DET TEKNISK-NATURVITENSKAPELIGE FAKULTET

MASTEROPPGAVE

Studieprogram/Spesialisering: Offshore Technology Marine and Subsea Technology	Vårsemesteret, 2014 Åpen / Konfidensiell
Forfatter: Sirak Zere Solomon	………………………………………… (signatur forfatter)
Fagansvarlig: Veileder(e): Sverre Kristian Haver	
Tittel på masteroppgaven: Engelsk tittel: Assessing slamming loads from breaking waves	
Studiepoeng: 30	
Emneord: Breaking waves Slamming loads Platform column Contour line	Sidetall: 96 + vedlegg/annet: ………… Stavanger, June 28,2014 dato/år

ABSTRACT

Recent studies show that, the load from breaking waves gives largest impact on the platform columns. If these impacts not carefully estimated and designed, they can cause a severe damage to the platform structures. The purpose of this report is to assess these slamming loads from breaking waves on platform columns. A focus is given on how to estimate these characteristic impact loads for design control of offshore platforms.

In the first chapters a brief review of breaking wave phenomenon and the requirements for the design of offshore structures particularly the limit state functions ALS and ULS is discussed. After these introduction chapters, how a wave dominated load corresponding to annual exceedance probability of q can be estimated is given. First for linear structural system is demonstrated using some collected data and assumptions, then it is extended to nonlinear systems. In nonlinear system time-domain simulation and model based analysis is introduced.

There are two methods of estimating the slamming loads; either using model test analysis or using recommended practice from DNV-RP-C205. On this report more focuses is given to model test analysis and a brief summary of the recommended practice is given. For elaborating the model test analysis a data from collected by MARINTEK for Heidrun TLP is used. The model test was performed few years ago in Trondheim and data were collected by 56 pressure sensors that were installed on the modeled Heidrun TLP column.

In model test analysis environmental contour line approach is mainly used in this report. There are four steps in contour line analysis in order to estimating the impact load from breaking waves. The first two steps are establishing contour line of the sea states (described by H_s and T_p) having the same probability of exceedance and identifying the worst sea state from the contour line. These two methods are already done by MARINTEK. The next two steps are establishing the 3-hour maximum impact pressure distribution and estimating the q probability extreme value using α value percentile. These two steps are mainly investigated on this report. The data obtained from MARINTEK are results of different realizations for the worst sea state. Using these data two approaches are used to establish the 3-hour maximum impact pressure on the platform column. Following the environmental contour line approach, the q probability impact events are obtained by estimating the 90 % and 95% value of the 3-hour extreme impact pressure distribution. The uncertainties related to estimating this impact load are fully discussed with the help of bootstrapping.

The result obtained from direct and indirect approaches is slightly different especially when we consider high α percentile from the distribution curve. Since indirect method considers the number of impact per test on estimating the impact load, it is relatively accurate than the direct approach.

Finally the investigation is extended to see how the area considered will affect the distribution of the 3-hour maximum values. The area is enlarged to vertical and horizontal directions by adding two sensors on their sides. It is found out that, as the considered sensor area increases the average pressure obtained decreases. On the other hand if the area of sensor decreases the impact pressure increases but there is possibility of missing the local peak force.

ACKNOWLEDGEMENT

I would like to thank my supervisor Prof. Sverre Kristian Haver for his contribution throughout the process of writing this report. He has given me encouragement and provided me valuable information and useful literatures during our meetings. Our meetings during the semester have been both inspiring and educational.

I would also like to thanks Professor Ove Tobias Gudmestad for his guidance and support during the entire master studies and for his lectures and for sharing his vast industrial experience.

Furthermore, I would like thank Professor Jan Terje Kvaløy for explaining statistical probability models and their corresponding uncertainties.

At last but not least, I would be happy and grateful to thank my dear wife Yirgalem H. Berhane for supporting me to continue my education and prepare this project, and taking care of my kids. All this work could have been impossible without her help.

MSc theses 2014

Title: **Assessing slamming loads from breaking waves**

Student: Sirak Zere Solomon

Background

Over the last few years several model tests have suggested that impact loads from breaking waves are larger than what is typically recommended by available standards. A challenge related to estimated impact loads is to scale model test data such that the loads represent adequate estimates for full scale loads. A proper assessment of this requires extensive complicated works which is not possible within the framework of a MSc thesis. Here we will there assume that standard Froude scaling is valid

In this project, focus is to be given to how to estimate characteristic impact loads for design control of offshore platforms. The basis for the work is literature studies and analysis of available model test data.

The topics that shall be given special focus are:

- Establish a scheme for estimating ULS and ALS impact loads.
- Short term modelling of impact pressures using various methods.
- Estimating extremes based on model test data

The model test data will be provided by reports and spread sheets

Below a possible division into sub-tasks is given.

1. Review briefly the requirements regarding design of offshore structures, in particular with respect to:
   * Overview over limit state functions to be considered.
   * Definition of environmental loads in ULS- and ALS limit state function.
   * Discuss relative importance of ULS and ALS.
2. Breaking wave phenomenon

3. Discuss how a wave dominated load corresponding to an annual exceedance probability of q can be estimated. This should be done for a generic linear case. Discussion should include short term analysis and long term analysis. Discuss how one could perform a long term analysis for a non-linear case.

4. Introduce the impact problem. What is making this a somewhat more challenging problem than the problem discussed in 3?

5. Assess the distribution function of the 3-hour maximum impact pressure based on two approaches:
   * Directly from observed 3-hour extremes.
   * Indirectly by considering all impacts above a certain threshold.
   * Establish uncertainties related to the estimated distribution functions.

6. Discuss possible methods for estimating ALS impact extremes. Discuss involved uncertainties.

7. Investigate how impact pressure is affected by areas considered.

8. Summarize the investigation in conclusions pointing out major learnings of this investigation.

The candidate may of course select another scheme as the preferred approach for solving the requested problem.

The work may show to be more extensive than anticipated. Some topics may therefore be left out after discussion with the supervisor without any negative influence on the grading.

The candidate should in his report give a personal contribution to the solution of the problem formulated in this text. All assumptions and conclusions must be supported by mathematical models and/or references to physical effects in a logical manner. The candidate should apply all available sources to find relevant literature and information on the actual problem.

The report should be well organised and give a clear presentation of the work and all conclusions. It is important that the text is well written and that tables and figures are used to support the verbal presentation. The report should be complete, but still as short as possible.

The final report must contain this text, an acknowledgement, summary, main body, conclusions, suggestions for further work, symbol list, references and appendices. All figures, tables and equations must be identified by numbers. References should be given by author and year in the text, and presented alphabetically in the reference list. The report must be submitted in two copies unless otherwise has been agreed with the supervisor.

The supervisor may require that the candidate should give a written plan that describes the progress of the work after having received this text. The plan may contain a table of content for the report and also assumed use of computer resources. As an indication such a plan should be available by early March.

From the report it should be possible to identify the work carried out by the candidate and what has been found in the available literature. It is important to give references to the original source for theories and experimental results.

The report must be signed by the candidate, include this text, appear as a paperback, and - if needed - have a separate enclosure (binder, diskette or CD-ROM) with additional material.

Supervisor: Sverre Haver, Statoil ASA.

Table of Contents

LIST OF FIGURES AND TABLES

List of Figures

List of Tables

LIST OF ABBREVIATIONS AND SYMBOLS

Abbreviations

ALS: Accidental Limit State

CDF: Cumulative Distribution Function

CT: Compliant Tower

DNV: Det Norsk Veritas

FLS: Fatigue Limit State

FPS: Floating Production and Systems

GBS: Gravity Based Structure

JONSWAP: Joint North Sea Wave Observation Project

NORSOK: Norsk Sokkels Konkurranseposisjon

PDF: Probability Density Function

PM: Pierson Moskowitz

POT: Peak Over Threshold

PSA: Petroleum Safety Authority Norway

RAO: Response Amplitude Operator

SLS: Serviceability Limit State

TLP: Tension Legged Platform

ULS: Ultimate Limit State

Symbols

a Wave amplitude

C Phase velocity of wave

$C_B^{(n)}$ Phase velocity of breaking wave height for n years

C_{pa} Coefficient of average slamming pressure

f Wave frequency in hertz

F_{Z3h} Cumulative distribution of 3-hour extreme response value

H Wave height

$H_B^{(n)}$ Most probable breaking wave height

H_s Significant wave height

$|h_{EX}|$ Transfer function

L Wave length

m Mass of an object in equation of motion

m_{3h} Expected number of waves in 3-hour

m_j j^{th} order moment spectra

P_n Poisson probability distribution

p_s Average slamming pressure

q Annual exceedance probability

R_d Total resistance load

S_d Total design load

S_{pm} Pierson Moskowitz energy spectra

$S_{\Gamma\Gamma}$ Response energy spectra

$S_{\Xi\Xi}$ Wave energy spectra

T Wave period

$T_B^{(n)}$ Period of n years breaking wave

T_p Peak wave period

T_z Average up crossing period

v is the relative horizontal velocity between water and column

w Wave frequency in rad/second

w_p Peak wave frequency

X Displacement of an object in equation of motion

X_e Environmental load

X_p Permanent load

X_v Variable load

$\dot{x}$ Velocity of an object in equation of motion

$\ddot{x}$ Acceleration of an object in equation of motion

Y Global crest height of the wave process

$\hat{Y}$ Characteristics largest wave crest height in a sea state

z Arbitrary response value

Z_p Peak response value

Z_{pq} q probability peak response value

Z_{3h} 3-hour extreme response value

$\hat{z}$ Characteristics largest response height in a sea state

Greek Characters

α Parameter in Weibull and Gumbel distribution

β Parameter in Weibull and Gumbel distribution

γ Parameter in Gumbel distribution

γ_p Safety factor for permanent load

γ_v Safety factor for variable load

γ_e Safety factor for environmental load

γ_m Safety factor for material

η Truncation wave height in joint H_s and T_p distribution.

λ Rate of occurrence of an event in Poisson probability distribution

μ mean in lognormal distribution

ρ Parameter in lognormal distribution

ρ	mass density of the fluid
σ_{Ξ}	Standard deviation of the wave process
σ_{Γ}	Standard deviation of the response process
ϕ^2	Variance in lognormal distribution

INTRODUCTION

Offshore structures are exposed to extreme loads from wind, current and waves. When designing those offshore structures, it is important to ensure that the structure can resist all the above environmental loads. Generally wave loads are the major loads that cause a severe damage on the vertical faces of offshore structures and in recent years, model tests have suggested that impact loads from breaking waves are larger than what is typically recommended by available standards. Therefore it is important to have the detail knowledge of the impact loading and the distribution of the pressure induced by the breaking waves.

When horizontal water particles acquire higher velocity than the phase velocity of a wave, then water on the top would move faster than the wave itself and result in a breaking wave. This action creates more irregular waves that propagate forward and break close to the shore or if there is a structure on the mid ocean during breaking, the waves impacts with the structure. This impact creates a load on a structure and can cause severe damage if not considered during design.

In the last few years different approaches and experiments were performed to estimate this impact load from breaking waves. Ochi & Tsai (1984) carried out experiment by generating breaking waves and measured the impact pressure at front face of circular cylinder. From this experiment they propose a statistical method to predict the magnitude of the impact pressure. Zhou, et al. (1991) conducted an experiment on measuring the distribution of pressure on vertical cylinder and they conclude that the largest pressure is essentially an inherent random phenomenon even with identical wave condition.

Nowadays impact loads from breaking waves on platform columns can be estimated by two general methods either by following the recommendation documents or by performing model tests. For Norwegian continental shelf a recommended practice document Det Norske Veritas (DNV) is available. DNV-RP-C205 (2007) is the most preferred and useful document in designing the impact load from breaking waves. Another method could be using the statistical analysis of model test results. On this report the impact load from breaking waves will be estimated using the model test analysis and a brief summary of recommended document will be given.

Considering impact loads from breaking waves, NORSOK N-003 defines the characteristic values of the impacts loads by specifying annual exceedance probabilities for the different design limit states. Impact loads from breaking waves fall in the category of environmental actions. For environmental actions the characteristic load values are defined by annual exceedance probability of 10^{-2} for ultimate limit state (ULS) and 10^{-4} for accidental limit state (ALS).

One of the simplified methods for estimating loads and response extremes corresponding to a given annual exceedance probability q is the environmental contour line method. This method estimates the load without having to carry out a full long-term analysis. In environmental contour line approach, the first step is to establish the contour lines of H_s and T_p corresponding to the same annual exceedance probability. From this contour line the worst sea state is identified using 3-hour tests of the selected sea state from the contour line. As the worst sea state is selected a large number of 3-hour tests (which represent each realization) must be done. From each of the 3-hour realization for the worsts sea state, the maximum impact pressure is identified and a reasonable probabilistic model can be established for the distribution of these 3-hour maximum impact pressure. Finally, from the 3-hour extreme distribution, the q probability is estimated by the fractile of α value.

On this report MARINTEK has identified the worst sea state (described by H_s and T_p) and they perform 109 model tests which represent the different realizations of the worst sea state. 56 sensors are installed on the modeled platform column to record the pressure impact from the 109 test runs. Based on these results, the distribution function of the 3-hour maximum impact load due to breaking waves is established. Direct and indirect approaches are used to establish the distribution and α=0.95 is used to estimate ALS impact load from the obtained distribution function.

1. REQUIREMENTS IN DESIGNING OF OFFSHORE STRUCTURES

1.1 General Types of Offshore Structures/Platforms

An offshore structure is a structure in offshore which has no fixed access to dry land and may be required to stay in position in all weather conditions. Offshore structures may be fixed to the seabed or floating. These floating structures may be moored to the seabed, dynamically positioned by thrusters or may be allowed to drift freely.

Offshore structures are employed in the exploration, production and transportation of offshore minerals as well as for transportation of people and products across nations. The structures used for the production of oil and gas are generally located at a particular site offshore while others are mobile. These structures are often at the mercy of the harsh environment of the ocean in the form of waves, wind, current and earthquake, and must survive the severest storm encountered during its lifetime (Chakrabarti, 2005). On this thesis our focus is given to the offshore structures or platforms used for the production, storage and offloading of hydrocarbons.

Offshore platforms are evolved from land-based facilities and were constructed on site. Knowledge of design was borrowed or extrapolated from traditional fields of engineering such as civil engineering and naval architecture. In general platforms can be classified according to their main distinguishing features as follows (Odland, 2012).

a) Fixed platforms

- Fixed steel platform (steel jacket)
- Fixed concrete platform (gravity base structure - GBS)
- Compliant tower
- Jack-up platform

b) Tension leg platforms

- Multicolumn tension leg platform
- Mono-column tension leg platform (mini-TLP)

c) Deep draft floaters

- Spar platform
- Multicolumn deep draft floater

d) Spread-moored floating platforms

- Semi-submersible platform
- Buoy-shaped platform (shallow draft buoy)
- Barge or ship-shaped platform

e) Single-point-moored platform

- Ship-shaped unit with internal or external turret
- Ship-shaped unit with disconnect able turret

f) Dynamically positioned platform

- Ship-shaped unit
- Semi-submersible unit

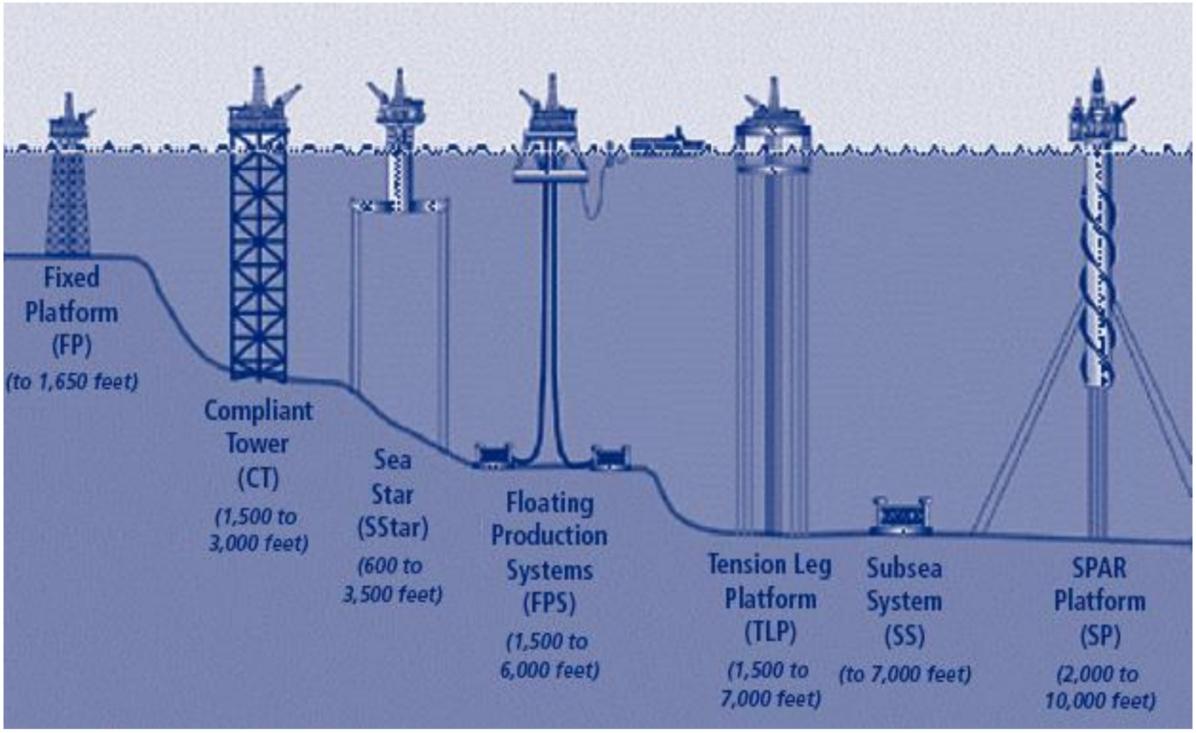


Fig. 1.1 Different types of offshore structures (platforms) (U.S. Mineral Management Service, 1999)

1.2 General Design Requirements and Principles of Offshore Structures

When designing offshore structures first and most important part of design process is safety. It is important that the structure can withstand all foreseen loads acting on the structure with an adequate safety factor. Higher safety is most often equal to larger costs. If an offshore structure was designed with cost optimization in focus, there is a large chance it would have insufficient safety level. To ensure that structural design is within the target safety level set by the authorities, the designer has to ensure that the design process is correct according to the rules and regulations for the location where the structure is going to be installed. Rules and regulations vary from country to country but a certain similarity is typically recognized. Since the report deals with offshore structures in Norwegian Continental Shelf, the Norwegian rules and regulation will be discussed here. Norwegian regulation hierarchy is shown in Fig.1.2.

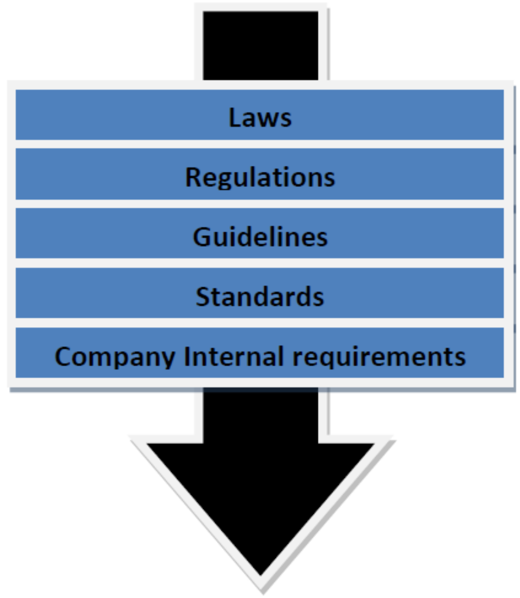


Fig. 1.2 Norwegian regulation hierarchy (Haver, 2013)

The Norwegian regulation Hierarchy is given by Petroleum Safety Authority Norway (PSA) based on the consideration to the health, safety and environmental issues related to the petroleum activity on the Norwegian Continental Shelf. It starts with the Laws and Acts that should be followed prior to the regulations, guidelines and standards, and finishes with the company internal (Haver, 2013).

The principal standard regarding design of offshore structures is the ISO 19900 petroleum and Natural Gas industry. The ISO 19900 series of standards addresses the design, construction, installation, integrity and assessment of offshore structures. The standard specifies general principles for the design and assessment of structures subjected to known or foreseeable types of actions. These general principles are applicable worldwide to all types of offshore structures including bottom-founded structures as well as floating structures and to all types of materials used including steel, concrete and aluminum.

Norsok standards are still referred to as the basic standards for design of offshore structures in the Norwegian Continental Shelf. Norsok N-001 gives the general provision and design principle to be used for design of offshore structures and N-003 is the principal standard when it comes to action and action effects.

According to NORSOK N-001 a structural system, its components and details should be designed according to the following listed principles:

- Structures and structural elements shall normally be designed with ductile resistance
- Structures shall be designed such that an unintended event does not escalate into an accident of significantly greater extent than the original event
- Structures shall be designed with the objective to minimize overall dynamic stress concentrations and provide a simple stress path
- Structures shall be designed such that fabrication, including surface treatment, can be accomplished in accordance with relevant recognized techniques and practices

- Design of structural details, selection of structural profiles and use of materials shall be done with the objective to minimize corrosion and the need for special precautions to prevent corrosion
- Adequate access for inspection, surveillance, maintenance and repair shall be provided
- Satisfy functional requirements as given in the Design Premises

1.3 Environmental Loads in ULS and ALS Function

The two basic codes and recommendations that give a detail explanation about environmental loads in Norwegian codes and standards are; first the NORSOK Standard N-003, Action and action effects; and second the recommended practice DNV-RP-C205, Environmental Conditions and Environmental Loads. These two standards defined the environmental load in limit state design. Therefore, we will first see what limit state is and its four categories.

Modern offshore design standards are based on the limit states design method. The principles of the limit states design method and the definitions of the four limit states categories are given in ISO 19 900.The term limit state shall be understood to mean that state where a structure or part of a structure no longer meets the requirements laid down for its performance or operation.

The limit state design is controlled by the following equation (Haver, 2013)

$$\gamma_p X_p + \gamma_v X_v + \gamma_e X_e \leq Y_c / \gamma_m \qquad (1.1)$$

Where, γ_p , γ_v and γ_e are safety factors for actions/loads and γ_m material safety factor

X_p, X_v and X_e are permanent, variable and environmental characteristics loads respectively.
Permanent actions are actions that will not vary in magnitude, position or direction during the time period considered. Examples are weight of structure, permanent weight of ballast and equipment, including mooring and risers, external hydrostatic pressure up to mean sea level and pretension.

Variable actions are actions whose variation originates from normal operation of the structure. Examples are people, stored goods, crane, helicopter, lifeboats, modules that can be removed, weight of gas and liquid in process plants etc.

Permanent and variable loads will not be discussed on this report. Since breaking waves are environmental load, our focus is going to be on the environmental loads. Environmental actions are loads induced by environmental process. Examples of causes for environment loads are; wave, wind, tide, earth quake, ocean current, snow, ice etc. For most structures waves are most important, but wind, currents and tide interacts with and enhances the effect of waves (Haver , 2006). In addition to the above load accidental loads are considered in the accidental limit state. They are actions caused by abnormal operation or extreme rare environmental loads (i.e. loads with annual probability of 10^{-4}). Some examples are fire, explosions, impacts from ships, dropped objected, helicopter crush and change of intended pressure difference.

The four limit state categories are:

- Ultimate limit state (ULS)

- Serviceability limit state (SLS)
- Fatigue limit state (FLS)
- Accidental damage limit state (ALS)

Since the breaking waves are more related to the ultimate limit state and Accidental damage limit state, more detail will be given to ULS and ALS while an introduction to SLS and FLS.

1.3.1 Ultimate Limit State

To satisfy the ultimate limit state, the structure must not collapse when subjected to the design load for which it is designed. A structure is deemed to satisfy the ultimate limit state criteria if all factored load (action) effects are below the factored strength. A magnification factor is used for the loads (actions) and a reduction factor is used for the strength/resistance of structural members. This general statement can be expressed by equation (1.2) (Odland, 2012) and Fig. 1.3.

$$S_d \leq R_d \qquad (1.2)$$

$S_d = S_k \gamma_f$ design load, action or action effect

$R_d = R_K / \gamma_m$ design strength or resistance

S_k characteristics load, action* or action effect*

R_k characteristics strength or resistance

γ_f partial safety factor for load, action or action effect

γ_m partial safety factor of materials

***Action**: external load applied to the structure (direct action) or an imposed or acceleration (indirect action)

***Action effect**: effect of actions on structure components (internal force, moment, stress or strain)

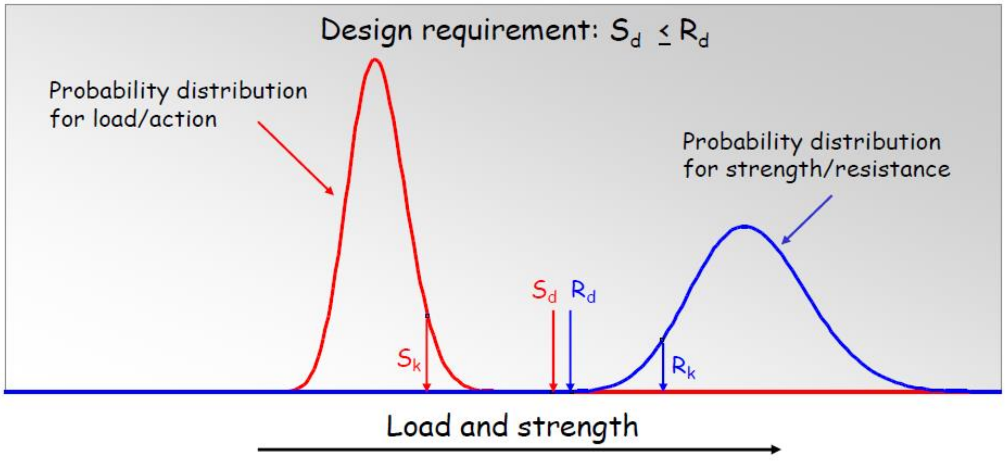


Fig. 1.3 Probability distribution of load/action Vs strength/resistance (Odland, 2012)

According to the NORSOK N-003, the environmental actions shall be determined with stipulated probabilities of exceedance. Characteristics environmental load is typically taken to be the value corresponding to an annual exceedance probability of 10^{-2}. It has to be checked for two scenarios. a) case when permanent and variable loads are governing b) the case when environmental load is governing. The loads are the same for both scenarios but their importance is adjusted by safety factor.

Table 1.1 Action factors to be used in the ultimate limit state scenarios (Haver, 2013)

Action combinations	Permanent actions	Variable actions	Environmental actions
a	1.3	1.3	0.7
b	1.0	1.0	1.3

1.3.2 Accidental damage limit state (ALS)

The ALS check ensures that the accidental action does not lead to complete loss of integrity or performance of the structure (NS-EN ISO 19900, 2013). In most cases all partial safety factors are set equal to 1.0. ALS is applied in connection with accidental loads like explosion, fire, collision, etc. However, in the Norwegian rule regime, abnormal environmental loads are also to be checked under the accidental limit state. Annual exceedance probability of 10^{-4} is in this case defining characteristics environmental load.

The ALS is checked in two steps:
a) ***Resistance to accidental actions.*** The structure should be checked to maintain the prescribed load carrying function for the defined accidental actions i.e. 10^{-4} annual exceedance probability.
b) ***Resistance in damaged condition.*** Following local damage which may have been demonstrated under a), or following more specifically defined local damage, the structure shall continue to resist defined environmental conditions without suffering extensive failure, free drifting, capsizing, sinking or extensive damage to the external environment i.e. 10^{-2} annual exceedance probability.
The structure need to withstand the characteristic environmental loads defined for the limit states. ALS corresponds to a characteristic environmental load effects with annual probability of exceedance not larger 10^{-4}. NORSOK N-003 (2007) contains an overview of different combinations that can be used to ensure that this requirement is satisfied. This overview is shown in Table 1.2.

Table 1.2 Combination of environmental actions with expected mean values and annual probability of exceedance 10^{-2} and 10^{-4} (NORSOK N-003, 2007)

Limit state	Wind	Waves	Current	Ice	Snow	Earthquake	Sea level [a]
Ultimate Limit State	10^{-2}	10^{-2}	10^{-1}	-	-	-	10^{-2}
	10^{-1}	10^{-1}	10^{-2}	-	-	-	10^{-2}
	10^{-1}	10^{-1}	10^{-1}	10^{-2}	-	-	m
	-	-	-	-	10^{-2}	-	m
	-	-	-	-	-	10^{-2}	m
Accidental Limit State	10^{-4}	10^{-2}	10^{-1}	-	-	-	m*
	10^{-2}	10^{-4}	10^{-1}	-	-	-	m*
	10^{-1}	10^{-1}	10^{-4}	-	-	-	m*
	-	-	-	10^{-4}	-	-	m
	-	-	-	-	-	10^{-4}	m

[a] m - mean water level
m* - mean water level, including the effect of possible storm surge
Seismic response analysis should be carried out for the most critical water level.

1.3.3 Serviceability Limit State (SLS)

Serviceability limit states for offshore structures are associated with:

- Deflections which may prevent the intended operation of equipment
- Deflections which may be harmful to finishes or non-structural elements
- Vibrations which may cause discomfort to personnel
- Deformations and deflections which may spoil the aesthetic appearance of the structure

Serviceability requirements will normally be defined by the operator for the specific project and in general all partial safety factors are set to 1.0.

1.3.4 Fatigue Limit State (FLS)

Structures are designed to withstand the presupposed repetitive (fatigue) actions during the life span of the structure. Design fatigue factors are applied for safety and with the objective to minimize life cycle costs, taking into account the need for in-service inspection, maintenance and repair.

1.4 Relative Importance of ALS and ULS

According to Norwegian Rules and Regulations NORSOK N-003 (2007) and PSA (2001), an offshore structure is to be controlled against overload failures by the Ultimate limit state (ULS) control and Accidental limit state (ALS) control .The ULS design control will most often govern the design against environmental loads. However, in bad-behaving problem (shape parameter changing abruptly for an annual exceedance probability well above 10^{-4}) the ALS controls the environmental design loads (Haver , 2006). To clarify the above statement let's see the uncertainties available in estimating design loads.

For offshore structures in estimating the characteristics load, both the epistemic uncertainty (uncertainty introduced by our lack of knowledge regarding an underlying deterministic phenomenon or parameter) and aleatory (uncertainties that are inherent random nature) variability of the environmental process affects the result. The epistemic uncertainty can be minimized by gaining more knowledge. Increased knowledge can be gained by collecting more data, executing research work and investigating in better equipment for monitoring the phenomenon.

However, the dominating source of uncertainty is due to the inherent randomness of the environmental process. This means that with very low annual probability, the structure can face loads significantly larger than the characteristics load even if epistemic uncertainties don't exist. To account the total variability associated with load and capacity, partial safety factors are introduced (γ_f and γ_m in equation 1.1). For steel structures on the Norwegian continental shelf γ_f =1.3 and γ_m = 1.15 are recommended factors.

In linear problem if we multiply the values corresponding to an annual exceedance of 10^{-2} by the load factor 1.3, the annual exceedance probability of γ_f x_c is usually lower than 10^{-4}. However, for non-linear response problem γ_f x_c will typically correspond to an annual exceedance probability higher than 10^{-4} (Haver , 2006) . This is illustrated in Fig. 1.4

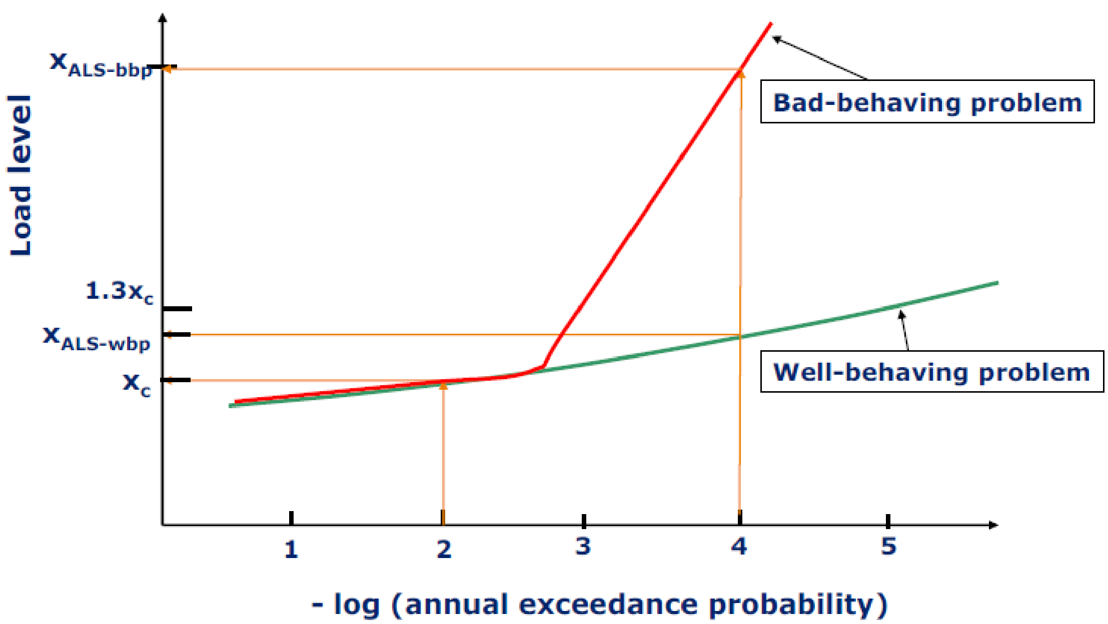


Fig. 1.4 Bad -behaved versus Well-behaved response problem (Haver , 2006)

As it is shown in Fig.1.4 for well-behaving system, γf xc will give a design load level corresponding to an annual exceedance probability typically around 10^{-4}. For the bad-behaving system, it is seen that γf xc corresponding to an annual exceedance probability much larger than 10^{-4}. Hence it can be difficult to obtain low annual failure probability. As an Example for old structures where the load pattern for one reason or the other is considerably changed, e.g. worsened wave conditions, reservoir subsidence, etc. one can very well foresee that a bad-behaving tail property is realized. Wave – deck impact is a mechanism that typically will result in a load –exceedance probability relation like the red curve in Fig. 1.4.

In addition to ULS, design codes require that ALS control has to be done against accidental loads. It requires that the structure has to resist accidental loads corresponding to an annual occurrence probability of 10^{-4}. In Fig. 1.4 for bad behaving nature the environmental load corresponding to annual exceedance of 10^{-4} is much larger than the design load γ_f x_c predicted by ULS design. This shows that excessive environmental load may be just as dangerous for the structure as a collision load. The Norwegian rules have introduced the ALS limit state to include such cases by the design process. It requires ALS must be applied to environmental loads.

As a conclusion the ALS control with respect to environmental loads is a convenient way of ensuring certain robustness against unforeseen environmental loads. Therefore on this report we will estimate the ALS extreme impact load from breaking wave.

2. BREAKING WAVE PHENOMENON

2.1 Breaking Wave Definition and Criteria

A breaking wave is a highly complex system. Even some distance before the wave breaks, its shape is substantially distorted from a simple sinusoidal wave (Brown, et al., 1989). In general a breaking wave can be seen as a wave that carries too much energy to remain stable and dissipates it in terms of turbulences or viscous heat. Dissipation of this energy can give large impact loads if it breaks on an offshore structure. In general this excess of energy can be caused due to decreasing of depth and interaction of waves and wind.

In decreasing depth, if the wave reaches shallower water depth, the ratio between amount of energy and depth gets larger, because the wave length decreases and the wave becomes steeper. This rise will cause the breaking.

In interaction of waves and wind in deep water the overlap of waves and the contribution of energy due to wind will cause an increase of energy that excites breaking.

More precisely, waves break due to their increased steepness, as a general criteria in deep water when the ratio of wave height to wave length is greater than 0.14 ($H/L > 0.14$) the wave will break and we get slamming load. The cause of this increase in steepness (see Fig. 1.5) is due to the rise of energy. However, it is pointed out that the onset of breaking may also be affected by the presence of the platform, i.e. the presence of a platform column in front of the wave may steepen the wave such that breaking is initiated. According to Stokes (1847), the necessary criteria to start an individual wave breaking are:

a) The particle velocity of fluid at wave crest equal the phase velocity
b) The crest of wave attains a sharp point with an angle of 120^0
c) The ration of wave height to wave length is approximately 1/7
d) Particle acceleration at the crest of the wave equal 0.5g (where g = gravitational acceleration)

It can also be initiated due change of water depth as approaching to shore Here waves breaking upon shore and deep water will be discussed.

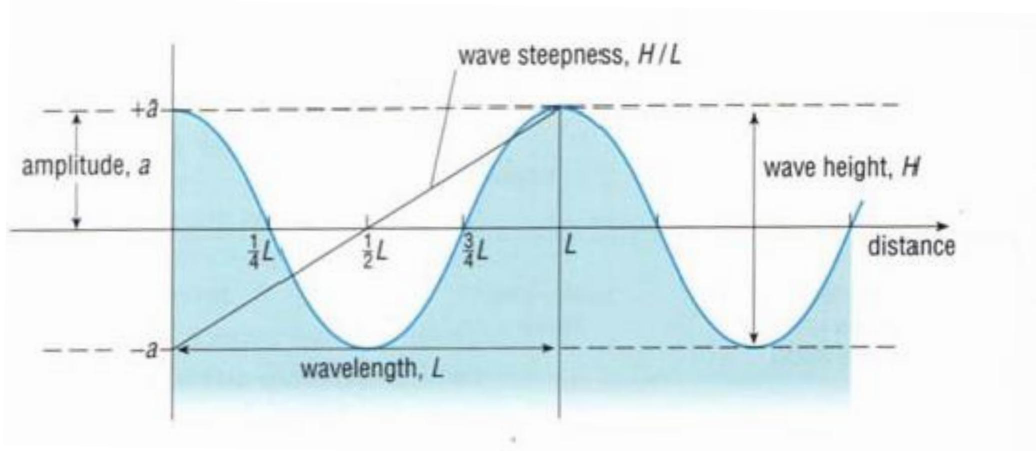


Fig. 2.1 Ocean wave showing its liner dimensions and shape (Brown, et al., 1989)

2.2 Waves Breaking Upon Shore

The most general sort of breaking wave is the breaking of water surface waves on a coastline. As a wave breaks, the energy it received from wind is dissipated. Some energy is reflected back out to sea, the amount depends on the slope of the beach. Most of the energy is dissipated as heat in the final small-scale mixing of foaming water, sand and shingle. Some energy is used in fracturing large rock or mineral particles into smaller ones, and yet more may be used to increase the height and hence the potential energy of the beach forms.
Here are four basic types of breaking water waves. They are spilling, plunging, collapsing, and surging. Each will be discussed in the following subtopics (Brown, et al., 1989).

2.2.1 Spilling

Spilling breakers (Fig. 2.2) are characterized by foam and turbulence at the wave crest. Spilling usually stats some distance from shore and is caused when a layer of water at the crest moves forward faster than the wave itself. Foam eventually covers the leading face of the wave. Such waves are characteristic of a gently sloping shoreline. Breakers seen on beaches during a storm, when the waves are steep and short, are of the spilling type. They dissipate their energy gradually as the top of the wave spills down the front of the crest, which gives a violent and formidable aspect to the sea because of the more extended period of breaking.

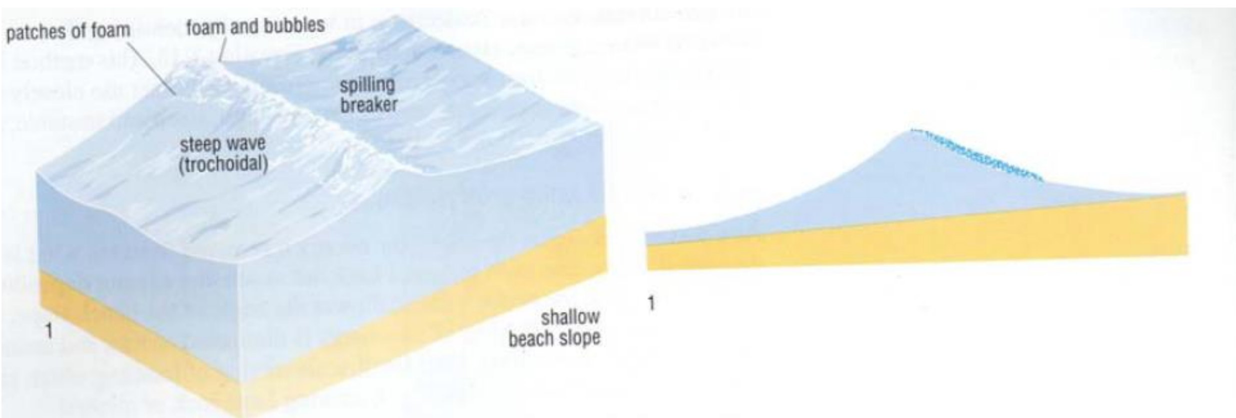


Fig. 2.2 spilling breaker (Brown, et al., 1989)

2.2.2 Plunging

Plunging breakers are the most spectacular type, Fig. 2.3. The classical form, much beloved by surf-riders, is arched, with a convex back and a concave front. The crest curls over and plunges downwards with considerable force, dissipating its energy over a short distance. Plunging breakers on beaches of relatively gentle slope are usually associated with the long swells generated by distant storms. Locally generated storm waves seldom develop into plunging breakers on gently sloping beaches, but may do so on steeper ones (Dalane, 2011).

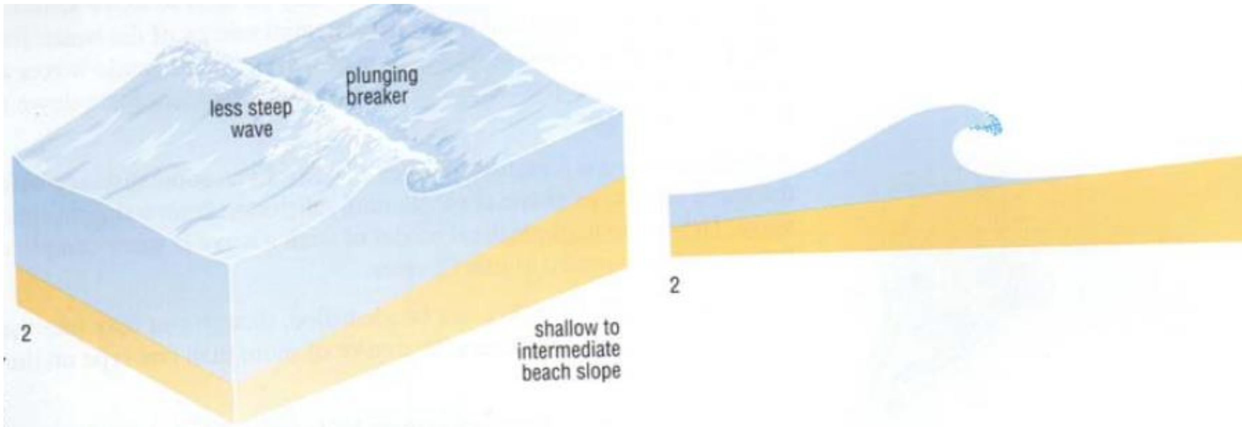


Fig. 2.3 Plunging breakers (Brown, et al., 1989)

2.2.3 Collapsing

Collapsing breakers are similar to plunging breakers, except that instead of the crest curling over, the front face collapses. Such breakers occur on beaches with moderately steep slopes, and under moderate wind conditions. Collapsing breakers are shown in Fig. 2.4.

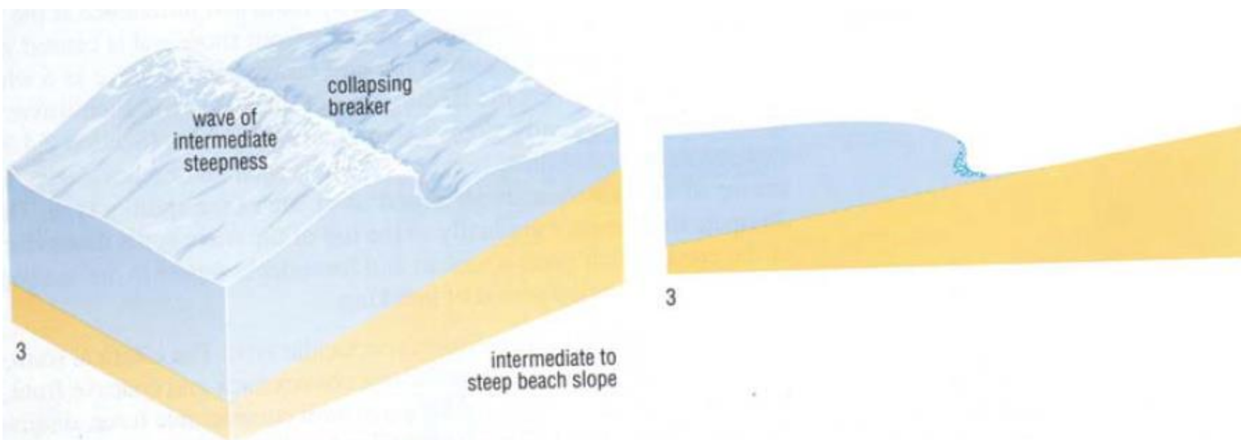


Fig. 2.4 Collapsing breakers (Brown, et al., 1989)

2.2.4 Surging

Surging breakers are found on the very steepest beaches, Fig. 2.5. They are typically formed from long, low waves, and the front faces crests remain relatively unbroken as the wave slide up the beach. The outcome is the rapid movement of the base of the wave up the swash slope and the disappearance of the wave crest. The front face and crest of the wave remain relatively smooth with little foam or bubbles, resulting in a very narrow surf zone or no breaking wave at all (Brown, et al., 1989).

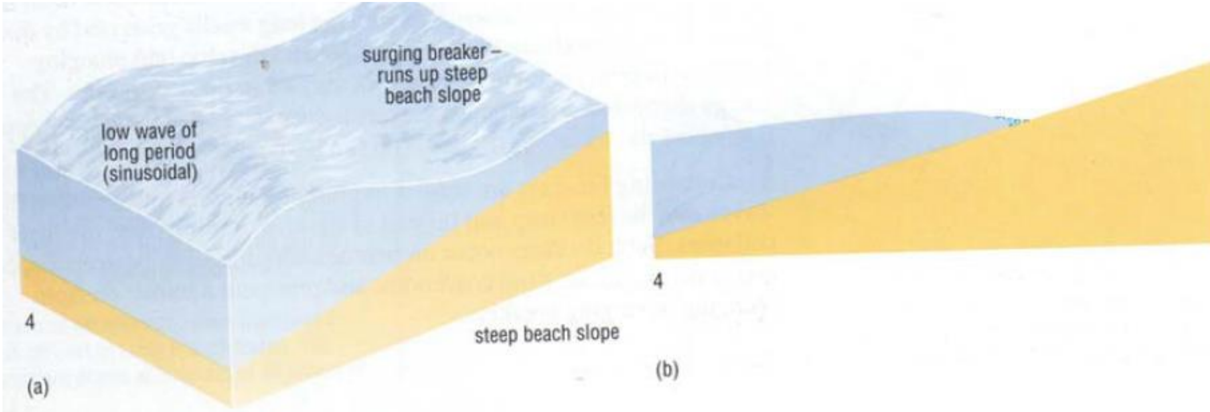


Fig. 2.5 Surging breaker (Brown, et al., 1989)

2.3 Wave Breaking on Deep Water

In deep water there are mainly two types of breaking waves, spilling breakers and plunging breakers. Both are waves that are very asymmetric and have an increasingly steeper crest front when they come close to their breaking point.

2.3.1 Spilling Breakers

These are the most common breaking waves in deep water. Their breaking point is reached when foam appears at the wave crest front. This foam is running down the wave front like an avalanche. This running process is rather regular so that after breaking most of the original wave motion is maintained. The evolution of a spilling breaker is outlined in Fig. 2.2

2.3.2 Plunging Breakers

Plunging breakers do not occur as often as spilling breakers. Plungers are at their breaking point when wave front is vertical, then a curly wave crest is propagating over an air gap (Brown, et al., 1989). This is the characteristic for plunging breakers. The shape of the breaking wave evolves from the fluid particles at the wave crest that outrun the rest of the wave. More precisely, a jet erupts from the wave crest when the wave has vertical front. The wave collapses and the jet hits the surface. This collapse causes turbulence and disturbance in the fluid, so that after the actual breaking process almost nothing of the original wave motion is maintained. The evolution of a plunging breaker is outlined in Fig. 2.3

2.4. Slamming force from breaking waves

The water particles of a sinusoidal linear wave move in circles as it is illustrated in Fig. 2.6. The corresponding velocity can be decomposed in vertical and horizontal direction. This horizontal velocity can increase tremendously due to the nonlinear behavior of the ocean surface. When these water particles acquire a higher horizontal velocity than the ****phase velocity*** of the wave, the water on the top would move faster than the wave itself and result in a breaking wave. This action creates more irregular waves that propagate forward and break close to the structure on the mid ocean during breaking. This water shooting is illustrated in Fig. 2.7. The impact load that hits the structure is also called slamming load and it will act as a load on the structure. Impact loads from breaking waves generates slamming pressure on the body and it is essentially unsteady hydrodynamic pressure resulting from direct contact between the body and the water (Lehn, 2003).

*Phase velocity, C is the velocity of wave expressed as the wave length, L over the period of wave, T. It is given by equation 2.1.

$$C = \frac{L}{T} \tag{2.1}$$

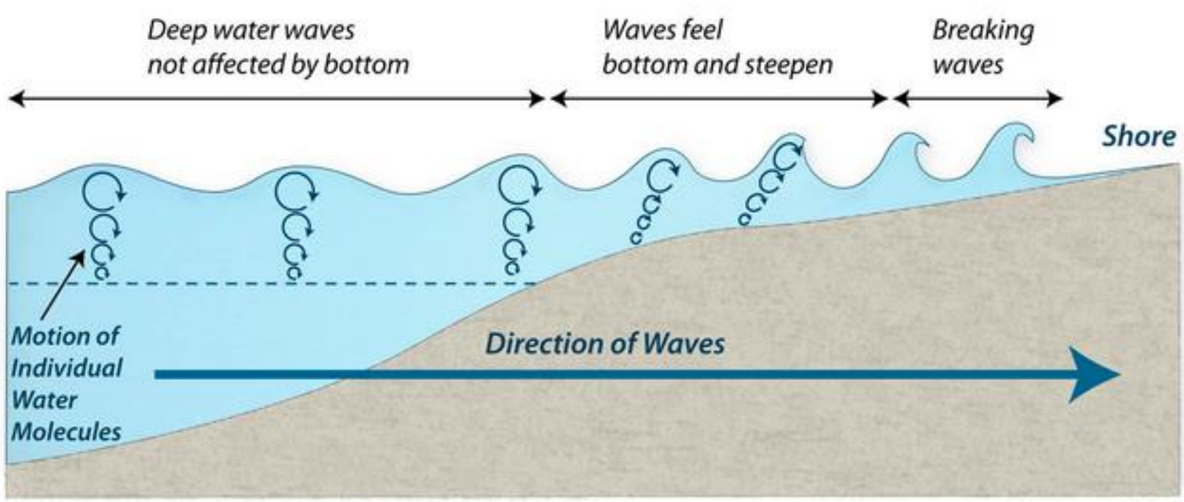


Fig. 2.6 Water particle orbital movement and breaks on shore (University of Maine System, 2003)

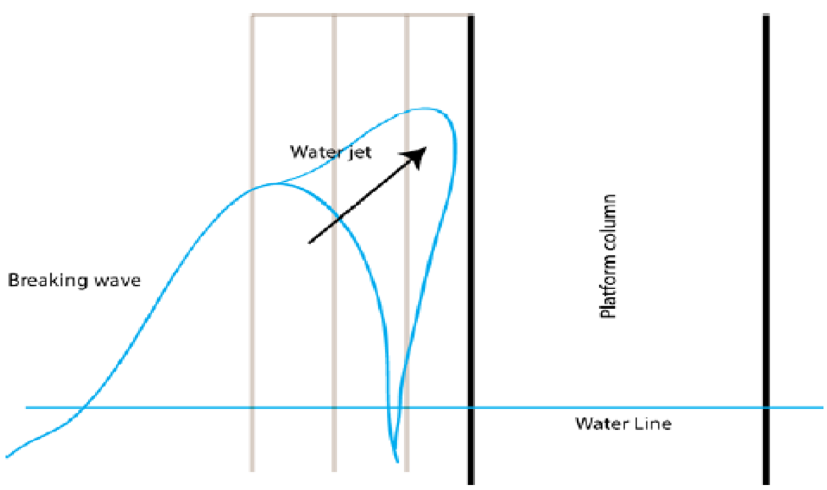


Fig. 2.7 Illustration of water jet shooting out of wave (Dalane, 2011)

3. PREDICTION OF A WAVE DOMINATED LOAD CORRESPONDING TO AN ANNUAL EXCEEDANCE PROBABILITY OF Q

Considering impact loads from breaking waves, NORSOK N-003 defines the characteristic values of the impacts loads by specifying annual exceedance probabilities for the different design limit states. Impact loads from breaking waves fall in the category of environmental actions. For environmental actions the characteristic load values are defined by annual exceedance probability of 10^{-2} for ultimate limit state (ULS) and 10^{-4} for accidental limit state (ALS).

Accordingly, the characteristic loads due to breaking waves are the loads with a return period of 100 years (ULS) and 10000 years (ALS). In the next chapter (chapter four) the impact pressure due to breaking waves on platform columns will be assessed using the observed 3-hour extreme values. Before we proceed to that approach let's see first on this chapter how a wave dominated load corresponding to an annual exceedance probability of q can be estimated.

Offshore structures must be designed to very low probabilities of failure due to environmental loads in addition to permanent and functional loads. These structures are required to be designed to exceed specific levels of reliability, expressed in terms of an **annual probability of failure or return-period** (Ewans & Jonathan, 2014). This requires specification of values of environmental variables with very low probabilities of occurrence. More specifically, it is to determine structural loading due to environmental forcing and a combination of environmental phenomena with a given return-period that is sought.

The goal is thus to design an offshore facility to withstand extreme environmental conditions that will occur during its lifetime with an appropriate optimum risk level. The level of risk is set by weighing the consequences of failure against the cost of over-designing (Ewans & Jonathan, 2014). In Norwegian rules and regulations, the characteristic loads are defined with reference to a given annual probability of exceedance q. A correct estimate of this characteristic value requires that the long-term variability in the weather condition as well as the short-term variability of the response given the sea state is accounted for. In order to establish a consistent estimate for a load corresponding to a given annual exceedance probability, some sort of a long term response analysis is in principle required.

3.1 Methods of predicting characteristic impact loads

It is important to note that according to Norwegian Rules and Regulations, the target annual exceedance probability refer to the load and not the environmental condition. This means that in connection with the ULS and ALS control against environmental loads, one should obtain reliable estimates for load/load effect corresponding to an annual exceedance probability of 10^{-2} and 10^{-4} respectively (Haver , 2006).

In general there are three commonly used approaches in estimating the wave dominated load corresponding to an annual exceedance probability of q (Naess & Moan, 2013).

i. The design wave approach
ii. The design sea state approach
iii. The full long term approach.

A brief introduction of the above three methods will be discussed on this chapter and a detail of the all sea state approach, particularly the short term extreme of 3-hour maximum value will be discussed. ***This approach will help as a base in assessing the distribution function for 3-hour maximum impact pressure due to breaking waves based on observed 3-hour extremes.***

3.1.1 Design Wave Approach

For the design of offshore structures where the load effects to be considered are primarily of a quasi-static nature, design wave approach is the convenient method (Naess & Moan, 2013). On this method the input is q probability wave height and its associated wave period. For example for extreme load effect for ULS and ALS design check can be estimated by the so called 100yr or 10000yrs wave approach ($H^{(100)}$ and $H^{(10000)}$) respectively. Estimation of load according to this format would in practice proceeds as follows. (1) The design wave height is established on the basis of available data relevant for the offshore location in question. (2) A suitable range of corresponding wave period is specified. (3) According to best practice, items 1 and 2 are combined to provide a range of wave profiles for which the corresponding load effects on the structure are established, either by numerical calculation using state-of-the-art computer code or in some cases model test.

The next question comes, how can we find the design wave height? NORSOK N-003 (2007) gives a guideline to take $H^{(100)} = 1.9 * H_s$, in case of accurate estimate is not available. H_s denote the significant wave height corresponding to an annual exceedance probability of 10^{-2}. The correspond wave period T is in the range of $\sqrt{6.5 * H^{(100)}} \le T \le \sqrt{11 * H^{(100)}}$.

In absence of more detailed documentation, the wave height, H_{10000} with annual exceedance probability 10^{-4} can be taken to be 1.25 times H_{100}, while the period is increased by 5 %, as compared to the period of H_{100}.

3.1.2 Short-Term Design Approach

In the short term design approach, it is necessary to calculate the extreme loads and responses during a short term storm condition. In many cases that would be done by assuming that the stochastic load or response process is stationary and Gaussian, where ***frequency domain method*** can be applied.

The short-term design approach is preferred for very complex problems, e.g. problems where time domain methods (numerical calculations and model tests) are required for solving the equation of motion. In recent years, the environmental contour line approach has been advocated as a rational basis for choosing the appropriate short term design storms leading to load and response extremes corresponding to a prescribed annual probability of exceedance.

Environmental contour line plots are convenient tools for complicated structural dynamic systems where a full long-term response analysis is extremely time consuming and costly. It makes it possible to obtain reasonable long term extremes by concentrating on a short-term consideration of a few sea states in the scatter diagram. The advantage of this method is that analysis of only few sea states are required. As the most unfavorable sea state along the q-probability contour line is identified, a proper estimate of the q-probability is taken as the α-fractile of distribution of the 3-hour extreme response. However, the challenge of this method is to know the value of α.

3.1.3 Long-Term Design Approach

The most consistent and accurate design approach to determine extreme loads for ULS and ALS design check is based on long-term statistics of response, but it may clearly be un economical method from computational point of view (Naess & Moan, 2013). This could be due to involvement of response calculation for sea states that contribute little or nothing to the design load or load effects. The simplified approaches described above need to be validated by the full long-term approach. The long-term design approach mainly consists of the conditional short term distribution and the long term variation of the wave climate. This means that many wave conditions must be analyzed and that the long-term distribution is obtained as a weighted sum of the short-term distributions. Three approaches are possible to estimate characteristic long term extreme loads. They are described in more detail on Naess & Moan (2013). These methods are based on:

- All peak values
- All short term extremes
- The long term extreme value

3.1.3.1 All peak values

In this approach all global peak values Z_p of Z(t) in a short term duration are considered ,where a global peak value Z_p is defined as the maximum value of Z(t) between two sequent zero-upcrossings. For each short term condition the conditional cumulative distribution function (CDF) of Zp, $F_{Zp|Hs,Tp}(z|h_s,t_p)$, has to be known. If the problem is a linear response system, i.e. a linear mechanical system, where the load is linear and Z(t) is Gaussian, the Rayleigh distribution is an often chosen stochastic model for, $Fz_{p|H_sT_p}(z\,|\,h_s,t_p)$.

The long term CDF of the global peak values Z_p can then be written as given in (3.1) (Naess & Moan, 2013), where $v_z^+(0\,|\,h_s,t_p)$ denotes the average zero-upcrossing frequency of Z(t) for the short term condition and $\overline{v_x^+(0)}$ is the long term average zero-upcrossing frequency of Z(t) given by (3.2).

$$Fz_p(z) = \frac{1}{\overline{v_z^+(0)}} \int_{h_s}\int_{t_p} v_z^+(0\,|\,h_s t_p)\; Fz_{p|H_sT_p}(z\,|\,h_s,t_p)\; f_{H_s,T_p}(h_s,t_p)dh_s dt_p \qquad (3.1)$$

$$\overline{v_z^+(0)} = \int_{h_s}\int_{t_p} v_z^+(0\,|\,h_s t_p)\; f_{H_s,T_p}(h_s,t_p)dh_s dt_p \qquad (3.2)$$

For q-probability peak value $Z_{p,q}$ can be obtained from equation (3.3), assuming that all Z_p are independent,

$$Fz_p(Z_{p,q}) = 1 - \frac{1}{365*24*3600s*\overline{v_z^+(0)}} \qquad (3.3)$$

3.1.3.2 All short term extremes

All short term extremes approach is discussed in detail in section 3.2 with a title all sea state extreme value approach.

3.1.3.3 Long term extreme value

By selecting only all the global extreme values $\hat{Z}_p = \hat{Z}_p(T)$ of a long term period T is considered. The CDF of $\hat{Z}_p$ is given in by equation (3.4)

$$F_{\hat{z}}(z) = \exp(-T\int_{h_s}\int_{t_p} v_z^+(z \mid h_s, t_p) f_{H_s,T_p}(h_s, t_p) dh_s dt_p) \qquad (3.4)$$

In equation 3.4 $v_z^+(z \mid h_s, t_p)$ denotes the average x-upcrossing frequency of Z(t) for the short term condition. When T is choose to be one year, i.e. T=365*24*3600s, the q-probability value $Z_{p,q}$ is calculated by solving (3.5).

$$F\bar{z}_p(Z_{p,q}) = 1 - q \qquad (3.5)$$

Here on this paper all short term (3-hour duration) extremes approach will be discussed in detail with the help of example and data. Other commonly used design approaches are explained in detail in Haver (2013) These are:

i. Reliability method
ii. Random Storm Approach or Peak Over Threshold (POT)
iii. All sea state approach
iv. Environmental contour line approach
v. Monte Carlo Simulation

3.2 All Sea State Approach

This is a long term analysis, on this section we will mainly focus on linear mechanical system and the analysis for nonlinear mechanical system will be discussed on section 3.3. By selecting the 3-hour maximum load as a target response, a long term analysis can conveniently be done. This means that the weather development is approximated by a sequence of stationary 3-hour events. Let's denote Z_{3h} as the 3-hour extreme value and its conditional distribution for a given sea state characterized by H_s and T_p, is denoted by $F_{Z_{3h}|Hs,Tp}(z \mid h_s, t_p)$. The long term variation in the wave climate can be described by joint probability density function of H_s and T_p, $f_{Hs,T_p}(h_s, t_p)$. Then the long term distribution of the 3-hour maximum is given by:

$$F_{\bar{Z}_{3h}}(z) = \int_{HsTp}\int \left[F_{Z_{3h}|Hs,Tp}(z \mid h_s, t_p)\right] f_{Hs,T_p}(h_s, t_p) dh_s dt_p \qquad (3.6)$$

As the long term distribution for the 3-hour maximum load is found, an estimate for the value corresponding to an annual exceedance probability of q can be obtained from the following relationship:

$$F_{Z3h}(Z_q) = 1 - \frac{q}{m_{3h}} \qquad (3.7)$$

Where m_{3h} is the expected annual number of 3-hour periods i.e. 365*24/3 = 2920.

Based on the long term distribution of the 3-hour maximum given by equation (3.6), en elaboration on seven tasks for further detail will follow. The first five tasks will be about short term description of linear response i.e. the conditional distribution of the 3-hour extreme value. The last two tasks will be about long term response distribution of the 3-hour maximum value. This illustration will help to understand the basic concepts of long-term analysis.

3.2.1 Short term description of linear response

For short term linear response, i.e. for each frequency there is a linear relationship between the response amplitude and wave amplitude. This relationship is characterized by the transfer function, $|h_{\Xi X}(f)|$. Hence the response spectrum is given by :

$$S_{\Gamma\Gamma}(f) = |h_{\Xi X}(f)|^2 \, S_{\Xi\Xi}(f) \qquad (3.8)$$

The transfer function is commonly referred to as the response amplitude operator or RAO as function of frequency, f or w. Where f is in H_z and w is in radians/ second based on equation 3.8 let's analyze each parts in the following five tasks.

TASK-1 ***In practice work the RAO or the transfer function is obtained by numerical analysis or model test.*** Here on this section for illustration purpose we will assume the response amplitude operator (RAO) is given by the functions shown on Fig. 3.1 We can generally describe RAO as a vector of number corresponding to a vector of frequency.

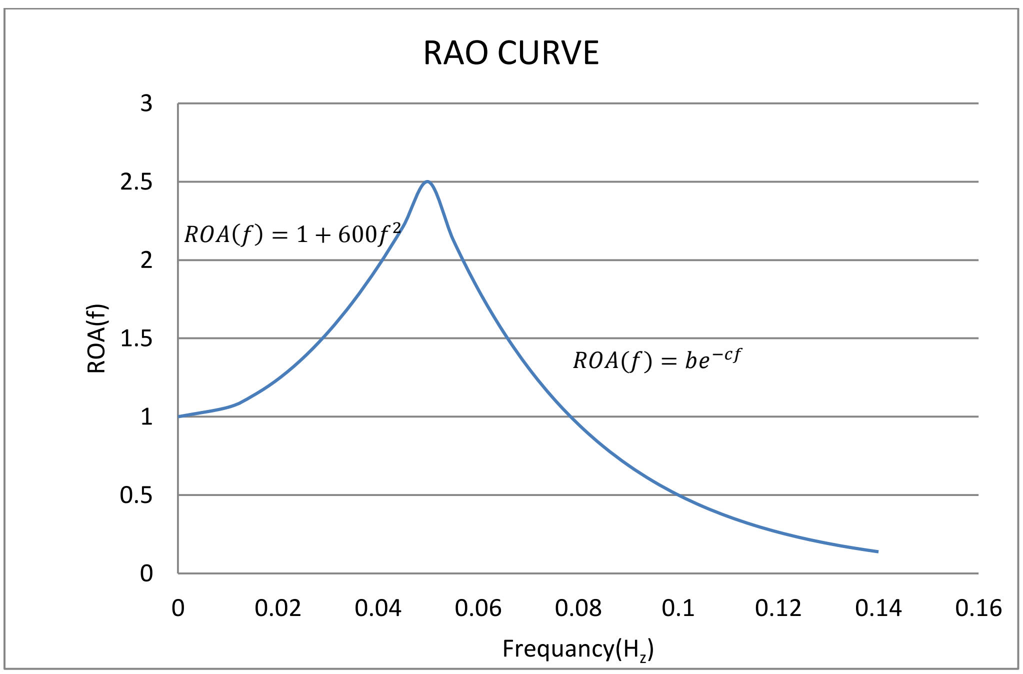


Fig. 3.1 RAO curve

By solving the mathematical equation, the RAO function for the two parts can be given by (a) and (b).

For f < 0.05Hz $\quad RAO(f) = 1 + 600 f^2 \quad$ (a)

For f > 0.05Hz $\quad RAO(f) = 12.5 * e^{-32.19 f} \quad$ (b)

These two RAO function will help us in calculating the response using equation (3.8), but let's first calculate the wave process spectra in task2.

<u>Task -2</u> calculating the wave spectrum for different values of H_s and T_p, for example let H_s = 10m and T_p = 12sec.

A random ocean wave process can be described by energy density spectrum, where the energy spectrum describes the energy content of an ocean wave process and its distribution over a frequency range of wave process.

There are several standard spectrum formulas that are used in the design of offshore structures. These formulas are derived from observed properties of ocean waves and are thus empirical in nature. The most commonly used spectrum formulas are Pierson Moskowitz (P-M), JONSWAP, ISSC model, Bretschneider and less used Ochi-Hubble (Chakrabarti, 2005). The three most commonly used wave spectra models in Norwegian Continental shelf are the P-M, JONSWAP and Torsethaugen spectrum (for combined sea state).

As an example, for the given H_s and T_p values, I chose a Pierson Moskowitz model to represent wave process spectrum. Note that the P-M is a special case of JONSWAP with the peakedness parameter value being one.

The Pierson Moskowitz model or formula is given in equation (3.9) and using this equation the wave spectrum can be plotted as shown in Fig.3.2

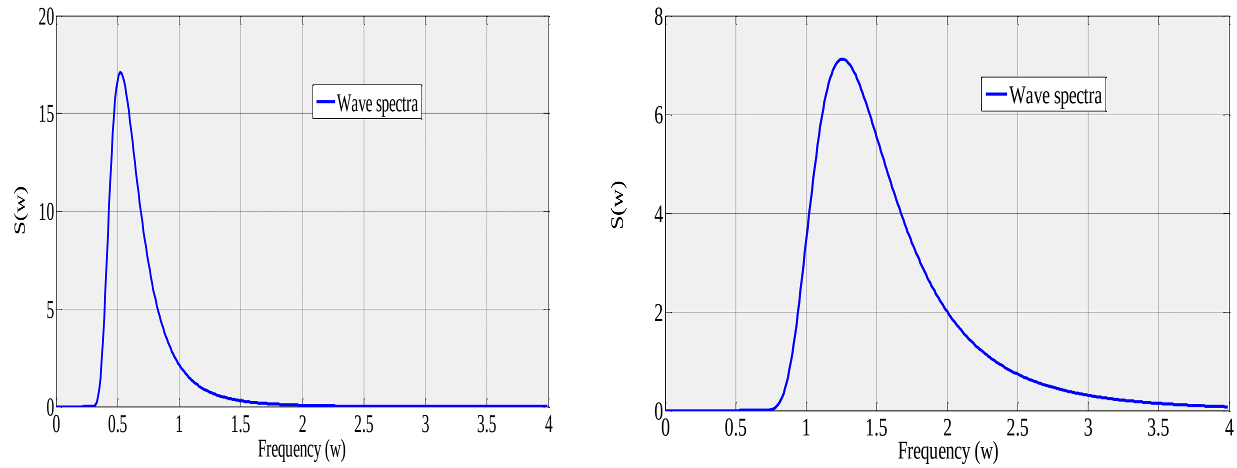
$$S_{pm}(w) = \frac{5}{16} H_s^{\,2} w_p^{\,4} w^{-5} \exp\left(\frac{-5}{4} \left(\frac{w}{w_p} \right)^{-4} \right) \qquad (3.9)$$

Let's calculate the value of peak wave frequency W_p for T_p = 12s

$$w_p = \frac{2\pi}{T_p} = \frac{2\pi\, rad}{12 s} = 0.523\, rad/\sec$$

$$S_{pm}(w) = \frac{5}{16} H_s^{\,2} w_p^{\,4} w^{-5} \exp\left(\frac{-5}{4} \left(\frac{w}{w_p} \right)^{-4} \right)$$

$$\underline{S_{pm}(w) = 2.338 w^{-5} \exp(-0.0935 w^{-4})}$$



(a) (b)

Fig. 3.2 Wave spectra for a given (a) H_s=10m and T_p=12s and (b) H_s=10m and T_p=5s

For combined sea state, a sea state that includes both wind sea component process and swells component process, Torsethaugen wave spectra represents the sea surface wave process. Fig. 3.3 shows a swell dominated sea state for JONSWAP and Torsethaugen wave spectra, for given H_s =10m and T_p= 20s. Details of the Torsethaugen spectrum is given in appendix A of DNV-RP-C205.

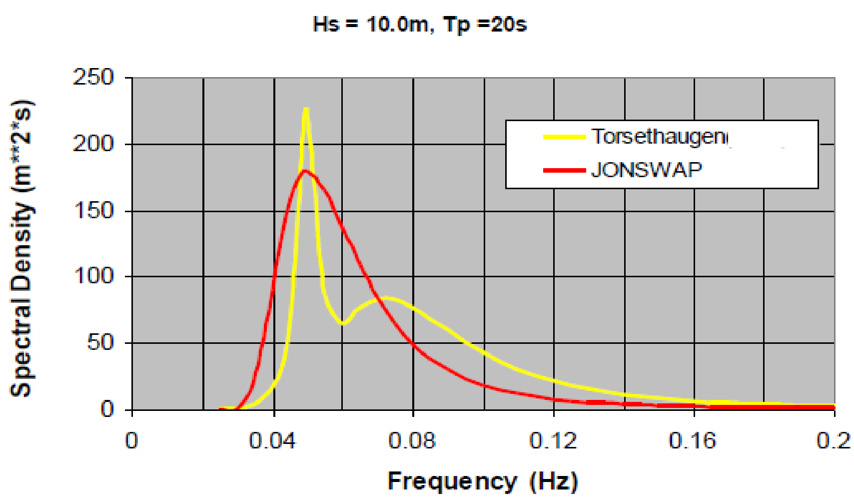


Fig. 3.3 Comparison of JONSWAP and Torsehaugen wave spectra model for swell dominated sea state (Haver, 2013)

<u>Task 3</u>: Response Spectra $S_{\Gamma\Gamma}(w)$ and wave spectra $S_{pm}(w)$ relationship assuming $RAO(w)$ of structure is known

For ULS consideration and often also for ALS considerations, structural deformation will essentially stay with in elastic regime and the structure can be modelled as a linear mechanical system. If for some reasons one would like to analyze a structure until collapse takes place, a computer program accounting for non-linear structural behavior must be adopted.

For linear mechanism the response quantity can be conveniently characterized by the transfer function. The transfer function, which is the ratio between the complex response amplitude and the wave amplitude, is a function of frequency and it gives both the amplitude scaling and phase shift of response relative to a wave component. The amplitude scaling or rather the absolute value of the transfer function, $|h_{\Xi X}(f)|$ is often referred to as the response amplitude operator, RAO (f) (Haver, 2013). Hence the response can be calculated as:

$$S_{\Gamma\Gamma}(w) = [RAO(w)]^2 S_{\Xi\Xi}(w) \qquad (3.10)$$

In our case since the response amplitude operator (RAO) is a combination of two equations, the response spectrum will also has two equations. The first is for the range of f < 0.05 H_z and the second one for f>0.05.

For f < 0.05 H_z or w < 0.314 rad/s

$$S_{\Gamma\Gamma}(w) = S_w(w)\,ROA^2$$
$$S_{\Gamma\Gamma}(w) = 2.338w^{-5}\exp(-0.0935w^{-4})(1+600f^2)^2$$

For f > 0.05 H_z or w > 0.314 rad/s:

$$S_{\Gamma\Gamma}(w) = S_w(w)RAO^2$$
$$S_{\Gamma\Gamma}(w) = 2.338w^{-5}\exp(-0.0935w^{-4})\left(12.5e^{-32.19\frac{w}{2\pi}}\right)^2$$

Based on the above equations we can plot the response spectra for given sea state. Fig. 3.4 shows the response spectrum for H_s=10m and T_p =12s

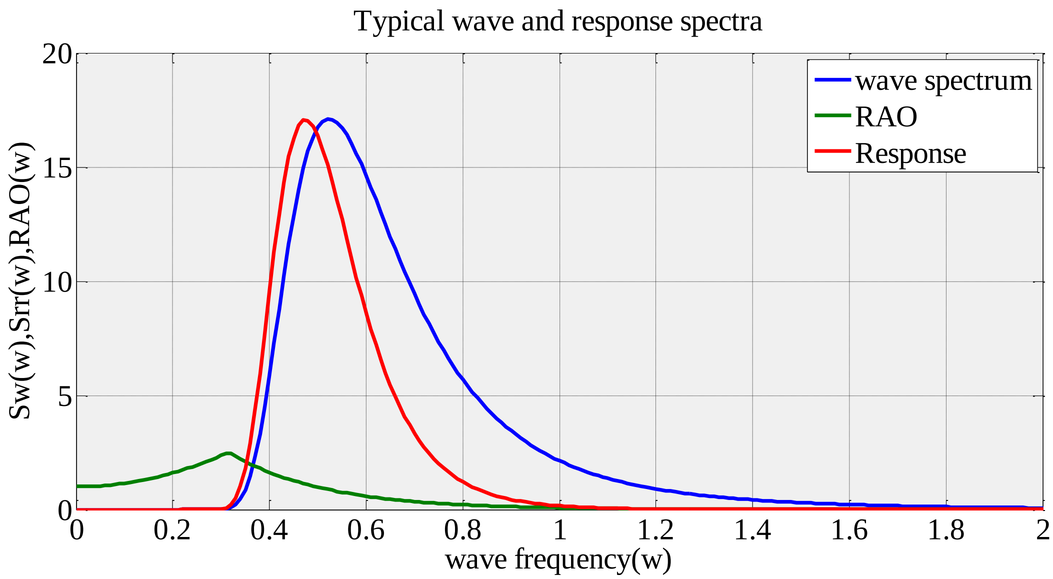


Fig. 3.4 Response spectra for a given sea state (H_s =10m and T_p = 12s) and RAO

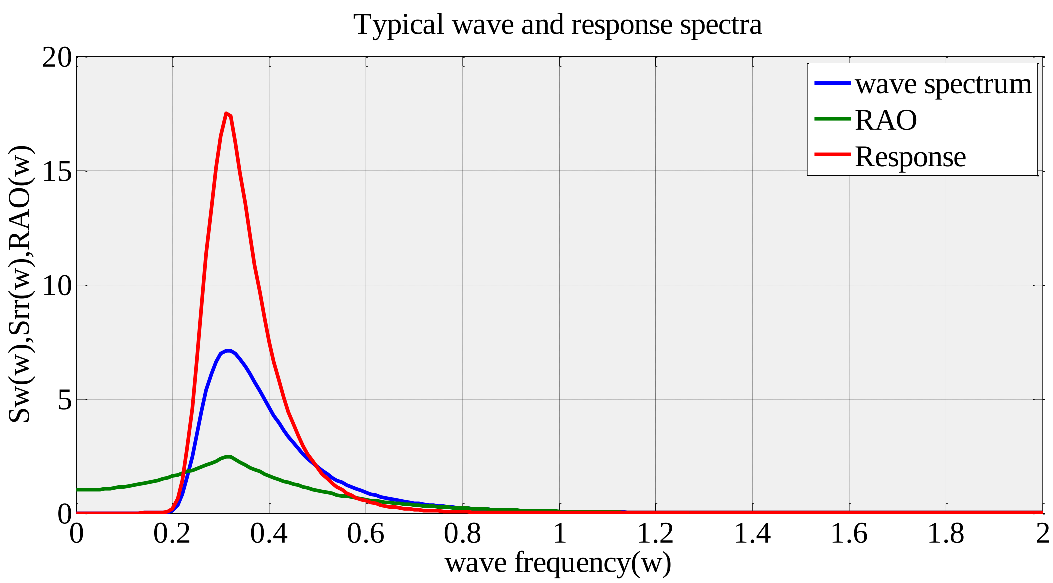


Fig. 3.5 Response spectra during resonance for a given sea state (H_s =5m and T_p = 20s) and RAO

Figure 3.5 shows that the response amplifies to multiple times for the values of H_s=5m and T_p= 20s. This could be the response during resonance.

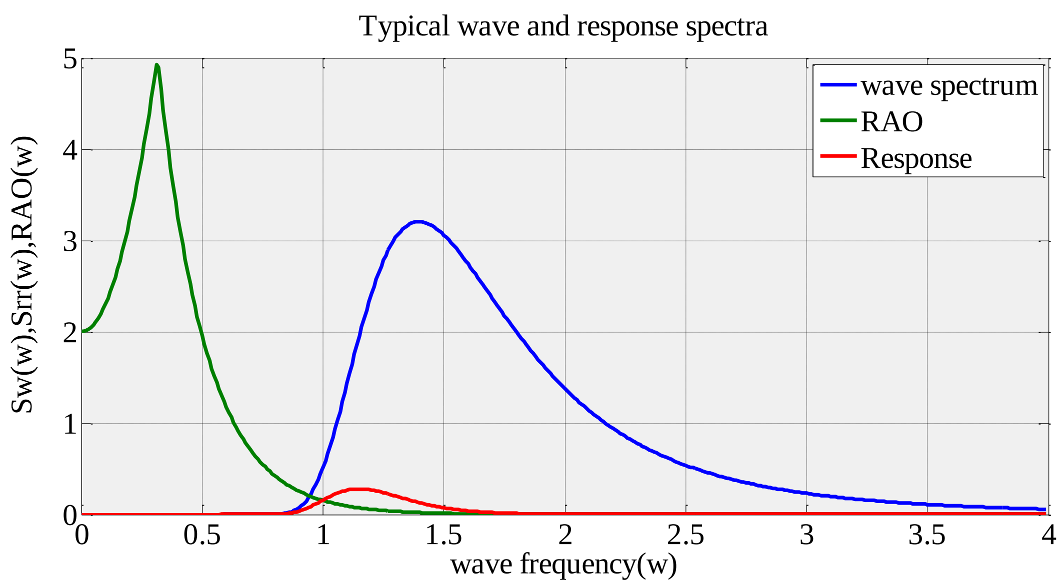


Fig. 3.6 Response spectra for a given sea state (H_s =5m and T_p = 4.5s) and RAO (Scaled by 2)

Figure 3.7, shows the response when frequency of the wave is away from the structures egen frequency, there is almost no response (red line) for given sea state (H_s=5 and T_p =4.5s) and RAO.

<u>TASK 4.</u> Estimating the Maximum peak value ($\tilde{Y}$) for a given sea state.

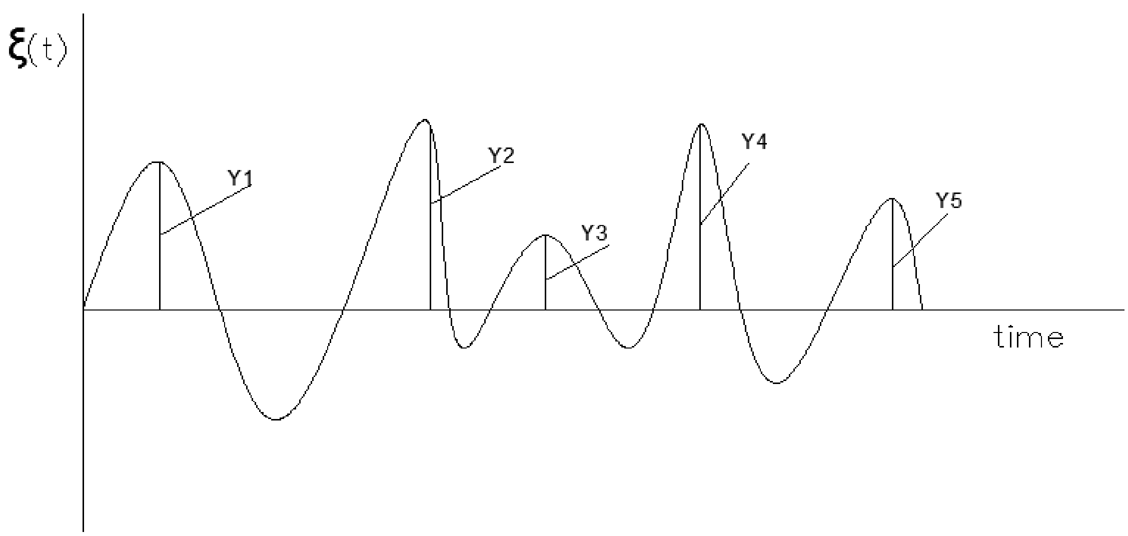


Fig. 3.7 Time history of measured wave height

Sea surface can be described as Gaussian process and stationary sea state condition when the parameters that vary with the process have shorter period than the typical wave period. Under the Gaussian assumption, global wave crests (largest crest between adjacent zero-up-crossings), Y, are well modelled by the Rayleigh probability distribution given by equation (3.11). This distribution is very important in our coming section in determining the conditional short term extreme value (section 3.2.2)

$$F_Y(y)=1-\exp\left(-\frac{1}{2}\left(\frac{y}{\sigma_\Xi}\right)^2\right) \qquad (3.11)$$

The only parameter involved is the standard deviation, σ_Ξ of the process. This can be found from the wave spectrum specified in task 2 for a given sea state.

The variance can be calculated using equation 3.12 :

$$\sigma_\Xi{}^2 = \int_0^\infty S(w)dw \qquad (3.12)$$

In task 2 we have already calculated S(w) from Pierson Moskowitz on equation (3.9') . Now in equation (3.2) the integral part $\int_0^\infty S(w)dw$ is the $zero^{th}$ moment spectra. Generally the j^{th} moment spectra can be calculated using equation (3.13).

$$m_j = \int_0^\infty w^j S(w)dw \qquad (3.13)$$

Now let's back to our target maximum peak value, the probability of exceedance for this maximum peak value can be obtained using equation (3.14).

$$1\text{-}F_Y(y)=\frac{1}{m_{3h}} \qquad (3.14)$$

Where $F_Y(y)$ is the cumulative distribution of peak values given in equation (3.11)

m_{3h} is the expected number of waves in 3-hour duration and it can obtained from equation (3.15)

$$m_{3h} = \frac{3*60*60}{t_{mo2}} \qquad (3.15)$$

t_{m02} is the average zero up crossing period and can be calculated from the spectra moments, m_0 and m_2

$$t_{mo2} = 2\pi\sqrt{\frac{m_0}{m_2}} \qquad (3.16)$$

The largest wave crest out of the expected number of waves is given by $\tilde{Y}$, where the probability of exceeding $\tilde{Y}$ can be given by:

$$\left(1 - F_Y(\tilde{Y})\right) = \frac{1}{m_{3h}} \tag{3.17}$$

Where $F_Y\left(\tilde{Y}\right)$ and m_{3h} are given in equation (3.11) and (3.15) respectively. By substituting these on equation (3.17) we get

$$\exp\left(-\frac{1}{2}\left(\frac{\bar{Y}}{\sigma_{\Xi}}\right)^2\right) = \frac{1}{m_{3h}}$$

$$\textit{and this can be approximated to} \;\Rightarrow\; \underline{\underline{\tilde{Y} = \sigma_{\Xi}\sqrt{2\ln\, m_{3h}}}} \tag{3.18}$$

Now let's back to our case, we have already calculated the response spectrum $S_{\Gamma\Gamma}(w)$ and we can calculate m_0 and m_2

$$m_0 = \int_0^{\infty} w^0 S(w)dw \;=\; \int_0^{\infty} S_{\Xi\Xi}(w)dw \quad \textit{if go back to our illustrative example}$$

$$\int_0^{\infty} w^0 S(w)dw = \int_0^{0.314} S_{\Xi\Xi}(w)dw \;+\; \int_{0.314}^{\infty} S_{\Xi\Xi}(w)dw$$

In the same manner as the above we can calculate second order moment m_2

$$m_2 = \int_0^{\infty} w^2 S_{\Xi\Xi}(w)dw \;=\; \int_0^{\infty} w^2 S_{\Xi\Xi}(w)dw \;=\; \int_0^{0.314} w^2 S_{\Xi\Xi}(w)dw \;+\; \int_{0.314}^{\infty} w^2 S_{\Xi\Xi}(w)dw$$

From the above solution we can obtain the variance and average zero up crossing period using MATLAB. The following result is obtained from MATLAB for our illustrative example i.e. H_s =10m and T_p =12s

Wave analysis results from MATLAB

Zero order moment (m_o) is ***6.247695e+000***

Second order moment (m_2) is ***3.321956e+000***

Average up crossing period (T_z) is ***8.616737e+000***

Number of up crossing cycles of wave (m3h) is ***1.253375e+003***

Standard deviation, $\underline{\underline{\sigma_\Xi}}$ *of wave process is* ***2.499539e+000***

N.B. The MATLAB calculations and programming (editor) is attached on the Appendix c

The maximum peak value $\tilde{Y}$ for H_s =10m and T_p =12sec is therefore 9.44m **(**from equation (3.18))

For the response, maximum peak value can be calculated in the same manner as the wave process. For the coming tasks the response process analysis will be performed

TASK 5: Calculate the distribution function for the Z_{3h} for given H_s and T_p

On the previous tasks (from task 1 to task 4) we analyzed the wave process and from wave spectra we obtained the characteristic largest wave crest out of the expected number of waves. Similarly we can obtain the characteristic largest response amplitude during 3-hour sea state from the response process. Let's assume that the amplitude process be expressed with Z values and the 3-hour maxima by Z_{3h}. As we already said our assumption is linear response and hence the global maxima (largest crest between adjacent zero-up-crossing) distribution is Rayleigh distribution (Haver, 2013) and can be given by equation (3.19).

$$F_{Z|Hs,Tp}(z\,|\,h_s,t_p) = 1-\exp\left[-\frac{1}{2}\left(\frac{z}{\sigma_\Gamma}\right)^2\right] \qquad (3.19)$$

For a 3-hour duration sea state the characteristic largest response amplitude $\tilde{Z}$ can be given by:

$$1-F_Z\left(\tilde{Z}\right)=\frac{1}{m_{3h}} \quad \textit{and this can be approximated to} \Rightarrow \quad \underline{\underline{\tilde{Z}=\sigma_\Gamma\sqrt{2\ln \overline{m_{3h}}}}}$$

But if we observe a number of realizations from the same short term sea state, $\tilde{Z}$ will be slightly different value for each realization. This shows that the largest response value by itself is a random variable and its distribution can be derived as follows:

If we assume that all crest tops are independent and identically distributed, the cumulative distribution of the largest peak value can be given by

$$F_{Z_{3h}}(z) = P(Z_{3h}\le z) = P((Z_1\le z)\cap(Z_2\le z)\cap \quad . \quad . \quad .\cap(Z_n\le z))$$

$$F_{Z_{3h}|Hs,Tp}(z\,|\,h_s,t_p) = \left[F_{z|Hs,Tp}(z\,|\,h_s,t_p)\right]^{m_{3h}}$$

$$F_{Z_{3h}|Hs,Tp}(z\,|\,h_s,t_p) = \left\{1-\exp\left[-\frac{1}{2}\left(\frac{z}{\sigma_\Gamma}\right)^2\right]\right\}^{m_{3h}} \qquad (3.20)$$

Equation (3.20) can be approximated by the Gumbel cumulative distribution as follows:

$$F_{Z_{3h}|Hs,Tp}(z\,|\,hs,tp)=\exp\left\{-\exp\left[-\left(\frac{z-\gamma}{\beta}\right)\right]\right\} \tag{3.21}$$

Where: the parameters β and γ can be obtained as follows (Haver, 2013):

$$\begin{aligned}\gamma &= \sigma_\Gamma\sqrt{2\ln m_{3h}}\\ \beta &= \frac{\sigma_\Gamma}{\sqrt{2\ln m_{3h}}}\end{aligned} \tag{3.22}$$

m_{3h} denotes the number of global peak values during short term condition (3 hours).
We have already calculated the values of m_{3h}, σ_Γ and $F_{Z_{3h}/Hs,Tp}(z/h_s,t_p)$ in **task 4**
The distribution for the largest of the crest is narrow and we don't expect large exceedance values, see Fig. 3.8

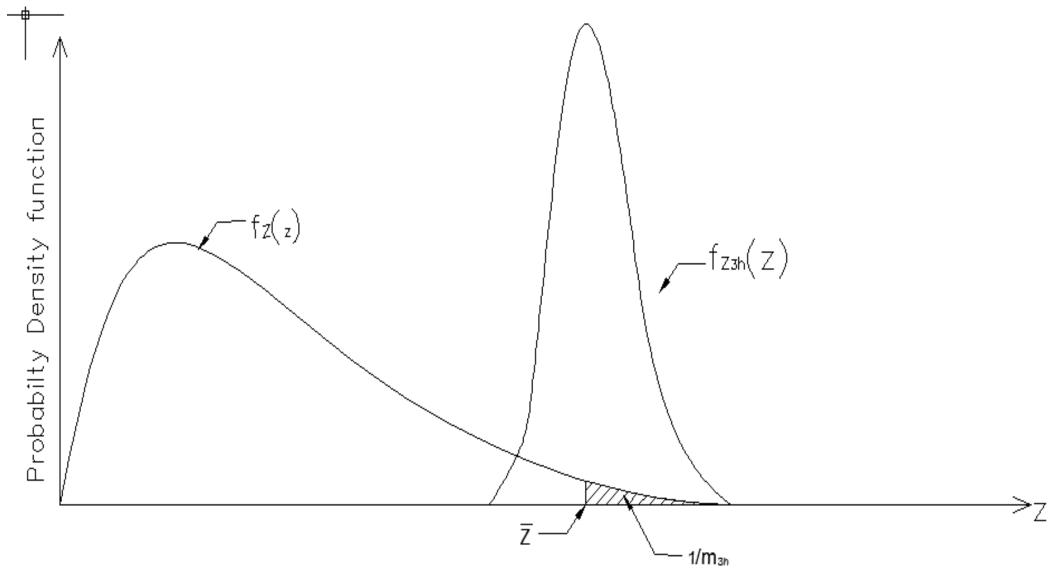


Fig. 3.8 Probability Density Distribution of the extreme value

Hence our short-term cumulative distribution of Z_{3h} for a given state is given by equation (3.21) and the long term distribution of Z_{3h} will be explained in section 3.2.2.

3.2.2 Long Term Description of Linear Response

In order to obtain an accurate estimate for the q-probability long term extreme value, we need to account for the effects of non-observed sea states. This can be done by fitting a joint probabilistic model to simultaneous observations of H_s and T_p. This is demonstrated by the following two tasks as continuation of the previous five tasks.

<u>TASK 6</u>: Calculate the long term distribution of the Z_{3h}

The long term distribution of the 3-hour maximum (or the marginal distribution of 3-hour maximum) is given by equation (3.6). It consists of the conditional distribution of 3-hour extreme value for a given sea state and the long term variation of the wave climate. The conditional part is already explained on the previous tasks particularly task 5 by equation (3.19). Now we will focus on the long term climate variation and the overall distribution of the 3-hour maximum.

The long term variation in the wave climate can be described by the joint probability density function for Hs and Tp. This joint function f (h_s,t_p), can be estimated from the joint values in a given scatter diagram. For an explanation and illustration let's take the data collected from Northern-North Sea from 1980-1983.

Table 3.1Joint frequency table H_s and T_p data Northern-North Sea, 1980 – 1983 (Haver & Nyhus, 1986)

H_{mo} \ T_p	< : 4.0	4.0 : 5.0	5.0 : 6.0	6.0 : 7.0	7.0 : 8.0	8.0 : 9.0	9.0 : 10.0	10.0 : 11.0	11.0 : 12.0	12.0 : 13.0	13.0 : 14.0	14.0 : 15.0	15.0 : 16.0	16.0 : 17.0	17.0 : 18.0	18.0 : 19.0	19.0 : 20.0	20.0 : <	SUM	AVERAGE
< 0.5	1	3	12	17	10	12	5	6	3	1	1								71	7.61
0.5- 1.0	16	68	121	133	96	91	78	38	24	8	2	1	1						677	7.33
1.0- 1.5	6	63	151	170	226	171	156	79	67	41	17	4	2	1					1154	8.07
1.5- 2.0		11	127	230	227	186	168	113	81	64	45	17	3	1	2		1	1	1277	8.63
2.0- 2.5		2	41	146	216	202	146	128	68	50	33	31	10	5	1	1	1	2	1083	9.12
2.5- 3.0			11	69	184	204	119	94	106	73	45	29	19	6	4	2		1	966	9.74
3.0- 3.5				22	92	207	120	102	81	71	47	33	19	6	3				803	10.18
3.5- 4.0				8	44	162	119	92	57	74	40	22	14	8	3	1			644	10.42
4.0- 4.5					16	103	114	75	60	43	18	18	10	5	5				467	10.51
4.5- 5.0				1	3	44	76	45	51	29	27	9	10	10	8	2			315	11.24
5.0- 5.5						18	60	69	50	23	13	10	5	4	4	1			257	11.11
5.5- 6.0					1	8	32	40	31	17	10	13	3	6	4	4			169	11.73
6.0- 6.5							6	28	21	22	6	10	2	4	2	2	2	1	106	12.42
6.5- 7.0							2	20	18	21	14	2	4						81	12.09
7.0- 7.5								3	9	15	13	3	1	1	1				46	12.80
7.5- 8.0									8	12	4	3	3						30	12.89
8.0- 8.5								3	5	11	4	5	3						31	12.86
8.5- 9.0									3	3	4	4	1						15	13.31
9.0- 9.5										1	4	2	3		1		1		12	14.85
9.5-10.0										3	1								4	13.10
10.0-10.5										1									1	12.96
10.5-11.0								1						1		1			3	15.09
SUM	23	147	463	796	1115	1408	1201	936	743	583	348	216	113	58	38	14	5	5	8212	
AVERAGE	.84	1.05	1.37	1.68	2.06	2.62	2.86	3.25	3.44	3.73	3.76	3.87	4.08	4.28	4.44	5.15	5.30	3.03		

Where: $f_{Hs,T_p}(h_s,t_p)$ can be calculated from

$$f_{Hs,T_p}(h_s,t_p) = f_{Hs}(h).f_{Tp/Hs}(t_p/h_s) \tag{3.22}$$

A joint probability model for a long term response analysis is given by Haver & Nyhus (1986). The joint modelling is based on product of probability distribution of significant wave height and the conditional distribution of T_p for given H_s. Haver and Nyhus have modeled the f (h_s) by log normal distribution for $h_s \le \eta$ and by a Weibull distribution for $h_s > \eta$ as shown in the following equations:

$$f_{Hs}(h_s) = \frac{1}{\sqrt{2\pi}\alpha h_s} \exp\left(\frac{(\ln h_s - \lambda)^2}{2\alpha^2}\right) \qquad ; \quad h_s \le \eta$$

(3.23)

$$= \frac{\beta}{\rho}\left(\frac{h_s}{\rho}\right)^{\beta-1} \exp\left(-\left(\frac{h_s}{\rho}\right)^{\beta}\right) \qquad ; \quad h_s > \eta$$

The parameters λ and α^2 are the mean and variance of the variable ln (H_s) respectively. The values of parameters are given in table 3.2.

Table 3.2 The values of the parameters in equation (3.23)

Parameters	λ	α^2	β	ρ	η
Value	0.836	0.376	1.547	2.822	3.27m

The conditional distribution of Tp for a given Hs is approximated by the log normal distribution and given as:

$$f_{Hs/Tp}(h_s / t_p) = \frac{1}{\sqrt{2\pi}\phi t_p} \exp\left(\frac{-(\ln t_p - \mu)^2}{2\phi^2}\right) \tag{3.24}$$

Where μ and ϕ^2 are the conditional mean and variance of $\ln(T_p)$ respectively. They can be calculated using the following functions which fitted to the point estimates for various significant wave heights.

$$\mu = 1.59 + 0.42\ln(h_s + 2) \tag{3.25}$$

$$\phi^2 = 0.005 + 0.085\exp\left(-0.13h_s^{1.34}\right)$$

By substituting the parameters on the joint model we can calculate the long term probability density distribution of each sea state, $f_{Hs,T_p}(h_s, t_p)$. For further details and analysis on the above equations and parameters please refer Haver & Nyhus (1986).

Now we have obtained the long term variation in the wave climate by equation (3.22) and we already shown that in task 5 how to calculate conditional distribution of the Z_{3h} in equation (3.20), and hence the long term CDF of the 3-hour largest peak value, Z_{3h} can be gained from equation (3.6).

Equation (3.6) can be calculated using MATLAB by representing the integral with summation. Equation (3.6) can be approximated with the following summation formula.

$$F_{Z3h}(z) = \sum_{hs}\sum_{tp} F_{Z_{3h}|Hs,Tp}(z \mid h_s, t_p) * f_{Hs,T_p}(h_s, t_p)\Delta h \Delta t \tag{3.26}$$

Based on the summation approximation given in equation (3.26), $F_{Z3h}(z)$ can be solved in MATLAB and the result of the cumulative distribution is shown in Fig.3.9.

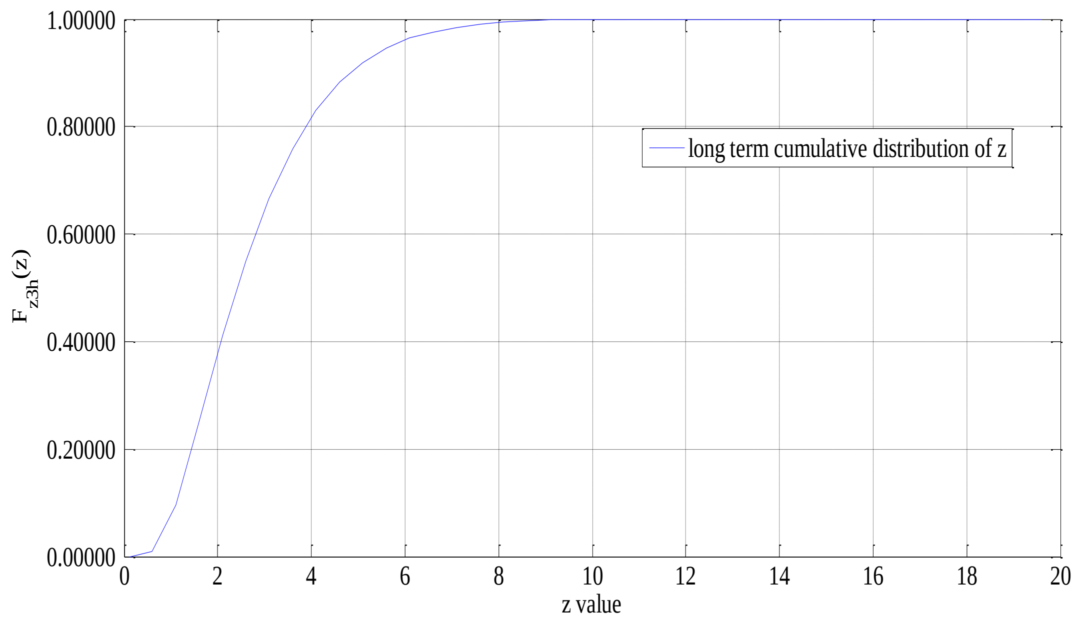


Fig. 3.9 Long term Cumulative distribution of Z

TASK 6: Finding q probability of exceedance of the extreme value.

In order to obtain the q-probability extreme value, we have to use equation (3.3) to find Z_q, but first let's say we are interested for $q = 10^{-2}$.

$$F_{Z3h}(Z_q) = 1 - \frac{q}{365 * \frac{24}{3}} \qquad \text{where } q = 10^{-2}$$

$$F_{Z3h}(Z_q) = \int\int_{HsTp} \left[F_{Z_{3h}/Hs,Tp}(z / h_s, t_p\right] f_{Hs,T_p}(h_s, t_p) dh_s dt_p$$

$$F_{Z3h}(Z_q) \quad = 1 - \frac{q}{365 * \frac{24}{3}} = \int\int_{HsTp} \left[F_{Z_{3h}/Hs,Tp}(z / h_s, t_p\right] f_{Hs,T_p}(h_s, t_p) dh_s dt_p$$

$$F_{Z3h}(Z_q) = 0.9999966$$

In order to obtain Z_q, we have to read the corresponding value of Zq for $F_{Z3h}(Z_q)$ in the cumulative distribution Fig. 3.10. However, this is not practical. It is too difficult to read a six digit value in the figure. It would be beneficial and easy to prepare a graph using values on the horizontal and vertical axis such that the graph appears to be close to straight line. Then it would be much easier to read values from the graph. The model used to determine the value on the axis would represent a mathematical model for cumulative distribution function of the measured data.

Let's assume the Gumbel cumulative distribution represents the distribution of the variable Z, because of its simplicity in reading. Gumbel distribution will be rearranged so that the x-axis will be the Z value if we plot it in Gumbel probability paper. If we had taken Weibull distribution it would be difficult to read the Z value because the x-axis is going to be ln(z).

$$F_{Z_{3h}}(z) = \exp\left\{-\exp\left[-\left(\frac{z-\gamma}{\beta}\right)\right]\right\}$$

$$\ln(F_{Z_{3h}}(z)) = -\exp\left[-\left(\frac{z-\gamma}{\beta}\right)\right]$$

$$-\ln(F_{Z_{3h}}(z)) = \exp\left[-\left(\frac{z-\gamma}{\beta}\right)\right]$$

$$\ln(-\ln(F_{Z_{3h}}(z))) = -\left(\frac{z-\gamma}{\beta}\right)$$

$$-\ln(-\ln(F_{Z_{3h}}(z))) = \frac{z-\gamma}{\beta}$$

$$-\ln(-\ln(F_{Z_{3h}}(z))) = \frac{z}{\beta}-\frac{\gamma}{\beta}$$

This is a linear function with parameters β and γ. Its graph will be straight line z value on the x-axis and $-\ln(-\ln(F_{Z_{3h}}(z)))$ as y value on the y-axis. From the MATAB analysis the cumulative curve shown in Fig 3.10 will be modeled to straight line as shown in Fig.3.11.

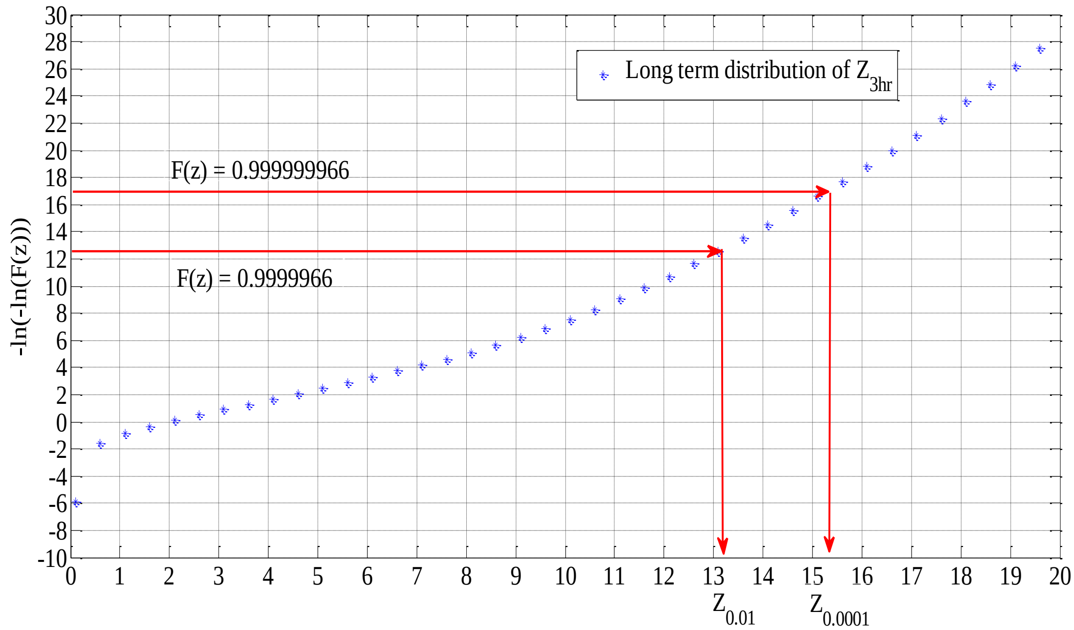


Fig. 3.10 Long term cumulative distribution of Z_{3h} on Gumbel probability paper

For the value of $F_{Z3h}(Z_{100}) = 0.9999966$ the value of $-\ln(-\ln(F_{Z_{3h}}(z)))$ = 12.59733 ≈12.6. If we read the corresponding value on figure 3.11, Z_{100} will be ≈ 13.2

Then the 10^{-2} probability of exceedance extreme value or Z_{100} is ≈ 13.2

Similarly for Accidental Limit State (ALS) design i.e. q = 10^{-4} $F_{Z3h}(Z_{10000}) = 0.999999966,$ the y-axis value on the Gumbel probability paper is $-\ln(-\ln(F_{Z_{3h}}(z)))$ =17.19 and its corresponding value (Z_{10000}) on the graph is ≈ 15.4

3.3 Non-Linear System

In estimating a wave dominated load corresponding to an annual exceedance probability of q for non-linear response problems, frequency domain solutions are no longer available, either a step-by-step time domain simulation or model test might be useful.
As we have seen on section 3.2 for estimating the load/response for linear system, the long term extreme value distribution consist the conditional short term extreme value distribution and the joint probability density function. Those methods used in obtaining the conditional short -term distribution in linear system are no longer useful in nonlinear mechanical system. This conditional short term extreme value distribution has to be modified or calculated using another methods.
In non-linear mechanical systems, specifically structures where the motions are so large that the stiffness property of the system must be updated during the analysis. This can be analyzed by solving equation of motion Eq. (3.27) in the time domain, but due to the non-linearity, the stiffness properties (and possibly damping properties) must be updated for each time step (Haver, 2013).

$$m\ddot{x}(t) + c(\dot{x}, x)\dot{x}(t) + k(\dot{x}, x)x(t) = F(t) \qquad (3.27)$$

Note that for linear mechanical system, $c(\dot{x}, x)\dot{x}(t) = c$ and $k(\dot{x}, x)x(t) = k$.
In order to solve the nonlinear mechanical system either the time domain simulation or model test has to be performed. Time simulation assessment for all sea state long term analysis, and model test for calibration of time domain analysis can be for obtaining characteristic loads. These two things will be discussed on the coming sub topics. Finally the importance of environmental contour line approach in relation to time domain analysis and model test will be presented.

3.3.1 Time domain simulation

By a time domain analysis we basically mean solving the equation of motion in the time domain by some step by step procedure. As an introductory illustration for time domain solution of equation of motion can be seen in the compendium by Haver (2013). This illustration gives a step by step solution for one degree of freedom motion. However, here we will discuss its application in the all sea state long term analysis for nonlinear response problem.
As we have seen on the liner mechanical system, an all sea state long term analysis is used to determine the long term extreme value; similarly we can use this method for non-linear system with the following conditions and modifications. The conditional short term cumulative probability in equation 3.2 has to be modified. In all sea state long term analysis of linear response, both the sea surface process and the response process under consideration can be modelled by Gaussian process. An important condition is that the response quantity must be of linear nature, i.e. the structural loading must be linearly related to the wave, and the mechanical system must be of a linear nature both regarding damping and stiffness. For a number of practical applications these conditions will be difficult to fulfill. For example let's see the complications that make it difficult in fixed platform and floating structures.

In fixed platforms, drag force dominates the load and it is proportional to the horizontal particle speed normal to the structure. This horizontal particle speed reaches maximum at the wave crest, and it is

underestimated by the Gaussian assumption, it will be necessary to model the surface as a non-Gaussian process. In floating structures the source of non-linearity could be global loading to ships due to slamming load, 2nd order loading and difference frequency type, none linear damping , nonlinear stiffness, etc.
However, the above complications will not destroy the validity of the integral equation given in equation 3.6. In global maximum approach instead of using Rayleigh distributed global maximum, Weibull distribution may be used. In 3-hour extreme value approach, still the Gumbel distribution can be used but the expression used for calculating the parameters γ and β have to be changed.

Gumbel parameters γ and β can be obtained using time domain simulation. For problems where numerical simulations of a sufficient accuracy can be carried out, the easiest way for doing a long term response analysis is to carry out k 3-hour simulations for large number of sea states (Haver, 2013). Provided the simulations carried out for sufficient number of sea state, one may establish response surface for the parameters γ and β. After obtaining these two parameters a long term distribution for the 3hour maximum value can be obtained by equation 3.6.

Baarholm et al. (2010) set a four step approach to investigate the characteristic load for nonlinear response using time-domain simulation. The investigation was motivated after a comprehensive analysis program showed that a need for verification of the percentile applied in 2007 for model test of Troll A platform for estimation of the q probability response. Troll A platform is a concrete based platform installed in 1995 in Northern North Sea about 300m depth. The model test was required in 2007 after the platform deck weight was needed to increase by 20,000 tons.

The verification process used by Baarholm et al. (2010) involves the following steps:

1) Executing a full long-term analysis. This requires that a sufficient number of 3-h time domain simulations are carried out for a sufficient number of different sea states. It is important that the numerical calculations reflect the underlying physics with sufficient accuracy.

2) Screening analysis to determine the worst sea state in view of the problem under consideration along the 10^{-2}- probability contour.

3) Establish the 3-h response extreme value distribution for this sea state.

4) The target percentile level is then obtained by the percentile giving the same result as the long-term analysis.

3.3.2 Model test analysis

NORSOK N-003 (2007) explains the main objective of model tests in e.g. wave basin or wind tunnel would be to confirm no important feature has been overlooked for temporary and in-place conditions. Model test analysis can also be used in estimating a wave dominated load corresponding to an annual exceedance probability of q for non-linear response problems. It is hard to imagine that one will use model tests for obtaining short term distributions for all sea states involved in a full long term analysis are used as long term method. The model tests will therefore typically be limited to some few sea states of

interest either for tuning purposes or load prediction purposes (Haver, 2013). For very complicated and nonlinear response problems, the case like the floating structures, numerical analysis might not give good results and possibly impractical. Under this circumstance, model test is the only option. The model test can help in obtaining the corresponding load/response, firstly by providing additional data for calibration of a numerical model, which are done in the all sea state analysis, secondly by providing model test data to come up with a characteristic load.
The detailed procedures for performing model tests, including the scaling of the model, etc. will not be covered here. Instead we merely focus on how one can estimate load corresponding to annual exceedance probability of q based on model test results.

We start by selecting some few sea states along the q-probability contour line for h_s and t_p. A next step would be to carry out 3-4 tests of 3-hour (full scale) duration. Based on these tests, one should select which sea state that is the worst sea state.

As the worst sea state is selected, the aim is to estimate the distribution function of the 3-hour extreme value for this sea state. When this distribution function is established, a proper first estimate of the q-probability value is the 90 percentile of this distribution for $q=10^{-2}$ and 95 percentile for $q = 10^{-4}$.The above mentioned approach is the environmental contour lines approach and it is discussed in section3.3.3.

3.3.3 Environmental contour lines approach

For very complicated response problems, particularly for floating structures, time domain simulation analysis with sufficient accuracy is not possible. At least there should be a model test for proper calibration. In order to do this proper calibration costly model tests and relatively cheap time domain simulation will be carried out to establish response surface for the parameters γ and β. To perform the full long term assessment will be costly and time consuming. For such situations environmental contour line approach will be convenient option

Environmental contour line approach is mainly suggested as simplified method for predicting reasonable estimate of corresponding value to a given return period. This method makes it possible to estimate long term extremes without carrying out a full long term analysis. Here the summary of the basic steps used in environmental contour lines approach will be given. For more information on this approach reference is made to Haver & Kleive (2004).

Using the environmental contour line approach, a reasonable estimate for the q-probability value (i.e. the response value corresponding to an annual exceedance probability of q) can be obtained by the following steps (Haver , 2006).

1. Establish the q-probability contour or surface for the involved metocean characteristics, e.g. significant wave height and spectral peak period.
2. Identify the most unfavorable metocean condition along the q-probability contour/surface.
3. Establish the distribution function for the 3-hour maximum response for the unfavorable metocean condition.

4. An estimate for the q-probability response value is now obtained by the α-quintile of this extreme value distribution. If, say two metocean characteristics are included, e.g. significant wave height and spectral peak period, an adequate value of α will typically be around. 0.90.

4. ASSESSMENT OF THE DISTRIBUTION FUNCTION FOR 3-HOUR MAXIMUM IMPACT PRESSURE

Considering impact loads from breaking waves, NORSOK N-003 (2007) defines the characteristic values of the impacts loads by specifying annual exceedance probabilities for the different design limit states. Impact loads from breaking waves fall in the category of environmental actions. For environmental actions the characteristic load values are defined by annual exceedance probability of 10^{-2} for ultimate limit state (ULS) and 10^{-4} for accidental limit state (ALS).

Accordingly, the characteristic loads due to breaking waves are the loads with a return period of 100 years (ULS) and 10000 years (ALS). In order to determine the impact load from breaking wave two approaches are introduced, a recommended practice document from DNV (DNV-RP-C205, 2007) and a statistical analysis of model test results. The first section of this chapter will focus on the recommendation and the second part on the model test analysis.

4.1 Estimation of Impact Load or Slamming Force due to breaking waves According to DNV.

NORSOK standard and DNV recommended Practice are the most useful documents in estimating the characteristics impact loads due breaking waves. NORSOK N-003 defines the characteristics load using annual exceedance probability of q i.e. 10^{-2} for ULS and 10^{-4} for ALS. It recommends that breaking waves that cause shock pressure on vertical surfaces must be considered in design. In estimating the shock pressure NORSOK recommends to refer DNV-RP-C205. Hence we will see here how to estimating the impact load (slamming pressure) due to breaking waves according to DNV-RP-C205.

DNV-RP-C205, gives guidance for modelling, analysis and prediction of environmental conditions as well guidance for calculating environmental load acting on structures. Accordingly DNV also shows how to estimate the slamming pressure due to breaking waves. This slamming pressure represents the average pressure on the strip of the platform column. The average slamming pressure, *Ps, i*s given by equation 4.1

$$p_s = \frac{1}{2}\rho C_{pa} v^2 \qquad (4.1)$$

Where p_s is the average slamming pressure

ρ is the mass density of the fluid

C_{pa} is the coefficient of average slamming pressure

v is the relative horizontal velocity between water and column

4.1.1 Slamming Coefficient

Slamming coefficient C_{pa} depends on the area that is subjected to the slamming force and the slenderness of the column. A column structure can be classified as slender or not based on the ratio of column diameter (D) to the wave length (λ). If this ratio is less than 0.2, it is considered to be slender. According to DNV-RP-C205 (2007) the coefficient C_{pa} the slamming coefficient for a smooth circular cylinder should not be smaller than 5:15. If the condition for slenderness is not fulfilled the coefficient should not be smaller than 2π.

4.1.2 Relative Impact Horizontal Velocity

Relative impact horizontal velocity, v according to DNV-RP-C205 (2007) impact horizontal velocity is the relative velocity of the water particle to the column structure, and can be calculated as1.2 times the phase velocity of the most probable highest breaking wave in n years. It is given in equation (4.2).

$$v = 1.2 * C_B^{(n)} \tag{4.2}$$

4.1.3 Phase Velocity

Phase velocity, $C_B^{(n)}$ of the most probable highest breaking wave in n years is calculated from the period corresponding to the most probable highest breaking wave in n years $T_B^{(n)}$, by the following equation (4.3):

$$C_B^{(n)} = \frac{g}{2\pi} T_B^{(n)} \tag{4.3}$$

Where: g is the gravitational acceleration.

4.1.4 Highest Breaking Wave Period

Highest breaking wave period, $T_B^{(n)}$ can be calculated from the breaking wave criterion i.e. the different global limiting steepness , ε (Stokes, 1847).

$$T_B^{(n)} = \sqrt{\frac{H_B^{(n)}}{g\varepsilon}} \tag{4.4}$$

Where: $H_B^{(n)}$ is the height of the most probable breaking waves

4.1.5 Highest Breaking wave height

Where $H_B^{(n)}$ is the height of the most probable breaking wave in n years and can be calculated from the most probable significant wave in n years, $H_s^{(n)}$.

$$H_B^{(n)} = 1.4 * H_s^{(n)} \tag{4.5}$$

The most probable largest significant wave height in n years, $H_s^{(n)}$, has to be determined by available metocean data. According to NORSOK N-003 (2007), characteristic impact pressures are defined as the impacts with a return period of 100 and 10,000 years, respectively. This is corresponding to choosing n equal to 100 and 10,000 years respectively.

4.2 Assessment of Uncertainties on DNV method

In estimating the impact load due to breaking wave according to DNV recommendation, it has some uncertainties that can affect the result. In calculating the slamming coefficient, C_{pa} the document recommends for a smooth circular cylinder should not be smaller than 5:15 and if this condition for slenderness is not fulfilled the coefficient should not be smaller than 2π. But this only explains the lower limit it doesn't indicate what should be the approximate value. Usually it is taken to be 2π but it depends on the configuration of area exposed to the slamming load.

The phase velocity is calculated assuming the wave is regular sinusoidal but in reality the sea wave is irregular and this will definitely change the result. Another assumption is when the wave hits the column it is assumed that the wave comes directly to the column at 90^0 but due to the heave and yaw motion of the platform (if it is floating) the angle can divert slightly from 90^0. This can also affect the impact load obtained by the DNV method.

5. PREDICTION OF IMPACT LOADS FROM MODEL TEST DATA

In addition to the recommended practice from DNV-CP-205 (2007), marginal distribution of the 3-hour maximum pressure can be estimated using model test. On this project we will see a model test analysis using the data obtained from the Heidrun TLP. It was performed at MARINETEK, Trondheim for STATOIL. The test was done using slamming sensors fixed on the modeled column of the platform. Each sensor measures the impact pressure due to breaking waves on the modeled column of the TLP.

From the results obtained by model test, the distribution function of the 3-hour maximum impact load due to breaking waves can be established using the all sea state analysis and environmental contour line methods as explained in chapter 3. In establishing the short term extreme distribution two approaches are used; the first one is indirectly by considering all impacts above a certain threshold and the second one is directly from observed 3-hour extremes.

Data used in this project are obtained from a model test done at MARINTEK for Statoil, but due to confidentiality, the data are normalized. The "pressure" values used here are scaled using the ratio of pressure measured over scaling factor ($\hat{P}$)

$$Pscaled \quad = \quad \frac{P}{\hat{P}}$$

Where P is the maximum pressure of an impact event.
$\hat{P}$ is a selected scaling value.

In the Appendix B Table B1, the full data merely Pscaled and time of impact are given and all the analysis done here are using this scaled pressure value. The actual pressure value can be obtained just by multiplying the result pressure by the scaling factor.

Pressure was measured by sensors representing a full scale area of 9 x 9 m^2. Sensors were installed at eight rows and seven columns. Positions of sensors are defined by row number and column number. Row number one is close to deck level, while row number eight is close to still water surface.

There are 109 realizations, each test represents one realization and lasts for 10800 seconds (3hours full scale duration). As a sample of the data obtained from Statoil, table 5.1 is shown here. In table 5.1 the first column shows that the test or run number, and the last column gives the time of an impact occurred. In some cases an impact event is merely observed in one sensor, while for others the impact is a severity that it covers in several sensors**.**

Here we will see the critical **sensor (4,4)** which is located at row 4 and column 4, at the center of the panels. It is expected that the maximum impact pressure hits sensor (4,4). All the impact pressure that hits this sensor in each test is recorded. The measured readings will be analyzed and a representing probability distribution model will be established.

Fig. 5.1 Heidrun TLP (Norsk Oljemuseum, 2010)

Table 5.1 Sample of the collected data from MARINTEK, full data is in Appendix B table B1

Run no.	Row	Col	P [scaled]	time [sec]
30020	3	4	2.375116	5522.394
30020	4	4	0.93116	5522.394
30020	3	5	2.952487	5522.394
30020	4	5	0.984187	5522.395
30020	3	6	0.872462	5522.405
30020	3	3	1.303997	6319.56
30020	3	4	1.01961	6319.564
30020	4	2	0.733999	7179.183
30031	4	4	0.64471	2288.546
30031	2	5	1.225942	3700.39
"				"
"				"

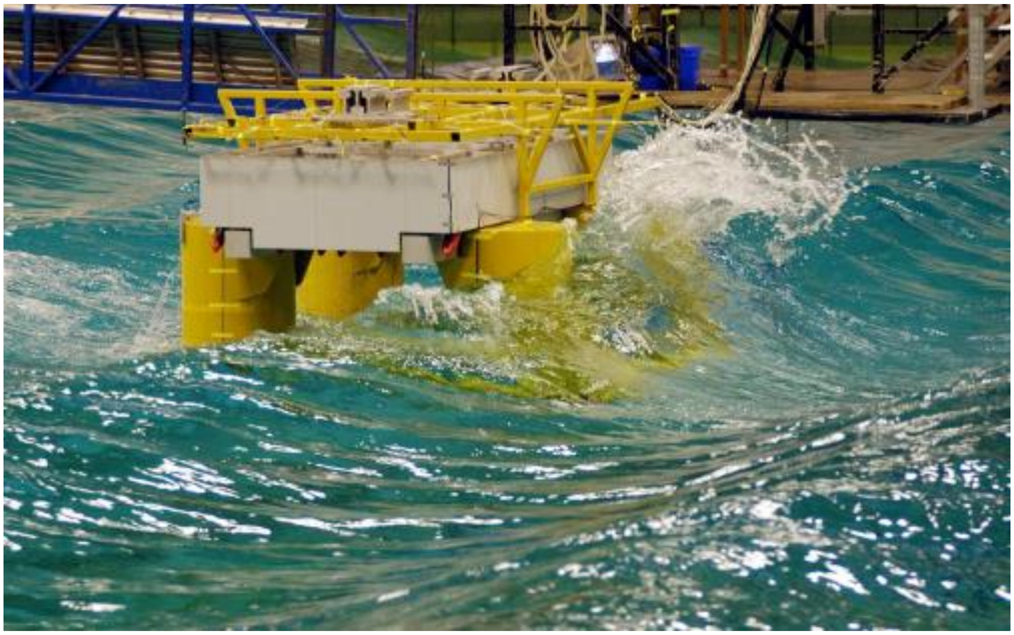


Fig. 5.2 Heidrun TLP model test (Statoil, 2003)

5.1 Indirect Approach

Our main target in model test analysis whether we use direct or indirect approach, is to calculate the marginal distribution of the 3-hour maximum pressure. After estimating this distribution' we can easily calculate different percentile of the pressure distribution.

In the indirect approach, a threshold is introduced so as to exclude events that would not represent impacts caused by breaking waves. Only impact pressure larger than 0.529 (scaled pressure value) are considered as slamming events. The sensors that were installed on the column surface are adjusted to record the pressure above this threshold value.

The 3-hour maximum pressure distribution using indirect approach can be generalized by equation (5.1). This distribution is obtained from the product of conditional distribution of the 3-hour maximum pressure and the long variation of the number of readings per test. Both these terms will be discussed on the following subchapters in detail.

$$F_{P_{3h}}(y) = \sum_{n=0}^{\infty} \underbrace{\left(F_{Y|N}(y \mid n)\right)^{n}}_{short term\ \text{variation}} * \underbrace{P_N(n)}_{long\ \text{variation} of\ n} \tag{5.1}$$

5.1.1 Conditional Distribution of the 3-hour maximum Impact Load

In order to obtain the conditional distribution of the maximum pressure within 3-hours duration, 109 tests or runs were performed to observe the pressure impact on the column sensor. As previously mentioned we are analyzing the impact pressure on sensor (4,4). The readings can be seen on Appendix B
From reading on table 5.3 i,e sample Table B on Appendix B , the graph on Fig. 5.3can be plotted

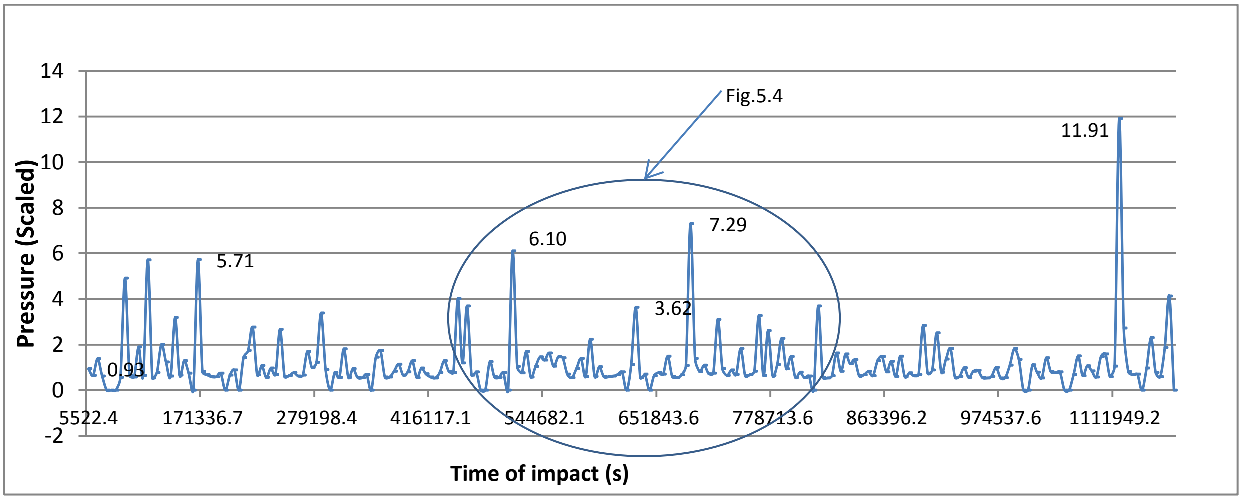


Fig. 5.3 Pressure reading on sensor (4,4) for all 109 tests or 327hrs.

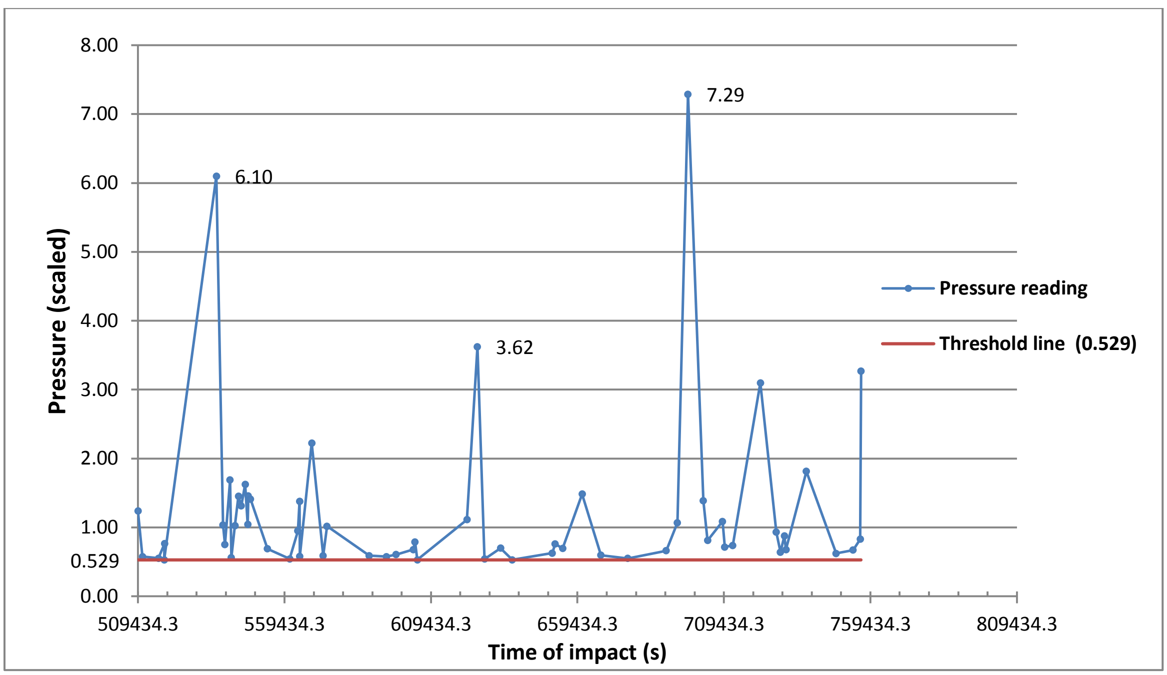


Fig. 5.4 Pressure reading with the threshold value of 0.529

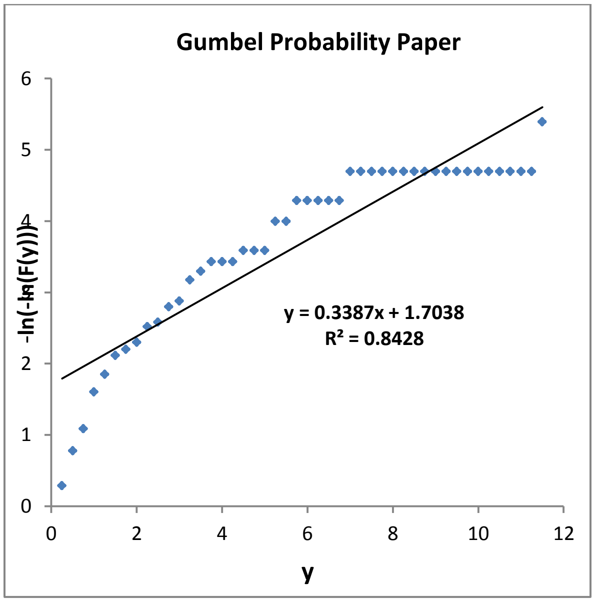
As it is indicated in Fig. 5.4 the readings above threshold value of 0.529 is selected and the distribution of these pressure values is analyzed. If we recall to chapter 3 section 3.2.1 task 4, the probability distribution of the global maximum was approximated by the Rayleigh distribution, equation 3.11. Here also we can find a probability distribution that fits the given pressure data. Three different possible probability fitting were plotted on their probability scales, then based on the coefficient of determination (R^2) the best fitted probability graph is selected.

Coefficient of determination is a measure of how well the regression line represents the data. If the regression line passes exactly through every point on the scatter plot, it would be able to explain all the variation (Mathbits, 2014). For example, if $R^2 = 0.950$, which means that 95% of the total variation in y can be explained by the linear relationship between x and y. The other 5% of the total variation in y remains unexplained.

The three probability distributions are Weibull, Gumbel and Pareto. As it is shown on Fig.5.5 and Fig. 5.6 the Weibull probability distribution fits better than the other two distributions. If coefficient of determination is observed Weibull has ~ 0.97 while Gumbel and Pareto distributions have 0.84 and 0.96 respectively. From this analysis Weibull cumulative probability distribution represents the pressure distribution better than the other distributions. Weibull cumulative probability distribution is given by equation (5.2)

$$F_{y|n}(y) = 1 - \exp\left\{-\left(\frac{y}{\alpha}\right)^{\beta}\right\} \tag{5.2}$$

Where y is the impact pressure measured by the sensors.
α and β are scale and shape parameter respectively.



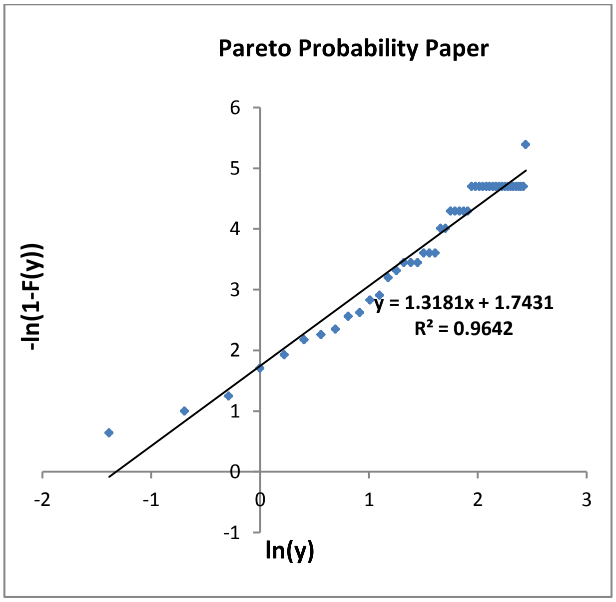


Fig. 5.5 Distribution fittings and Empirical data plotted on Gumbel and Pareto Probability Papers.

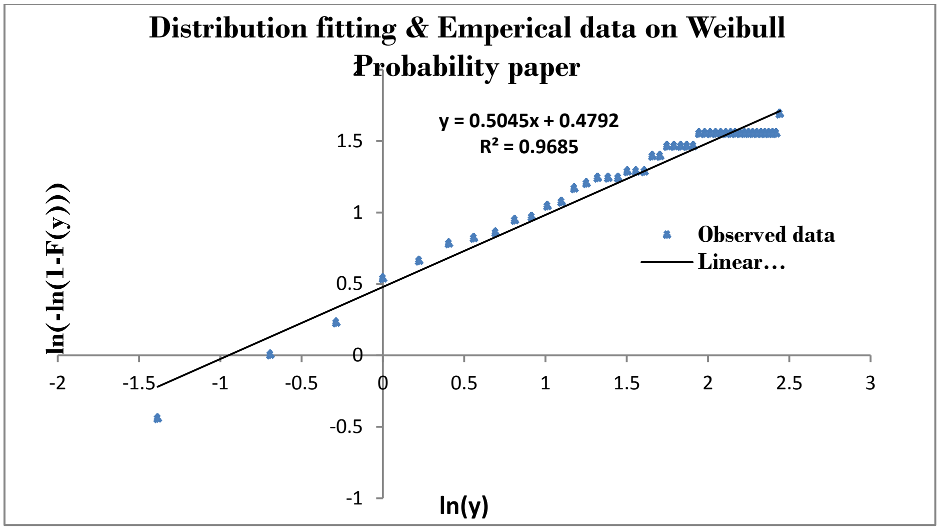


Fig. 5.6 Distribution fitting and Empirical data plotted on Weibull Probability Paper.

The parameters α and β in Weibull probability distribution can be obtained using the Least Square Method. The detail calculation of the Least Square Method is given in Appendix A, Table A2. Hence α=0.379647 and β = 0.5050 by substituting α and β on equation (5.2), we will obtain equation (5.2')

$$F_{p|n}(y) = 1 - \exp\left\{ -\left(\frac{y}{0.379647} \right)^{0.5050} \right\} \qquad (5.2')$$

5.1.2 Probability Density Distribution of number of hits (reading) per test

In estimating the marginal distribution of 3-hour maximum pressure, after investigating the short term distribution, it is important to estimate the probability distribution of the number of hits per run. Those two parts are the main cores of equation (5.1).

As it has been mentioned earlier a threshold was introduced to avoid events that would not represent impacts caused by breaking waves. The impacts which are higher than the threshold value (0.529) are shown on Fig. 5.2 and 5.3. The number of those readings or hits in each test is varies from zero, no reading at all to 7 readings/hits. The number hits in each test is collected and shown on Table 5.2. For the detail observation of the number of readings in each sensor and each test please refer Appendix B

Table 5.2 Probability distribution for number of readings per test/run vs theoretical Poisson distribution

Number of reading/run (n)	Number of reading (n) in all tests for sensor(4,4)	Probability of n (n / N)	Theoretical Poisson dist. $P(n) = \frac{e^{-\lambda t}(\lambda^* t)^n}{n!}$
0	20	0.1835	0.1341
1	31	0.2844	0.2707
2	22	0.2018	0.2694
3	12	0.1109	0.1813
4	16	0.1468	0.0911
5	5	0.0459	0.0366
6	2	0.0183	0.0123
7	1	0.0092	0.0035
	N = Σn = 109		

λ * = the rate of events occurring per unit time, in our case the average number of hits per test. We have 109 tests/runs and a total reading of 219. Therefore, $\lambda = 219/109 = 2.00917 \approx 2$.
From table 5.1 the probability density distribution based on the number of hits recorded on each run and the theoretical Poisson distribution can be plotted as shown in Fig.5.7. The bar marked by red color represents the distribution based on actual data and green one represents the empirical Poisson distribution. The empirical distribution doesn't fit exactly to the observed data distribution but if can fairly represent the observed data distribution. The empirical distribution represents the variation probability in the 3-hour maximum marginal distribution (5.1) and is given by equation (5.3).

$$P(n) = \frac{e^{-\lambda t}(\lambda\, t)^n}{n!} \tag{5.3}$$

Where n = the number of hits/reading above threshold per test/run, it ranges from 0 to infinity
λ = the average number (rate) of hits/reading per test/run , here $\lambda \approx 2$
t = the time interval, here t =3hrs

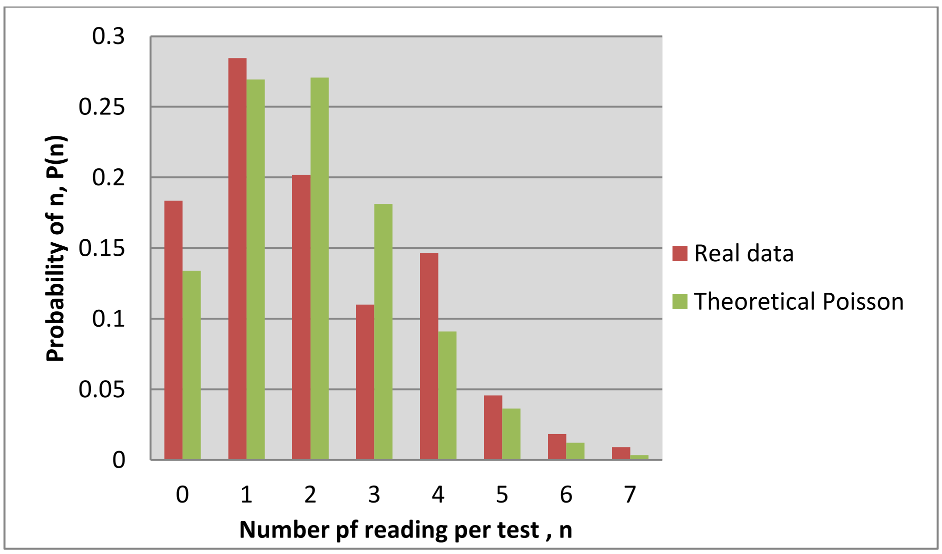


Fig. 5.7 Probability distribution of the observed number of data and the theoretical Poisson distribution

5.1.3 Marginal Distribution of the 3-hour maximum impact pressure

As it is mentioned in the beginning of this chapter the main target is to establish the marginal distribution of the 3-hour maximum pressure. This can be calculated using equation (5.1). The two parts of this equation are the short term (conditional distribution of pressure for a given number of hits) and the probability of the number of hits. The first part is already calculated on section 5.1.1 and the second part is calculated in the previous section, 5.1.2

By inserting the values equation (5.3) and (5.4) in equation (5.1) , we can get the cumulative distribution values of the 3-hour maximum ($F_{P_{3h}}(y)$).This is given by equation (5.1')

$$F_{P_{3h}}(y) = \sum_{n=0}^{\infty} \underbrace{\left(1 - \exp\left\{-\left(\frac{y}{0.379647}\right)^{0.5050}\right\}\right)^n}_{short\,term\,\text{variation}} * \underbrace{\frac{e^{-2}(2)^n}{n!}}_{long\,\text{variation}\,of\,n} \tag{5.1'}$$

Using MATLAB the values of $F_{P_{3h}}(y)$ are calculated and when these values are plotted on the Weibull probability paper, it almost fits to straight to straight line as shown in Fig.5.6. This shows that Weibull cumulative distribution fits best for the data calculated using equation (5.1').

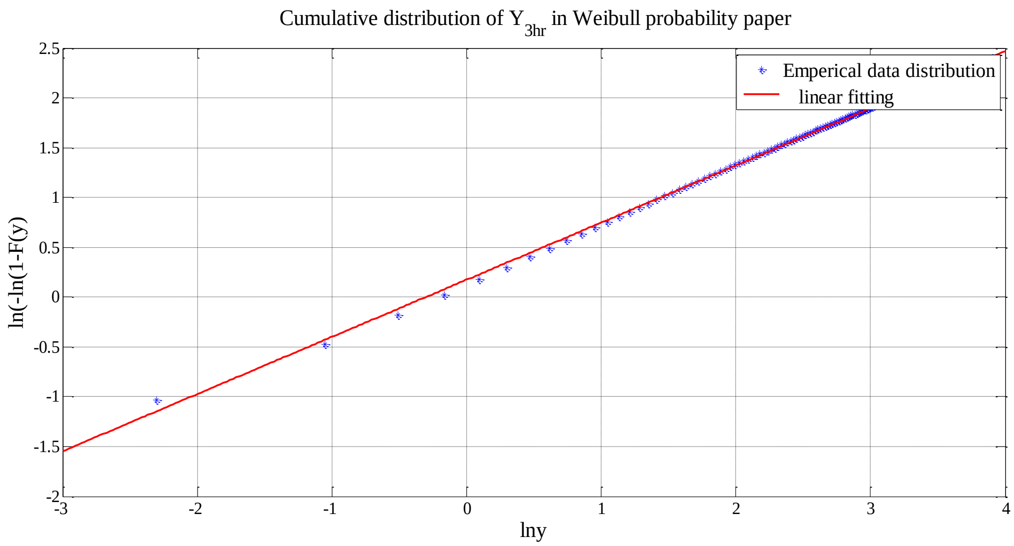


Fig. 5.8 Empirical data distribution and fitting line on Weibull probability paper.

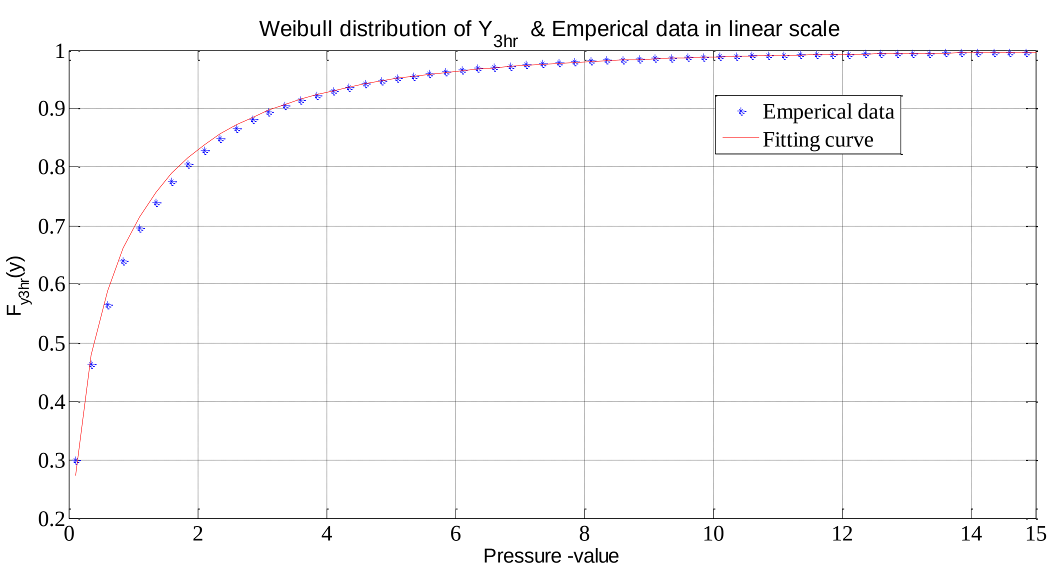


Fig. 5.9 Cumulative Distribution fitting of Y_{3hr} in linear scale (Indirect Approach)

Finally when the marginal distribution of 3-hour maximum pressure is established, different α percentile can be calculated from Fig 5.9. The percentiles for α = 0.5, 09, 0.95 and 0.99 is given in Table 5.2. These values will be used later for comparing with the values obtained in section 5.2 in the results and discussion part, chapter 6.

Table 5.3 Percentile values pressure distribution from the marginal distribution of 3-hour maximum pressure

F(y)	y	$P_{threshold}$	P_{scaled}
0.5	0.3896105	0.5209	0.910510501
0.9	3.1580522	0.5209	3.67895224
0.95	4.9960099	0.5209	5.516909935
0.99	10.571041	0.5209	11.0919411

5.2. Directly from observed 3-hour extremes approach

On this approach all the extreme values from each test or sea state is collected and the distribution of these collected extremes values will be investigated. These data are checked to fit for different probability distribution functions. The cumulative distribution that fits best for the given data will be selected and analyzed to find the marginal distribution of 3-hour maximum pressure. In the same procedure as the indirect approach, the α percentiles will be calculated.

The extreme value of each run/test from the model test is shown in Appendix A. Based on this data different probability distribution will be checked. There are 20 runs which have extreme value less than P_{scaled} = 0.529 and 89runs more than 0.529.

From the Table A1 on Appendix A Weibull, Gumbel, Frechet and Pareto cumulative probability curve can be plotted. As it is shown in Fig. 6.1 Frechet and Pareto distribution fits better than the other graphs, but Frechet is the best from the four possible fittings.

As it is shown in Fig. 5.10 Frechet probability distribution fits best to the observed data , it has coefficient of determination (R^2)= 0.9783 closest to one. The cumulative probability distribution of Frechet represents the distribution of the largest 3-hour extreme value. The cumulative distribution function of Frechet distribution is given by:

$$F_X(x) = \exp\left[-\left(\frac{\lambda}{x}\right)^k\right] \qquad (5.4)$$

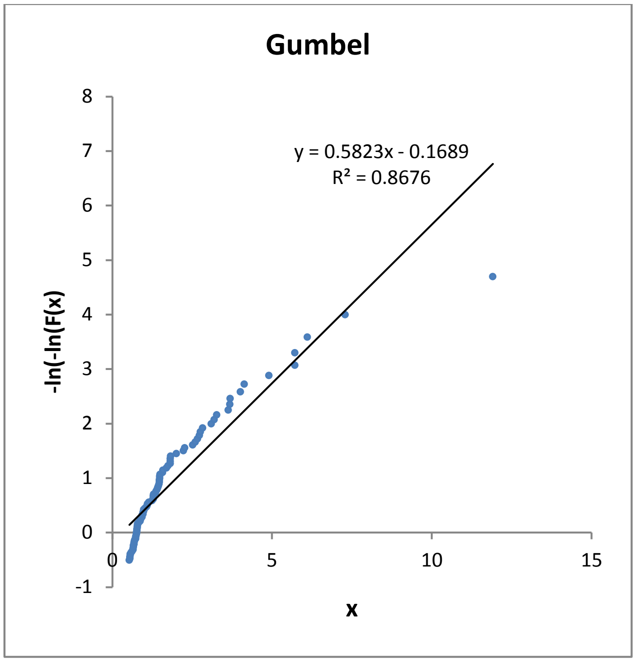
Where λ and k are the distribution parameters. Here we will estimate the Frechet parameters using a Least Square Method.

From least square method

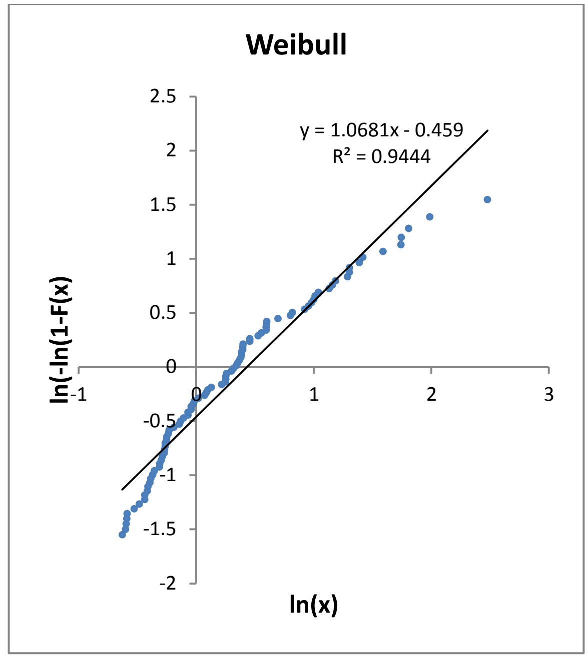
λ = 0.7795

k = 1.5681

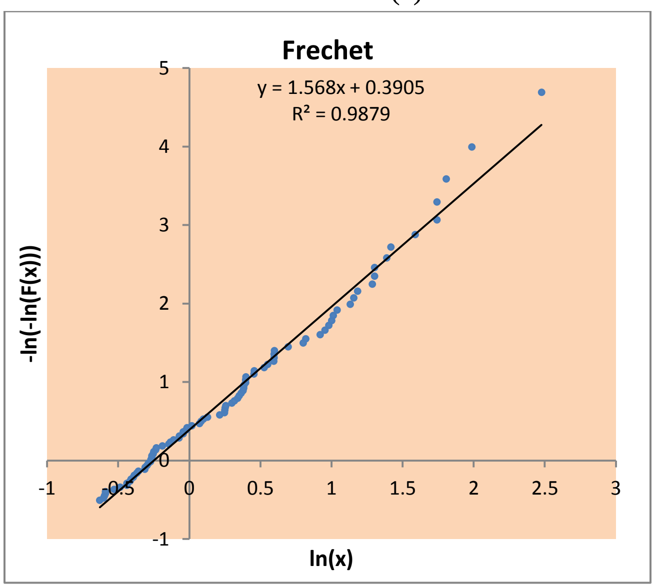
Benjamin & Cornell (1970) indicate that for parameter K < 2, the variance is unbounded and this shows that the distribution is **fat tailed** i.e. where the probability density f(x) goes to zero only very slowly as $x \to \infty$. This indicates that there is high chance of deviation from the mean value. It is further discussed in the uncertainties of Model test analysis in section 5.3.



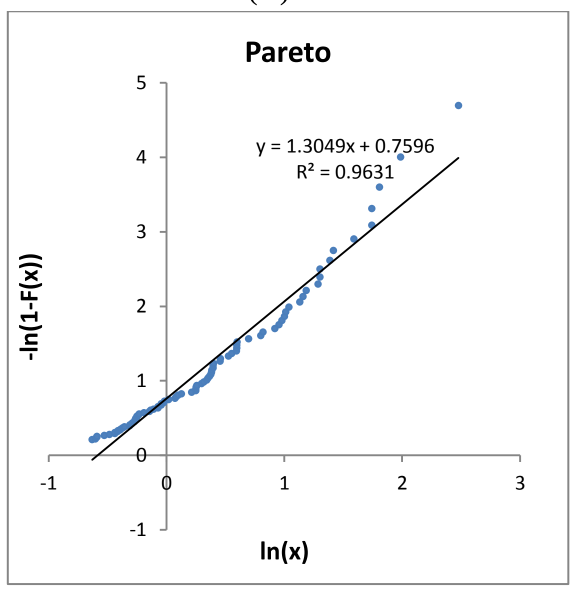
(a)



(b)



(c)



(d)

Fig. 5.10 Distribution fitting of the largest slamming on (a) Gumbel (b) Weibull (c) Frechet and (d) Pareto probability papers

Hence, the Frechet distribution fitting and empirical data can be plotted in linear scale as shown in Fig.5.11.

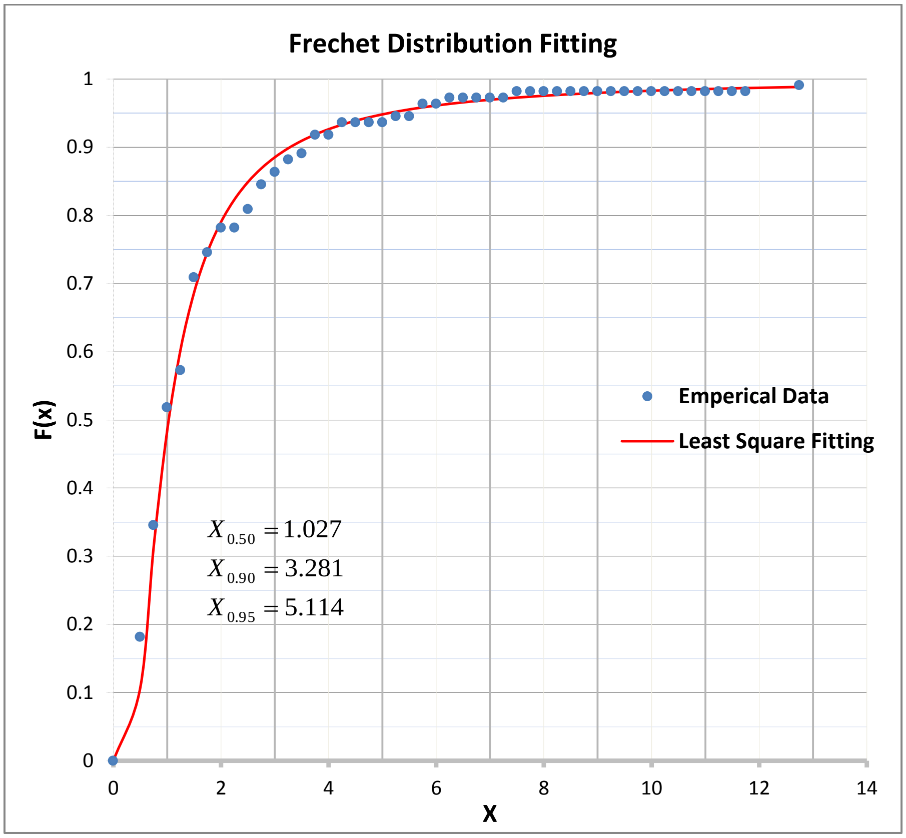


Fig. 5.11 Cumulative distribution fitting to the largest slamming load (Direct Approach)

Based on Fig. 5.11 we can estimate the α percentiles of the distribution. For α =0.5 , α = 0.90 and α = 0.95 the scaled pressure is estimated. The result is shown on Table 5.4.

Table 5.4 The percentile values from Frechet probability paper

F(x)	x	P_{scaled}
0.5	1.027	1.027
0.9	3.281	3.281
0.95	5.114	5.114

5.3 Uncertainties in the model test analysis

There are uncertainties related to estimating the impact load due to breaking waves using model test approach. The main uncertainties will be discussed below.

During model test, the wave impact loads were detected by panels/transducers. The wave impacts were recorded as the responses of the panel and not as the explicit external forces. When slamming occurs, the panel of the transducer will shrink or grow as reaction to the pressure or suction force of the slam. This dynamic characteristic can lead to significant amplification if the sensors' own natural period are close to the excitation period. It is usually tried to reduce this by designing the panels to have high natural frequency but there is still dynamic effects. These effects in the slamming sensors if not removed from all sensors, it can affect the results. On this report the removal of this dynamic effect is not done due to limited time and scope of the report. For more detail and analysis on the method to estimate the impact load Lehn (2003) can be referred.

In the direct approach of estimating the marginal 3-hour extreme distribution, a Frechet distribution is estimated to be the best fit but Frechet distribution follows fat-tailed distribution. This means that there is variability in the value. The following note is obtained from Benjamin & Cornell (1970.

The variance of Frechet distribution can be found as:

$$\text{Variance} = \lambda^2\left[\Gamma\left(1-\frac{2}{k}\right)-\Gamma^2\left(1-\frac{1}{k}\right)\right] \quad \text{for } k > 2 \tag{5.5}$$

However, our parameter k (1.6225) is less than 2, this indicates that the variance is not bounded for the distribution. The distribution is **Fat-tailed distribution**. A fat-tailed distribution looks like Normal distribution but the parts far away from the average are thicker, f(x) goes to zero only very slowly as $x \rightarrow \infty$. This shows that there is a higher chance of huge deviations.

Another uncertainty is created due to the panel size of the sensor. If too small panel is selected, it is possible that this sensor measures a local peak impact force that is not representing the impact force averaged over the entire strip. On the other hand it is likely that too small panels will miss the local peak force. Therefore a correct size of panel must be selected in order the above events not to happen. The effect of area considered on the impact pressure is discussed on chapter 7.

6. ESTIMATING ALS IMPACT PRESSURE AND INVOLVED UNCERTAINTIES

According to NORSOK N-003 (2007) an offshore structure should resist the characteristics environmental loads defined for the limit states, one of these limit states is the Accidental Limit State (ALS) i.e. the load effect corresponding to the annual probability of exceedance not larger than 10^{-4}. A correct estimate of this characteristic value requires that the variability in the weather condition as well as the short-term (3-hour) variability of the response it is accounted for (Baarholm, et al., 2010). In order to establish a long variation of the three hour extreme distribution, some sort of long term response analysis is required. This is done by approximating the whether development by sequence of stationary 3-hour events. However, to establish each 3-hour conditional response distribution is very time consuming and uneconomical. For economical and fare estimating the environmental contour line approach can be used. On this report both the long term all sea state analysis and environmental contour line analysis will be discussed. Due to confidentiality and lack of data the value for ALS extreme impact load will not be estimated. However, the detail procedures will be discussed in section 6.2.

6.1 ALS Impact Extremes Using Environmental Contour line analysis

In section 3.3 environmental contour line method is discussed for non-linear system and we will use the steps here in order to estimate the ALS impact extreme.

The first step is to establish the q-probability contour/surface for the involved sea state condition, represented by significant wave height and spectral peak period. MARINTEK has already established 10^{-4} probability contour line. After establishing the contour line, model tests were performed in order to identify the most unfavorable sea state condition along the q-probability contour line. From these two steps they obtain the critical or most unfavorable sea state (H_s and T_p). These values are not given on this project, as early mentioned due to confidentiality.

However, the realizations for this most unfavorable sea state are given. 109 realizations were done in order to establish the marginal 3-hour extreme impact load distribution.

In chapter five direct and indirect approaches were done so as to create the marginal 3-hour extreme impact load distribution from the obtained data. The distribution is given in chapter five by Fig.5.10 and Fig. 5.11. This is the third step in estimating ALS impact load using environmental contour line method.

The next or final step is to estimate for the 10^{-4} probability response value by the α-percentile of this extreme value distribution. According to NORSOK N-003 (2007), a percentile between 90th and 95th should be used when predicting characteristic values for this kind of problem. Since using the 95th percentile gives the worst result, this has been used.

Table 6.1 ALS impact extreme pressure due to breaking wave

α Value	Direct approach	Indirect approach
0.9	**3.281**	**3.679**
0.95	**5.114**	**5.517**
N.B The values given are scaled pressure		

As we can see on table 6.1, the direct method gives less value than the indirect approach. As we have seen in the indirect approach in chapter 5, all the maximum pressure and the probability distribution of the number of impacts per run are considered. Therefore indirect approach is better estimate than the direct approach, while direct approach is an approximate.

According to the result in Table 6.1 and the explanation in the above paragraph our ALS impact load or pressure is **5.517.** The actual pressure value can be obtained by multiplying this scaled pressure (5.517) by the scale factor which can be obtained from the owners of the data (STATOIL).

6.2 All sea state or long term analysis

Another and consistent estimation of the 10^{-4} probability of exceedance impact load is the all sea state approach. This method is explained in detail in section 3.2 for linear system with some numerical values and section 3.3 for nonlinear system. Here we will only indicate the main steps of the procedure used to estimate the ALS load.
The long term 3-hour extreme response distribution can be calculated using equation 3.6. The equation contains the conditional short term 3-hour maximum distribution and the long term weather variation.

$$F_{\bar{Z}_{3h}}(z) = \int\int_{HsTp} \left[F_{Z_{3h}|Hs,Tp}(z \mid h_s, t_p)\right] f_{Hs,T_p}(h_s, t_p) dh_s dt_p \qquad (3.6)$$

From equation 3.6 the main challenge is to estimate the conditional distribution of the 3-hour maximum, particularly for nonlinear structure system like our TLP platform. It is generally accepted that Gumbel distribution is represents the 3-hour extreme distribution. To estimate the Gumbel parameters α and β, time domain analysis should be applied to generate time series of the relevant responses for a large number of sea states. Baarholm et al. (2010) have prepared response surface as shown in Fig. 6.1 for their project Troll A platform. Response surfaces should be established for the parameters, $\alpha(H_s,Tp)$, $\beta(H_s,Tp)$, of the Gumbel distributions. Using the response surface, long-term distribution given by equation 3.6 is established.

As the long term distribution for the 3-hour maximum load is established, an estimate for the value corresponding to an annual exceedance probability of q (10^{-4}) can be obtained from the following relationship:

$$F_{Z3h}(Z_q) = 1 - \frac{q}{m_{3h}} \qquad (3.7)$$

Where m_{3h} is the expected annual number of 3-hour periods i.e. 365*24/3 = 2920.

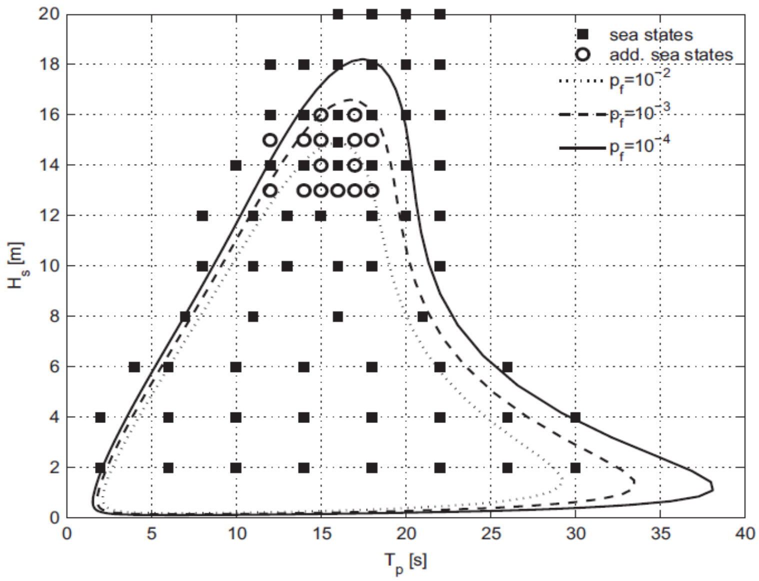


Fig. 6.1 Large number of sea states for long term analysis to create a response surfaces Troll A (Baarholm, et al., 2010)

Our final result from equation 3.7 is to calculate the Z_q i.e. the extreme value which has annual probability of exceedance 10^{-4}. The procedure how to find this is explained in section 3.2, task 6 and the value is read from the marginal long term 3-hour extreme load distribution (Fig.3.11). This value (Z_q) is the impact pressure load due to breaking wave corresponding to an annual exceedance probability of 10^{-4}.

6.3 Uncertainties

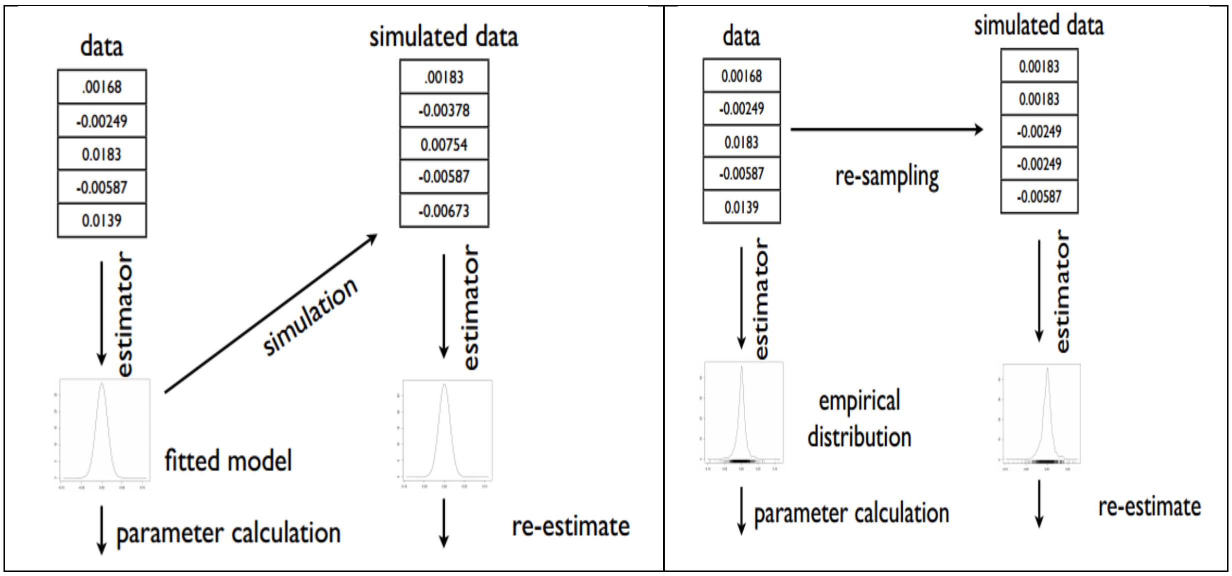
Before we discuss about uncertainty, it is important to remember that the environmental contour method is an approximate method. There is uncertainty regarding the choice of **α**. This **α** can only be checked using long-term analysis. To perform the long-term analysis using model tests is very expensiv; however it can be fairly done using time-domain simulation. Baarholm et al. (2010) has analyzed the long-term analysis using time-domain simulation to check the 90 percentile used in the model test for Troll A platform. They found out that the percentile average α = 0.80 which is lower than the assumed percentile(0.90) In Kleiven & Haver (2004) values up to 0.98 are found necessary, while in Sødahl et al. (2006) **α** =0.57 is found for a riser problem in the touch down region. This shows that there is a considerable uncertainty on selection of α.

The second main uncertainty is, if we assume that the data from the model test is correct, uncertainty in estimating the probability model and the parameters of the model can be created. These uncertainties can be investigated using the bootstrapping method. It is a statistical resampling method that can be used to determine the confidence band of a statistical variable. In the following subtopic this method will be discussed in detail.

Bootstrapping

In statistics, bootstrapping is a method for assigning measures of accuracy (defined in terms of bias, variance, confidence intervals, prediction error ...etc.) to sample estimates. It is a statistical resampling method which is assumed that there is an equal probability for each of the measured extreme values to occur. All values are picked randomly with replacement, from the measured sample. A new data set is then created. This type of bootstrapping is Non-parametric bootstrapping and there is another type of bootstrapping called Parametric (Model-based) bootstrapping.

In parametric bootstrapping, a new set of samples is created using the distribution established based on the measured (original) values. The cumulative distribution F(x) is set to values from 0 to 1, and then a new sample is generated using the fitted model by inserting the drawn random numbers. The procedures for both parametric and non-parametric bootstrapping are shown in Fig.6.2



(a) (b)

Fig. 6.2 Schematic for parametric (model-based) and non-parametric bootstrapping (Carnegie Mellon University, 2011)

In the estimation of ALS load for α =0.95, the scaled pressure value is 5.114 (Table 6.1). Now let's find out the 90% confidence interval for the scaled pressure of 5.114. In direct approach we have 109 tests/ runs and each test has one extreme value, therefore we get 109 observed data. Using bootstrapping we can regenerate many new samples. If we run the loop to 100 times, we can get 100x109(10900) sample data. Here we will use the model based boot strapping .The Frechet distribution, Eq.(5.4), is set as the "true" distribution for this method. New sample values will then be generated based on this distribution using the Monte Carlo method. This method is described in Haver (2013). A new sample is generated by drawing random numbers between 0 and 1 for the cumulative function, F(x). The x-values, in this case are the impact pressures, are then found using the inverted cumulative distribution, $F_x^{-1}(x)$.

We assign F(x) = r = [0,1] hence x can be calculated using equation (6.1)

$$X_i = \frac{\lambda}{(-\ln r_i)^{\frac{1}{k}}} \tag{6.1}$$

Where r = $r_{1}, r_2, r_3 \ldots r_{109}$ by substituting these values on equation (6.1) we can get new 109 x values .

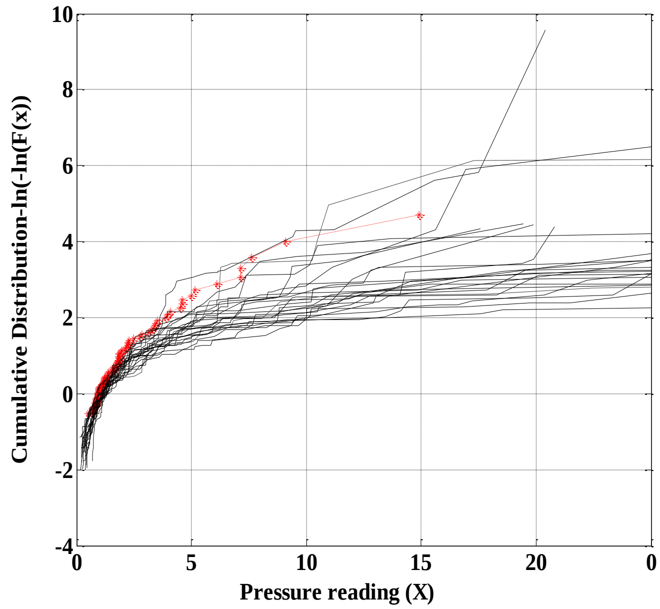


Fig. 6.3 Sample generation using the Monte Carlo simulation and true data are shown by red color.

From this it can be seen that the confidence band gets wider for higher probabilities. It can therefore be concluded that the uncertainty rises for higher probability levels. As discussed in section 5.3, Frechet probability model has a **Fat-tailed distribution**, f(x) goes to zero only very slowly as $x \rightarrow \infty$. This shows that there is a higher chance of huge deviations. In Fig. 6.2 as the values of x increase there is very wide band, this shows there is high uncertainties for large probability values.

The 90% confidence band and mean value at the 95th percentile are given in Table 6-2.

Table 6.2 90% confidence band and mean value from the Monte Carlo method at the 95th percentile

X_{3h} , 10^{-4} (Pscaled)	90% Confidence Band		
	Lower Limit (Pscaled)	Upper limit (Pscaled)	Mean (Pscaled)
5.114	2.301	9.301	5.801

7. EFFECT OF SLAMMING AREA CONSIDERED ON IMPACT PRESSURE

7.1 Theoretical background

In reference to DNV-RP-C205 (2007) the impact force due to breaking waves on platform columns if calculated using equation 7.1. As then the average pressure per strip or exposed area will be F/A. This basic relationship shows that impact pressure is indirectly proportional to the impact area. On this chapter we will investigate the area considered effect on the average impact pressure due to breaking waves.

$$F_I = \frac{1}{2}\rho C_{pa} A v^2 \qquad (7.1)$$

In general when selecting the panel size in the model test, there are 56 pressure sensors arranged in such a way that they can record the impact in large area of 9x9m^2 (Full scale). The arrangement of the sensors is in eight rows by seven columns. The arrangement is shown in Fig. 7.1, where sensor (4,4) as the center core of the arrangement.

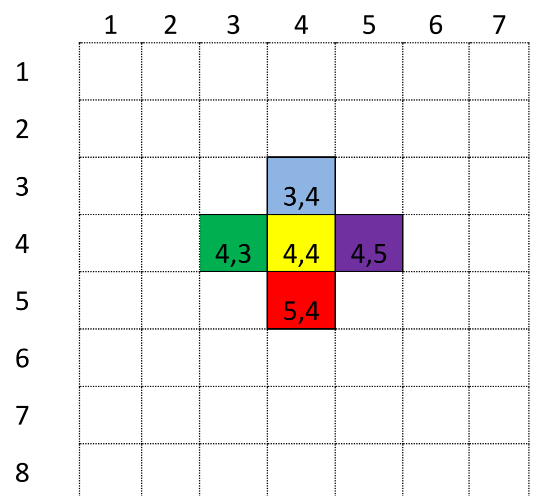


Fig. 7.1 The arrangement of 56 sensors in eight row and seven columns

Previously on chapter five we established the marginal 3-hour extreme load distribution based on sensor (4,4). This is the impact pressure on a specific area by a single sensor, how will be the impact if we consider three sensors at a time. Meaning by including the data from neighboring sensors to panel (4,4) i.e. one on top and one on bottom. In the same way for the horizontal section one to the left and one to the right. Here on this chapter we will consider these two cases to investigate the effect of considered area on the impact pressure. The first case is in the vertical direction (Blue, Yellow and Red) sensors and the second one will be in the horizontal direction sensors (Green, Yellow and Violet).

7.2 Sensors on Vertical Direction

Sensors 3,4 ; 4,4 and 5,4 when the impact pressure on them for each test is recorded and those impacts that occurred at the same is averaged. The maximum average pressure for each test and event is selected and their distribution is checked to fit different probability models. The Weibull, Gumbel, Frechet and Pareto distributions are tested and the result is shown on Fig. 7.2. Most of the data are crowded between 0 and two. Values greater than two are little bit scattered, specially one data (x=11.9) is far apart from the other values. For that particular sea state further investigation has to be done to answer what makes it that big impact?

Fig. 7.2 Vertical direction lined sensors (3,4); (4,4)and (5,4)

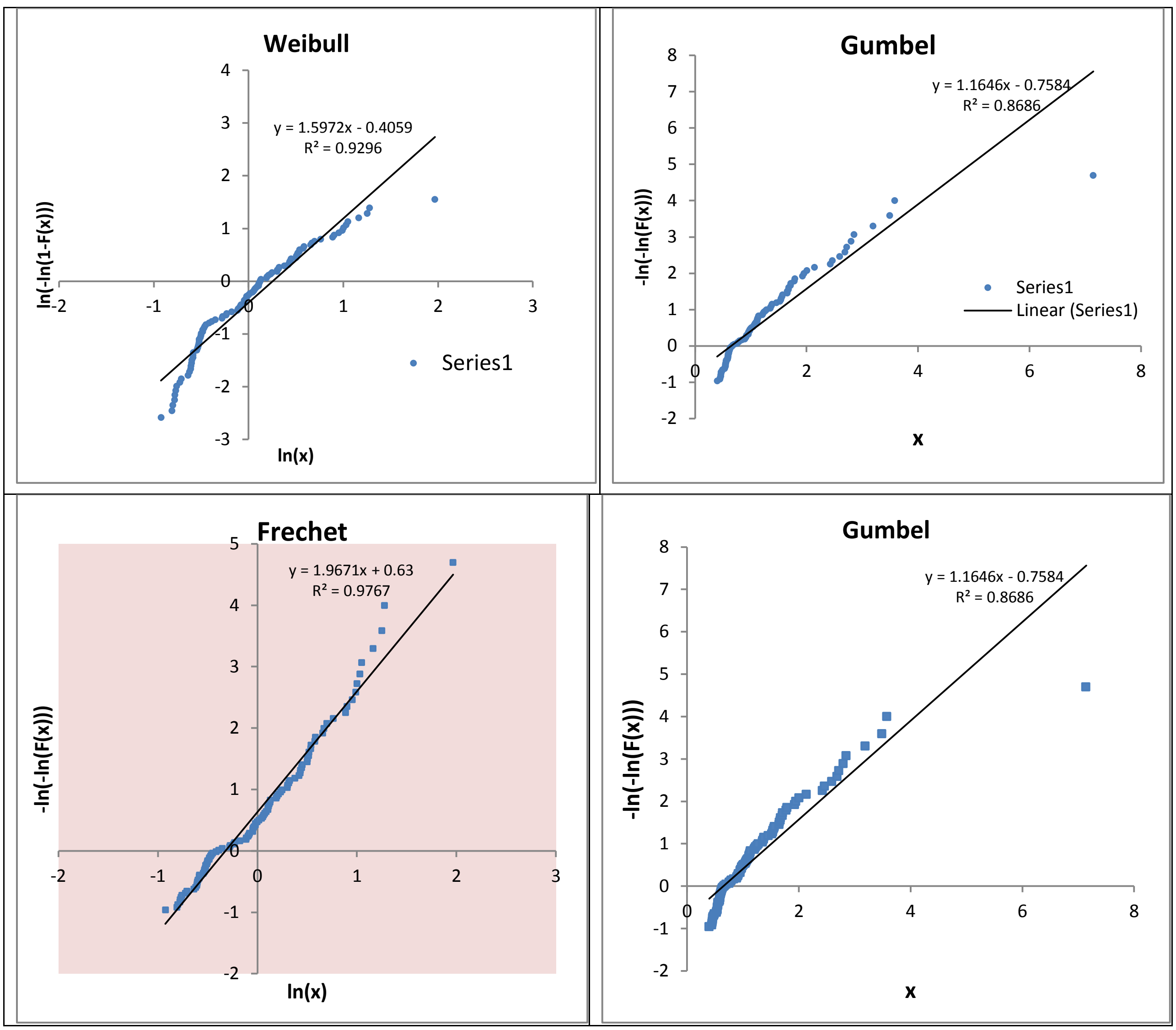


Fig. 7.3 Different probability distribution models for fitting the data distribution

From Fig.7.2 Frechet probability distribution fits best for the data distribution. If we plot the cumulative

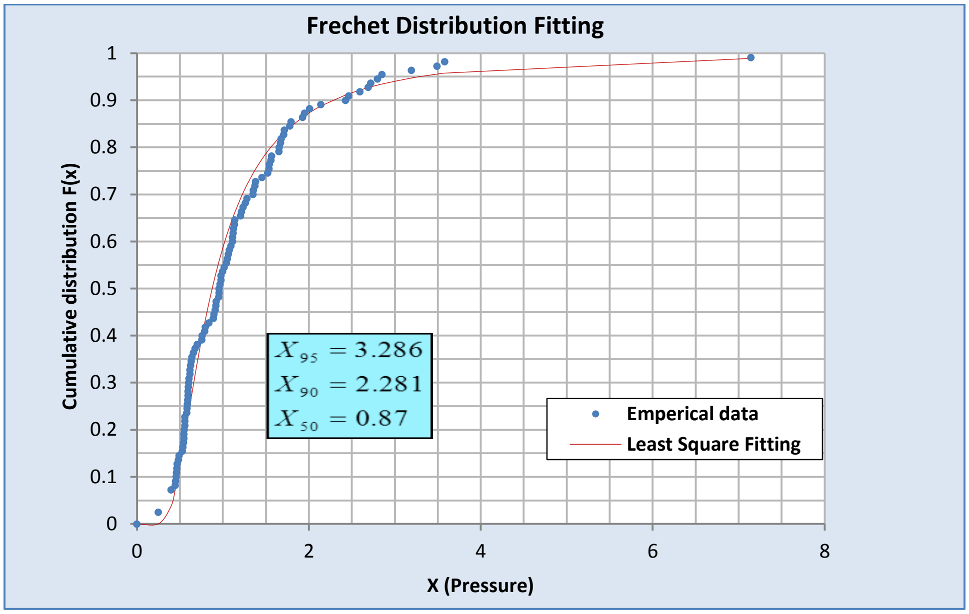


Fig. 7.4 Frechet distribution fitting and empirical data plotted in linear scale for vertical sensors

7.3 Sensors on Horizontal Direction

In horizontal direction the next close sensors to sensor (4,4) are (4,3) and (4,5). If we group all the data reading from these sensors and averaged the impacts that hit at the same time, the marginal distribution of the 3-hour maximum can be established. Fig.7.4 shows the alignment of the sensors and marked with green, yellow and Violet colors.

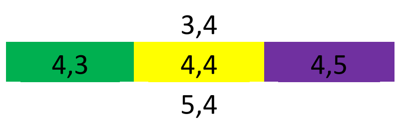


Fig. 7.5 Horizontal direction lined sensors (4,3); (4,4)and (4,5)

Selected probability models are fitted to the data using a least square procedure. A Frechet probability model fits best and is shown in Fig.7.6. The trail for other probabilistic models is shown in Fig. 7.6

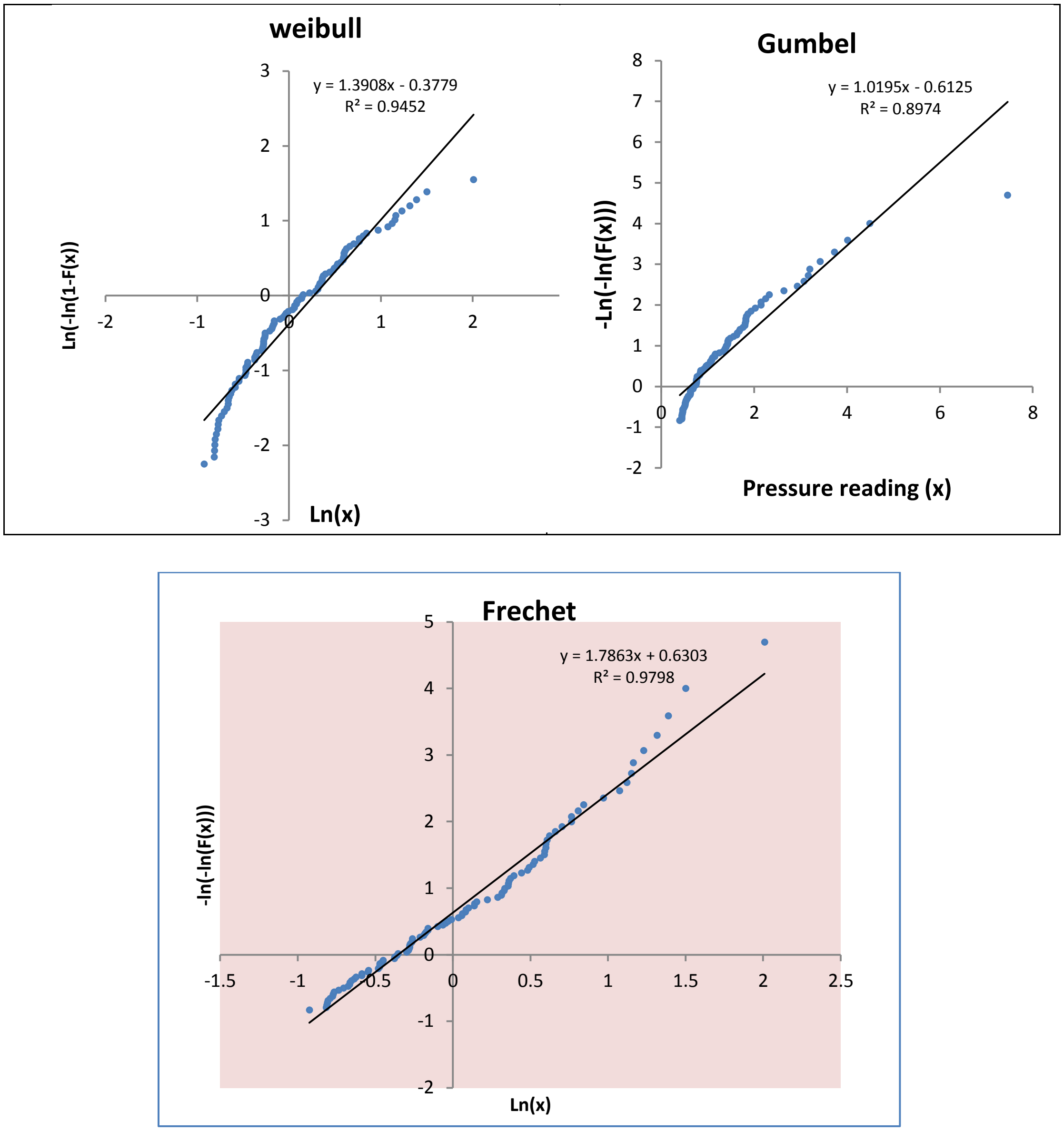


Fig. 7.6 Different probability distribution models for fitting the empirical data

Based on Fig 7.6 we can estimate the Frechet distribution parameters and plot the cumulative probability distribution on linear scale. It is shown in Fig.7.7 and the α percentiles can be estimated accordingly.

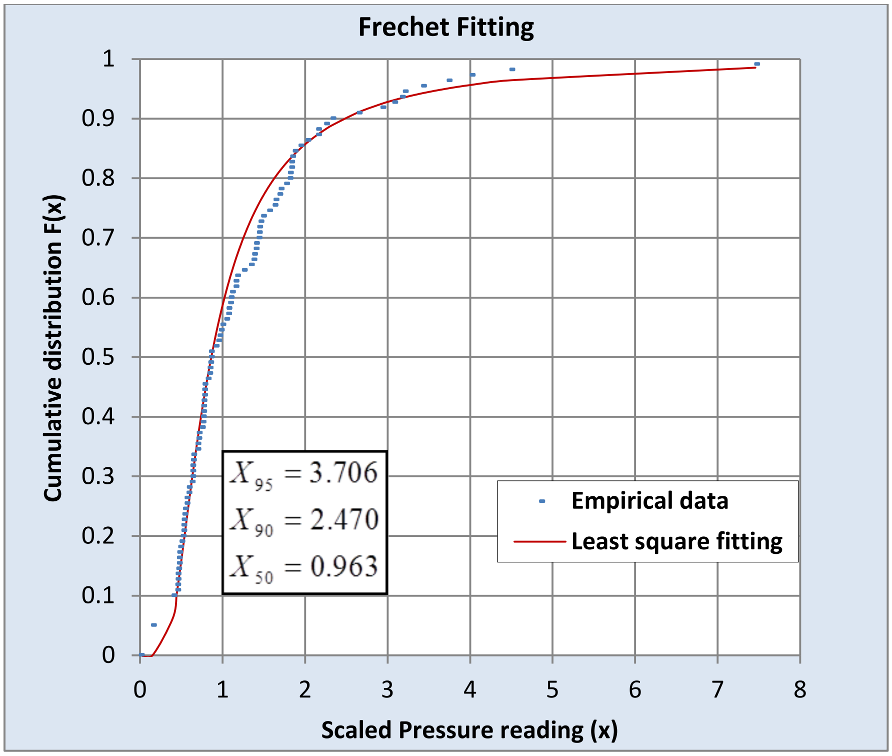


Fig. 7.7 Frechet distribution fitting and empirical data plotted in linear scale for horizontal sensors

8. RESULTS AND DISCUSSION

On this section we will see two parts, the first is in the approaches used for estimating the marginal 3-hour extreme value distribution and the second part will be on the investigation of the effect of area considered on the impact pressure.

As it is explained in the abstract part and in the introduction part of chapter five, on this project environmental contour line method is used to estimate extreme impact load due to breaking wave. Breaking wave impact load is categorized under the environmental load and due it's extremity it will be treated under the accidental limit state analysis. The final target of this project is to estimate this ALS load due to breaking wave. To estimates this, model test is done in Marintek, Trondheim for Statoil and environmental contour line method approach is used. The contour line of sea states with $q=10^{-4}$ are plotted and the worst sea state is estimated by modeling few sea states along the 10^{-4} contour line. After estimating the worst sea state, 109 realizations are tested and the data is shown in appendix B. Using these data two approaches are used to establish the marginal distribution of the 3-hour extremes load and corresponding α value loads.

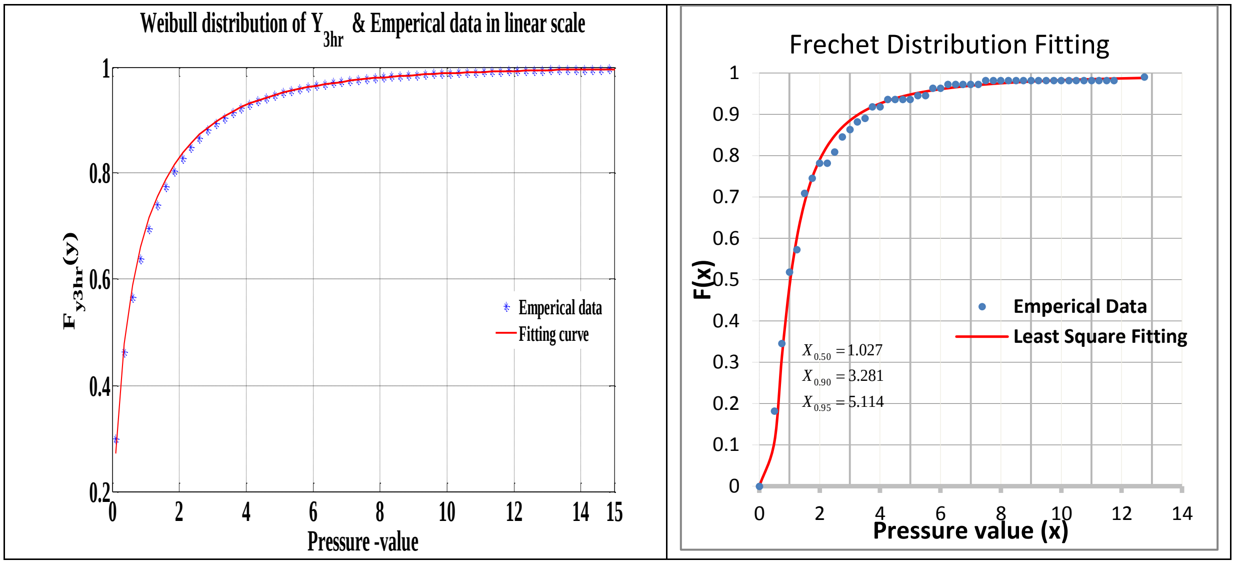
For the above two approaches the impact pressure measured on sensor (4,4) are used to estimate the extreme load; however, the investigation is extended to the neighboring sensors to check the effect of area considered on the impact pressure. On the second section of this chapter the results obtained will be discussed.

8.1 Direct and indirect approaches on estimating the 3-hour marginal extreme value impact pressure

In direct approach estimation the 3-hour extreme pressure is estimated by selecting all the 3-hour extremes from each test and fitted to a probabilistic model. The best fitted model to the numerical data is Frechet cumulative distribution. Using the fitted model 10^{-4} probability values are estimated using different α values (Table 8.1). The challenge is here which value of alpha to select. The right value can only be estimated using the long term all sea state analysis but on this project we used environmental contour line method and NORSOK N-003 (2007) recommends to take alpha = 0.9 or 0.95. We will check out for alpha = 0.5, 0.90, 0.95 and 0.99 for both direct and indirect approach.

As it can be seen on table 8.1 the pressure values are different from direct and indirect method. For lower values of α there is little difference between the result, for example for α = 0.5 there is only a difference of 0.117 but as the value of α increases the difference increases. For example when α=0.99 the difference is 2.894 it is huge amount. This could be due to the reason that Frechet distribution is a fat tailed distribution and we have to remember also direct approach is an approximate estimate.

Direct method is approximate approach because it only picks the extreme value from each test and estimates the distribution of these values to a probabilistic model. However, it doesn't consider the probability of the extreme in each test. Is it very rare to occur or it has relatively high probability of occurrence. This probability will affect the result when we calculate the marginal 3hour extreme impact pressure. In indirect method, by considering all impacts above a certain threshold, the maximum values within each test are considered but are they many? is their value close to the extreme value? and so on . All these questions are answered by the indirect method. First it includes all the maximum values then it considers also how many maximum impacts can occur within each test , this is elaborated by Poisson distribution in equation 5.1. Therefore we can conclude that indirect approach is relatively precise result than the direct approach even though we have to check out with the long term (all sea states) analysis.



(a) (b)

Fig. 8.1 Cumulative distribution for 3-hour extreme impact load (a) Indirect (b) Direct approach

From the above cumulative distribution the estimated 3-hour extreme value can be calculated. Table 8.1 shows the value of the extreme for different values of α, starting from 0.5 to 0.99 for both approaches.

Table 8.1 Estimated 3-hour extreme value

α - value	Estimated 3-hour extreme Pressure (scaled)		Difference
	Direct Approach	Indirect Approach	
0.5	1.027	0.910	↓ 0.117
0.90	3.281	3.679	↑ 0.398
0.95	5.114	5.517	↑ 0.403
0.99	13.968	11.092	↓2.894

8.2 Effect of area considered on the impact pressure

On chapter five the analysis was done for impact pressure based on a single sensor (4,4) and in chapter seven the investigation extended to neighboring sensors to see the effect of area considered on estimating impact pressure. On this section we will discuss on the impact pressure obtained from the single sensor, vertical sensors and horizontal sensors.

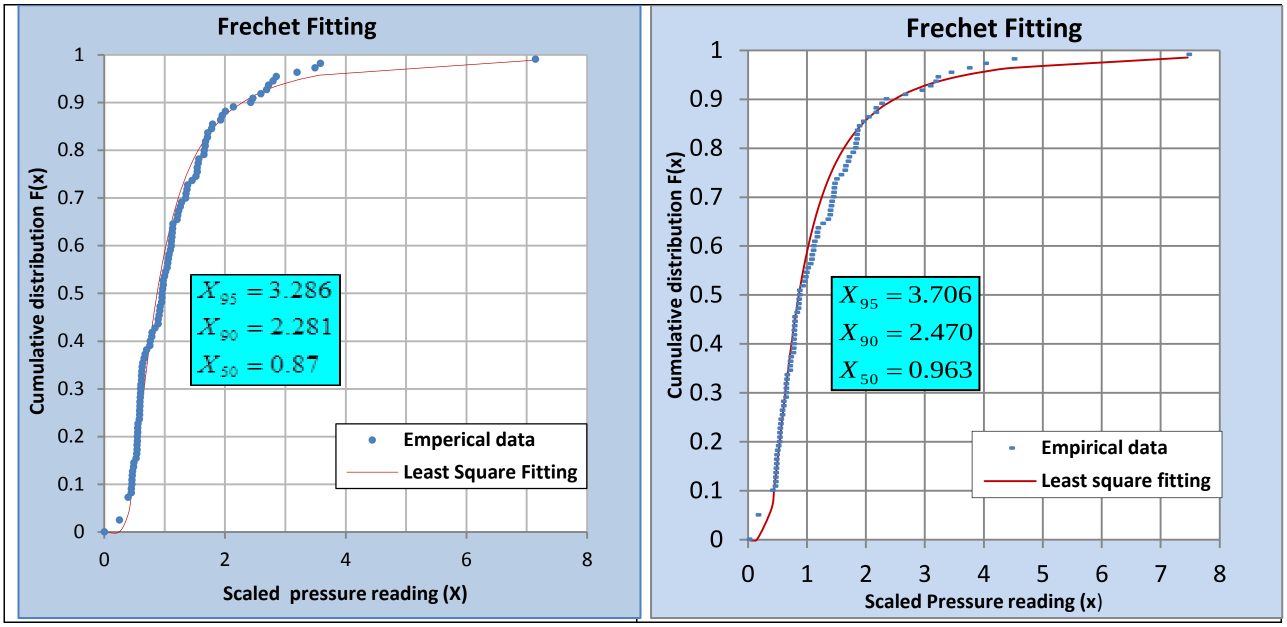
For smaller values of α the impact pressure on a single sensor (4,4) is almost the same with the impact pressure on three sensors. For example when α =0.5, the impact pressure in sensor (4,4) is 0.910 but in three horizontal and vertical sensors, it is 0.870 and 0.963 respectively . However, when α value increases, there is a significant difference on the estimated average impact pressure. For example when α= 0.95 the average impact pressure from sensor (4, 4) is 5.517. While in vertical and horizontal sensors is 3.286 and 3.706 respectively. These values are summarized in table 8.2.

Table 8.2 Summary of the estimated impact pressure on single sensor (4,4) , vertical & horizontal sensors

	Single sensor (C)	Vertical sensors (a)	Horizontal (b)	Remark (Difference)	
α- value	Sensor (4,4)	(3,4); (4,4) & (5,4)	(4,3); (4,4) & (5,4)	(c) –(a)	(c) -(b)
0.5	0.910	0.870	0.963	0.04	0.053
0.9	3.679	2.281	2.470	1.398	1.209
0.95	5.517	3.286	3.706	2.231	1.811
0.99	11.092	7.526	9.229	3.566	1.863

From the above analysis it is clearly shown that the average impact pressure reduces as the area considered increases. As the value of alpha increases, the difference between average impact pressure measure in a single sensor and three sensors increases.

The estimated average impact pressure on the three vertical and horizontal sensors is almost the same for most α values. For α=0.5, 0.90 & 0.95 there is only of average 0.35 but for very large α value like 0.99, there is a slight difference. In vertical direction sensors((3,4); (4,4);(5,4)) the average impact pressure is 9.229 and in horizontal sensors((4,3); (4,4);(5,4)), it is only 7.526, there is a difference of 1.694. The results obtained from the vertical versus horizontal direction sensors are given in Fig.8.3.



(a) (b)

Fig. 8.2 Cumulative distribution fitting of the impact pressure (a) vertical and (b) horizontal sensors

9. CONCLUSSION AND RECOMMENDATIONS

9.1 Summary

In this report slamming loads from breaking waves on platform column are assessed. Statistical analysis of model test results, i.e. stochastic analysis based on model test result is used. In addition to stochastic analysis, a recommendation from the DNV (Det Norske Veritas) is also discussed with its uncertainty. Environmental-contour line method is mainly used in estimating the slamming load using the model test data obtained from Statoil. On this method two approaches are used to establish a distribution of the marginal 3-hour maximum impact load. The results from these two approaches are analyzed and 95 percentile level of the distribution is used to obtain the ALS design slamming load.

In environmental-contour line method, four principal steps are used to estimate the impact load from breaking waves. However, the first two steps, establishing of contour line and identifying the worst sea state are already performed by Marintek and the last two steps (establishing of 3-hour maximum distribution and estimating of the q-probability response value by the α-quintile) are analyzed here. Direct and indirect approached are used to establish the 3-hour maximum impact distribution. Both approaches give almost the same result; however, indirect approach gives slightly higher values than the direct approach.

After obtaining the slamming load from breaking wave using the data from a single sensor, the investigation is extended to see how the area considered affects the slamming load. The sensors on the top, bottom, left and right of the sensor are included in estimating new slamming load. This new slamming load is compared with the impact load obtained from single sensor data.

In order to understand the above methods and investigations, the theories behind them are discussed in the beginning chapters. Breaking wave phenomenon, limit state analysis (specifically ALS) and prediction of a wave dominated load corresponding to annual exceedance q are some of the theories that are discussed in the first three chapters. Mainly in chapter-3 a wide explanation of the short-term and long-term analysis is given. All sea state long-term analysis is elaborated for linear structural system using numerical values and assumptions. A scatter diagram for Northern-North Sea, 1980 – 1983 is used in the analysis of joint distribution of wave height and peak period. From these statistical data a long-term analysis is performed and the ALS load from the impact load is estimated.

9.2 Conclusion

From the analysis and investigations done on this report, the following three main conclusions can be pointed out.

- When estimating the distribution of the 3-hour maximum pressure, two approaches can be used. First one is by directly collecting the observed 3-hour extremes, this is an approximate value it doesn't consider the number of impacts per event and the probability of this extreme impact

- within 3-hour event. The second approach is indirectly by considering all impacts above a certain threshold. This method selects all maximum impacts within each test (3-hour duration) and it includes the number impacts per event. On this report the result obtained from both approaches is slightly different especially when we consider high α percentile from the distribution curve. Therefore indirect method is preferred to direct method.

- In estimating the impact pressure due to breaking waves, the size of the considered area is important in estimating the average impact pressure per unit area. The area considered affects the impact pressure. As the considered sensor area increases the average pressure obtained decreases. On the other hand if the area of sensor decreases the impact pressure increases but there is possibility of missing the local peak force.

- In estimating the impact pressure from breaking wave, two general methods can be used. The DNV recommendation and the model test analysis. Environmental-contour line analysis is the preferred model test and is widely used in the offshore industry.

9.3 Recommendation for further work

When estimating the maximum impact pressure on offshore structures from breaking waves, there are some uncertainties defined in section 6.3. The main uncertainties are selecting of α-value and selecting best fitted probability model that fits the data distribution.

The α–values selected here are based on the recommendation from NORSOK N-003 (2007), α=0.9 for ULS ($q= 10^{-2}$) and α = 0.95 for ALS ($q=10^{-4}$). However, previous studies show that the actual value is quite different from this value. It can only be checked using the long-term analysis. Baarholm et al. (2010) has investigated for Troll A platform, and they find out that α value around 0.8 which was previously assumed in the model test to be 0.9. With same principle used in Troll A, long-term analysis using time-domain simulation has to be done for this report in order to check the value of α.

The second main uncertainty is selection of probability model that fits the distribution of the 3-hour maximum impact pressure. It is find out that Frechet probability model fits best; however, Frechet probability model has **fat-tailed** end. Meaning that f(x) goes to zero only very slowly as $x \rightarrow \infty$, this indicates that there is a higher chance of huge deviations. As it is shown on the bootstrapping analysis, there is high uncertainty for high cumulative probabilities. Therefore a further investigation has to be done to find out the best fitting probability model or to minimize the uncertainty of the Frechet model.

The third investigation or further work that has to be done is on the model test. From the model test numbers 61850 and 61460 the scaled impact pressures are 11.9 and 7.28 respectively. But why do we get those two big numbers while the average reading is around 1.5? Those two tests must be carefully checked and if the model test is correct, a theory or principle must be investigated to find out that at what conditions maximum impact pressure is created.

REFERENCES

Baarholm, G. S., Haver, S. K. & Økland, D. O., 2010. Combining contours of significant wave height and peak period with platform response distributions for predicting design response. *Journal of Marine Structures*, Vol. 23(No. 2), pp. 147-163.

Benjamin, J. R. & Cornell, C. A., 1970. *Probabilty, Statistics and Decision for Civil Engineers.* 1st ed. New York: McGraw- Hill.

Brown, J., Colling, A. & Park, D., 1989. *Waves, Tides and Shallow-Water Process.* 1st ed. Oxford: The Open University.

Carnegie Mellon University, 2011. *Statistics.* [Online]
Available at: http://www.stat.cmu.edu/~cshalizi/402/lectures/08-bootstrap/lecture-08.pdf
[Accessed 2 May 2014].

Chakrabarti, S. K., 2005. *Offshore Structure Analysis.* 1st ed. Illinois: ELsevier.

Dalane, E., 2011. *Estimation of ALS wave-column impact loads for a semi-submersible and a simplified assessment of the associated structural response.* Trondheim: Msc Thesis, NTNU.

DNV-RP-C205, 2007. *Environmental Conditions and Environmental Loads.* Høvik: Det Norske Veritas.

Ewans, K. & Jonathan, P., 2014. Evaluating environmental joint extremes for the offshore industry using the conditional extremes model. *Journal of Marine Systems*, Volume 130, pp. 124-130.

Haver , S. K., 2006. *Reasons for Focusing More on Prediction of the Very Extreme Sea States*, Stavanger: Statoil.

Haver, S. K., 2013. *Prediction of Characteristics Response for Design Purpose*, Stavanger: Statoil.

Haver, S. & Kleive, G., 2004. Environmental Contour lines for Design-Why and When?. *OMAE*, Volume 1, pp. 337-345.

Haver, S. K. & Nyhus, K. A., 1986. A Wave Climate Descriptin for Long Term Response Calculation. *OMEA*, Vol. 4(No. 5), pp. 27-34.

Kleiven, G. & Haver, S., 2004. *Metocean Contour lines for Design Purpose, Correction for Omitted Variability in the Response Process.* Toulon, ISOPE.

Lehn, E., 2003. *On Water Impact Loads. Technical Report P_09.03*, Trondheim: MARINTEK.

Mathbits, 2014. *Mathbits.* [Online]
Available at: http://mathbits.com/MathBits/TISection/Statistics2/correlation.htm
[Accessed 15 February 2014].

Naess, A. & Moan, T., 2013. *Stochastic Dynamics of Marine Structures.* 1st ed. New York: Cambrigde University Press.

Norsk Oljemuseum, 2010. *Norsk Oljemuseum.* [Online]
Available at: http://www.norskolje.museum.no/modules/module_123/proxy.asp?I=2248&C=230&D=2
[Accessed 10 May 2014].

NORSOK N-003, 2007. *NORSOK STANDARD - Action and Action effects,* OSLO: Standards Norway.

NS-EN ISO 19900, 2013. *Petroleum and natural gas industries - General requirements for offshore structures*, Oslo: Standards Norway.

Ochi, M. K. & Tsai, C. H., 1984. Prediction of impact pressure induced by breaking waves on vertical cylinders in random seas. *Journal of Applied Ocean Research,* Vol. 6(No. 3), pp. 157-165.

Odland, J., 2012. *OFF550-Offshore Field Development.* Stavanger: UIS.

PSA, 2001. *Regulations Relating to Design and Outfitting of Facilities etc in the Petroleum Activities,* Stavanger: Petroleum Safety Authority Norway,.

Sødahl, N., Hagen, Ø., Steinkjer, O. & Chezhian, M., 2006. *Calculation of Extreme Nonlinear Riser Response.* Houston, DOT2006.

Statoil, 2003. *Statoil.* [Online]
Available at: http://waves.ist.utl.pt/XOffshore/NTNU/Microsoft%20PowerPoint%20-%20February%20_7th_HeidrunTLP_ModelTest_Dec_2003.pdf
[Accessed 10 January 2014].

Stokes, G. G., 1847. On the theory of oscillatory waves. *Journal of Transactions,Cambridge Philosophical Society*, Vol.8(No.4), pp. 441-455.

U.S. Mineral Management Service, 1999. *U.S. Mineral Management Service.* [Online]
Available at: http://www.atp.nist.gov/eao/grc04-863/chapt4.htm
[Accessed 10 March 2014].

University of Maine System, 2003. *SMS-491.* [Online]
Available at: http://misclab.umeoce.maine.edu/boss/classes/SMS_491_2003/Week_6.htm
[Accessed 10 April 2014].

Zhou, D., Chan, E. S. & Melville, W. K., 1991. Wave impact pressures on vertical cylinders. *Journal of Applied Ocean Research,* Vol.13(No.5), pp. 220-234.

Appendix A

TABLES OF PROBABILISTIC MODEL AND LEAST SQUARE METHOD FITTING

Table A1 Data analysis calculation for different probability fitting from directly observed extreme approach

Pressure values (x)	n	F(x)=n/(N+1)	ln(x)	Weibull	Gumbel	Frechet	Pareto
				ln(-ln(1-F(x)))	**.-ln(-ln(F(x)**	**.-ln(-ln(F(x)**	**.-ln(1-F(x))**
0.5327763	21	0.1909091	-0.62965	-1.551905143	-0.5043796	-0.50437965	0.211844
0.5471653	22	0.2	-0.603	-1.499939987	-0.475885	-0.475885	0.2231436
0.5503812	23	0.2090909	-0.59714	-1.449991648	-0.447877	-0.44787697	0.2345722
0.5542049	24	0.2181818	-0.59022	-1.401882956	-0.4203055	-0.42030547	0.2461331
0.5548745	25	0.2272727	-0.58901	-1.355458281	-0.3931256	-0.39312565	0.2578291
0.5894736	26	0.2363636	-0.52853	-1.310580145	-0.3662972	-0.36629718	0.2696636
0.6155803	27	0.2454545	-0.48519	-1.267126479	-0.3397835	-0.33978353	0.2816398
0.6447098	28	0.2545455	-0.43895	-1.224988364	-0.3135514	-0.31355145	0.2937611
0.6451660	29	0.2636364	-0.43825	-1.184068185	-0.2875705	-0.28757047	0.3060312
0.6588687	30	0.2727273	-0.41723	-1.144278086	-0.2618126	-0.26181256	0.3184537
0.6634270	31	0.2818182	-0.41034	-1.105538681	-0.2362518	-0.23625179	0.3310325
0.6746081	32	0.2909091	-0.39362	-1.067777973	-0.210864	-0.21086404	0.3437715
0.6791324	33	0.3	-0.38694	-1.030930433	-0.1856268	-0.18562676	0.3566749
0.6914003	34	0.3090909	-0.36904	-0.994936222	-0.1605188	-0.1605188	0.369747
0.6993276	35	0.3181818	-0.35764	-0.959740519	-0.1355202	-0.13552018	0.3829923
0.7308422	36	0.3272727	-0.31356	-0.925292949	-0.110612	-0.11061199	0.3964153
0.7338061	37	0.3363636	-0.30951	-0.891547085	-0.0857762	-0.0857762	0.4100209
0.7431111	38	0.3454545	-0.29691	-0.858460017	-0.0609956	-0.06099557	0.4238142
0.7501270	39	0.3545455	-0.28751	-0.825991978	-0.0362535	-0.03625355	0.4378005
0.7611638	40	0.3636364	-0.27291	-0.794106012	-0.0115341	-0.01153414	0.4519851
0.7627856	41	0.3727273	-0.27078	-0.762767689	0.01317815	0.013178153	0.4663739
0.7676021	42	0.3818182	-0.26448	-0.731944849	0.03789841	0.03789841	0.4809727
0.7688594	43	0.3909091	-0.26285	-0.701607374	0.06264139	0.062641388	0.4957877
0.7755790	44	0.4	-0.25415	-0.671726992	0.08742157	0.087421572	0.5108256
0.7782890	45	0.4090909	-0.25066	-0.642277094	0.11225324	0.112253243	0.5260931
0.7892364	46	0.4181818	-0.23669	-0.613232575	0.13715054	0.13715054	0.5415973
0.7937104	47	0.4272727	-0.23104	-0.584569694	0.16212752	0.162127519	0.5573456
0.8270742	48	0.4363636	-0.18986	-0.556265939	0.1871982	0.187198202	0.573346
0.8658254	49	0.4454545	-0.14407	-0.528299911	0.21237664	0.212376639	0.5896065
0.8756590	50	0.4545455	-0.13278	-0.50065122	0.23767695	0.237676951	0.6061358
0.8963140	**51**	**0.4636364**	**-0.10946**	**-0.473300383**	**0.26311339**	**0.263113392**	**0.6229429**
0.9311602	52	0.4727273	-0.07132	-0.446228737	0.28870039	0.288700394	0.6400374

0.9320435	**53**	**0.4818182**	**-0.07038**	**-0.419418357**	**0.31445262**	**0.314452624**	**0.6574291**
0.9575885	54	0.4909091	-0.04334	-0.392851977	0.34038503	0.340385035	0.6751287
0.9578419	55	0.5	-0.04307	-0.366512921	0.36651292	0.366512921	0.6931472
0.9766571	56	0.5090909	-0.02362	-0.340385035	0.39285198	0.392851977	0.7114963
0.9841080	57	0.5181818	-0.01602	-0.314452624	0.41941836	0.419418357	0.7301885
1.0183564	58	0.5272727	0.01819	-0.288700394	0.44622874	0.446228737	0.7492366
1.0751785	59	0.5363636	0.072487	-0.263113392	0.47330038	0.473300383	0.7686547
1.0874233	60	0.5454545	0.083811	-0.237676951	0.50065122	0.50065122	0.7884574
1.1048390	61	0.5545455	0.09970	-0.212376639	0.52829991	0.528299911	0.8086601
1.1358902	62	0.5636364	0.127417	-0.187198202	0.55626594	0.556265939	0.8292794
1.2389258	63	0.5727273	0.214245	-0.162127519	0.58456969	0.584569694	0.8503328
1.2820350	64	0.5818182	0.248449	-0.13715054	0.61323258	0.613232575	0.871839
1.2841016	65	0.5909091	0.250059	-0.112253243	0.64227709	0.642277094	0.8938179
1.2856406	66	0.6	0.251257	-0.087421572	0.67172699	0.671726992	0.9162907
1.2916564	67	0.6090909	0.255925	-0.062641388	0.70160737	0.701607374	0.9392803
1.3483485	68	0.6181818	0.298881	-0.037898410	0.73194485	0.731944849	0.9628107
1.3739209	69	0.6272727	0.317669	-0.013178153	0.76276769	0.762767689	0.9869083
1.4054899	70	0.6363636	0.340386	0.011534137	0.79410601	0.794106012	1.0116009
1.4207317	71	0.6454545	0.351172	0.036253546	0.82599198	0.825991978	1.0369187
1.4411644	72	0.6545455	0.365451	0.060995570	0.85846002	0.858460017	1.0628942
1.4588983	73	0.6636364	0.377682	0.085776197	0.89154708	0.891547085	1.0895625
1.4671225	74	0.6727273	0.383303	0.110611987	0.92529295	0.925292949	1.1169614
1.4701947	75	0.6818182	0.385395	0.13552018	0.95974052	0.959740519	1.1451323
1.4844464	76	0.6909091	0.395042	0.160518796	0.99493622	0.994936222	1.1741198
1.4861258	77	0.7	0.396173	0.185626759	1.03093043	1.030930433	1.2039728
1.4907071	78	0.7090909	0.399251	0.210864036	1.06777797	1.067777973	1.2347445
1.5758554	79	0.7181818	0.454798	0.236251791	1.10553868	1.105538681	1.2664932
1.5787763	80	0.7272727	0.45665	0.261812562	1.14427809	1.144278086	1.299283
1.6916831	81	0.7363636	0.525724	0.287570468	1.18406819	1.184068185	1.3331845
1.7373296	82	0.7454545	0.552349	0.313551448	1.22498836	1.224988364	1.3682759
1.8105526	83	0.7545455	0.593632	0.339783534	1.26712648	1.267126479	1.4046435
1.8141242	84	0.7636364	0.595603	0.366297181	1.31058015	1.310580145	1.4423838
1.8151781	85	0.7727273	0.596184	0.39312565	1.35545828	1.355458281	1.4816045
1.8204886	86	0.7818182	0.599105	0.420305467	1.40188296	1.401882956	1.5224265
2.0063588	87	0.7909091	0.696322	0.447876974	1.44999165	1.449991648	1.5649861
2.2257442	88	0.8	0.800091	0.475884995	1.49993999	1.499939987	1.6094379
2.2679492	89	0.8090909	0.818876	0.50437965	1.55190514	1.551905143	1.6559579
2.5105309	90	0.8181818	0.920494	0.533417353	1.60609005	1.606090045	1.7047481
2.5970100	**91**	**0.8272727**	**0.954361**	**0.563062064**	**1.66272868**	**1.662728678**	**1.7560414**
2.6660064	92	0.8363636	0.980582	0.593386848	1.72209281	1.722092808	1.8101086

2.7214214	**93**	**0.8454545**	**1.001154**	**0.624475876**	**1.78450064**	**1.784500643**	**1.867267**
2.7514842	94	0.8545455	1.01214	0.656426993	1.85032811	1.850328111	1.9278916
2.8262282	95	0.8636364	1.038943	0.689355082	1.92002379	1.920023791	1.9924302
3.0990895	96	0.8727273	1.131108	0.723396539	1.994129	1.994128995	2.061423
3.1823488	97	0.8818182	1.15762	0.758715332	2.07330531	2.073305309	2.135531
3.2700750	98	0.8909091	1.184813	0.795511384	2.15837318	2.158373178	2.2155737
3.6235985	99	0.9	1.287468	0.834032445	2.25036733	2.250367327	2.3025851
3.6817237	100	0.9090909	1.303381	0.874591383	2.35061866	2.350618656	2.3978953
3.6826737	101	0.9181818	1.303639	0.91759220	2.46087944	2.460879442	2.5032558
4.0060489	102	0.9272727	1.387805	0.963570737	2.58352259	2.583522595	2.6210388
4.1255921	103	0.9363636	1.417210	1.013261429	2.72187466	2.721874657	2.7545702
4.9020098	104	0.9454545	1.589645	1.067713430	2.88080724	2.880807244	2.9087209
5.7117955	105	0.9545455	1.742533	1.128508398	3.06787262	3.067872615	3.0910425
5.7143229	106	0.9636364	1.742976	1.198212044	3.29572254	3.295722537	3.314186
6.0986044	107	0.9727273	1.80806	1.281452621	3.58807417	3.58807417	3.6018681
7.2871954	108	0.9818182	1.986119	1.388125979	3.99817264	3.998172645	4.0073332
11.9110452	109	0.9909091	2.477466	1.547664709	4.6959176	4.695917599	4.7004804

From the above tabulated data the four probability models can be checked for best fitting. It can be plotted **x** versus **–ln (-ln (F(x)** for Gumbel probability paper and **ln(x)** versus **ln(-ln(1-F(x))** for Weibull probability model. We can plot on the same way for the other probability models. As an example we can plot the Frechet probability model.

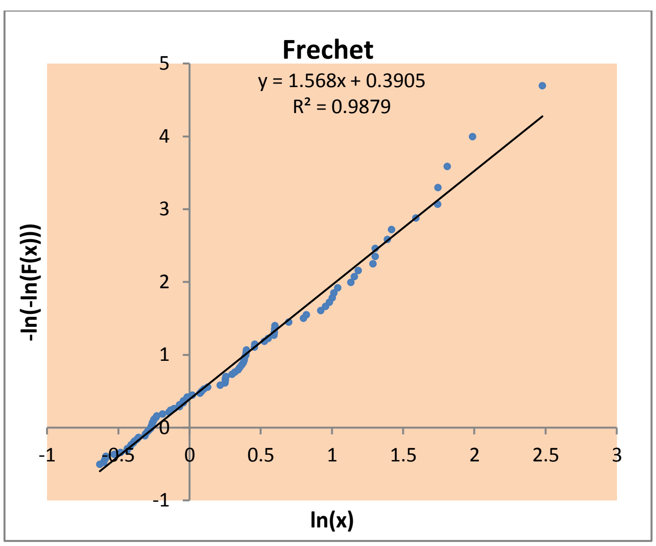


Fig. A.1 Distribution fitting of the largest slamming pressure on Frechet probability paper

Table A2 Least square method analysis in estimating the parameters in the Weibull distribution

ln (x) == X	Y = ln(-ln(1-F(x))	$(X_i - \overline{X})$	$(Y_i - \overline{Y})$	$(X_i - \overline{X})(Y_i - \overline{Y})$	$(X_i - \overline{X})^2$
-1.38629436	-0.446228737	-2.923695126	-1.711729297	5.004574602	8.54799319
-0.69314718	-0.000821946	-2.230547946	-1.266322506	2.824593064	4.975344137
-0.28768207	0.223537730	-1.825082837	-1.041962830	1.901668479	3.330927364
0	0.533417353	-1.537400765	-0.732083207	1.125505282	2.363601112
0.223143551	0.656426993	-1.314257214	-0.609073567	0.800479329	1.727272024
0.405465108	0.776914507	-1.131935657	-0.488586053	0.553047975	1.281278331
0.559615788	0.814538512	-0.977784977	-0.450962048	0.440943916	0.956063461
0.693147181	0.854034451	-0.844253584	-0.411466109	0.347381738	0.712764115
0.810930216	0.940169043	-0.726470549	-0.325331517	0.236343766	0.527759458
0.916290732	0.963570737	-0.621110033	-0.301929823	0.187531642	0.385777673
1.011600912	1.039809531	-0.525799853	-0.225691029	0.118668310	0.276465486
1.098612289	1.067713430	-0.438788476	-0.197787130	0.086786713	0.192535327
1.178654996	1.162026105	-0.358745769	-0.103474455	0.037121023	0.128698527
1.252762968	1.198212044	-0.284637797	-0.067288516	0.019152855	0.081018675
1.321755840	1.237712388	-0.215644925	-0.027788172	0.005992378	0.046502734
1.386294361	1.237712388	-0.151106404	-0.027788172	0.004198971	0.022833145
1.446918983	1.237712388	-0.090481782	-0.027788172	0.002514323	0.008186953
1.504077397	1.281452621	-0.033323368	0.015952061	-0.000531576	0.001110447
1.558144618	1.281452621	0.020743853	0.015952061	0.000330907	0.000430307
1.609437912	1.281452621	0.072037147	0.015952061	0.001149141	0.005189351
1.658228077	1.388125979	0.120827312	0.122625419	0.014816500	0.014599239
1.704748092	1.388125979	0.167347327	0.122625419	0.020521036	0.028005128
1.749199855	1.457455108	0.211799090	0.191954548	0.040655799	0.044858854
1.791759469	1.457455108	0.254358704	0.191954548	0.048825301	0.064698350
1.832581464	1.457455108	0.295180699	0.191954548	0.056661278	0.087131645
1.871802177	1.457455108	0.334401412	0.191954548	0.064189872	0.111824304
1.909542505	1.457455108	0.372141740	0.191954548	0.071434299	0.138489475
1.945910149	1.547664709	0.408509384	0.282164149	0.115266703	0.166879917
1.981001469	1.547664709	0.443600704	0.282164149	0.125168215	0.196781584
2.014903021	1.547664709	0.477502256	0.282164149	0.134734018	0.228008404
2.047692843	1.547664709	0.510292078	0.282164149	0.143986103	0.260398005
2.079441542	1.547664709	0.542040777	0.282164149	0.152944474	0.293808204
2.110202132	1.547664709	0.572812435	0.282164149	0.161627133	0.328114086
2.140066163	1.547664709	0.602665398	0.282164149	0.170050569	0.363205583
2.169053007	1.547664709	0.631652935	0.282164149	0.178229813	0.398985431
2.197224577	1.547664709	0.659823812	0.282164149	0.186178624	0.435367463
2.224623552	1.547664709	0.687222787	0.282164149	0.193909633	0.472275158

2.251291799	**1.547664709**	**0.713891034**	**0.282164149**	**0.201434456**	**0.509640408**
2.277267285	1.547664709	0.739866520	0.282164149	0.208763807	0.547402467
2.302585093	1.547664709	0.765184328	0.282164149	0.215907585	0.585507056
2.327277706	1.547664709	0.789876941	0.282164149	0.222874955	0.623905581
2.351375257	1.547664709	0.813974492	0.282164149	0.229674420	0.662554474
2.374905755	1.547664709	0.837504990	0.282164149	0.236313883	0.701414608
2.397895273	1.547664709	0.860494508	0.282164149	0.242800700	0.740450798
2.420368129	1.547664709	0.882967364	0.282164149	0.249141735	0.779631365
2.442347035	1.685218173	0.904946207	0.419717613	0.379821888	0.818927752
Σ =69.183034	**56.94752521**			**17.76338567**	**35.17461716**

Mean x value is $\bar{x} = \frac{\sum x}{n}$ Mean y value is $\bar{y} = \frac{\sum y}{n}$

b= β*ln(α) and $a = \frac{\sum(x_i - \bar{x})(y_i - \bar{y})}{\sum(x - \bar{x})^2}$ and $b = \bar{y} - a\bar{x}$

By substituting the values in the table on the above formulas we will get α and β values.

Appendix B

NORMALIZED PRESSURE DATA FROM MARINTEK

Table B1 Sample of the collected data from Marinetek*

Run no.	Row	Col	P [scaled]	time [sec]
30020	3	4	2.375116	5522.394
30020	4	4	0.93116	5522.394
30020	3	5	2.952487	5522.394
30020	4	5	0.984187	5522.395
30020	3	6	0.872462	5522.405
30020	3	3	1.303997	6319.56
30020	3	4	1.01961	6319.564
30020	4	2	0.733999	7179.183
30031	4	4	0.64471	2288.546
30031	2	5	1.225942	3700.39
"				"
"				"

*The above table is a sample of the first observations from the data given by MARINTEK. It is a 60 page data collection and there is no need to print out here. It is on the attached CD under the excel file name Normalized Pressure data.

APPENDIX C

MATLAB AND DOCUMENTS IN CD

On the attached CD the following documents are available

- Assessing slamming loads from breaking waves (pdf format)
- Normalized Pressure Data (Microsoft Excel file)
- Best-fitted graph (Excel file for MATLAB calculation)
- Normalized_pressure_2 (Excel file for MATLAB calculation)
- Wave_Spectra_final (MATLAB script)
- Poisson_pdf, Poisson probability density function (MATLAB script)
- Multi sea state (MATLAB script)