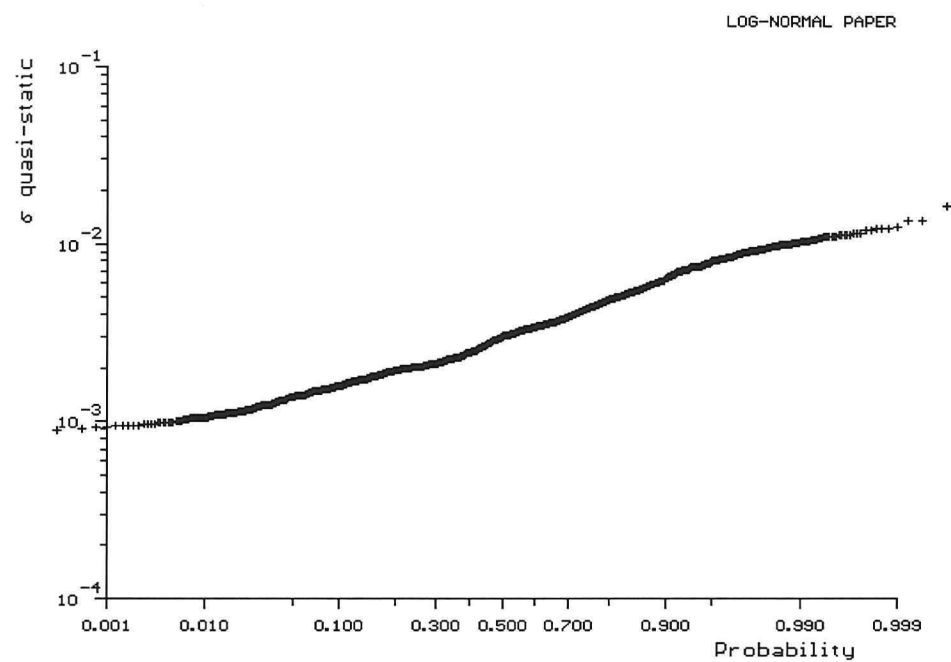
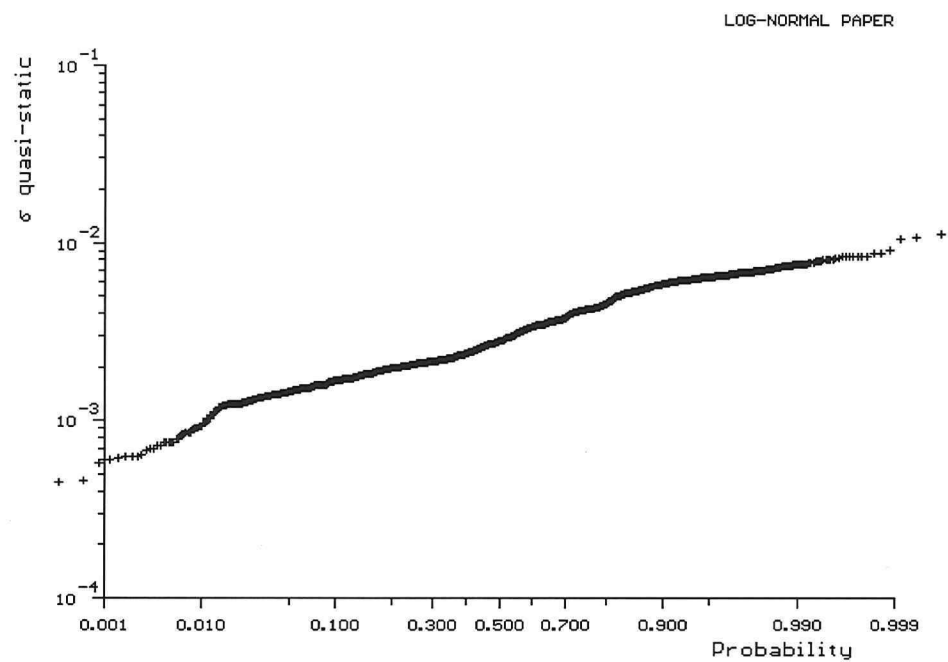


Figure E.1 Quasi static maxima on Log Normal paper. $h_{\text{loe}} = 0.30$ m. $s_p = 0.02$.

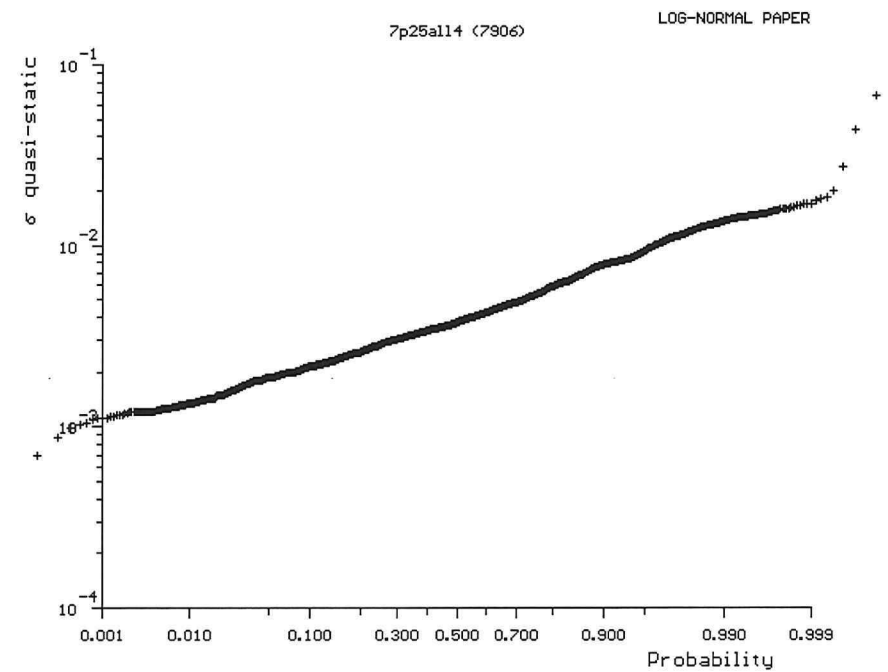
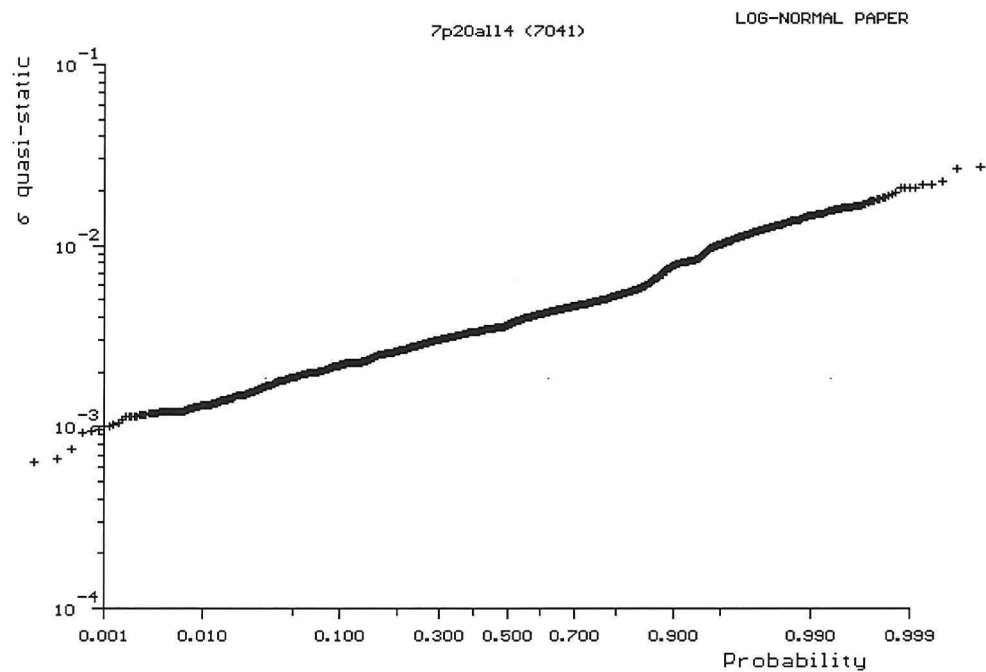
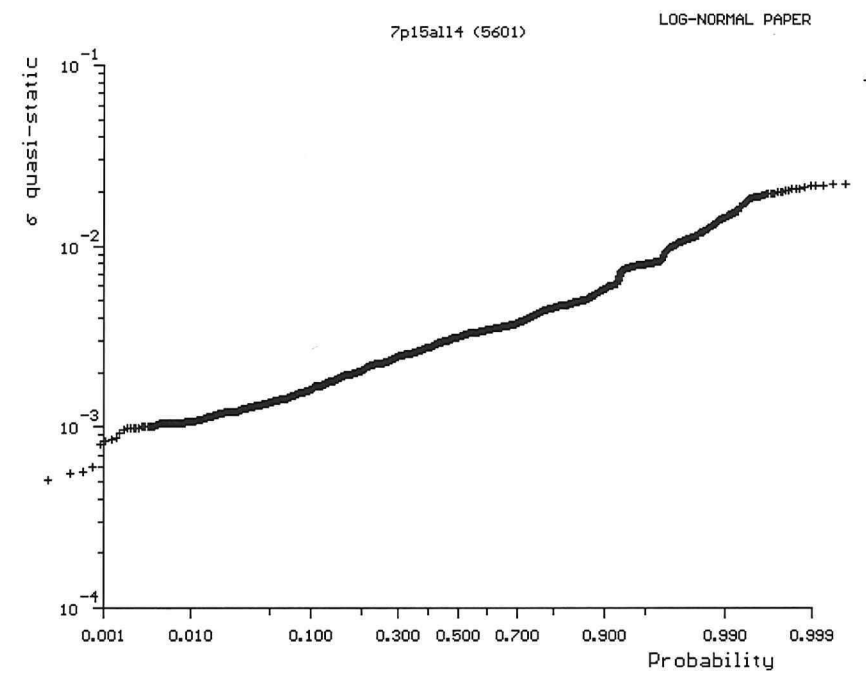
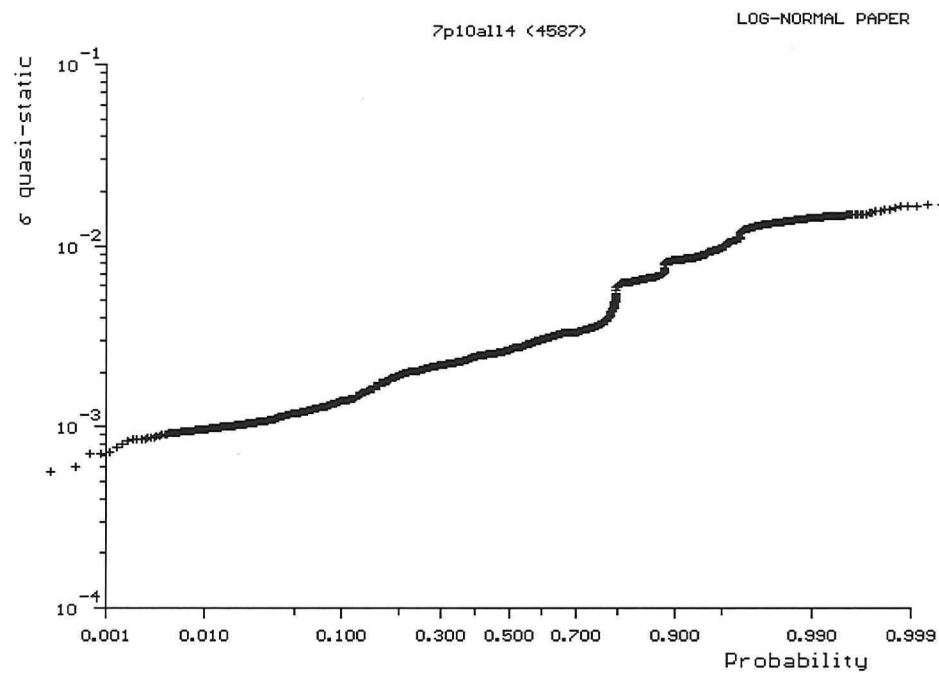


Figure E.2 Quasi static maxima on Log Normal paper. $h_{\text{loc}} = 0.30$ m. $s_p = 0.04$.

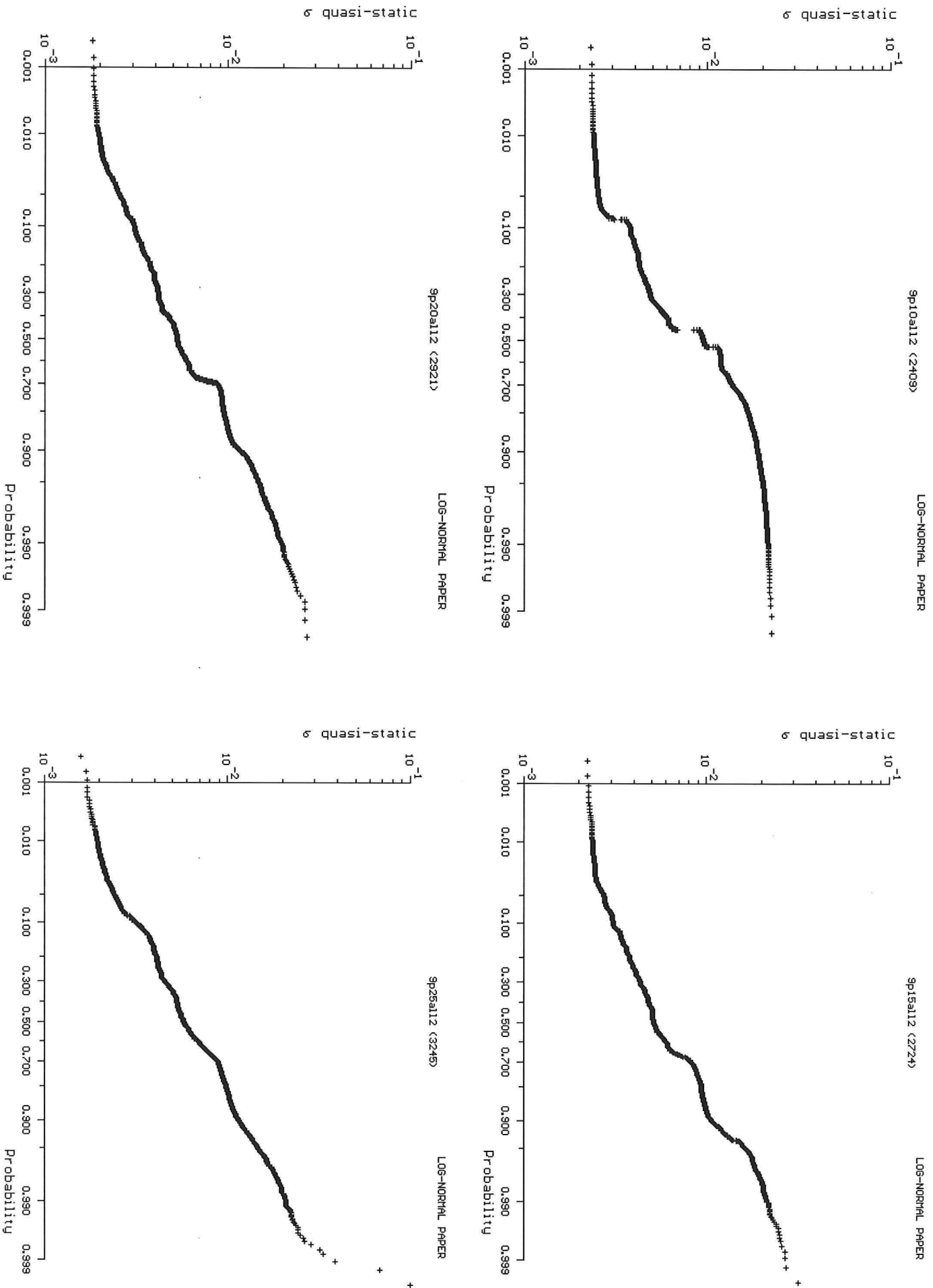


Figure E.3 Quasi static maxima on Log Normal paper. $h_{loc} = 0.50$ m. $s_p = 0.02$.

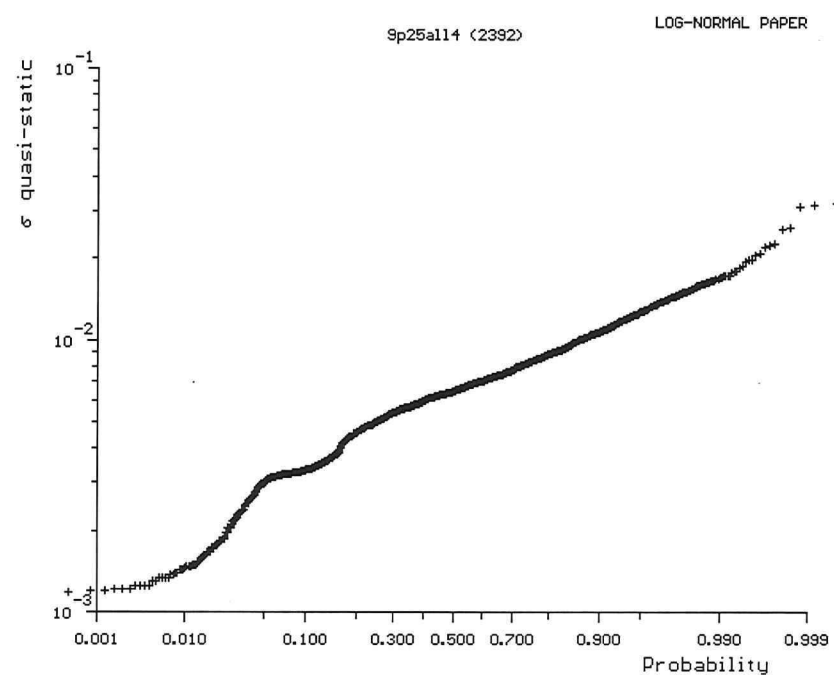
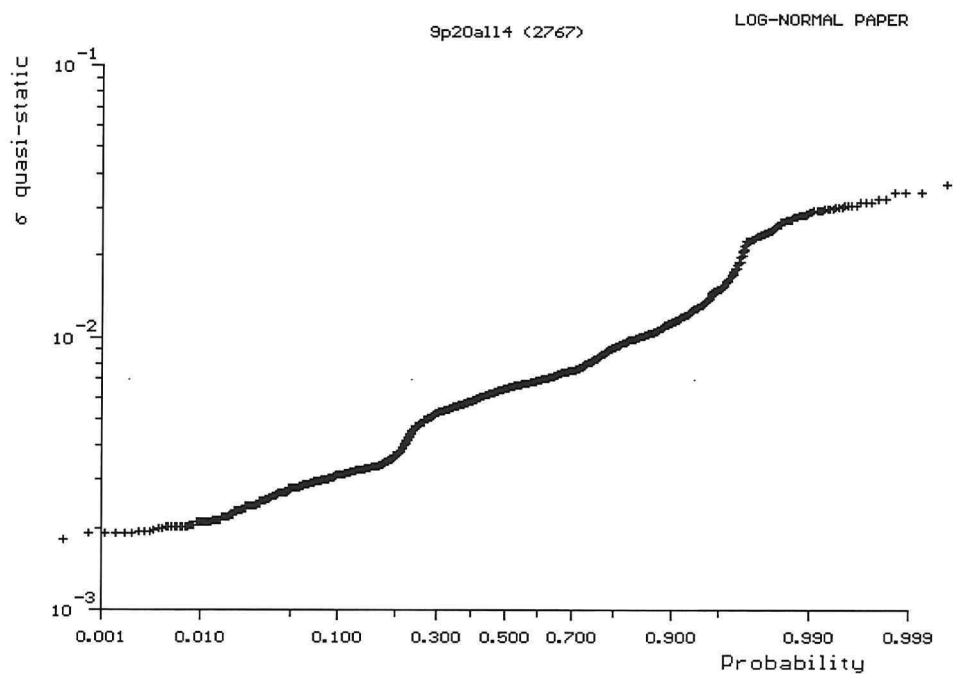
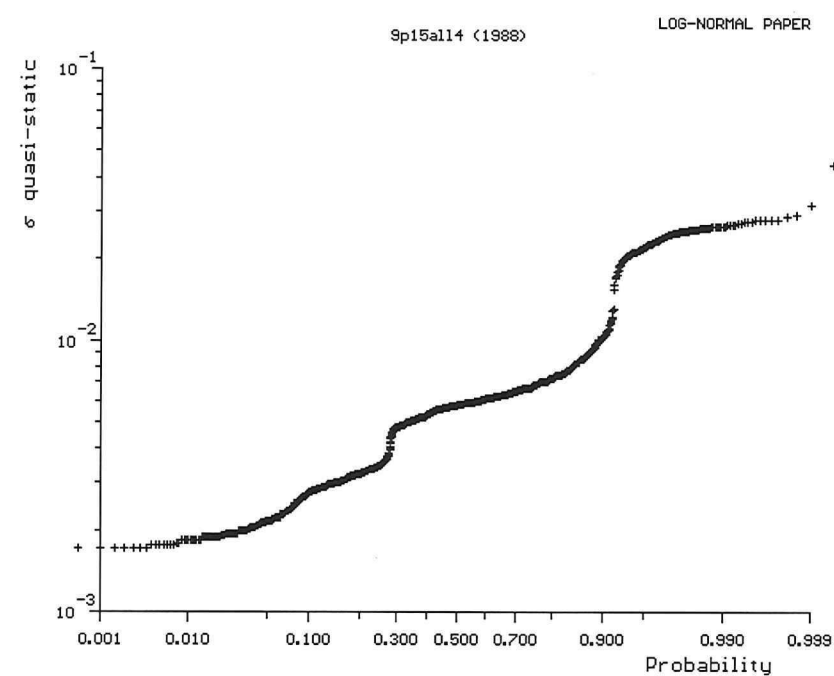
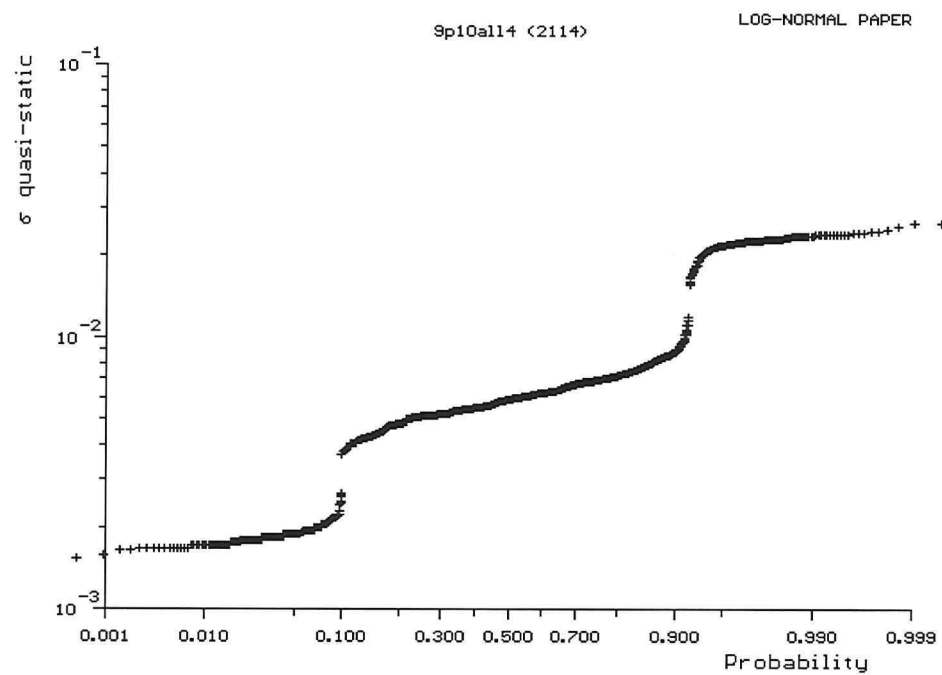
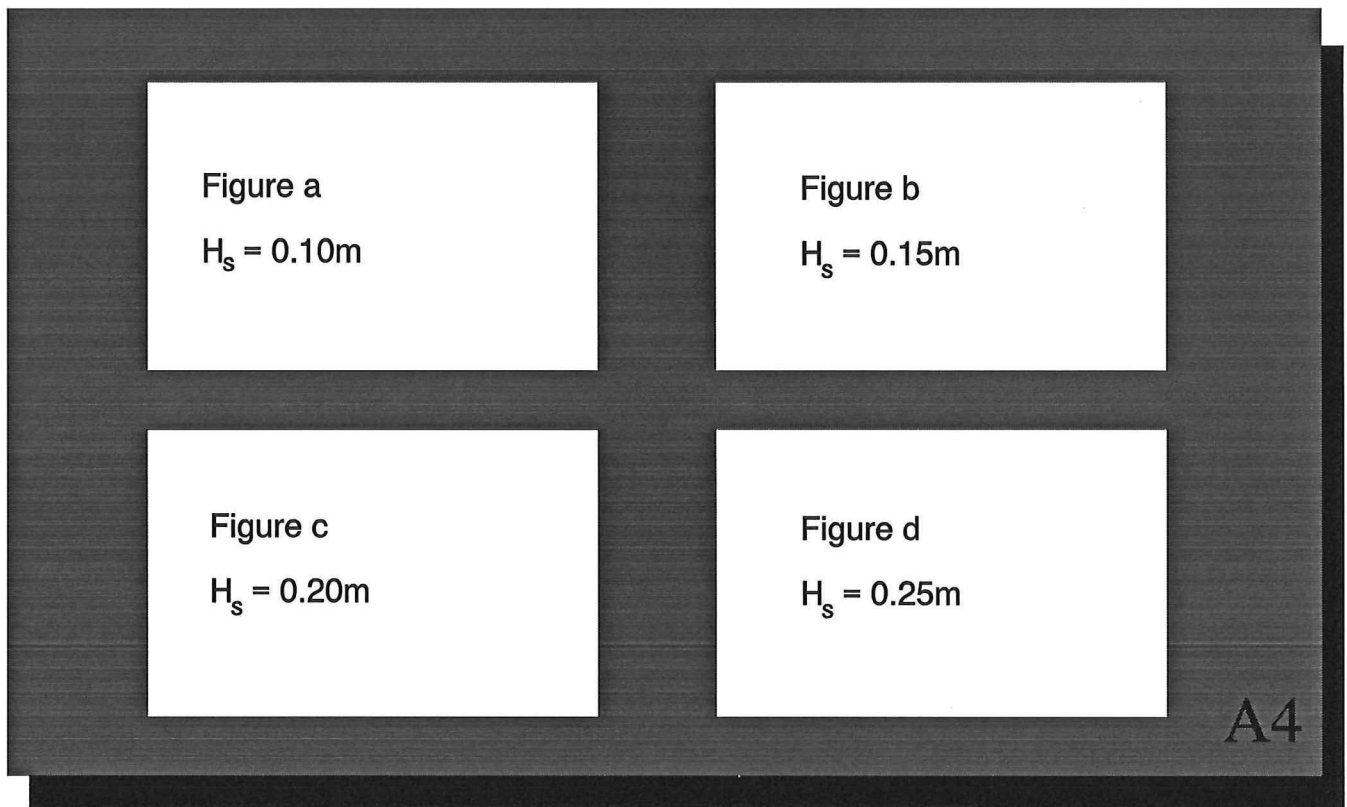


Figure E.4 Quasi static maxima on Log Normal paper. $h_{toe} = 0.50$ m. $s_p = 0.04$.

Appendix D

Each Figure contains 4 histograms. Within these four histograms only the significant wave height, H_s , changes according to the below presented scheme :



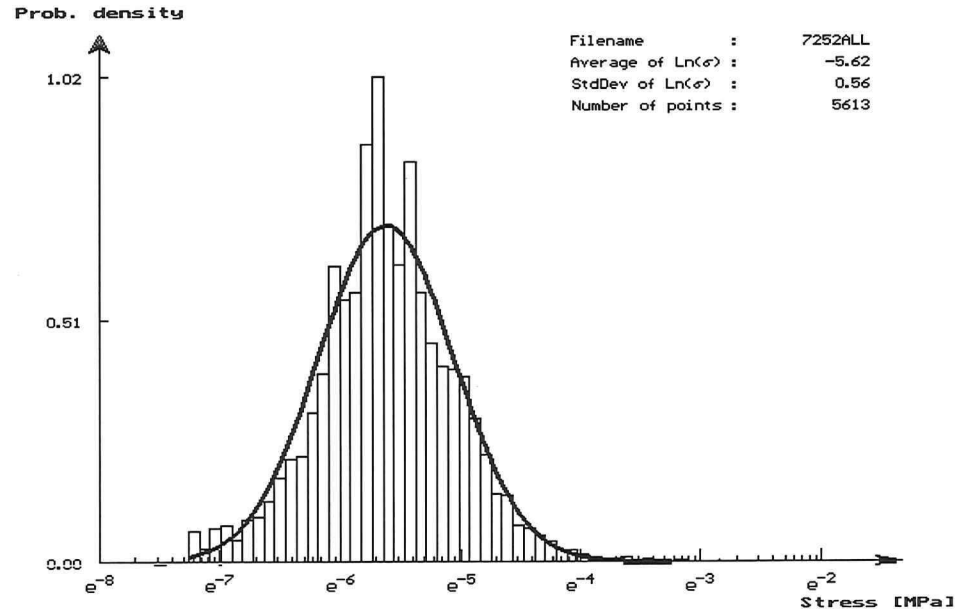
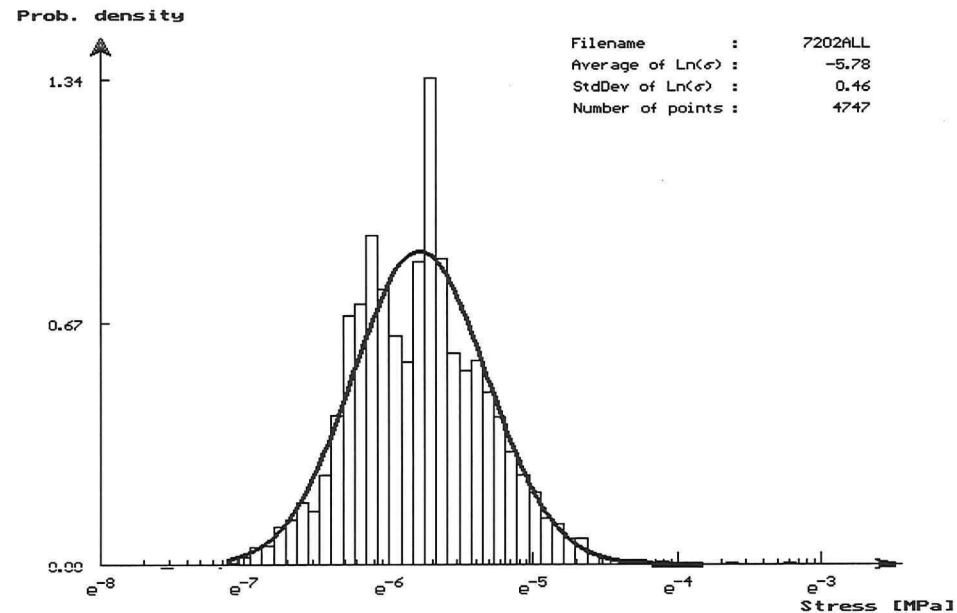
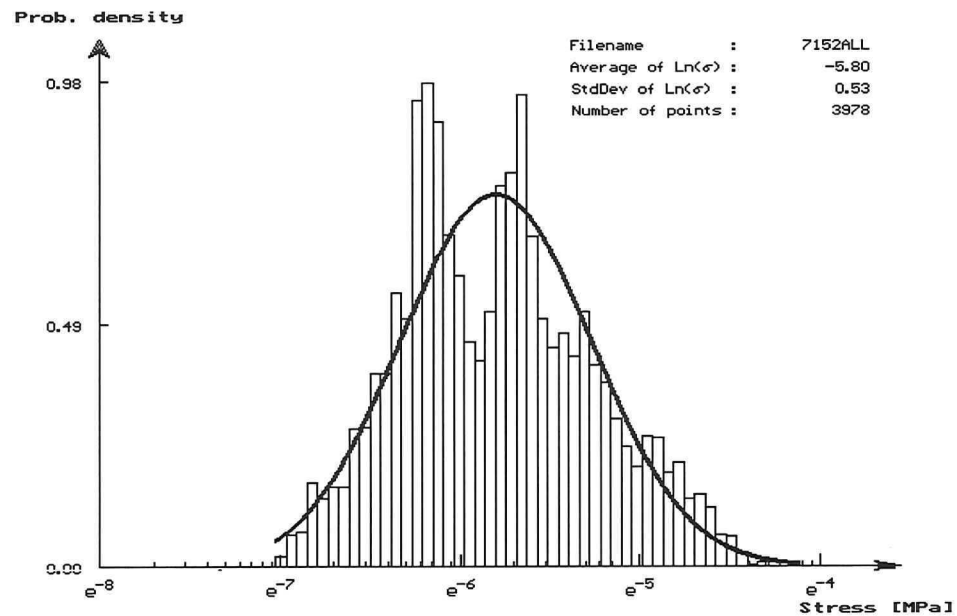
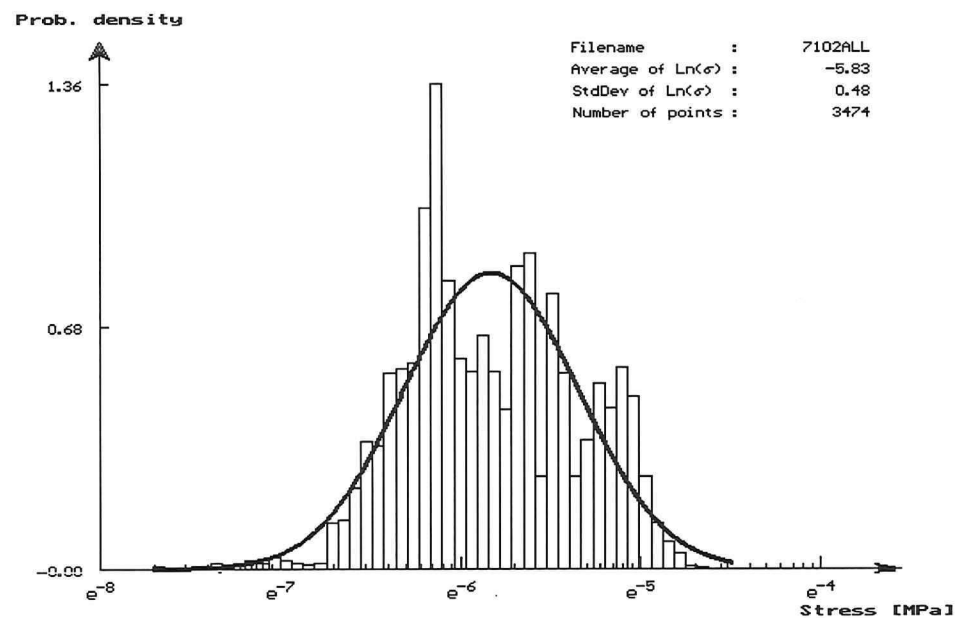


Figure D.1 Different H_s , $h_{loc} = 0.30$ m and $s_{op} = 0.02$

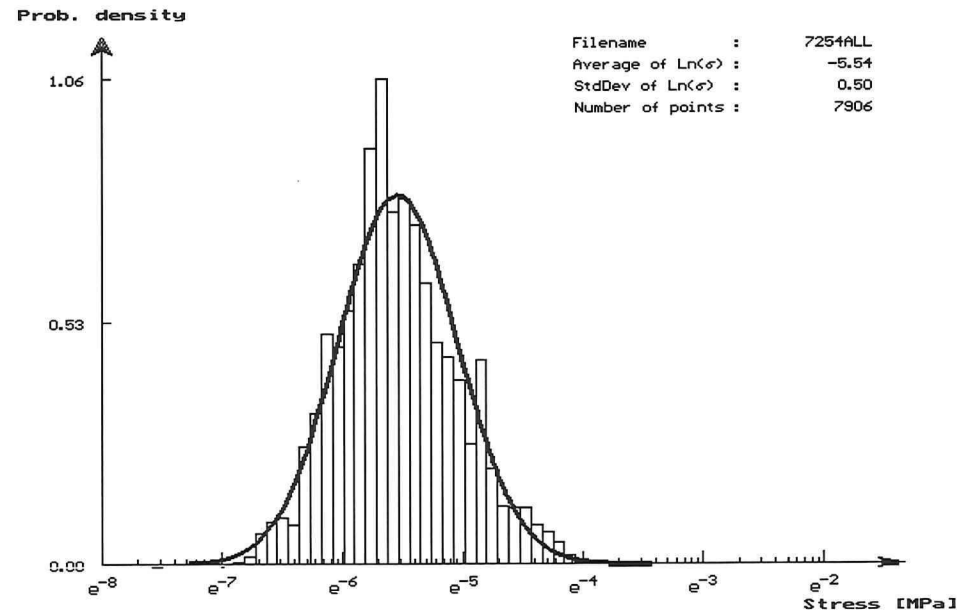
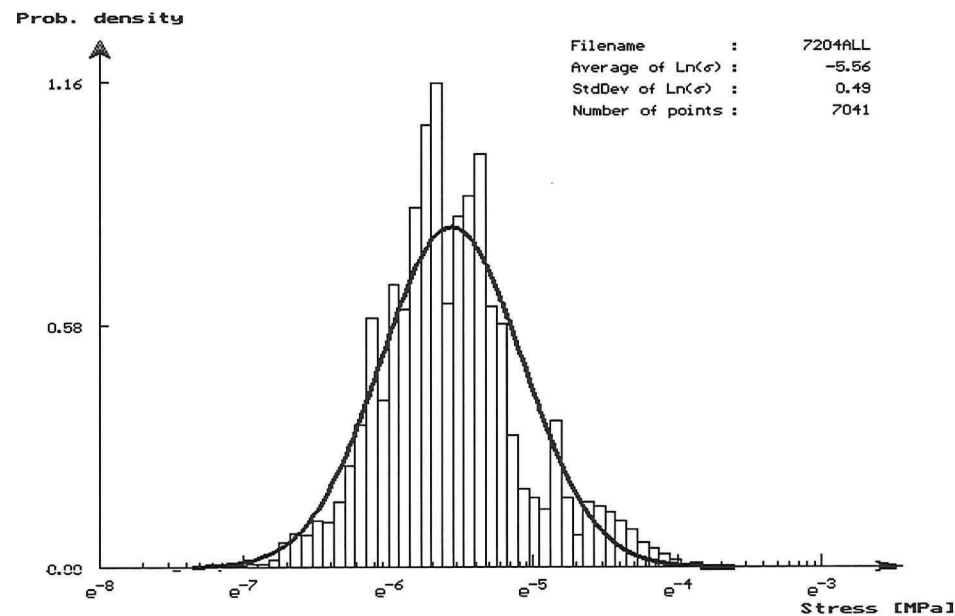
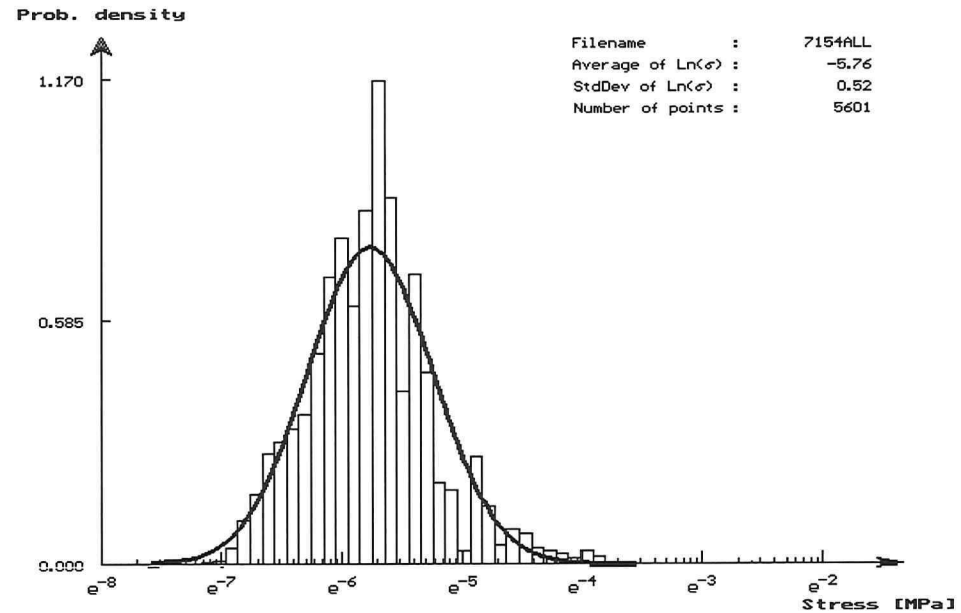
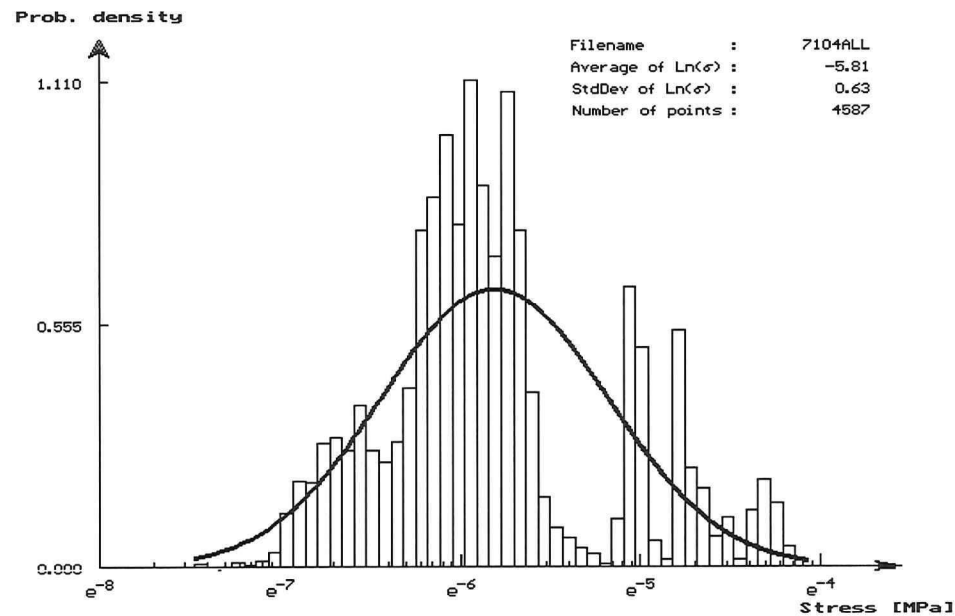


Figure D.2 Different H_s , $h_{loc} = 0.50$ m and $s_{op} = 0.04$

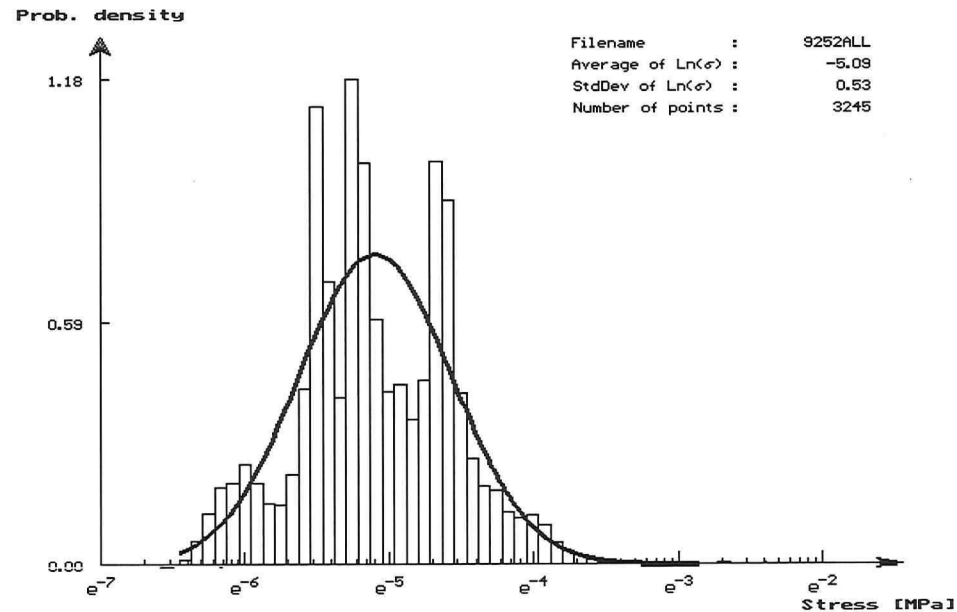
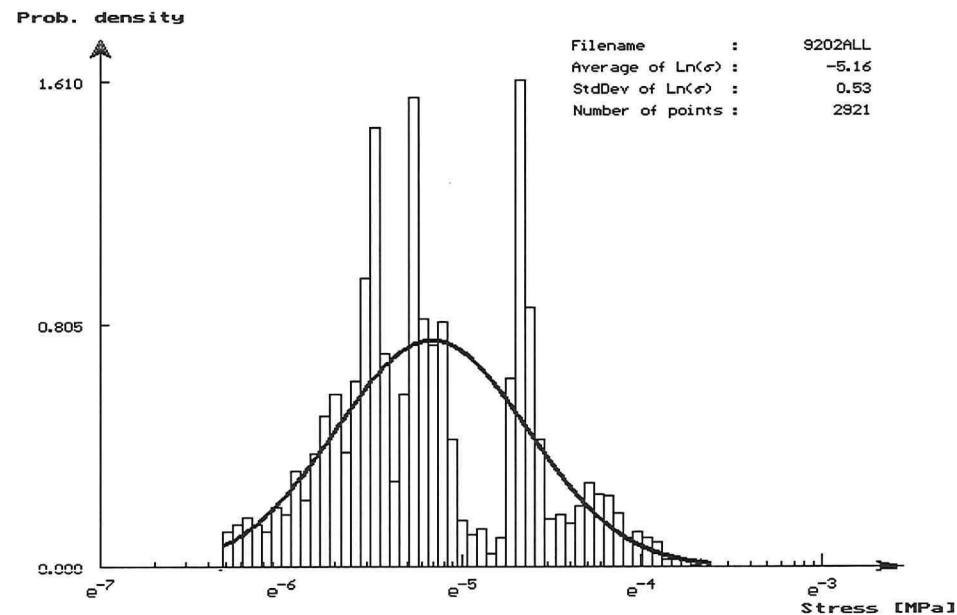
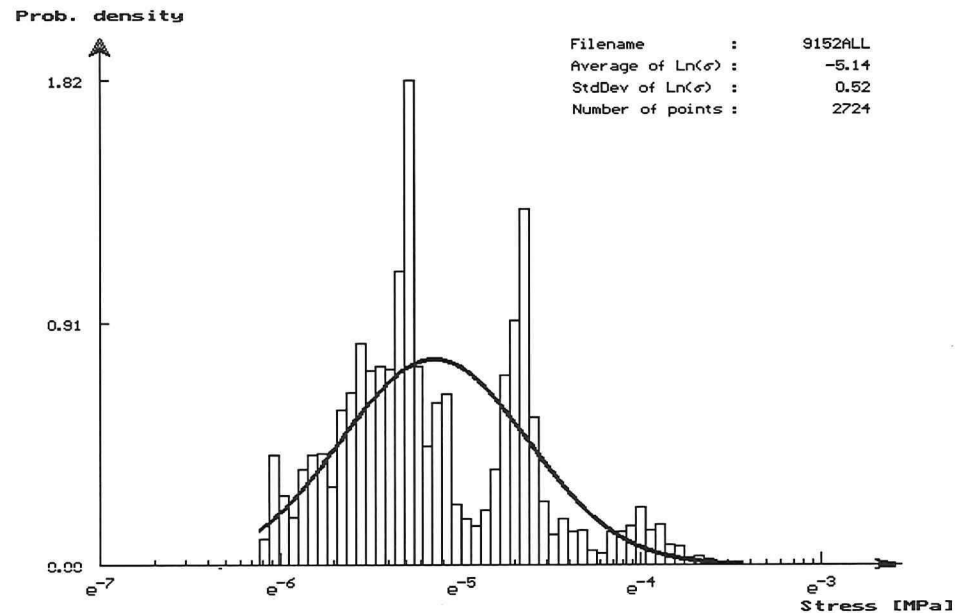
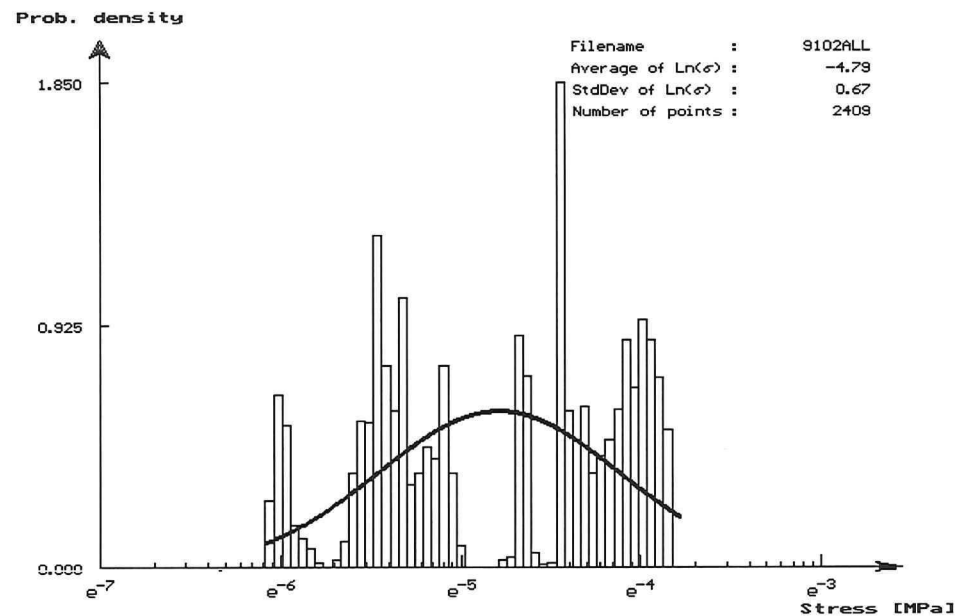


Figure D.3 Different H_s , $h_{loc} = 0.50$ m and $s_{op} = 0.02$

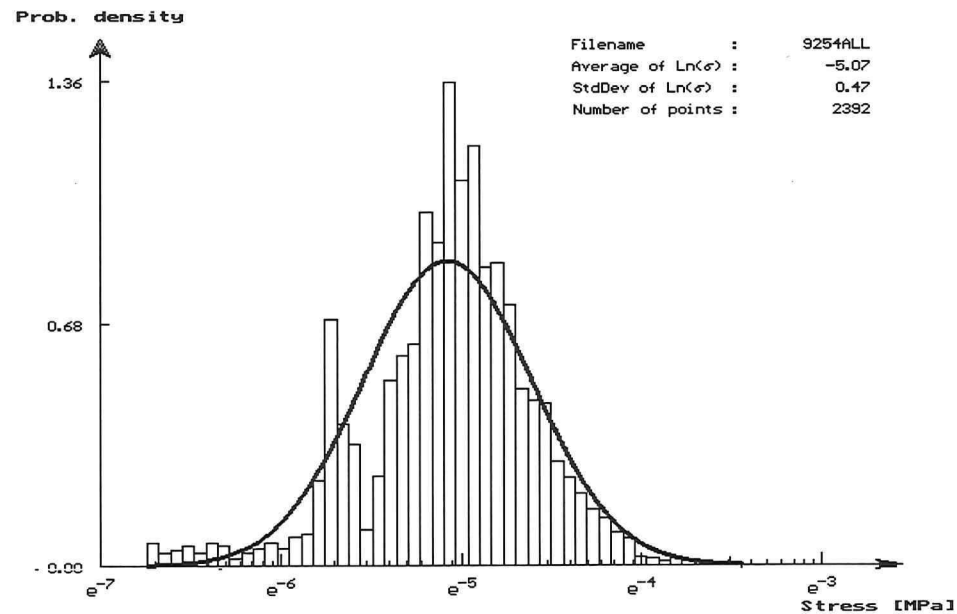
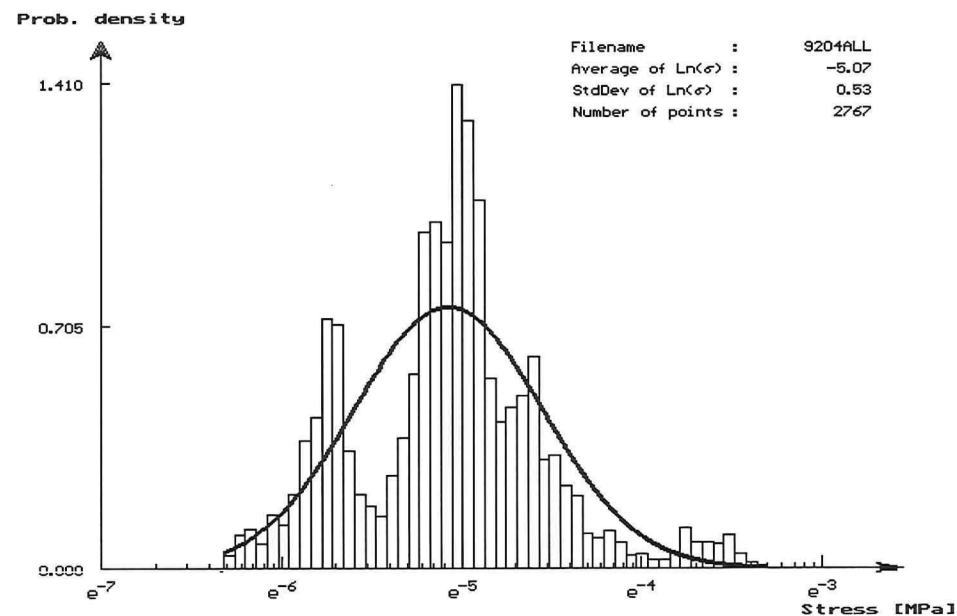
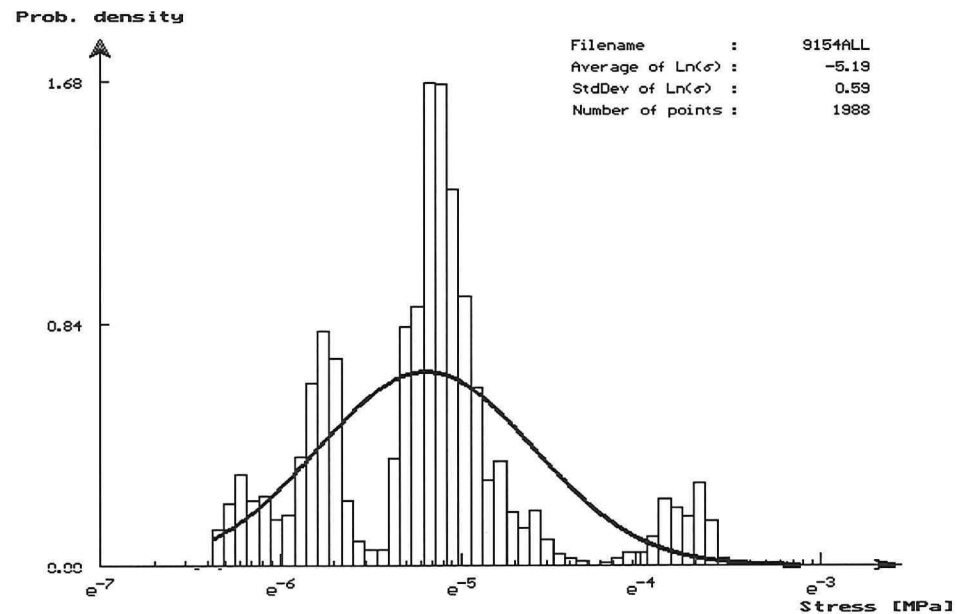
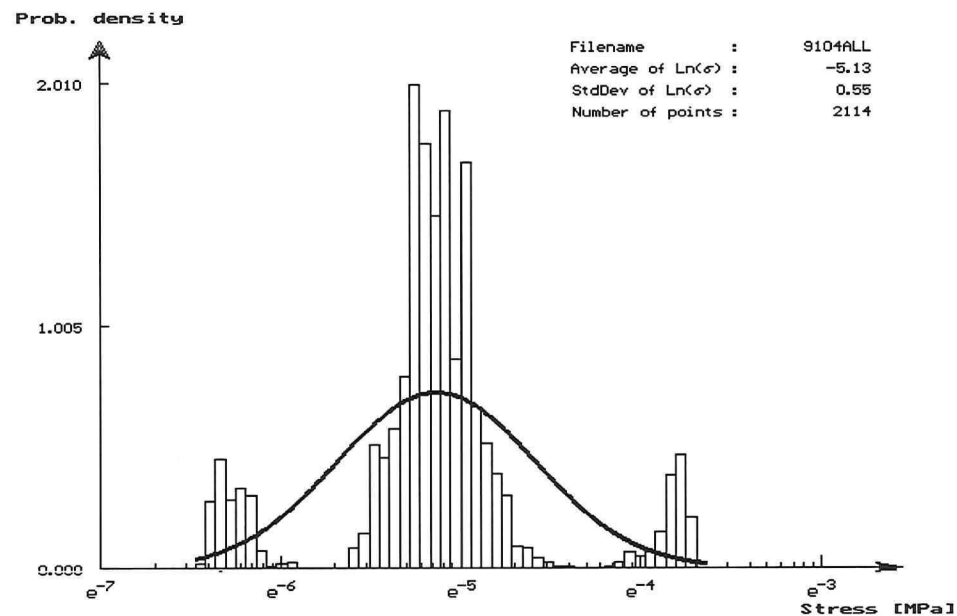


Figure D.4 Different H_s , $h_{toe} = 0.50$ m and $s_{op} = 0.04$