NTNU
Norwegian University of Science and Technology
Department of Marine Technology

M.Sc.THESIS

Title:	**Delivered:**
Analysis and Design of Offshore Jacket Wind Turbine	June 14, 2010
	Availability: Open
Student: Peng Li	**Number of pages:** 94

Abstract:

Offshore wind is an attractive source of renewable energy. In order to harvest this abundant energy source wind turbine farms are needed. In order to extend the application of offshore fixed wind turbine (OFWT) in deep water where winds are stronger and steadier, there are research works on fixed wind turbine which is ongoing on larger water depth like 70-100m. The focus in this thesis is analyses to support design of OFWT jacket substructure piled to the seabed, with a particular focus the modeling and ultimate capacity behavior of such facilities subjected to extreme wind and wave forces.

During modelling, Genie and USFOS are applied to re-design and modify the finite element model of wind turbine substructure given by Aker Solutions. New design jacket substructures are both for the intermediate water depth 70 meters and 100 meters with the soil and pile modelling based on the Ekofisk data.

Based on design load case, both of the static pushover analysis and API design code check is performed on the jacket substructure model to check the ultimate capacity. The pushover analysis is performed through nonlinear finite elemnt code USFOS. The API code check is based on hand calculation.The response of the jacket substructures under design load case is most interested in, which could indicate the critical position on jacekt, furthermore the effect of the direction and size of the wind and wave load. Meanwhile, through comaprission of results, the two methods could confirm each other, which could help to get the deeper understanding of ULS of offshore jacket structure.

Keyword:

Offshore fixed wind trubine
Ultimate Limite State
USFOS

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Scope

MASTER THESIS IN MARINE TECHNOLOGY

SPRING 2010

FOR

STUD. TECHN. PENG LI

Design and Analysis of Offshore Jacket Wind Turbine

Background

Wind offshore is an attractive source of renewable energy. To harvest this abundant energy source wind turbine farms are needed. Up to now wind turbines on land, usually supported on rocks, have been utilized. Offshore wind turbines can be supported on the sea bed or, in the future, also be floating. The focus in this thesis is analyses to support design of OFWT jacket substructure piled to the seabed, with a particular focus the modelling and analysis of the dynamic behavior of such facilities subjected to wind and wave forces.

Objective

The purpose of this work is to design the offshore fixed wind turbine for intermediate water depth and make the ULS (ultimate limit state) design check for an offshore jacket wind turbine under the combined extreme wind and wave loads by applying both the design load and the direct analysis method.

Project tasks

1. Literature reading: ultimate strength analysis, DNV design standard for offshore wind turbines (OS-J101), API load and resistance factor design, wind turbine analysis, linear wave theory and Morison forces
2. Use USFOS to re-build and modify the FE model of jacket wind turbine given by Aker Solutions, add the soil and pile modeling based on the Ekofisk data. Design the new jacket substructure for 100 meters water depth.
3. Eigenvalue analysis of the entire model including the wind turbine, the tower and the jacket supporting structure, make comparison for the cases with and without the soil-pile model, original 70 meters water depth jacket substructure and new design jacket substructure for 100 meters water depth. The first two or three global modes are focused on.

4. Perform the pushover analysis. Define and analyze the design load case given in the Aker Solutions report. Wind force acting on the rotor will be simplified as a nodal force in 6 DOFs on the top of the tower. Find the most critical load case for the jacket substructure. Determine the failure mode and the location. Determine the redundancy of the jacket by comparison with the design load.
5. Perform the pushover analysis for the jacket substructure with the pile foundation and environmental conditions. Find the most critical load case for the jacket. Determine the failure mode and the location. Determine the redundancy of the jacket by comparison with the design load. Focus on the different response of the substructure due to soil-pile interaction.
6. Apply API LRFD design code check for both of the jacket substructure in different water depth and with or without pile foundation. The utilization ratio for critical cylindrical component of the jacket is focused on. Determine the critical load case and the location. Determine the accuracy of the check by comparison with the results from pushover analysis.
7. Discussions:
   -On the comparison of two ultimate state analysis methods, pushover analysis and API design code check, in terms of global capacity and component strength.
   -On the failure mode of the jacket structure, the response of jacket under design load case.
   -On the effect of the main wind force and wave load direction on the response of jacket
8. Reporting

The work scope could be larger than anticipated. Subject to approval from the supervisor, topics may be deleted from the list above or reduced in extent.

In the thesis the candidate shall present his personal contribution to the resolution of problem within the scope of the thesis work.

Theories and conclusions should be based on mathematical derivations and/or logic reasoning identifying the various steps in the deduction.

The candidate should utilize the existing possibilities for obtaining relevant literature.

The thesis should be organized in a rational manner to give a clear exposition of results, assessments, and conclusions. The text should be brief and to the point, with a clear language. Telegraphic language should be avoided.

The thesis shall contain the following elements: A text defining the scope, preface, list of contents, summary, main body of thesis, conclusions with recommendations for further work, list of symbols and acronyms, reference and (optional) appendices. All figures, tables and equations shall be numerated.

Scope

The supervisor may require that the candidate, in an early stage of the work, present a written plan for the completion of the work. The plan should include a budget for the use of computer and laboratory resources that will be charged to the department. Overruns shall be reported to the supervisor.

The original contribution of the candidate and material taken from other sources shall be clearly defined. Work from other sources shall be properly referenced using an acknowledged referencing system.

The thesis shall be submitted in two copies:

Signed by the candidate

The text defining the scope included

In bound volume(s)

Drawings and/or computer prints which cannot be bound should be organized in a separate folder.

Torgeir Moan

Supervisor

Co-supervisor: Dr. Zhen Gao, CeSOS

Deadline: 14.06.2010

Preface

This report is the result of the Master Thesis for stud. techn. Li Peng at The Norwegian University of Technology and Science (NTNU), Spring 2010.

Prior to the thesis work I have obtained certain knowledge and experience about the original substructure model and non-liner finite element code USFOS since the work herein is the continuation of the Project Thesis in last semester about analysis of OFWT jacket substructure in 70 meters water depth. The new jacket substructure and pile foundation is re-build at the beginning of the work. Further investigation of ultimate limit state of structure on the basis of the previous pushover analysis is assessed. The fully integrated analyses have 7 load cases for the both of the substructure in 70 meters and 100 meters water depth with or without pile foundation, which is time consuming. Therefore, the API design code check is only applied to the API 2A LRFD code by hand calculation since the principle behind different API codes is same.

Better master one than engage with ten. It is very meaningful to concentrate on one item at a time. The ULS design check about the offshore wind turbine has been an interesting and rewarding task. The experience obtained will be very useful in my work situation after graduation. It's a bit regretful I didn't get access to FLS of OFWT due to time limitation. I will strongly recommend this as Master Thesis topic for future Master students.

It's such an honor I could finish my mater under supervision by Prof. Torgeir Moan. During thesis work, the problems were frequently discussed with various persons. I hereby extend my hearty thanks to following people.

- Master Thesis supervisor Prof. Torgeir Moan (NTNU) for guidance and motivation
- Master Thesis co-supervisor Post Dr Gao Zhen(Cesos) patiently answer all my questions
- Prof. Jørgen Amdahl (NTNU) for help with the USFOS
- Post Dr Nilanjan Saha(Cesos) for design of pile foundation

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June 8, 2010

Summary

Offshore wind is an attractive source of renewable energy. In order to harvest this abundant energy source wind turbine farms are needed. Up to now, the fixed wind turbines are applied for water depth 20-30m, and the support structure are typical monopile and tripod structures. In order to extend the application of offshore fixed wind turbine (OFWT) in deep water where winds are stronger and steadier, there are research works on jacket wind turbine which is ongoing on larger water depth like 70-100m. The focus in this thesis is analyses to support design of OFWT jacket substructure piled to the seabed, with a particular focus on the modeling and ultimate capacity behavior of such facilities subjected to extreme wind and wave forces.

During modelling, Genie and USFOS are applied to re-design and modify the finite element model of wind turbine jacket substructure given by Aker Solutions. New design for both of the intermediate water depth 70 meters and 100 meters, add the soil and pile modelling based on the Ekofisk data. Moreover, the eigen period and eigen modes of jackets in both of water depth for cases with and without the soil-pile interaction are estimated with special focuses on the first two or three global modes.

The load and resistance factors from API and DNV codes are applied to convert the characteristic environmental condition and load cases given by Aker Solution to design load cases. Based on design load case, both of the static pushover analysis and API design code check is performed on the jacket model to check the ultimate capacity. The ultimate resistance of the jacket cylindrical component must sustain the design load factor by API design code. Alternatively, the global strength could be assessed from the static pushover analysis where the design loads from the wind or wave are incremented up to complete collapse of the jacket. Inclusion of the results from both of ULS method, the jacket substructures for different water depth including pile foundation could undertake the extreme design loads with certain conservativeness. The response of the jacket substructures under design load case is most interested in, which could indicate the critical position on jacket, furthermore the effect of the direction and size of the wind and wave load. The response of the jacket with pile foundation is another place focus on due to complicated soil-pile interaction in different soil layers. The API design code check and pushover analysis could confirm each other by comparing the response, the hot stops in pushover analysis were also the position with largest utilization ratio.

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1. Introduction

1.1 General

The urgent need to deal with the climate change means that we need to apply renewable resources as soon as possible. Wind power, the important part of renewable energy, as the conversion of wind energy into a useful form of electricity by using wind turbines have got good application on land. However, the disadvantages, like limited available land space, high variation in wind speed, visual disturbance and noise, may limit or slow down the large-scale development of on-land wind turbines. Offshore wind turbines are less obtrusive than turbines on land, as their apparent size and noise is mitigated by distance. Up to now, the fixed wind turbines are applied for water depth 20-30m, and the support structure are typical monopile and tripod structures. In order to extend the application of offshore fixed wind turbine (OFWT) in deep water where winds are stronger and steadier, there are research works which is ongoing on larger water depth like 70-100m. For this increasing water depth, we need stronger support structures like jacket and gravity base to replace the monopile and tripod structure considering the dynamic response on structure. Except the extreme wind loads, considering the increasing water depth, the wave and current loads will increase significantly. The jacket substructure which could provide adequate ultimate strength capacity becomes a good alternative. For the depth more than 100-300m, floating wind turbines might be the only choice. However, more research work and experimental tasks need to be done before the floating concepts become commercially competitive. Therefore, OFWT will still domain in the growth of offshore wind energy in the following several years. In this thesis we will focus on the design and analysis of the OFWT substructure in the intermediate water depth 70 meters to 100 meters. The non linear finite element code USFOS is applied to perform the analysis to inspect the ultimate capacity of the supporting jacket considering the extreme environmental conditions.

1.2 Motivation and Objective for the Study

Both applicability and economical efficiency requires that offshore fixed wind turbines with large-dimension blades go to the deeper water. However, complicated deep water and wind state bring a new problem, that we need more accuracy design and analysis, to maintain the ultimate strength of the offshore wind turbine. Compared with offshore oil and gas platforms, offshore wind turbine substructure requires more ultimate capacity for the extreme wind load. The combination of the extreme wind and wave loads increases the offshore wind turbine failure probability under extreme environmental condition. Furthermore, the jacket substructures are normally piled to the sea floor. Therefore, the ultimate capacity of the substructure with pile foundation is a matter of special importance due to the increasing base shear and overturning moment from wind and wave. The motivation for the initiation of this thesis may be started as the ultimate limit state check for the designed OFWT jacket substructure in 70 meters water depth subjected to extreme wind and wave forces. Applying the design of wind turbines piled to the seabed, with a particular focus on the response of the substructure considering the soil-pile interaction. Moreover, we design and extend the application of the OFWT in 100 meters deeper water depth and perform analysis to verify applicability of the design. Also there is another consideration to establish an in-depth knowledge about ultimate limit state analysis since this work deals ultimate strength of substructure with both of the ULS methods, pushover analysis and API design code check.

1.3 Scope of the Work

In this thesis, integrated analysis is performed by focuses on nonlinear finite element codes USFOS. First, the OFWT substructure for both of the water depth 70 meters and 100 meters are built in GENIE, a helpful analysis computer software components of the SESAM. Afterwards, the models are imported to USFOS with extreme external forces acting on the complete jacket and tower structure to estimate the ultimate capacity. The pushover analysis is performed for models with several environmental condition and load cases from Aker solutions report. Furthermore, the API design code check for the component of the substructure is applied and make the comparison with results from previous pushover analysis. The organization of this work is to establish as follow:

1. Literature reading: ultimate strength analysis, DNV design standard for offshore wind turbines (OS-J101), API load and resistance factor design, wind turbine analysis, linear wave theory and Morison forces
2. Use USFOS to re-build and modify the FE model of jacket wind turbine given by Aker Solutions, add the soil and pile modeling based on the Ekofisk data. Design the new jacket substructure for 100 meters water depth.
3. Eigenvalue analysis of the entire model including the wind turbine, the tower and the jacket supporting structure, make comparison for the cases with and without the soil-pile model, original 70 meters water depth jacket substructure and new design jacket substructure for 100 meters water depth. The first two or three global modes are focused on.
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8. Reporting

2. Ultimate Strength Analysis

2.1 Introduction

The capacity of the offshore structure to yield and redistribute loads can contribute significantly to their ability to tolerate damage and to survive extreme events. Usually this is assessed on the basis of stochastic methods or the design wave approach where focus is placed on the effect from hydrodynamic load. The resistance of the offshore fixed wind turbine (OFWT) substructure to extreme environmental loads including extreme gust and extreme wave is obvious an important safety aspect. However, except the extreme sea condition, for the OFWT substructure, the designer needs to consider further about the extreme wind condition which bring significant load on the substructure. Especially for the OFWT in the shallow and intermediate water depth where the wind is dominant, the response of the substructure depends greatly on the design wind load.

Traditional design of the structures usually relies on component behavior. The ultimate resistance of the component must sustain the design load which factor by the different standards. Alternatively, the global strength may be assessed form the static pushover analysis where the design loads from the wind or wave are incremented up to complete collapse of the structure. In this thesis, both of the global or local analyses are applied to check the ultimate capacity of the OFWT jacket substructure. The global strength of the jacket substructure is assessed first by static pushover analysis. The API design code checking is performed considering the each component strength afterwards.

2.2 Ultimate limit states (ULS)

A limit state is a condition beyond which a structure or structural component will no longer satisfy the design requirements [11] .The following limit states are usually considered in practice:

1) Ultimate limit states (ULS) correspond to the maximum load carrying resistance

2) Fatigue limit states (FLS) correspond to failure due to the effect of cyclic loading

3) Accidental limit state (ALS) correspond to damage to components due to an accidental event or operational failure.

In this section, we focus on the ultimate capacity of the offshore fixed wind turbine support structure. Therefore, the FLS and ALS are not addressed herein. For failure of structure under ultimate load, there are several practical examples [11]:

1) Loss of structural resistance

2) Failure of components due to brittle fracture

3) Loss of static equilibrium of the structure

4) Failure of critical components of the structure caused by exceeding the ultimate resistance or the ultimate deformation of the components.

5) Transformation of the structure into a mechanism

There are several criteria related to the ULS. The occurrence of first yielding at highest stressed point in a structural component is often employed as a measure of structural capacity [12]. For design analysis of offshore fixed wind turbine in this thesis, generally, failure takes place as the loss of structural resistance due to excessive yielding or buckling of the component. Correspondingly, if the component check is not full filled the ULS, yielding or buckling happen at the design load, it is assumed that the structure is no fit for the purpose. Thus, the ultimate strength capacity of structural elements performing in yielding and buckling shall be assessed using a rational and justifiable engineering approach.

However, many components are redundant. For example, they are capable of redistributing stresses and loads over a cross section when some part starts to yield [12]. In such case, first yielding is a conservative criterion for the ULS of structure system. Redistribution of the loads may also take place within components causing the residual strength. The jacket is complicated steel system, the reserve capacities might also due to system effects.

For reserve capacity, the ultimate resistance of structure should be assessed as the maximum load the structure could sustain. The residual capacity check shall consider both post yielding and buckling. The structural design criteria to prevent the ULS are based on plastic collapse. There is also a simplified ULS design rely on estimation of the ultimate strength of critical component, usually from their elastic buckling strength adjusted by a simple plasticity correction. This is represented by the Figure 2.1.

The buckling strength is represented by the point A in Figure 2.1. Based on the strength at point A, the designer doesn't need to consider the detail information of post-buckling behavior. The true ultimate strength is represented by the point B. This graph also shows the typical load-deformation relationship for the jacket structure under pushover analysis which will be certificated later in the pushover analysis of practical problems.

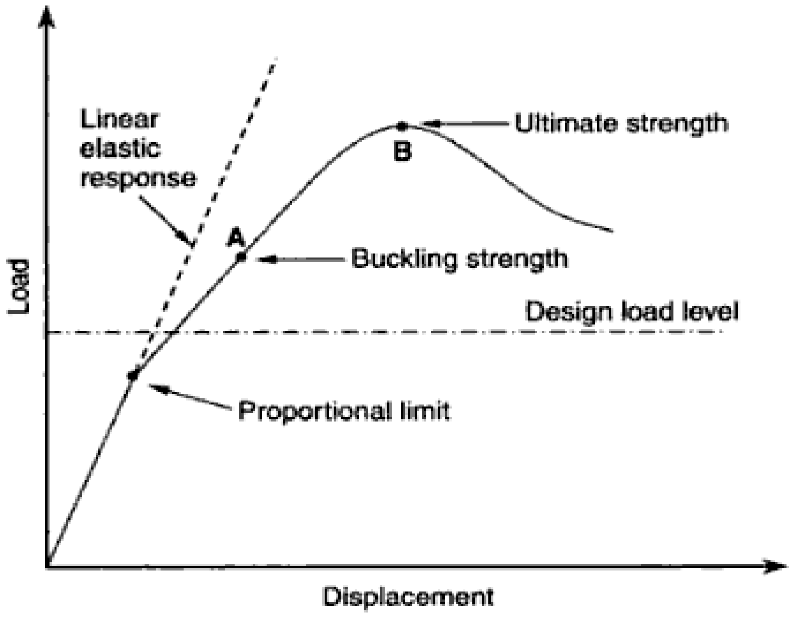

Figure 2.1 Buckling strength in load-displacement relationship diagram

2.3 Safety Factor method

The partial safety factor method is a design method by which the target safety level is obtained as closely as possible by applying load and resistance factors to characteristic values of the governing variables and subsequently fulfilling a specified design criterion expressed in terms of these factors and these characteristic values [11].

To satisfy the ultimate limit state, the structure must not collapse when sustain to the peak design load which is the criterion for design. A structure is considered to satisfy the ultimate limit state criteria if all factored bending, shear and tensile or compressive stresses are below the factored resistance calculated for the section under consideration. Safety factor is used for the external loads and reduction factor for the resistance of members. The safety level of a structure or a structural component is considered to be satisfactory when the design load effect S_d does not exceed the design resistanceR_d, the corresponding equation $S_d \leq R_d$ is the design criteria [11]. The design criterion is also known as the design inequality. The corresponding equation $S_d = R_d$ forms the design equation. The design load and design resistance generated from the characteristic load and material resistance [11]. The good design is deemed as fulfill the design equation precisely.

2.4 Design Load and Design Resistance

The variables is governed by the design factor method consist of the design loads acting on the structure or effects in the structure and design resistance of the structure or strength of the materials in the structure. In this thesis, the design load effect S_{di} is obtained from a structural analysis for the particular design loads F_{di}, where the design load action F_{di} is obtained by multiplication of the characteristic load F_{ki} by a specified load factor γ_{fi}. According to the partial safety factor format, the design load effect S_d resulting from simultaneous occurrence of n independent loads F_i, i = 1….n, can be taken as

$$S_d = \sum_{i=1}^{n} S_{di}\,(F_{di})$$

, where $S_{di}\,(F_{di})$ denotes the design load effect corresponding to the characteristic load F_{ki}. The resistance R_d against a particular load effect S is, in general, a function of parameters such as geometry, material properties, environment, and load effects themselves, the latter through interaction effects such as degradation. The approaches to establish the design resistance R_d is to divide the characteristic R_k by a specified material factor γ_m that $R_d = \frac{R_k}{\gamma_m}$[11]. In practical analysis, the design resistance is certificated by load ratio from pushover analysis or utilization ratio from API design coed checking which will be discussed in next paragraph.

2.5 Different Code for ULS

In this thesis, we apply two different design codes for the static pushover analysis and design code check respectively. The original design of the 70 meters water depth OFWT substructure is based on the DNV Offshore Standard DNV-OS-J101, Design of Offshore Wind Turbine Structures. This offshore standard provides principles, technical requirements and guidance for design, construction and in service inspection of offshore wind turbine structures. Therefore, we apply the factor from this code to perform initial pushover analysis, to consider the global ultimate capacity of the substructure. Because of our focus on the response of the jacket, the critical part of the substructure, the code check for the critical position of the jacket is introduced based on API 2A-LRFD, Recommended Practice for Planning, Designing and Constructing Fixed Offshore Platforms, which is the generally follows guidelines during design of the jacket substructure. The factors introduced in the both of the rules are tabulated as follow.

Code	Load type	Factor
API LRFD	Dead Load	1.3
(100 years Return period at Gulf of Mexico)	Evironmental Load	1.35
	Pile resistance factor	0.8
	Pile resistance factor	0.7
DNV-OS-J101	Dead Load	1.1
(100 year Return period)	Evironmental Load	1.35
	Axial pile material loads	1.25
	Lateral pile material loads	1.15
	Lateral pile total stress	1.25

Table 2.1 Load and resistance factors from API and DNV code

3. Design and Modeling of Substructure

3.1 Current Offshore Wind Design

3.1.1 Current Offshore Wind Energy Status

Offshore wind farms are common and growing fast in Europe. From 1992 to 2005, the global installed wind capacity increased from 2500 MW to 59200 MW. This corresponds to a yearly increase of 30 percent. More than 75 percent of the new wind capacity is installed in Europe. The capacity will grow from 577 MW in 2009 to 1000 MW within 2010 [1] .Over 100 GW of offshore wind power is currently being planned by European utilities, developers, and governments, mostly in the North Sea. Denmark, The Netherlands, and the United Kingdom all have several active installations. Especially Denmark and the United Kingdom have installed large offshore wind facilities to take advantage of consistent winds. Many offshore wind farms are under construction and the largest of these is the 500 MW Greater Gabbard wind farm in the UK (Figure 3.1). New offshore wind farms which are proposed include the 1,500 MW Atlantic Array and the 1,000 MW London Array, both in the UK.Once operational this 100 GW plus would supply 10% of Europe’s electricity [2]. Norway also has a lot of potential with respect to wind power and this can be used in the production of electricity. By the end of 2009, there was installed 420 MW of wind power in Norway, producing slitghly more than 1 TWh a year, or slightly less of 1 percent of the total electricity production in Norway[16]. Statoil, GE and Lyse plan to build two to four demonstration turbines, each with a capacity of around 4 megawatts. The official target for Norway is a production of 3 TWh annually by 2010, which can be achieved by the installation of approximately 1000MW of wind power capacity. The potential for wind power in Norway is however much greater than this and totally there is applied for 66 TWh with wind power in Norway.Considering the technology today, almost all the existed wind farms are in shallow waters which is less than 30 meters water depth, off the coasts of Europe. However, the performance of wind-farms improves greatly in deep waters, where winds are stronger and steadier. There are several new deepwater concepts with advanced technology. Jacket foundation has been proposed several years for the intermediate water depth project in New Jersey and Rhode Island. The jacket foundation technology has been licensed from OWEC Tower AS and has been deployed at the Beatrice offshore wind project in Scotland. Offshore Wind Power Systems of Texas has designed the Mobile Self-Installing Platform (MSIP), a three legged platform able to be towed out to sea and lowered into place [2]. The New deep-water, floating-turbine technologies are alternative choice which only recently beginning to be deployed in Norway. The first large-capacity floating wind turbine is the Hywind, a 2.3 MW turbine in 220-meter deep water in the North Sea, which became operational in September, 2009. In May 2009, consultancy Frost

& Sullivan estimated that installed capacity of offshore wind power would grow to 18,769 MW by 2015. Figure 3.2 graphically shows the typically offshore wind design nowadays and prospect in future.


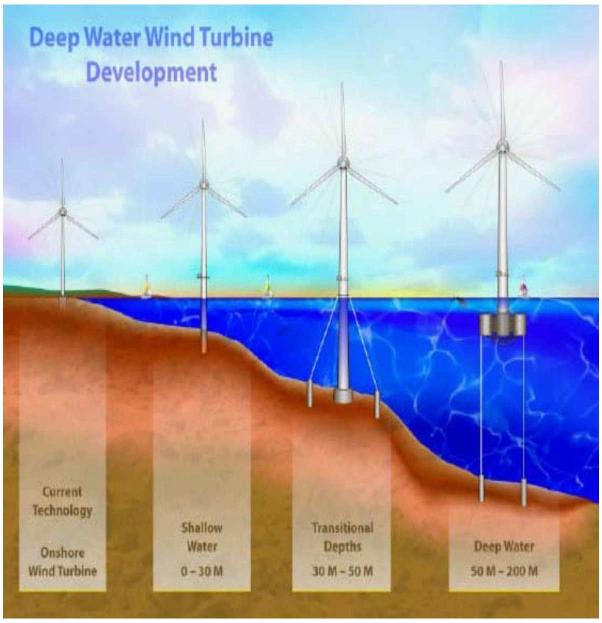

Figure 3.1 500 MW Greater Gabbard wind farm Figure3.2Typically offshore wind design nowadays

3.1.2 Substructure of Fixed Offshore Wind Turbine

The floating concepts have huge potential. However, it is extremely expensive to build and there is a key technology challenge we going to face. The fixed offshore is therefore the optimality option considering the economics and existent experience. Many of the proposed concepts utilize designs borrowed from the oil and gas industry. The common foundations used for current fixed offshore wind projects are shown in the Figure 3.3 below

1. Monopile: Consists of a steel pile which is driven approximately 10 to 20 meters into the seabed.
2. Gravity foundation: Currently used on most offshore wind projects, the gravity foundation consists of a large base constructed from either concrete or steel which rests on the seabed. The turbine is dependent on gravity to remain erect.
3. Tripod foundation: Designs tend to rely on technology used by the oil and gas industry. The piles on each end are typically driven 10 to 20 meters into the seabed, depending on soil conditions. This technology is generally used at deeper depths and has not been used on many projects to date.

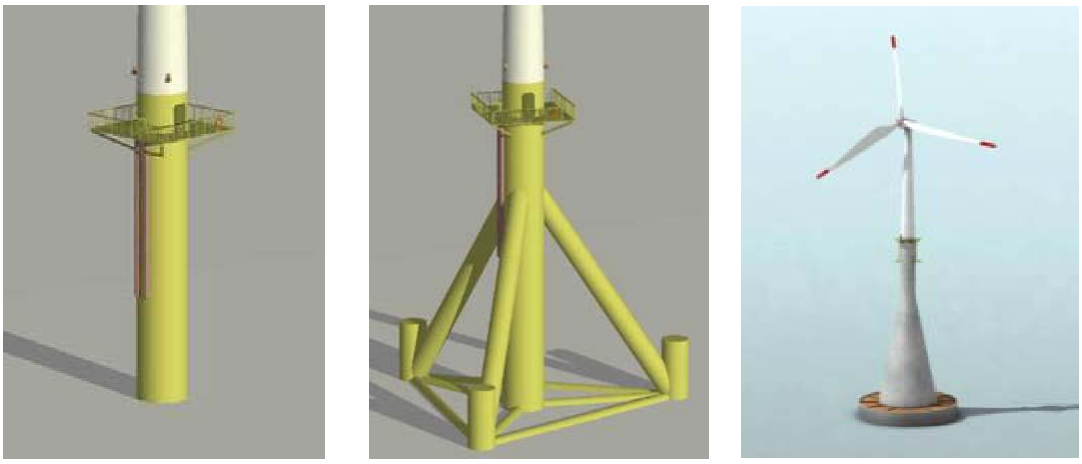

Figure 3.3 Foundations used for current fixed offshore wind turbine in shallow water

(Source: NREL)

4. Jacket foundation: These platforms are built on concrete and steel legs anchored directly onto the seabed, supporting a deck with space for interface and turbine tower. Compared with other substructure, Jacket support substructure has the potential to operate in the deep water with good applicability and economical efficiency.

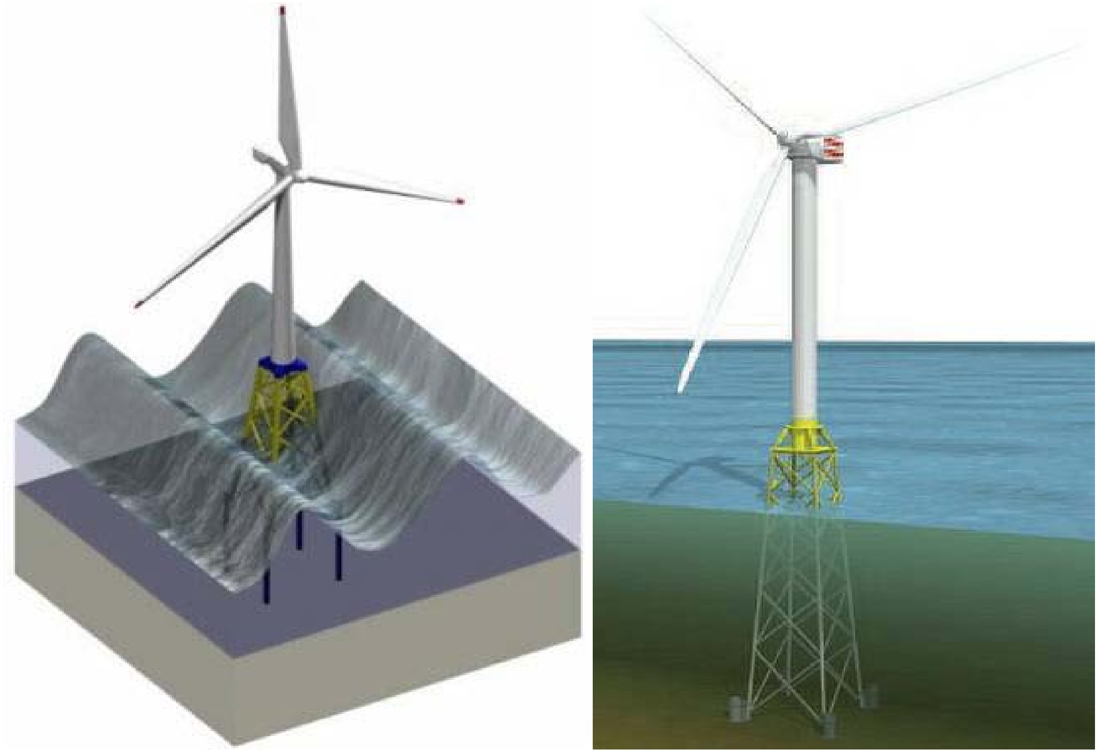

Figure 3.4 Fixed offshore wind turbine with jacket foundation (Source: DWIA)

Different offshore fixed wind turbine substructure has its particular strength and allowable weakness. Such as monopole which is advantages in shallow water depth because of fabrication and installation simplicity. However, with the increasing water depth, the dynamic loads due to larger current speed and wave height require more foundation design with large ultimate capacity. The economical efficiency is also important reason when the design going to the deep water. There are some proposed deep water designs about offshore fixed wind turbine. Such as

1. Tripod tower
2. Guyed monopile
3. Full-height jacket
4. Submerged jacket with transition to tube tower
5. Enhanced suction bucket or gravity base

These designs have been applicable for the water depth up to 60 meters in the offshore oil and gas industry. What we are interested in is the practical and feasible application of the full height jacket supporting substructure. Therefore, in this thesis, we will focus on the design and analysis of the OFWT jacket substructure to investigate the possibility and potential of the OFWT jacket substructure.

3.2 70 Meters Water-depth OFWT Jacket Description

3.2.1 General Investigation

For the offshore fixed jacket wind turbine, the main substructure that contains the tower and foundation which accounts for 25% of total wind farm costs. Therefore the design of sub structure became a main factor for the total project. In this chapter, the applied support jacket foundation is designed and modeled for the both of the water depth 70m and 100m.The original 70 meters water depth substructure was designed by Aker Solutions for Ekofisk oil field of Norway. The average working depth in this place is 70 meters to 75meters [3].The site of Ekofisk oil field is at the coordinate (Latitude: 56° 32' 60 N, Longitude: 3° 4' 60 E) shown in Figure 3.5. Figure 3.6 is the side view and front view of the designed OFWT substructure for 70 meters water depth including wind turbine, tubular tower, interface and corresponding jacket supporting substructure. Also, the sketch of the 3D perspective is attached in APPENDIX A, so we can get some visual experience about the OFWT substructure in this thesis.

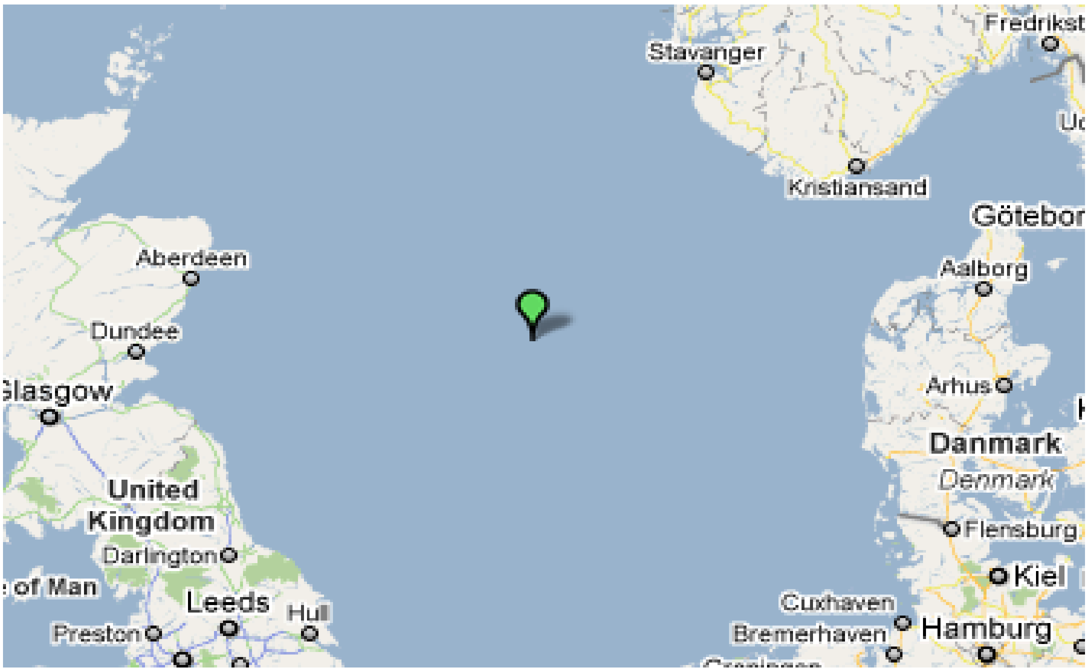

Figure 3.5 Ekofisk oil field of Norway coordinate (Source: Google earth)

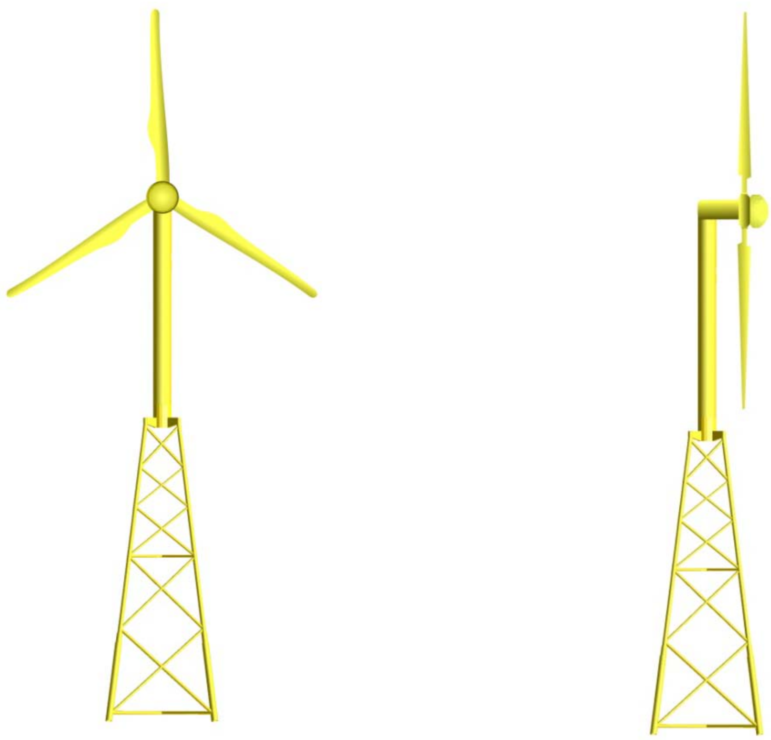

Figure 3.6 Front view and side view of the OFWT for 70 meters water depth

Figure 3.7is the complete substructure neglecting wind turbine applied in this thesis. A rough sketch on the left including a tubular tower, middle section and jacket foundation with piles, designed and certified as it will be finally installed at the seabed. The interface and pile foundation are magnified graphed on the right. It designed based on potentially supporting heavy turbines such as today is 5 MW NREL turbine or higher power turbine in 70 meters of water depth for energy production in extremely difficult areas and demanding soil conditions.

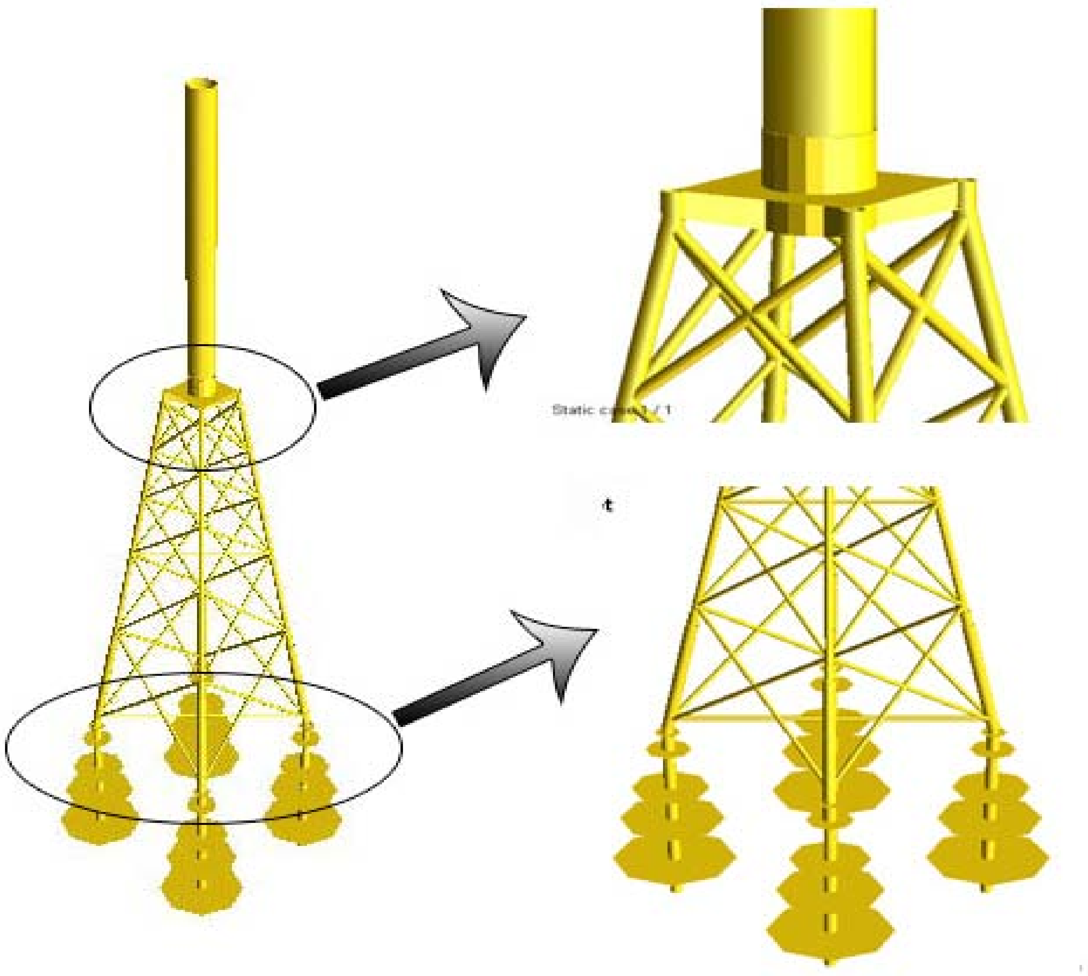

Figure 3.7 Jacket and tower model with magnified interface and soil-pile model

3.2.2 FEM Model

The USFOS analysis module is a finite element program based on an updated Lagrangian formulation. For the jacket substructure, the basic structural unit used in USFOS is the two-node beam. It is used to model an entire structural member; beams as well as beam columns. [7] Consequently, the OFWT substructure is modeled by 1286 elements. The large jacket system is modeled by 492 beam elements. Corresponding to interface, there are 794 shell elements indicated with quadrilateral shell element and triangular shell element. The pile foundations are modeled by equivalent elements. The equivalent stiffness of the piles is computed and used. Soil-structure interaction is modeled by linear springs in related to the soil condition and dimension of the pile foundation. Figure3.13 shows the major finite element meshed for the jacket and interface.

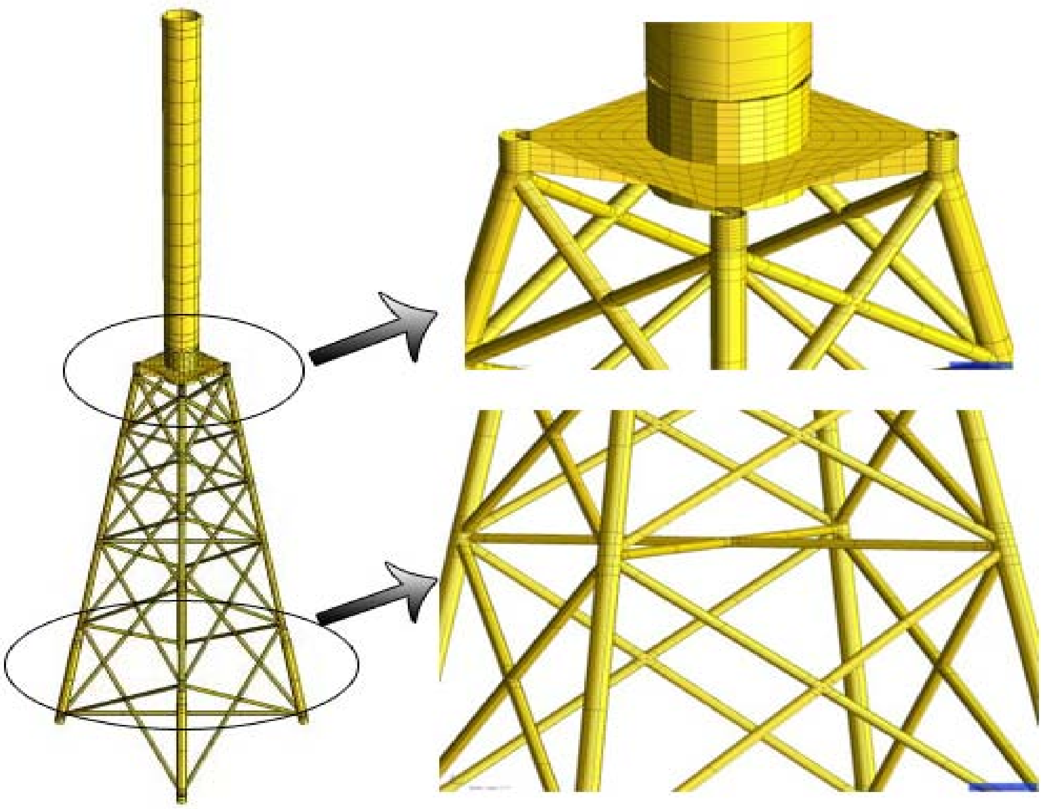

Figure 3.8 Finite element model of OFWT jacket

The basic philosophy behind USFOS is to use a very coarse finite element mesh, and still obtain reliable and accurate results. USFOS requires only one finite element per physical element of the structure. [8] Due to the small amount of the elements, the USFOS could perform the collapse analysis efficiently for space frame structures with allowable accuracy.

3.2.3 Material

The wave and current loads are acting on the jacket substructure with the moments from the wind force. In this thesis we focus on the design and analysis of the jacket, therefore we neglect the plastic deformation of the tubular tower. Elastic material is similarity applied for the tower column. The material of both of jacket and interface are plastic with yield strength. That means the plastic deformation will happen when sufficient load is applied to the substructure. Considering the high compressive stresses could happen in the jacket beam elements, the buckling failure mode is also taken into account. Table 3.1 shows the material character of the tower, interface and jacket. The description of the Nomenclature is attached below table.

Place	Element	E-mod N/m²	Poisson	Yield N	Density Kg/m3	ThermX
Jacket	Beam	2.10E+11	3.00E-01	4.20E+08	12670	1.200E-05
Interface	Shell	2.10E+11	3.00E-01	4.20E+08	7850	1.200E-05
Tower	Beam	2.10E+11	3.00E-01	Infinite	7850	1.200E-05

Table 3.1 Material of the Structure

The parameter description is shown as follow [6]:
E-Mod: Modulus of elasticity
Poiss: Poissons ratio
Yield: Yield stress (Relevant for plastic material)
Density: Material Density
ThermX: Thermal Expansion Coefficient

3.2.4 Dimensions

This jacket structure fixed at the base for water depth of 70meters and the heights of jacket and tower are 89.5meters and 63.5 meters, respectively. The dimension of the jacket foundation on the sea bed is 32 meters square with pile battered on each corner of jacket. The dimension is decided during the design of substructure carried out by Aker solution. In order to make the rigidity connection in USFOS, several theoretical rigidity beams has been added at the connection between tubular tower and interface, the legs and interface respectively. Since the dimensions and mass of the theoretical rigid beams are very small, we could neglect the rigid connection beam during analysis. Therefore in this section, we don’t cover the detail of the rigid beam. The Figure 3.9 shows the main dimension of the 70 meters water-depth substructure including jacket interface foundation at sea bed and tubular tower.

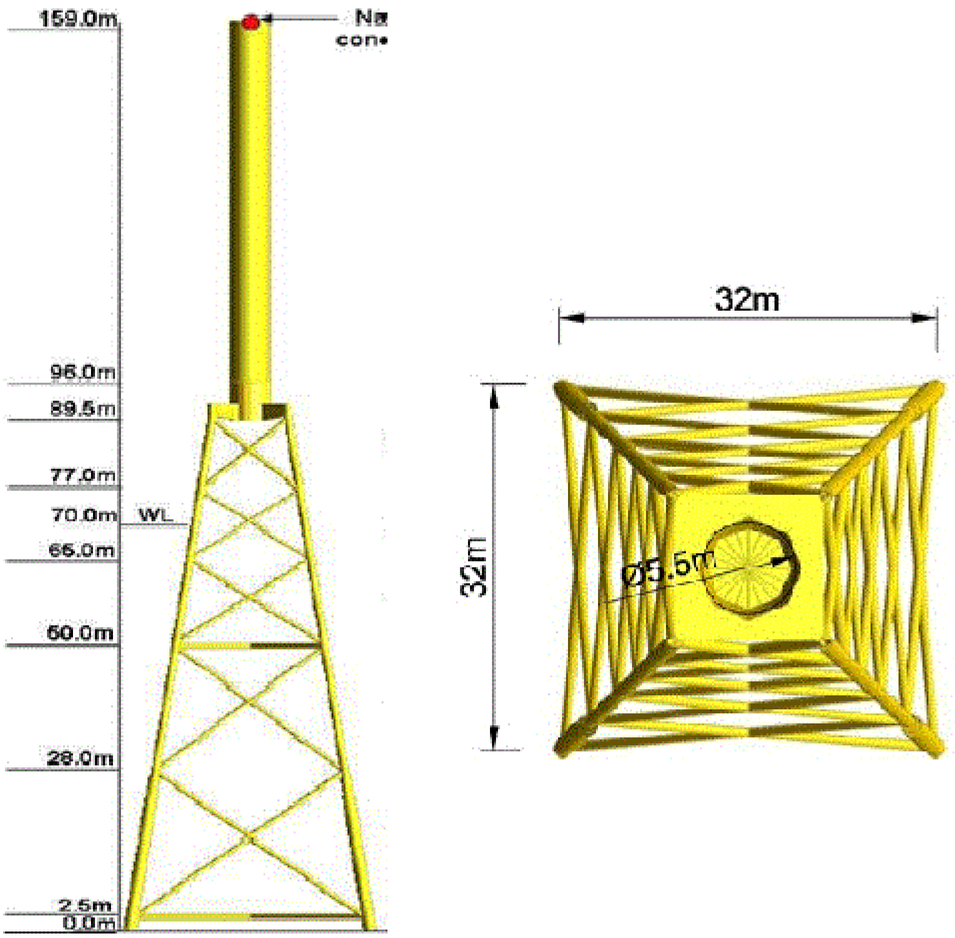

Figure 3.9Main Dimension of the 70m water-depth jacket substructure

The jacket is composed of five layers of vertical braces and two horizontal braces at 2.5 meters and 50 meters respectively. The horizontal bracing are not required as top plate and foundation provides adequate rigidity. Tower structure is modeled as pipe structures with constant diameter of 5.5 meters. From top to bottom, the thickness of the tubular tower increase from 0.022 to 0.034 meter. The height of the interface is 6 meters between the tower and jacket. Leg diameters range from 1.8 meters with thickness 0.04 meter at foundation level to 1.17 meters with thickness 0.03 meter at elevation of interface. Vertical braces diameters range from 0.8 to 0.6 meter with constant thickness 0.02 meter from the first storey to interface. Horizontal brace diameter dimension starts from 0.7 to 0.95 meter with the thickness 0.02 and 0.025 meter respectively. The larger dimension brace located on the first storey corresponds to the smaller on the third storey. The Figure3.9 and Figure 3.10 shows the main dimension of legs and braces. The color represents the different thickness of tubular elements.

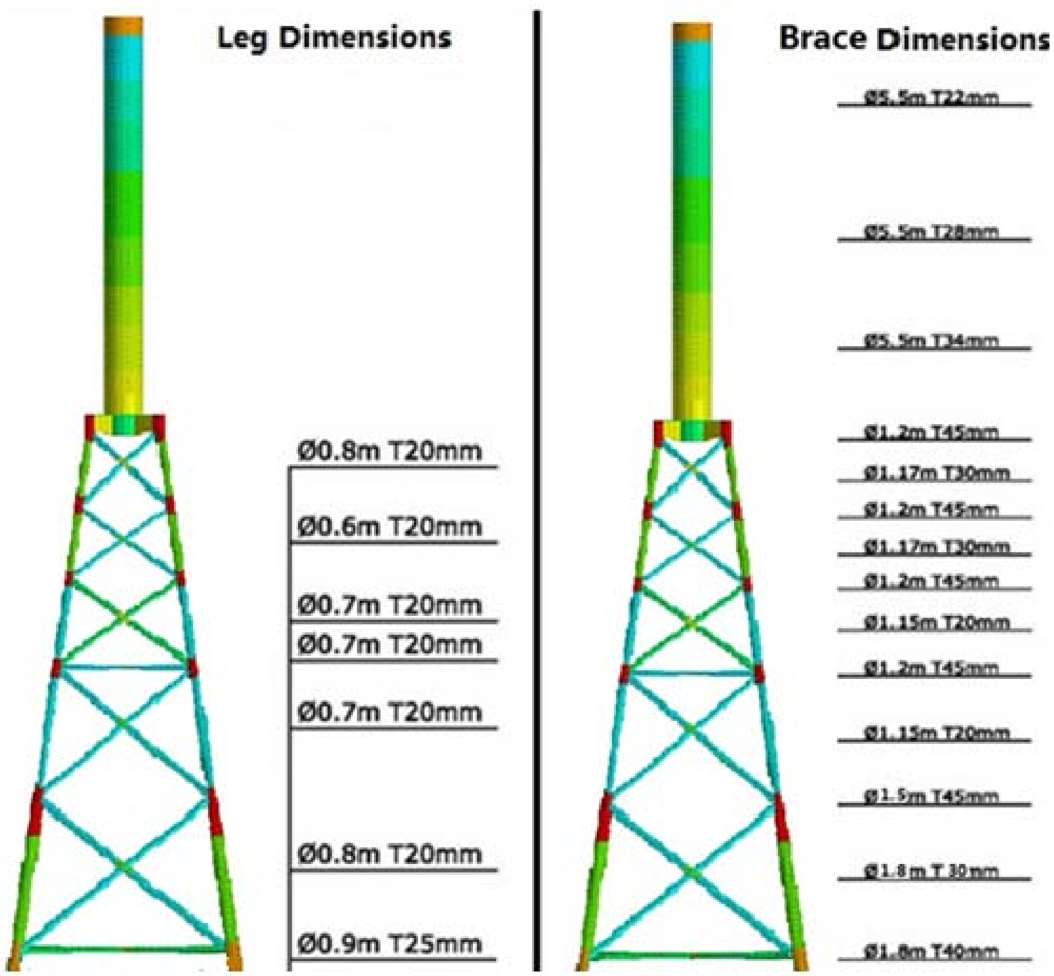

Figure 3.10 70 meters water-depth OFWT jacket legs and braces dimension description

3.2.5 70 Meters Water-depth Fixed End Jacket Eigen Mode

A key feature of the offshore structure is the resonance associated with the certain eigen period and the corresponding eigen mode. At the eigen period the substructure oscillate most easily which will display significant resonance, this would bring great damage to the substructure especially during long time operation. Some of the eigen mode has strong effect on the bottom of substructure will damage the foundation. To avoid the maximum dynamic stress from the maximum vibration velocity, designer need to ensure the model eigen period come as far as possible to those environmental period. Therefore the eigen period of the 70 meters water-depth jacket substructure is calculated and tabulated below:

Eigen Mode	First	Second	Third	Fourth	Fifth	Sixth
70m Jacket Eigen Period	2.86923	2.8635	0.630274	0.593213	0.588716	0.573928

Table 3.2 Eigen Value of the 70meters and 100meters water-depth OFWT fixed bottom jacket

We observe that the first eigen period is approximately 2.8 seconds, which is far less than typical wave period. That means for the normal wave load, the hydrodynamic vibration rarely occur. However, we still need to consider the eigen mode of the jacket to understand the possible vibration movement. The majority of the response depends on the global eigen mode of the jacket, therefore in this paper we focus on the global eigen mode and make comparison between different models. The Figure 3.11 below shows the first to third eigen mode movement of the supporting jacket. The Figure 3.12 shows the fourth to sixth Eigen mode movement of the supporting substructure. The red color in eigen period table represent the global eigen mode corresponding eigen period. The jacket oscillates globally during first and second, fourth and fifth eigen period.

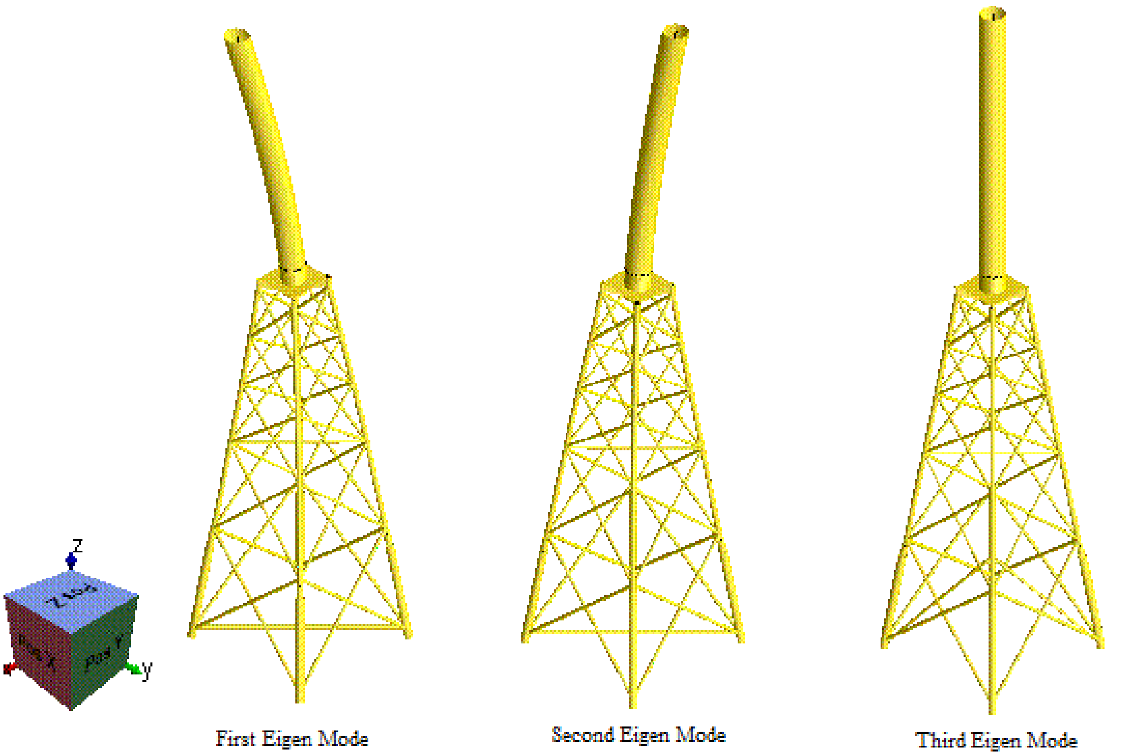

Figure 3.11 70 meters water depth fixed OFWT jacket's oscillation in first to third eigen modes.

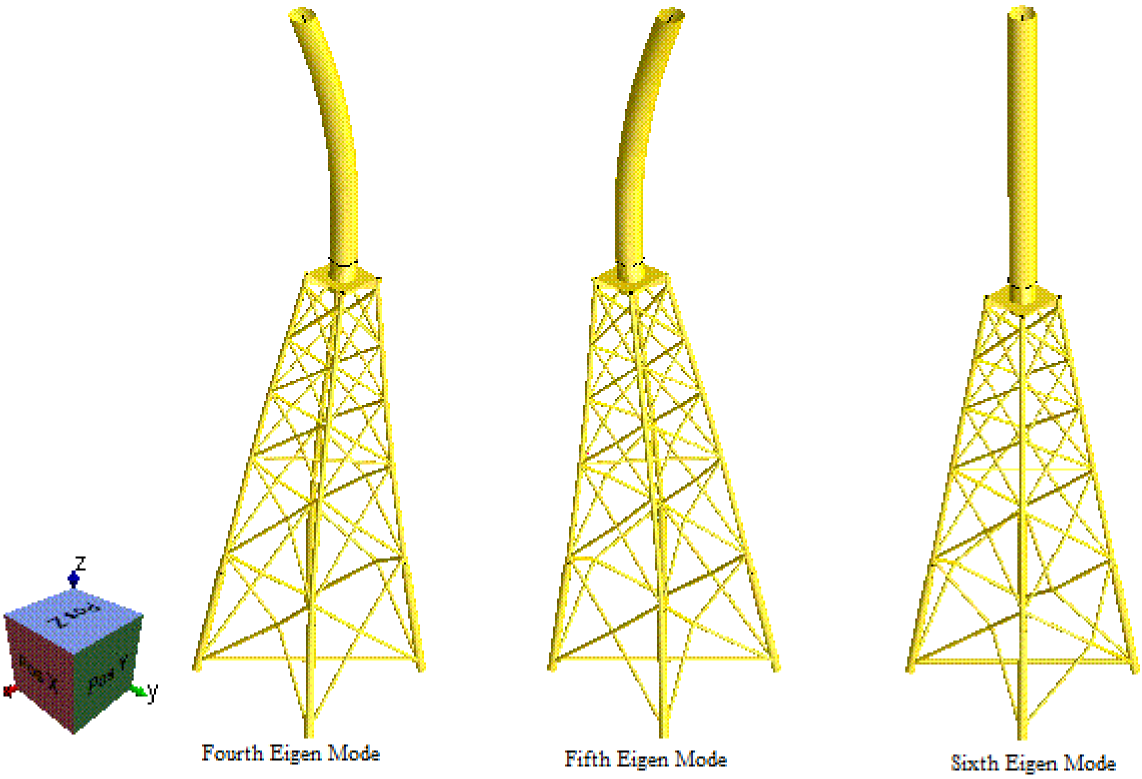

Figure 3.12 70 meters water depth OFWT fixed bottom jacket's oscillation in fourth to sixth eigen modes.

We observed that the jacket is globally oscillating in the first eigen period, the tubular tower move in the x direction with the jacket keeps stationary. The global oscillation of the second eigen mode is as same as first one in the y direction. The fourth and fifth eigen mode of the substructure are global bending. The different between the first and fourth global eigen mode is that for the later one, the jacket oscillating with the bending of the tubular tower. Except for direction, the fourth and fifth global eigen modes are same. In contrary, corresponding to the third eigen mode, the oscillation is locally located on the horizontal braces on the first storey. The horizontal braces vibrate up and down with the other components of the jacket keeps stationary. The local oscillation of the vertical braces in the first storey is the eigen mode during sixth eigen period. The movement of the braces indicates that the oscillation of opposite braces at same storey is section symmetry with the opposite direction of motion.

3.2.6 Wind Turbine

The NREL 5MW wind turbine is selected to operate on the top of the steel tower which is supported by steel jacket substructure in this thesis. The properties of NREL wind turbine are tabulated in Table 3.3. The graph next to it is the drift of the nacelle.

Rating	5MW
Rotor orientation Upwind	3 blades
Cut-In Wind Speed	3m/s
Rated Wind Speed	11.4m/s
Cut-Out Wind Speed	25m/s
Rotor / Hub diameter	126m / 3m
Concentrated Mass at Top-Nacelle	295 M kg
Concentrated Mass at Top-Turbine	115 M kg
Blade Diameter	126m

Table 3.3 Properties of NREL wind turbine

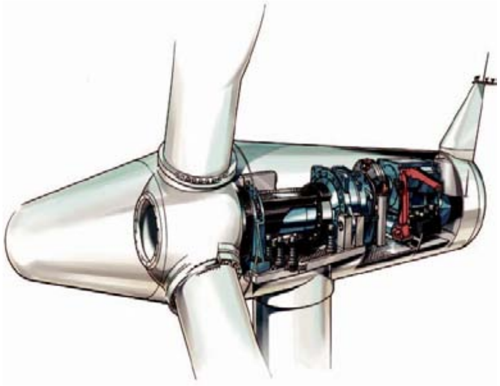

Figure 3.13 Draft of NREL Nacelle

Wind turbine could endure a lot of changes, including extreme wind speed and direction, extreme wind gusts etc. during their lifetime. This thesis is focused on the design and analysis of the jacket substructure, so we don't make in-depth research on the turbine technology. The point we interested in is the mass of the nacelle and loading reversals due to their own weight. In analysis, the nacelle is represented as a point mass on the top of the tower. Meanwhile, the length of the blade will affect the height of the tubular tower.

3.3 Design of the 100 Meters Water-depth OFWT Jacket

3.3.1 General Description

The precious work focused on the analysis and development of wind turbines in water-depths 70 meters. Actually the intermediate water-depth involves water-depth from 50 to 100 meters. In order to extend the application of offshore wind turbine in deeper water, the jacket foundation is redesigned. Design is based on water depth of 100 meters and the heights of jacket and tower are 89.5 and 93.5 meters respectively which extended from the original 70 meters water-depth jacket. Also the design requirements and parameters must be defined before the design phase started. For further details, the API RP 2A LRFD [14] is the reference which provides the requirements. In order to increase the height of the jacket, one storey is adding on the bottom of the previous jacket with the height 30 meters. The legs of the jacket are elongate in the same direction to make the part of the whole. For the commonly encountered case of main piles located inside the jacket legs, leg inside diameter is sized to accommodate pile driving and grouting operation [10]. Since the leg diameter increase from top to the bottom, the diameter of the adding legs connected to the old bottom is slightly larger than the upper jacket. Jacket leg wall thickness is sized to resist the axial force and bending stresses and deformations exerted by intersecting braces [10]. The jacket bracing patterns adopt the old 70 meters fully X-braced pattern which provides the high horizontal stiffness, ductility, and redundancy. The selection of the bracing size is depends on the pipe diameter to thickness ratio of members, which recommended in API RP 2A LRFD.

3.3.2 Dimensions

Figure3.15 is the complete substructure with main dimensions designed for 100 meters water-depth including a tubular tower, middle section and jacket foundation compared with original 70 meters jacket.

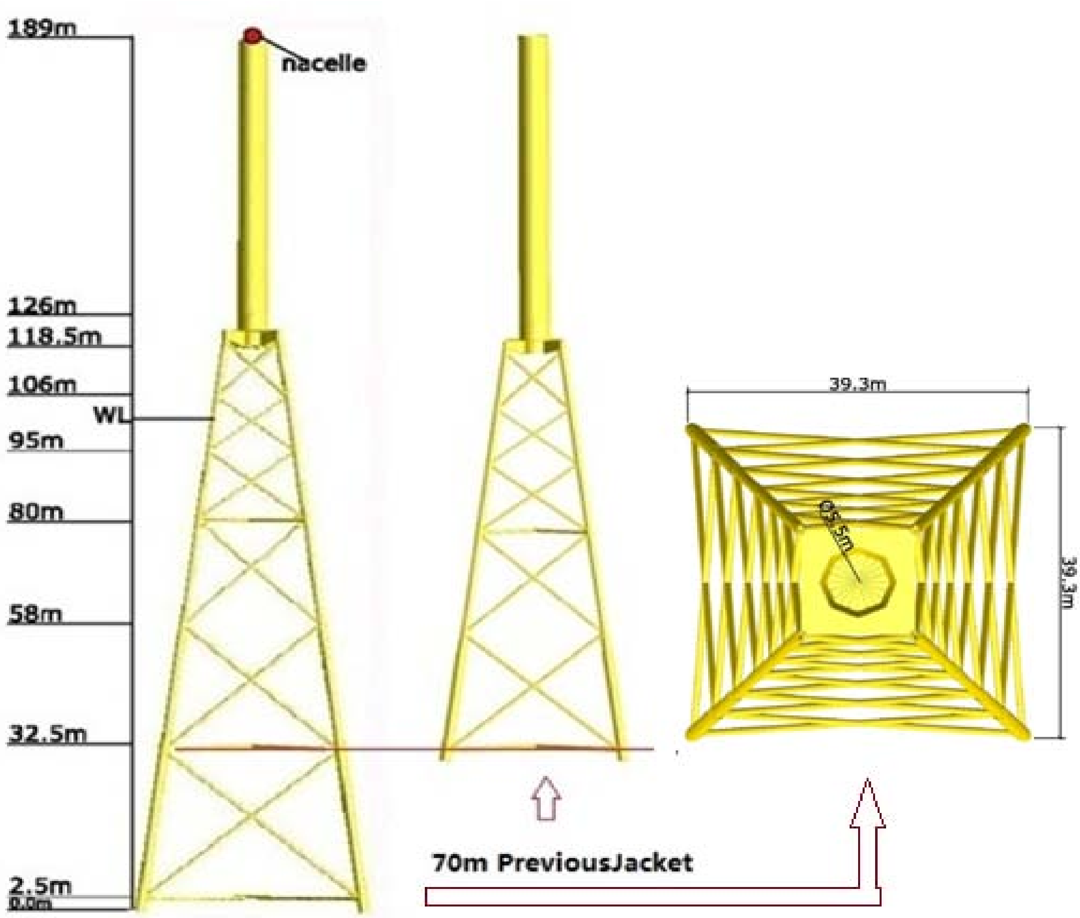

Figure 3.14100 meters water-depth jacket main dimension descriptions

The jacket is extended from water-depth70 meters to 100 meters, the dimensions of the leg and braces adjust accordingly in accordance with the requirement mentioned in last section. The dimension of the jacket foundation on the sea bed is 39.3 meters square. The jacket is composed of six layers of vertical braces and two horizontal braces at 2.5 meters and 80 meters respectively. The horizontal bracing are not required as top plate and foundation provides adequate rigidity. The graph 3.16 and 3.17 shows the main dimension of legs and braces. The color represents the different thickness of tubular elements.

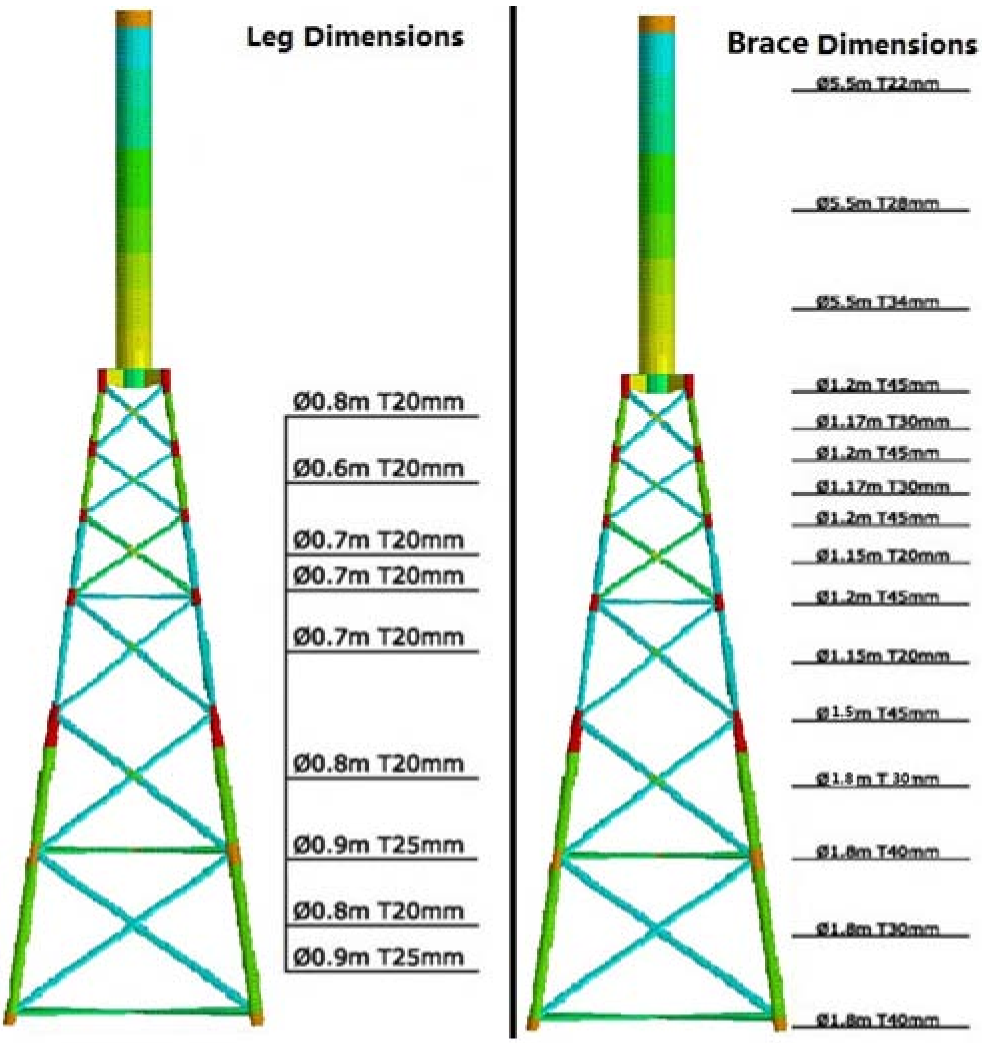

Figure 3.15 100 meters water-depth OFWT jacket legs and braces dimension description

Leg diameters range from 1.8 meters with thickness 0.04 meter at foundation level to 1.17 meters with thickness 0.03 meter at elevation of interface. Tower structure is modeled as pipe structures with constant diameter of 5.5 meters. From top to bottom, the thickness of the tubular tower increase from 0.022 to 0.034 meter. The height of the interface is 6 meters between the tower and jacket. Vertical braces diameters range from 0.8 meter to 0.6 meter the dimension 0.8 meter with constant thickness 0.02 meter from the first storey to interface. Horizontal brace diameter dimension starts from 0.7 to 0.95 meter with the thickness 0.02 and 0.025 meter respectively. The larger dimension brace located on the first and second storey corresponding to the smaller on the fifth storey.

3.3.3 100 Meters Water-depth Fixed End Jacket Eigen Mode

Eigen value analysis was done for the designed 100 meters water depth jacket substructure to find the mode shapes and the eigen period of structure. The results are compared with the 70 meters water depth jacket foundation and tabulated below.

Eigen Mode	First	Second	Third	Fourth	Fifth	Sixth
100m Jacket Eigen Period	2.94885	2.94342	0.976301	0.821159	0.795772	0.795656
70m Jacket Eigen Period	2.86923	2.8635	0.630274	0.593213	0.588716	0.573928

Table 3.4 Eigen Value of the 70meters and 100meters water-depth OFWT fixed bottom Jacket

Table 3.4 shows the eigen period of both of OFWT jacket substructures in 70 meters and 100 meters water depth. There is 25% difference between third eigen period for both of substructures. However, the eigen period is still small which rarely happens in the environmental condition. The red color represent the eigen period with corresponding global eigen mode. The Figure 3.16 shows the first to third mode shapes of the 100 meters water depth supporting jacket. We observed that the movement of jacket is global, mainly in the tower with the jacket keep stationary in the first and second eigen modes which have different direction. Figure 3.17 shows the fourth to sixth eigen mode shape of the 100 meters water depth supporting jacket. The sixth and fifth eigen mode of the substructure are global bending. The difference between the fifth and first global eigen mode is that the jacket oscillate with the bending of the tubular tower. Compared with eigen mode shapes of 70 meters water depth jacket in third to fifth eigen period, we observe that the sequence and shapes of the global eigen mode remain the same. In similar, the position of the local movement remains at the first storey of the jacket. Corresponding to the third and fourth eigen mode, the oscillation is locally locate on the horizontal and vertical braces on the first storey. The third eigen mode is the horizontal brace vibrate up and down. The fourth eigen mode is the horizontal vibration of the horizontal braces in the different direction.

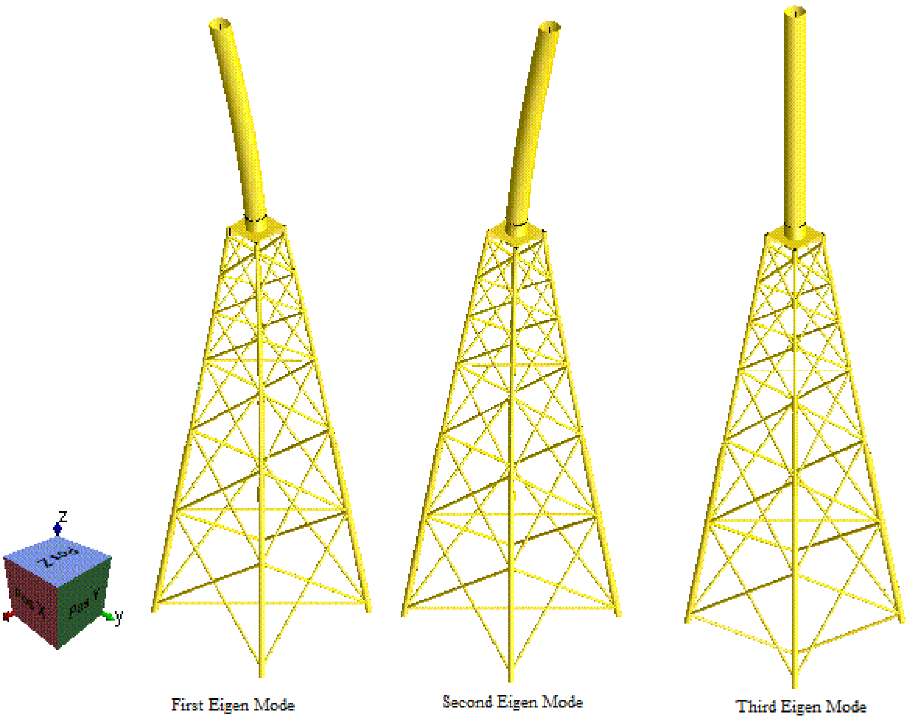

Figure 3.16100 meters water-depth OFWT fixed bottom jacket's oscillation in first to third eigen modes.

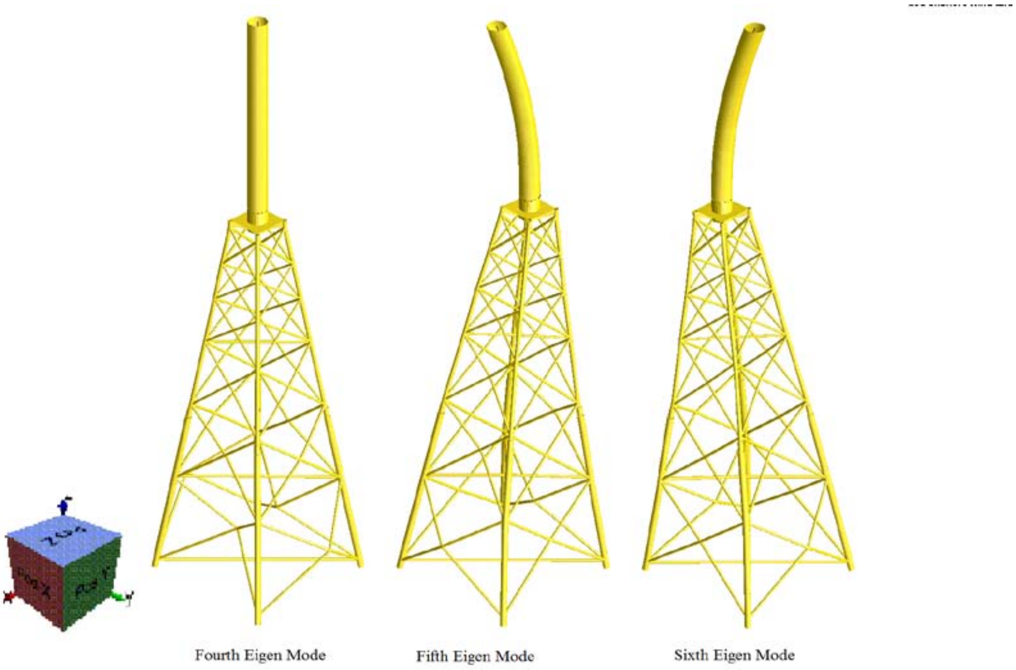

Figure 3.17100 meters water-depth OFWT fixed bottom jacket's oscillation in fourth to fifth eigen modes.

3.3.4 Others

The material of the redesigned jacket remains the same as original 70 meters water depth jacket foundation. Because of small environmental condition modified for the increasing 30 meters water depth, for100 meters jacket substructure we adopt the same NREL turbine and corresponding characteristic wind load case. However, with the increasing water depth, the environmental loadings lead to greater forces, especially the bending moment, in the jacet substructure. It is necessary to ensure the adequacy and efficiency of the design. Therefore, the ultimate capacity analysis and code check will be applied after the design activities. This is going to be performed and document in the following graph.

3.4 Design of Pile Foundation

Jackets structures are normally piled to the sea floor. The purpose of the pile is to take the axial bearing actions due to gravity load and overturning moment caused by environmental actions and to support the global base shear force from the environmental actions. The axial bearing action is resisted by external and internal shaft friction plus the end bearing of the pile annulus. Alternatively, the end bearing pressure is assumed to act over the entire cross section of the pile with no internal friction [12].When the pile foundations are taken into account, the failure modes of the pile need to be considered as the part of the whole structure failure. The most likely failure modes of the foundation are pile pull-out in tension and punch through in compression and excessive pile bending due to insufficient later strength of the soil. These locally soil-pile failures might lead the global collapse of the jacket. In order to find the effect of pile foundation on substructure response, the pushover analysis with design pile foundation will be performed in next chapter. The piles foundation used in analysis is designed by Dr Nilanjan. The purpose of the analysis is to check the correctness of the design and understand the soil-pile interaction effect on the ultimate capacity of the whole substructure and mechanism of the failure process. In this section we focus on the dimension of the piles and eigen period and eigen mode of the pile foundation jacket in 70 and 100 meters water-depth.

3.4.1 Pile Foundation for 70 Meters Water-depth Jacket

The pile foundation of 70 meters water depth OFWT jacket has been designed by Dr Nilanjan Saha in his paper Design of Piles Using LRFD Method [13]. The dimension of the pile is determined by the axial and lateral bearing capacity which is calculated by load resistance factor design method API-2R-LRFD [14]. First, the pile length and diameters are assumed. The diameter of the leg is 1.17 meters to 1.8 meters and inside piles then would have to be less than 1.17 m which is extremely small. Therefore, the diameters of piles are larger than the diameter of the leg to satisfy design criteria. The minimum pile wall thickness is driven by the equation form the API LRFD codes based on assumed diameter and length, the minimum piling wall thickness used should not be less than 0.025 meter. The design dead loads and environmental loads are applied to the pile foundation afterwards to calculate the axial and lateral loads the piles should undertake. According to the calculation, the assumed length and diameter could undertake the design load case, the design is accepted. The dimensions are primarily chosen based on axial loading condition. Four piles need to be installed at the base of each leg of jacket. The length of the pile foundation connected with each leg ends is 45 meters with the constant thickness 25 millimeters. From bottom to the -2 meters soil depth, the diameter of the pile is 2 meters with the other depth 1.8 meters diameters.

3.4.2 70 Meters Water-depth Pile Foundation Jacket Eigen Mode

As we have discussion in previous section, a key feature of the offshore structure is the resonance associated with the certain eigen period and the corresponding eigen mode. At the eigen period the substructure oscillate most easily which will display significant resonance, this would bring great damage to the substructure. Especially some of the eigen mode has strong effect on the bottom of substructure will damage the pile foundation. Therefore the eigen period of the 70 meters jacket substructure is estimated and tabulated.

Eigen Mode	First	Second	Third	Fourth	Fifth	Sixth
70m Jacket Eigen Period	2.9454	2.93996	0.77028	0.767569	0.642638	0.633542

Table 3.5 First six eigen period of the 70 meters water-depth pile foundation jacket

We observe compared with fixed bottom substructure the eigen period increase a little, the first eigen period is approximately 2.95 second, which is larger than 2.8 from fxied bottom jacket but still less than typical wave period. The Figure 3.20 below shows the first to third eigen mode movement of the supporting jacket. Figure 3.21 shows the fourth to sixth Eigen mode movement of the supporting substructure. The red color represents the eigen period with the global eigen mode. We observed that the jacket is globally oscillating in the first to fourth eigen period. The tubular tower move in the x and y direction respectively with the jacket keeping stationary in the first and second eigen modes. The third and fourth eigen mode of the substructure are global bending in x and y direction, the jacket oscillating with the bending of the tubular tower. That means the global eigen mode remain the same as the fixed bottom jacket in the same water depth. The pile foundation doesn’t affect the eigen movement of the jacket. Corresponding to the fourth eigen mode, the oscillation is locally located on the vertical braces on the first storey. The opposite vertical braces at first storey oscillate as section symmetry. The local up and down oscillation of the horizontal braces in the first storey is the eigen mode during sixth eigen period.

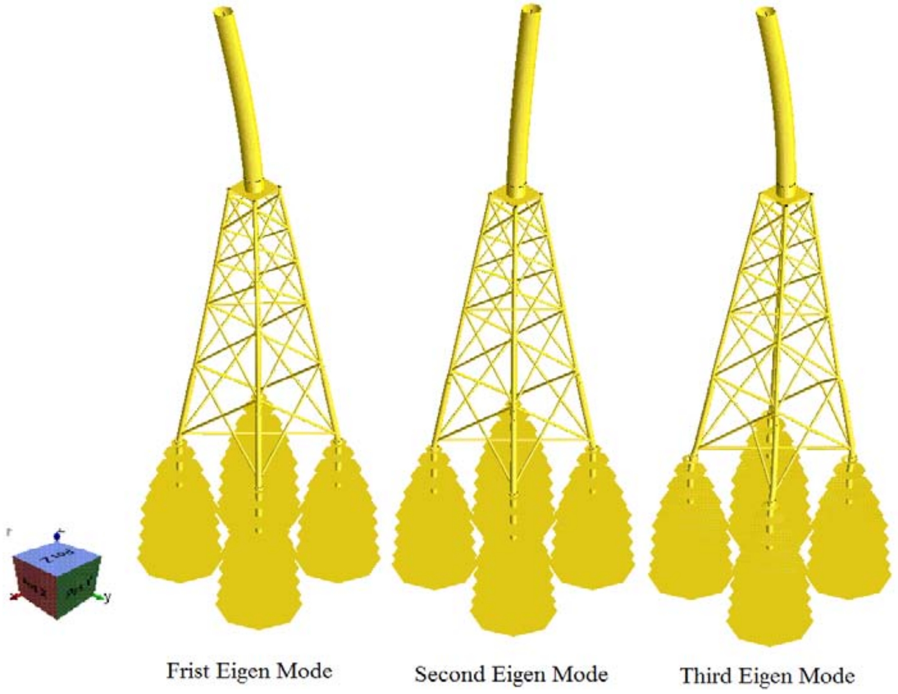

Figure 3.18 70 meters water-depth pile foundation jacket's oscillation in first to third eigen mode.

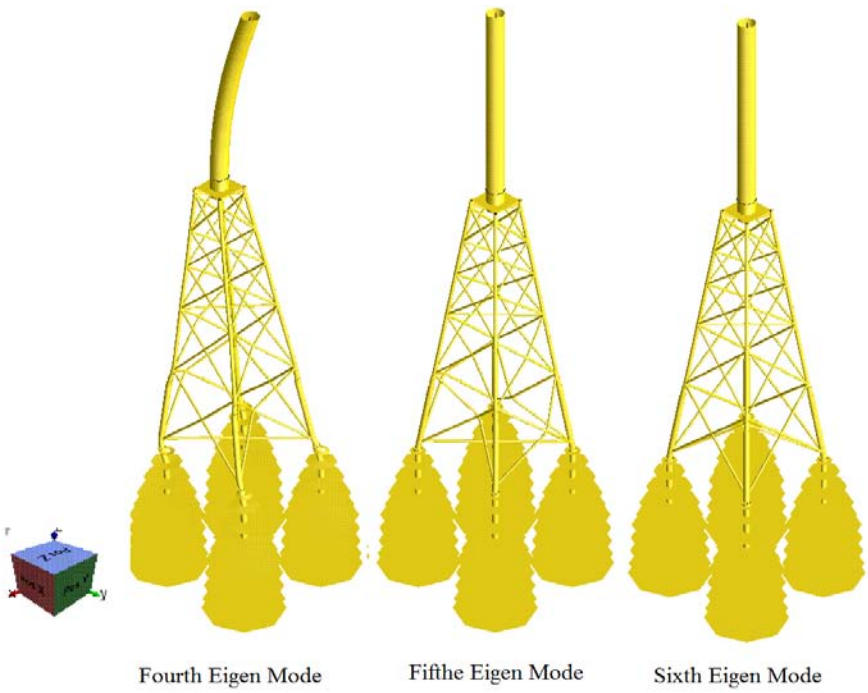

Figure 3.19 70meters water-depth pile foundation jacket's oscillation in fourth to Sixth eigen mode.

3.4.3 Pile Foundation for 100 Meters Water-depth Jacket

The pile foundation for 100 meters water depth jacket adopts the same diameter and thickness from the design of piles from 70 meters water depth. In order to support the larger functional loads and environmental load, the length of the pile foundation connected with each leg ends increases to 55 meters with constant thickness 25 millimeters. From bottom to the -2 meters soil depth, the diameter of the pile is 2 meters with the constant 1.8 meters diameters at other depth.

3.4.4 100 Meters Water-depth Pile Foundation Jacket Eigen Mode

In the similar way, we perform the eigen value estimation for the 100 meters water-depth OFWT jacket considering the soil pile interaction.

Eigen Mode	First	Second	Third	Fourth	Fifth	Sixth
100m Jacket Eigen Period	3.04236	3.03732	1.00871	1.00671	0.98211	0.88899

Table 3.6 First six eigen period of the 100 meters pile foundation jacket

Table 3.6 shows the eigen period of 100 meters water depth pile foundation substructure. Red color shows the eigen periods relating to the global eigen mode. Compared with eigen period of fixed end jacket at same water depth, the variation is small. The corresponding eigen mode is plot as follow. Figure 3.22 is the first to third eigen mode. Figure 3.23 shows the fourth to sixth eigen mode. We observed that the jacket is globally oscillating in the first to fourth eigen period, adopt the same mode shape as 70 meters water depth jacekt. The tubular tower move in the x and y direction respectively with the jacket keeping stationary in the first and second eigen modes. The jacket oscillating with the bending of the tubular tower in the third and fourth eigen modes. The fifth and sixth eigen modes are local oscillation at first storey horizontal and vertical braces respectively. The horizontal braces vibrate up and down with the other components of the jacket keeps stationary. The movement of the vertical braces indicates that the oscillation of opposite braces at same storey is section symmetry with the opposite direction of motion. Compared with fixed bottom substructure at same water depth, first and second global eigen modes remain the same. For other eigen modes, the sequence of the eigen mode alter without changing movement. Compared with the 70 meters water depth pile foundation jacket, the global eigen modes are exactly same with small variation in eigen period.

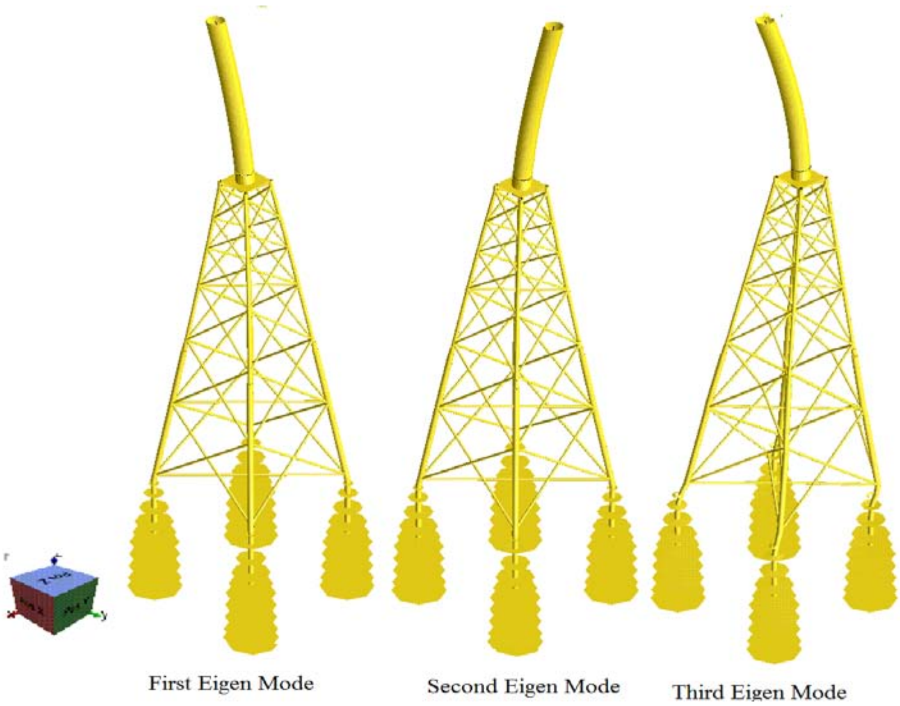

Figure 3.20100meters water-depth pile foundation jacket's oscillation in first to third eigen mode.

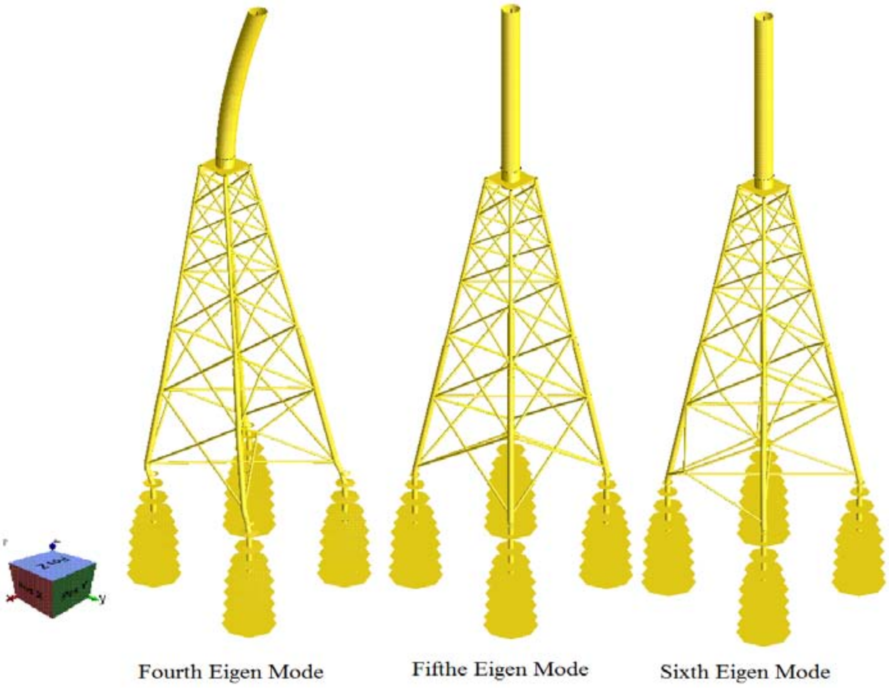

Figure 3.21100meters water-depth pile foundation jacket's oscillation in fourth to sixth eigen mode.

4. Environmental Conditions and Design Loads

4.1 Environmental Conditions

4.4.1 Introduction

Environmental conditions cover natural phenomena, which may contribute to structural damage, operation disturbances or navigation failures. The environmental conditions in this thesis are picked from the data of the Design basis for NTNU [5]. The most important phenomena for offshore wind turbine are from three main areas:

- Wind
- Waves
- Current

This chapter will focus on these three environmental condition and corresponding load generated. Of course, there are many other phenomena, which may be very important in the specific cases, including:

- Tides
- Ice
- Earthquake
- Soil Conditions
- Temperature
- Fouling
- Visibility

For these phenomena, except soil condition, which is of importance for the jacket substructure, will be mentioned in following, others are not covered by this report. The environmental phenomena are usually described by physical variables of statistical nature. The statistical description should reveal the extreme conditions as well as the long- and short-term variations. If a reliable simultaneous database exists, the environmental phenomena can be described by joint probabilities. The environmental design data should be representative for the geographical areas where the structure will be situated, or where the operation will take place.

4.1.2 Wind conditions

Mean wind speed found for offshore wind turbine are often considerably higher than those found on land. For representation of wind climate, a distinction is made between normal wind conditions and extreme wind conditions [11]. The normal wind conditions generally concern recurrent structural loading conditions, while the extreme wind conditions represent rare external design conditions. Normal wind conditions are used as basis for determination of primarily fatigue loads, but also extreme loads from extrapolation of normal operation loads. Extreme wind conditions are wind conditions that can lead to extreme loads in the components of the wind turbine and in the support structure and the foundation. This average wind conditions could be modeling through the empirical equilibrium equation with the mean wind condition and parameters mentioned in the DNV offshore standards DNV-OS-J101.

Because wind speed varies with time also varies with the height above the ground or the height above the sea surface. Particularly in this thesis case, the wind turbine located above the sea surface about 90 meters. It is thus essential to distinguish between offshore standers and wind turbine related standers when wind speed is taken into account. For these reasons, in the offshore wind foundation design, interest is focused on the extreme wind speed, since this is the design driving cases.

The wind direction is defined as the direction from which the wind is blowing at a given location. It is normally measured in tens of degrees from 0 degree to 360 degrees. The plot below could indicate the direction of wind in this thesis. The positive X directoin along X axes is 0 degree with counterclockwise increase degree to 90 degrees at positive Y axes. In this thesis, since the jacket is axially symmetric, the incidence angle larger than 45 degrees could be simplified as angle in 0 to 45 degrees. For example, 120 degrees could be simplified as 30 degrees measured counterclockwise from the positive half of X axis.

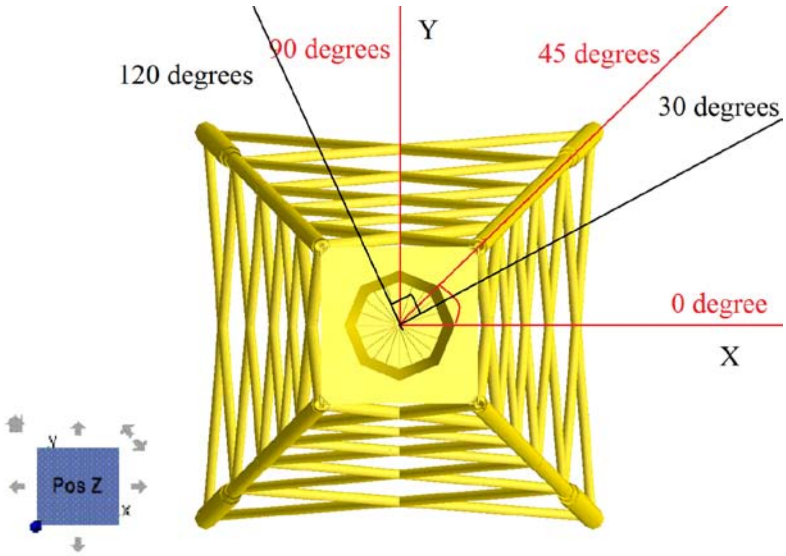

Figure 4.1 Definition of wind direction

In this thesis, the extreme wind speeds averaged over 1hour, at a height of 10 meters above LAT for different return periods are shown in table 3.1.Presnetly we consider the extreme wind speed 35.3m/s to a return period of 100 years at Ekofisk offshore complex [5]. Model the corresponding wind force in HAWC2.

	Return Period (Years)			
Wind speed	1	10	100	10000
V1hour,10m(m/s)	28.8	32.2	35.3	40.8

Table 4.1 Extreme Wind Speed for Different Return Period

Return Period	*N*	*NE*	*E*	*SE*	*S*	*SW*	*W*	*NW*
1 year	22.6	21.4	24.6	23.3	24.8	25.8	27.5	25.3
10 year	26.3	26.0	28.5	26.2	28.3	28.9	31.2	29.6
100 year	29.4	30.0	31.9	28.7	31.3	31.5	34.4	33.3

Table 4.2 Extreme wind speeds and direction

Calculations of design wind speed and loads to be performed in accordance with NORSOK N-003. The variation with height and duration shall be taken into account by use of the following formula:

$$u(z,t) = U(z)\cdot(1-0.41\cdot I_u(z)\cdot\ln(\frac{t}{t_0}))$$

$$U(z) = U_0\cdot(1+0.0573\cdot(1+0.15\cdot U_0)^{0,5}\cdot\ln\left(\frac{z}{10}\right))$$

$$I_u(z) = 0.06\cdot(1+0.043\cdot U_0)\cdot\left(\frac{z}{10}\right)^{-0.22}$$

Where t is the duration and z is the elevation.U_0 = 1 hour mean wind speed at 10 meters above sea level, t_0 = 3600s.

4.1.3 Wave Condition

Ocean waves are irregular and random in shape, height, length and speed of propagation. A real sea state is best described by a random wave model. The wave climate is represented by the significant wave height H_s and the spectral peak period T_p[13]. In the short term, typically over a 3-hour period, stationary wave conditions with constant H_s and constant T_p are assumed to prevail. Wave statistics are to be used as a basis for representation of the long-term and short-term wave conditions. Empirical statistical data used as a basis for design must cover a sufficiently long period of time. About the long term and short term conditions and corresponding stochastic analysis will be mentioned in Chapter 3.The modeling of the wave and prediction of wind parameter defined in the DNV offshore standards DNV-OS-J101.The wave parameters using in this project is as follows. Omni directional wave-parameters with respective return period are presented in Table 4.3.

Return period (Years)	*Hmax (m)*	*THmax (sec)*	*Cr (m)*	*Hm0 (m)*	*T02 (s)*
1	17.3	12.4	11.5	9.6	9.9
10	21.3	13.5	14.2	11.7	10.9
50 (interpolated)	24.1	14.1	15.9	13.1	11.6
100	25.2	14.4	16.8	13.8	11.9
1000	29.1	15.2	19.6	15.8	12.7
10000	33.0	15.9	22.4	17.7	13.6

Table 4.3 Estimations of single directional wave-parameters

H_{max}: Maximum individual wave-height

T_{Hmax}: Wave-period for Hmax

Cr: Max wave-crest-elevation wrt, LAT

H_{m0}: Significant wave-height (3 hours)

T_{02}: Zero-crossing period

The wave shall be calculated using the Stokes 5th order wave theory including a wave kinematic factor of 0.95.

4.1.4 Current

In the similar way, the Current statistics are to be used as a basis for representation of the long-term and short-term current conditions. Since the dominated of the dynamic force is drag force calculated by Morison’s Equation, the current speed and modeling is corresponding important.

The current profile at Ekofisk may be considered to be constant over the water-depth until 3 m above seabed with omni-directional speed. The same speed is assumed down to seabed for substructure designing, the speeds are given in Table 4.4.

Return period:	1 year	5 years	10 years	50 years	100 years
Current speed (cm/s)	54	65	69	80	85

Table 4.4 Extreme values for current

4.1.5 Soil Conditions

For offshore wind farm, the soil stratigraphy and range of soil strength properties shall be assessed within each group of foundations or per foundation location, as relevant. As the jacket foundation structure is used in this project, the soil condition is an important input to site selection. The soil investigations shall provide all necessary soil data for a detailed design. The soil properties at Ekofisk offshore site are estimated from API rules indicated in the Table 4.5.

Depth(m)		Type of Soil	Density	δ	Φ
From	To		(kg=m3)		
0	-1.524	Scour	913	–	–
-1.524	2.134	Sand	913	35	30
-2.134	-8.534	Sand	1020.4	38	33
-8.534	-20.117	Sand	1020.4	35	30
-20.117	-23.47	Sand	1020.4	38	33
-23.47	-53.34	Clay	1020.4	–	–
-53.34	-74.066	Sand	1020.4	30	25
-74.066	-106.68	Clay	1020.4	–	–

Table 4.5 Soil properties at Ekofisk offshore site

4.2 Environmental Loads

For the structure meet the ultimate limit state (ULS) design conditions, the load cases shall in general be determined according to DNV offshore standards DNV-OS-J101. Thus, the temporary design conditions which cover the design conditions during transport, assembly, maintenance, repair and decommissioning of the wind turbine structure are taken into account. Also, we need consider the operational design conditions, the basis for selection of characteristic loads and load effects specified in DNV-OS-J101.These will be discuss later in charter pushover analysis.

3.2.1 Permanent Loads

Permanent loads are loads that will not vary in magnitude, position or direction during the period considered. In this project, the permanent loads mainly refer to gravitational and inertia loads which are the loads from gravity, vibration, rotation and seismic activity. The external and internal hydrostatic pressure of a permanent nature also should be considered. In the USFOS since the turbine and nacelle structure could not be applied well, we simplified the turbine and nacelle structure as a point mass in 6 degree of freedom at the top of the tower.

3.2.2 Environmental Loads

Environmental loads are loads caused by environmental phenomena. Environmental loads to be used for design shall be based on environmental data for the specific location and operation in question, and are to be determined by use of relevant methods applicable for the location/operation taking into account type of structure, size, shape and response characteristics. Environmental loads may vary in magnitude, position and direction during the period under consideration, and which are related to operations and normal use of the structure. For the offshore wind turbine substructure design, the majority affect from wind, wave and current, are the only characteristic loads and load effect shall be determined as design loads before design.

3.2.3 Wind Loads

Wind generated load on the rotor and tower have been considered as a wind loads produced directly by the inflowing wind as well as indirect loads that result from the wind-generated motions of the wind turbine and the operation of the wind turbine.

For design of the supporting structure and the foundation, a number of load cases for wind turbine loads due to wind load on the rotor and on the tower shall be considered, corresponding to different design situations for the wind turbine. The load cases shall be defined such that it is ensured that they capture the 50-year load or load effect, as applicable, for each structural part to be designed in the ULS. The characteristic load cases in this project provided by ‘Aker Solution Design Basis’.

OPERATING CONDITIONS						
Load condition	Fx [MN]	Fy [MN]	Fz [MN]	Mx [MNm]	My [MNm]	Mz [MNm]
Production 1	0.83	-0.06	-4.2	5	5	3
Production 2	0.4	-0.06	-4.2	6	16.8	7
Extreme Gust 1	-1.6	0.11	-4.2	-1.5	-15	-5
Extreme Gust 2	1.46	0.08	-4.2	0.96	-10.8	-3.4

Table 4.6 Forces from rotor acting on hub for wind from west for operating conditions

SHUT-DOWN CONDITIONS						
Load condition	Fx [MN]	Fy [MN]	Fz [MN]	Mx [MNm]	My [MNm]	Mz [MNm]
50 years wind, reduced wave	1.4	0.41	-4.2	-6.93	4.62	14.03
50 years gust, reduced wave	1.08	0.33	-4.2	-4.1	3.3	10.8
50 years wave, reduced wind	0.65	0.2	-4.2	-2.5	2	6.5

Table 4.7 Forces from rotor acting on hub for wind from west for shut-down conditions

Wind turbine loads during power production and selected transient events shall be verified by load measurements that cover the intended operational range. We can generate the estimate wind force in the practical operation condition through the simplified monopile in HAWC2 as point rotor shaft load locate on the nacelle in time series.

4.2.4 Wave and Current Loads

For calculation of wave loads, a recognized wave theory for representation of the wave kinematics shall be applied. The wave theory shall be selected with due consideration of the water depth and of the range of validity of the theory. Methods for wave load prediction shall be applied that properly account for the size, shape and type of structure. At this project depth, and with this rather large wave height, the wave loads might be dominating. Both viscous effects and potential flow effects may be important in determining the wave-induced loads on a wind turbine support structure. Wave diffraction and radiation are included in the potential flow effects. Figure below can be used as a guidance to establish when viscous effects or potential flow effects are important. It refers to horizontal wave-induced forces on a vertical cylinder, which stands on the seabed and penetrates the free water surface, and which is subject to incoming regular waves.

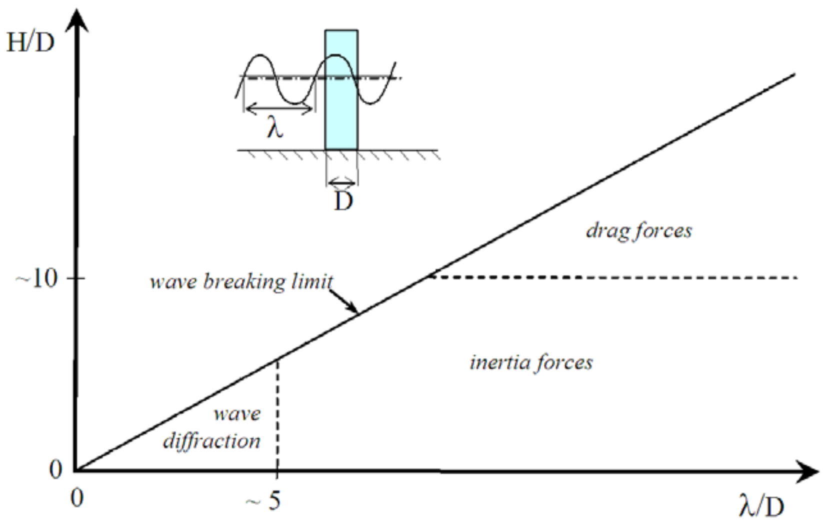

Figure 4.1 Dominating force in damping

Morison's equation can be applied to calculate the wave and current Loads. By this equation, in an oscillatory flow with flow velocity $u(t)$, the Morison equation gives the inline force parallel to the flow direction:

$$F = C_m \rho \frac{\pi}{4} D^2 \dot{u} + C_d \frac{1}{2} \rho D u |u|.$$

Here, Cd and Cm are drag and inertia coefficients respectively which have been determined for both the jacket and monopile. The wave and current velocity has been document previous. In this project, for the jacket structure, the wave length is much larger than the jacket dimensions, so the dominating force should be the inertia forces. In case the body moves as well, with velocity $v(t)$, the Morison equation becomes:

$$F = \underbrace{\rho V \dot{u}}_{a} + \underbrace{\rho C_a V \left(\dot{u} - \dot{v}\right)}_{b} + \underbrace{\frac{1}{2}\rho C_d A \left(u - v\right)\left|u - v\right|}_{c}.$$

This equation should be applied to the dynamic analysis of the OFWT jacket. Since the analysis in this thesis is base on static pushover, we still adopt the former equation only considering the flow velocity.

4.2.5 Combined Loads and Load Effects

In a short-term period with a combination of waves and fluctuating wind, the individual variations of the two load processes about their respective means can be assumed uncorrelated. This assumption can be made regardless of the intensities and directions of the load processes, and regardless of possible correlations between intensities and between directions.

- Linear combination of wind load and wave load or of wind load effects and wave load effects.
- Combination of wind load and wave load by simulation.
- For linear superposition, the combined load effect in the structure due to concurrent wind and wave loads may be calculated by combining the separately calculated wind load effect and the separately calculated wave load effect. This method may be applied to concept evaluations and in some cases also
- to load calculations for final design, for example in shallow water or when it can be demonstrated that there is no particular dynamic effect from waves, wind, ice or combinations. The design combined load effect is expressed as

$$S_d = \gamma_{f1} S_{wind,k} + \gamma_{f2} S_{wave,k}$$

- In this equation, Swind,k denotes the characteristic wind load effect and Swave,k denotes the characteristic wave load effect. It is a prerequisite for using this approach to determine the design combined load effect. The separately calculated value of the characteristic wave load effect Swind,k is obtained for realistic assumptions about the equivalent damping that results from the structural damping and the aerodynamic damping. For the ULS design conditions, the design load cases shall in general be according to DNV. This document describes in total 32 load cases to be relevant for the different operating and shut down cases of the turbine. However, detailed knowledge about generator and blade behavior is required to develop all these cases, and only the cases assumed to be governing have therefore been investigated. These cases are:

- Four operating conditions with accompanying forces acting on the rotor, combined with a reduced wave load (70%) of the maximum 50 year wave. The reduced 50 year wave is combined with the 5 year current.
- Three shut down cases. Two of these cases combine the reduced 50 years wave with wind load and reduced rotor loads. For the last case the 50 years wave is combined with wind and reduced rotor loads.
- For all three cases, the wave is combined with 5 year current.

The combined load case in this project is tabulated below:

	Wind conditions		Wave conditions	
	vhub [m/s]	αwind,hub [°]	hwave [m]	αwave [°]
Operating conditions				
Production 1	12	0	0.71·Hmax	0
Production 2	30	0	0.71·Hmax	0
Extreme Gust (2 cases)	20	0	0.71·Hmax	0
Shut-down conditions				
50 years wind, reduced wave	50	30	0.71·Hmax	90
50 years gust, reduced wave	63	30	0.71·Hmax	90
50 years wave, reduced wind	55	30	Hmax	90

Table 4.8 Combined wind and load case

4.2.6 Soil-Pile Interaction Force

The geotechnical design of foundations shall consider both the strength and the deformations of the foundation structure and of the foundation soils. The soil condition has been determined in previous discussion about environmental condition. Therefore, in this thesis, for the jacket substructure, pile foundation is applied for existing soil condition. The purpose of pile is to take the axial bearing actions due to gravity and overturning moment from environmental actions and to support the global base shear force from the environmental actions. The interaction force between soil and pile include external and internal shaft friction and end bearing load. The nonlinear load-displacement relationships for soil-pile interaction are conveniently modeled with piece-wise linear springs. This approach has been implemented in USFOS.As USFOS can be used to model soil-pile interaction force, therefore we use the estimate Ekofisk soil properties to calculate the interaction force. According to the eigen value analysis in last chapter we also notice that the eigen periods may vary slightly depending on pile dimensions, type of interactions and soil types.

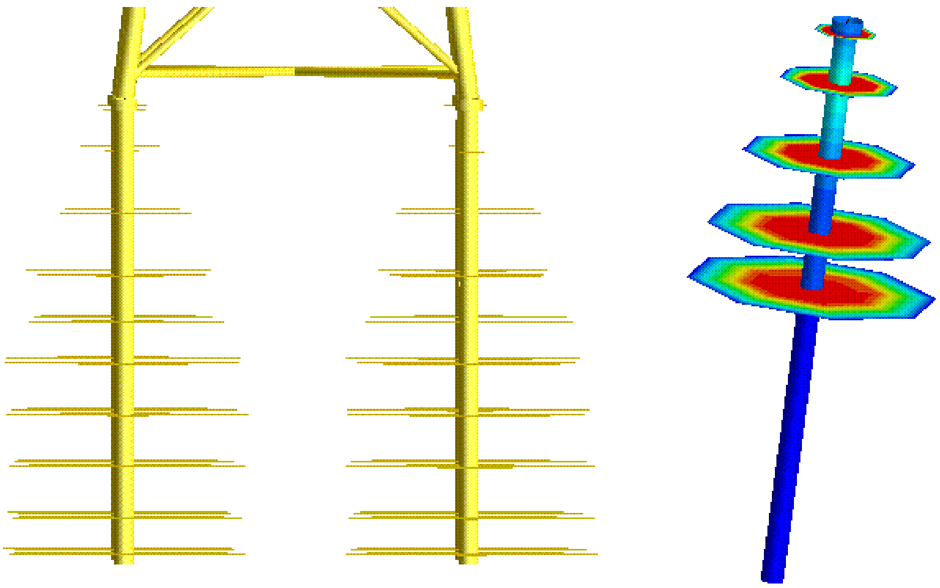

Figure 4.2 Pile foundation non-linear spring model in USFOS

5. Pushover Analysis for OFWT Substructure

5.1 Objective

As discussed in Chapter 2 the initial failure seldom represents the capacity of a structure. This implies that structure processes strength reserves beyond those recognized in traditional design. Therefore, except the design strength, we also interest in the strength reserved. Static pushover analysis is a typical approach to identify the ultimate capacity. By this method, the ultimate capacity and redundancy of the structure could be identified. In this project, considering the characteristic load cases have been provided, we apply pushover analysis to identify the ULS capacity and corresponding reserved capacity based on characteristic load cases.

Pushover analysis is normally carried out in the following way. First, functional actions including permanent actions and live actions are incremented up to design value (partial safety factors for functional actions are normally equal to unity). Second, characteristic environmental actions are increased proportionally until global collapse takes place [12].

The complete substructure including tower, middle section and jacket foundation with piles which have been investigated in Chapter 3 is introduced in this analysis. Using the software USOFS, the pushover analysis is done for the ULS capacity of OFWT substructure with the characteristic load case given by the report “Design basis for NTNU”. In analysis the functional loads are applied first up to their design values, the environmental loads are incremented up to collapse afterwards. A load ratio of design environmental loads represents how many times the design environmental load jacket could bear. This factor should be larger than 1 which proves the jacket could survive beyond the design environmental loads and reserve loads of jacket which shows the conservativeness. The functional loads include gravity, buoyancy and design wave and current loads. The characteristic wind load is simulated as rotor force on the top of the tower.

5.2 Loads and Environmental Conditions

Loads and environmental conditions are defined by “Aker Solution Design Basis” [5].These cases have been discussed already in chapter 4. The NREL 5MW wind turbine is supported with the 70 meters and 100 meters steel jacket substructure. The mass of the nacelle and loading reversals due to their own weight is applied as functional load correspondingly. The

ULS load factor and resistance factors are applying to the characteristic load cases during the input part by multiplying the characteristic load case. Pushover analysis is carried out with waves coming from a direction parallel to a symmetry axis. For the characteristic wave loads, USFOS will step through the actual wave and identify the worst wave position where causing the highest base shear or overturning moment. The hydrodynamic forces from this wave phase and position are saved in memory to be used in the pushover analysis as functional load. Wind loads are simplified as rotor force on the top of tower which is provided by the design wind load case in “Aker Solution Design Basis” [5]. Wind loads are applied as concentrated rotor forces and moments on the tower top node. The pushover analysis is carried for each load case with respect to increasing rotor load. The characteristic load cases are tabulated below, the red color represents the critical load case in pushover analysis which will be specified in next section.

<table>
<tr><th>Operating Conditions</th><th>Wind Speed</th><th>Wind Direction</th><th colspan="2">Wave Speed</th><th colspan="2">Wave Direction</th></tr>
<tr><td>Production 1</td><td>12</td><td>0</td><td colspan="2">0.71·Hmax</td><td colspan="2">0</td></tr>
<tr><td>Production 2</td><td>30</td><td>0</td><td colspan="2">0.71·Hmax</td><td colspan="2">0</td></tr>
<tr><td>Extreme Gust</td><td>20</td><td>0</td><td colspan="2">0.71·Hmax</td><td colspan="2">0</td></tr>
<tr><td>Extreme Gust</td><td>20</td><td>0</td><td colspan="2">0.71·Hmax</td><td colspan="2">0</td></tr>
<tr><td>Shut-down conditions</td><td colspan="6">Wind Load on Rotor</td></tr>
<tr><td>50 years wind, reduced wave</td><td>50</td><td>30</td><td colspan="2">0.71·Hmax</td><td colspan="2">90</td></tr>
<tr><td>50 years gust, reduced wave</td><td>63</td><td>30</td><td colspan="2">0.71·Hmax</td><td colspan="2">90</td></tr>
<tr><td>50 years wave, reduced wind</td><td>55</td><td>30</td><td colspan="2">H max</td><td colspan="2">90</td></tr>
<tr><td>Operating Conditions</td><td colspan="6">Wind Load on Rotor</td></tr>
<tr><td>Production1</td><td>0.83</td><td>-0.06</td><td>-4.2</td><td>5</td><td>5</td><td>3</td></tr>
<tr><td>Production2</td><td>0.4</td><td>-0.06</td><td>-4.2</td><td>6</td><td>16.8</td><td>7</td></tr>
<tr><td>Extreme Gust1</td><td>-1.6</td><td>0.11</td><td>-4.2</td><td>-1.5</td><td>-15</td><td>-5</td></tr>
<tr><td>Extreme Gust2</td><td>1.46</td><td>0.08</td><td>-4.2</td><td>0.96</td><td>-10.8</td><td>-3.4</td></tr>
</table>

Shut-Down Conditions						
50 years wind, reduced wave	1.4	0.41	-4.2	-6.93	4.62	14.03
50 years gust, reduced wave	1.08	0.33	-4.2	-4.1	3.3	10.08
50 years wave, reduced wind	0.65	0.2	-4.2	-2.5	2	6.5

Table 5.1 Brief summary of wind and wave conditions for operating and shut-down conditions

5.3 Pushover Analysis for 70 Meters Water-depth OFWT Jacket

The analysis is performed for all the 7 characteristic wind load cases on rotor corresponding to operating and shut down cases separately. The procedure of the analysis has been discussed in the last section. As we have mentioned in Chapter 2 the following safety factor requirement should be satisfied according to ULS, the design resistance should be larger than design load.

$$\varepsilon_k S_k \leq R_c/\beta_c$$

Where the load factor ε_k=1.35 and resistance factor β_c=1.25 from the DNV-OS-J101 [7] are applied to convert the characteristic load cases form Aker Solution's reports to design load cases. During pushover analysis design environmental actions are increased proportionally with load ratio ω until global collapse takes place. Therefore the load ratio ω could be identified as

$$\text{Load ratio } \omega = \frac{R_c/\beta_c}{\varepsilon_k S_k}$$

The load ratio could indicate how many times of the design load jacket could undertake when the most critical element starts to yield and when the jacket reaches the ultimate capacity. The load ratio should be larger than 1 and the larger part could show the conservativeness of the design based on characteristic load cases.

5.3.1 Results

The results of pushover analysis for 70 meters water depth OFWT fixed bottom jacket are tabulated below. The load ratio which represent how many times of the design load acting on jacket is documented when the most critical element starts to yield buckle and when the whole structure collapses.

Result for operating condition:

Production 1	Initial Yield	First Buckle Member	Ultimate collapse load
Failure place	1239 YIELD END2	1239 BUCKLE	842 PLAST END2
Load Ratio	3.451851852	4.973037037	5.143703704

Table 5.2 70 meters water-depth fixed bottom jacket critical load ratio for Production 1

Production 2	Initial Yield	First Buckle Member	Ultimate collapse load
Failure place	1239 YIELD END2	1239 BUCKLE	842 PLAST END2
Load Ratio	5.133037037	7.434666667	7.611851852

Table 5.3 70 meters water-depth fixed bottom jacket critical load ratio for Production 2

Extreme Gust1	Initial Yield	First Buckle Member	Ultimate collapse load
Failure place	867 YIELD MID	867 BUCKLE	1239 PLAST END1
Load Ratio	2.379851852	3.131259 259	3.301333333

Table 5.4 70 meters water-depth fixed bottom jacket critical load ratio for Extreme Gust1

Extreme Gust2	Initial Yield	First Buckle Member	Ultimate collapse load
Failure place	1240 YIELD END2	1240 BUCKLE	841 PLAST END2
Load Ratio	2.336592593	3.179851852	3.406222222

Table 5.5 70 meters water-depth fixed bottom jacket critical load ratio for Extreme Gust2

Result for shut down condition:

50years wind reduced wave	Initial Yield in Jacket	First Buckle Member	Ultimate collapse load
Failure place	1240 YIELD END2	1240 BUCKLE	841 PLAST MID
Load Ratio	1.801481481	2.536888889	2.650074074

Table 5.6 70 meters water-depth fixed bottom jacket critical load ratio for 50years wind reduced wave

50years gust reduced wave	Initial Yield	First Buckle Member	Ultimate collapse load
Failure place	1240 YIELD END2	1240 BUCKLE	841 PLAST END2
Load Ratio	2.334222222	3.291259259	3.428148148

Table 5.7 70 meters water-depth fixed bottom jacket critical load ratio for 50years gust reduced wave

50years wave reduced wind	Initial Yield	First Buckle Member	Ultimate collapse load
Failure place	1240 YIELD END2	1240 BUCKLE	841 PLAST END2
Load Ratio	3.792592593	5.377777778	5.706666667

Table 5.8 70 meters water-depth fixed bottom jacket critical load ratio for 50years wave reduced wind

In the results, we observe for the first member yielding and ultimate capacity load, the smallest load ratio is 1.80 and 2.65 respectively from the shut down design load case 50 years wind reduced wave (shown in red color). That means for the same safety factor, this load case gives worst bending moment and compression stress of the jacket element. Therefore, the load case 50 years wind reduced wave is defined as critical load case in the next description. A critical load ratio with respect to ultimate capacity is then determined from this load case. According to the critical load ratio, which is higher than 1, the substructure could undertake deign load cases. That means the structure can pass the ULS pushover design check. The critical elements which have significant effect on response of jacket are plotted as Figure a in APPENDIX B.

5.3.2 Critical Load Case Sensitivity Study

If we focus on the shut down design load case 50 years wind reduced wave, which is the most critical load case we find previously, that the rotor force needs to be twice as the design load when the yielding happens. It implies a load ratio of 1.8, and it is a bit higher than the required 1. As we mentioned, that could prove that the structure can pass the ULS pushover design check, and present some conservativeness. In order to understand the where the reserved strength of the jacket from and failure mechanism of the substructure, the sensitive study of the critical design load case is performed to understand the response of the jacket collapse. In this way, the critical place of the jacket failure and stress distribution could be identified. The failure mechanism for the jacket in 50 years wind reduced wave load case shows in Figures 5.1. Corresponding Figure 5.2 shows the load-deformation relationship of the jacket under load case 50 years wind reduced wave.

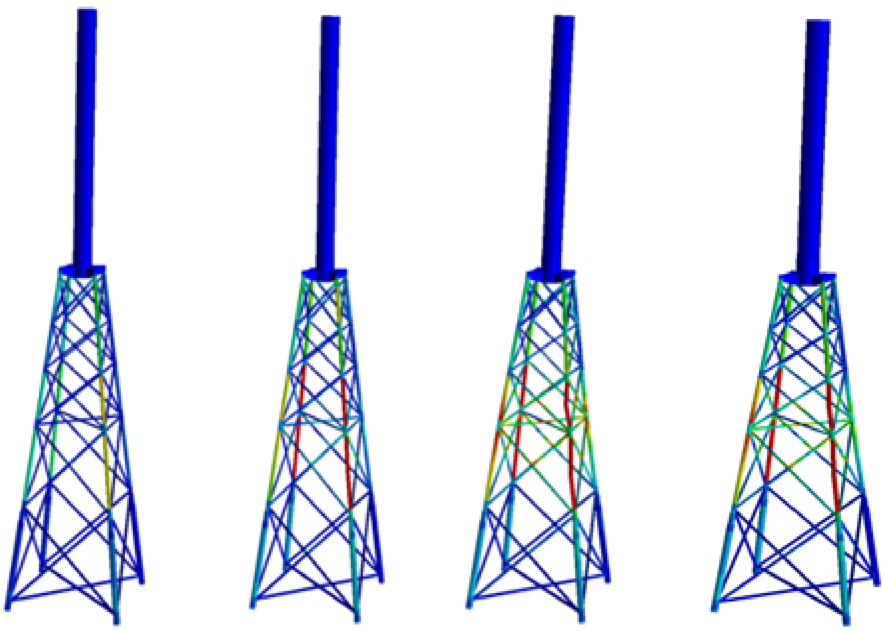

Figure 5.1 Failure mechanism for70 meters water-depth jacket under 50 years wind

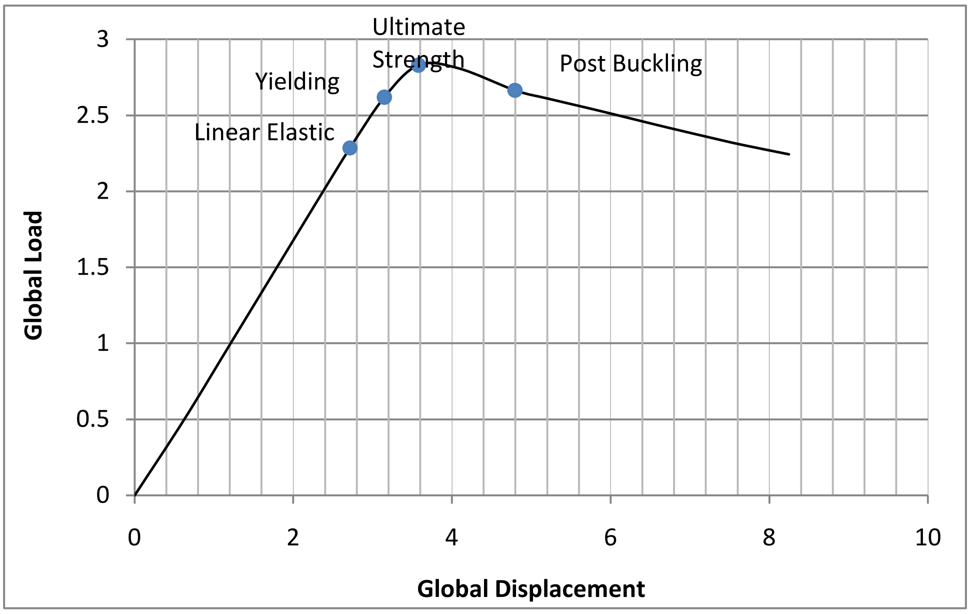

Figure 5.2 Load-deformation relationship of the70 meters water-dpeth fixed bottom jacket under 50 years wind

We observe that the response is virtually linear until the first yielding happened at the end of element 1239 and 1240(compression legs in second storey Figure a, APPENDIX B). The end of compression legs on the third storey yield in rapid sequence. After first members group yielding, the collapse occurs immediately, the jacket reach its ultimate capacity fast as shown in the figure 5.2 from yielding point to ultimate strength point. That means compared with ultimate strength, the reserved strength is pretty small. The post-peak reduction in capacity is significantly slow, due to the redistribution of the loads very fast in the x-braces after compression leg element buckling. The loss of force in the post buckling range is accompanied by the corresponding drop of force in the tension legs. By further loading, buckling occur at the horizontal x-braces due to the load redistributed, the jacket loss ability to redistribute loads. Subsequently, the other leg elements buckle or yield due to compression loads. After a series of loads redistribution, the collapse analysis end at load ratio 2.2 which is still larger than the design load.

We also observe the first buckling starts form the compression leg on the second story and the collapse of the whole jacket closely related to the plastic deformation of tension leg component on the same height. Therefore the sensitive study of these two leg elements is performed to find relationship of stress and deformation in both of elements. The stress-deformation relationship of both of the compression and tension legs in the same story is plot below.

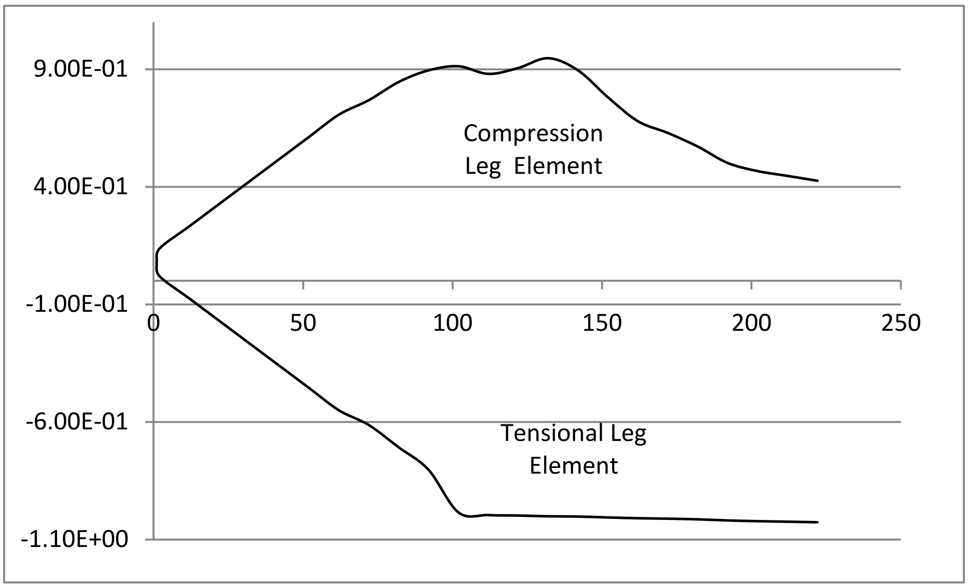

Figure 5.3 Stress-deformation relationship for both of the compression and tension legs under 50 years wind

Figure 5.3 shows the stress-deformation relationship of the critical element where the failure first happens at the compression leg and tension leg on the second storey. It indicated that the stress increasing in the compression leg element is almost equal to the stress increase in tension leg element during the elastic deformation. With the load increasing, the tension leg element is reaching its yield capacity. In contrary, the compression leg element doesn't reach the yield force due to buckling. This is illustrated in the peak of curve in stress deformation relationship diagram. By further load increasing, the compression element could not take more increasing load, the stiffness decreases. The further increasing load is redistributed in the x braces. The post buckling capacity reduction of the compression leg does not impair the load carrying of the tension leg as long as the horizontal brace does not fail. The load deformation responses for other load cases are tabulated in the APPENDIX C.

5.3.3 Effect of Wind Direction

We turn the attention back to all observations and make the compression on the position and load ratio when the initial member failure and jacket achieve the ultimate capacity. The base shear and overturning moment caused by 7 design wind load cases are tabulated below. Considering the symmetry of the jacket, for the combined load direction larger than 45 degrees, the combined load is translated to the same face of the jacket as we mentioned in the Chapter 4 about load direction.

Operating Condition	Base Shear	Overturning Moment	Load Direction
Production1	8.32E+05	1.32E+08	4 degrees
Production2	4.05E+05	6.43E+07	9 degrees
Extreme Gust1	1.60E+06	2.55E+08	4 degrees
Extreme Gust2	1.46E+06	2.33E+08	3 degrees
Shut Down Condition	Base Shear	Overturning Moment	Load Direction
50 years wind, reduced wave	1.46E+06	2.32E+08	16 degrees
50 years gust, reduced wave	1.13E+06	1.80E+08	17 degrees
50 years wave, reduced wind	6.80E+05	1.08E+08	17 degrees

Table 5.9 base shear and overturning moment for characteristic load cases

Combining the pushover analysis results, we observe that the safety factor is much depends on the combined load of the horizontal wind loads on rotor. The wind moment on rotor make a little progress to the collapse of jacket. For the load cases with the same direction, such as load case Production 1 and Extreme Gust1, the base shear of the load case Production 1 is the double of the Extreme Gust1, the load ratio of the first member failure and ultimate capacity have the same proportion. This could be also reflected from load case 50 years wind reduced wave and 50 years wave reduced wind.

The results also indicate that the direction of the wind force affect the structure failure mechanism and ultimate strength significantly. The base shear and overturning moment of the load cases Extreme Gust 2 is equal to load case 50 years wind reduced wave. However the load ratio of Extreme Gust 2 is almost 1.5 times than that of 50 years wind reduced wave because of different load direction. In order to find the effect of the wind load direction, in this case, we apply pushover analysis with the critical load case 50 years wind reduced wave at different incidence angle. Considering the symmetry of the jacket, the direction only rotates from 0 degree, 15 degree, 30 degree and 45 degree respectively.

		First Yield	First Buckle	Ultimate Capacity
0 degree	Load Ratio	2.300444	3.114074	3.201185
	Failure Place	1240	1240	1240
16 degree	Load Ratio	1.8234074	2.5451852	2.6654815
	Failure Place	1240	1240	841
30 degree	Load Ratio	1.707852	2.434963	2.531556
	Failure Place	1240	1240	841
45 degree	Load Ratio	1.656889	2.200889	2.439704
	Failure Place	1240	1240	841

Table 5.10 Load ratio under different wind load direction

The table 5.10 shows the load ratio for first member failure and ultimate capacity of the jacket with the 50 years wind reduced wave at different incidence angle. The failure place and element number is indicated in figure a. We find the ultimate strength of the jacket various significantly according to different wind load directions. The load ratio is largest during jacket is against the wind load on the 0 degree. The load ratio decreases with the increase of incidence angle until the wind load direction is located on the 45 degree of the jacket where the load ratio reaches the smallest value. Since the jacket substructure is axially symmetric, the incidence angle larger than 45 degrees could be simplified as angle in 0 to 45 degrees with same load ratio. It is easy to understand that the wind force sustain by two couple of legs will stay stable compared with wind force only undertake by one couple when wind direction is 45 degrees. In practice, the wind load always composite from different direction. Therefore, for OFWT jacket substructure design, avoiding the annual most frequent wind direction which is located in the region of 45 degrees of the jacket is important.

5.3.4 Failure Position

The typical failure position is plotted in figure a, APPENDIX B. From the table 5.9, we could also observe that the first failure mostly start from the element 1240, compression leg on the second storey. The ultimate capacity depends on load ratio jacket could sustain until plastic deformation at tension leg element. In proof of this observation, we focus on the position of failure in the results of pushover analysis. In 7 load cases, the position of first yield of the jacket is always located on the compression leg element on the second story, like element 1234, 1240 and 867. There is also the first place buckling in rapid sequence. With the load increasing, the jacket is reaching their ultimate capacity until the tension leg element 841, 842 and 1239 plastic deformed. We define the compression leg element on the second story as hot spot of the 70 meters water depth OFWT jacket, where the first yield and buckling happened, and make the compression with the hot spot in 100 meters water depth OFWT jacket in following section.

5.3.5 Discussion

The jacket structure can pass the ULS design check, and present some conservativeness. The sensitivity study shows the compression leg on second storey is the most critical place where the yielding and buckling first happen. It also indicated that jacket is sensitive to the direction of the wind load. We need to consider the regime of the wind and variable wind direction during substructure design to avoid the annual most frequent wind direction which is located on the 45 degrees of the jacket.

5.4 Pushover Analysis for 100 Meters Water-depth OFWT Jacket

The pushover analysis for 100 meters water-depth OFWT jacket is performed in the same way as supporting jacket for 70 meters water-depth. We adopt the same environmental condition, operating and shut-down wind load cases as 70 meters water-depth since the wind speed and wave height alters little during intermediate water-depth. Adopt the same NREL 5MW wind turbine on the top of the column tower, so the turbine mass doesn't change.

5.4.1 Results

The load ratio which represent how many times of the design load acting on jacket is documented when the most critical element starts to yield and when the whole structure collapses. This pushover analysis indentifies if the structure will pass the ultimate limit state also could indicate the reserve resistance of the jacket and conservativeness of design.

Result for operating condition:

Production 1	Initial Yield	First Buckle Member	Ultimate collapse load
Failure place	1261 YIELD END2	1261 BUCKLE	881 PLAST END1
Load Ratio	3.537777778	5.00562963	5.163259259

Table 5.11 100 meters water-depth fixed bottom jacket critical load ratio for Production 1

Production 2	Initial Yield	First Buckle Member	Ultimate collapse load
Failure place	1233 YIELD END2	1261 BUCKLE	881 PLAST END1
Load Ratio	5.141925926	7.265777778	7.636148148

Table 5.12 100 meters water-depth fixed bottom jacket critical load ratio for Production

Extreme Gust1	Initial Yield	First Buckle Member	Ultimate collapse load
Failure place	881YIELD MID	881 BUCKLE	1233 YIELD END2
Load Ratio	2.360296296	3.108148148	3.274074074

Table 5.13 100 meters water-depth fixed bottom jacket critical load ratio for Extreme Gust1

Extreme Gust2	Initial Yield	First Buckle Member	Ultimate collapse load
Failure place	1262 YIELD END2	1262 BUCKLE	880 PLAST END2
Load Ratio	2.392888889	3.380148148	3.402074074

Table 5.14 100 meters water-depth fixed bottom jacket critical load ratio for Extreme Gust2

50years wind reduced wave	Initial Yield in Jacket	First Buckle Member	Ultimate collapse load
Failure place	1232 YIELD END2	1262 BUCKLE	880 PLAST MID
Load Ratio	1.822814815	2.551111111	2.667259259

Table 5.15 100 meters water-depth fixed bottom jacket critical load ratio for 50years wind reduced wave

50years wave reduced wind	Initial Yield	First Buckle Member	Ultimate collapse load
Failure place	1232 YIELD END2	1262 BUCKLE	880 YIELD END2
Load Ratio	2.36562963	3.341037037	3.468444444

Table 5.16 100 meters water-depth fixed bottom jacket critical load ratio for 50years gust reduced wave

50years wave reduced wind	Initial Yield	First Buckle Member	Ultimate collapse load
Failure place	1232 YIELD END2	1262 BUCKLE	880 PLAST END2
Load Ratio	3.920592593	5.436444444	5.723851852

Table 5.17 100 meters water-depth fixed bottom jacket critical load ratio for 50years wave reduced wind

5.4.2 Critical Load Case Sensitivity Study

First, we still focus on the most critical design load case 50 years wind reduced wave. According to the results, we find that for the rotor force, the load ratio keeps the same level around 2. That means the wind load needs to be twice as the design load when the yielding happens. The jacket could bear double of characteristic environmental loads and show certain conservativeness. This could indicate that the design is acceptable. In order to identify the process of the jacket collapse and find the difference in the failure position and stress distribution, the sensitive study of the critical design load case is performed. The failure position is indicated in the figure b, APPENDIX B.

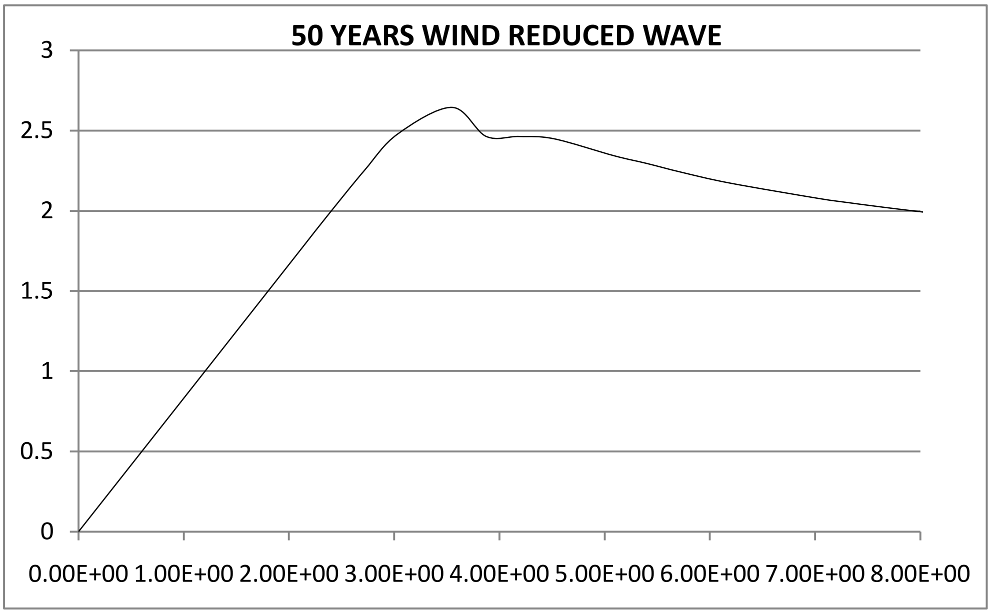

Figure 5.4 Load-deformation relationship of the 70 meters water-depth OFWT fixed bottom jacket under 50 years wind

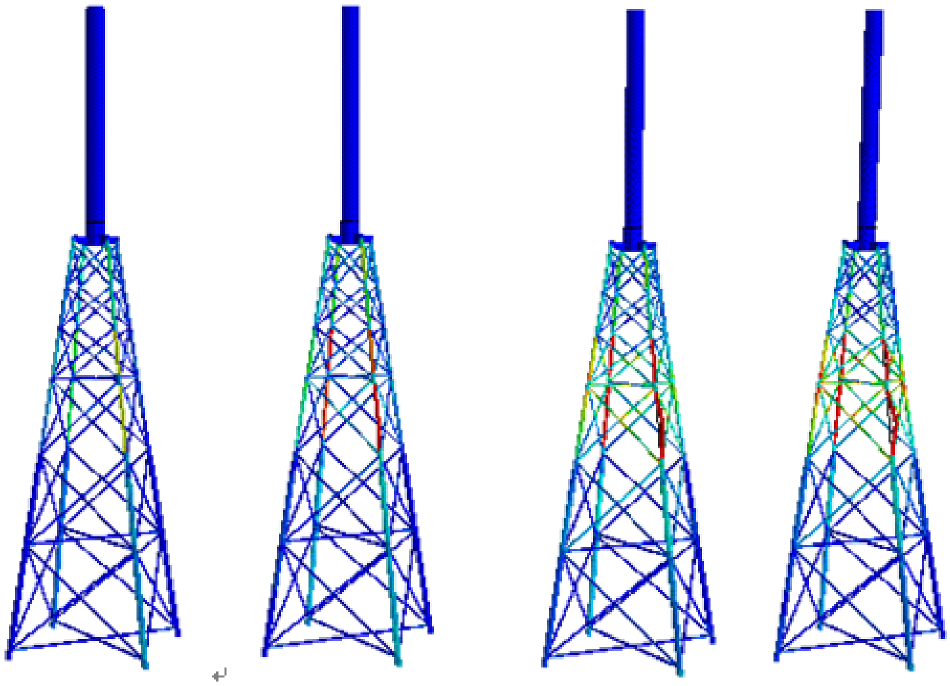

Figure 5.5Failure mechanism for100 meters water-depth fixed bottom jacket under 50 years wind

The load displacement relationship is plotted in figure 5.4.Figure 5.5 shows the pushover failure mechanism for jacket under 50 years wind reduced wave load case. The failure position is indicated in the figure b, APPENDIX B. The response is virtually linear until the first yielding happened at the element 1262 and 1263 the compression leg on the third story. The compression legs on the fourth storey fail in rapid sequence. By further load increasing, tension leg element at same storey yield, because of the load redistribution, the horizontal X-brace starts to fail afterwards. Subsequently, the other legs start buckling due to compression. After first members group buckling, the collapse occurs immediately. The post buckling capacity reduction of the compression leg does not impair the load carrying of the tension leg due to the stress redistribution in the horizontal and vertical braces. The failure of these braces is one of the fundamental reasons the jacket collapse finally. In the end, the jacket reaches the ultimate strength when the tension leg element at the third storey failure. The jacket could not take any increasing load. The load deformation responses for other load cases are tabulated in the APPENDIX C.

5.4.3 Comparison with 70 Meters Water Depth OFWT Jacket

By comparing with the ULS analysis result with the original jacket for 70m water depth, we observe that the load ratios for different load cases remain the same level. The size and direction of the combined horizontal wind loads is the main factor that determines the how many time the design load jacket could undertake. Also we notice that compared with the failure mechanism with original 70 meter water depth jacket, the failure place and process is exactly similar. For all load cases, the ultimate capacity depends on load ratio jacket could sustain until tension legs element 880 on the fourth storey fail. For shut down environmental condition, the first failure mostly starts from the element 1232, compression leg on the third storey which corresponds to the compression legs on the second storey of the 70 meters water depth jacket. That means for both of the jacket, the hot spots are same in these load cases. For the operating condition, except production 2, the first failure place is generally located on the fourth floor element 1262 of the jacket with the first element buckling on the third storey. Actually, no matter which storey fail fist, third or fourth storey, the other left always fail in rapid sequence. In analysis they almost fail at the same load ratio. Therefore, we define the compression legs on both of the third and fourth storey are hot spots.

5.4.4 Discussion

The new design 100 meters water-depth OFWT jacket structure can pass the ULS design check, and present some conservativeness. The load ratio for the different load cases mostly remain in the same region, that means the new design for the deeper water adopt the same safety factor during the original design based on DNV-OS-J101 [7] .The sensitivity study shows the compression leg on third and fourth storey is the most critical place where the yielding and buckling first happen. It also indicated that jacket is still sensitive to the direction of the wind load as previous 70 meters water-depth jacket. Therefore, we also need to consider the regime of the wind and variable wind direction during substructure design to avoid the annual most frequent wind direction which is located on the 45 degrees of the jacket.

5.5 Pushover Analysis for 70 Meters Water-depth OFWT Jacket with Pile Foundation

As we have mentioned in Chapter 3 and Chapter 4, jackets structures are normally piled to the sea floor. The purpose of the pile is to take the axial bearing actions due to gravity load and overturning moment caused by environmental actions and to support the global base shear force from the environmental actions. The nonlinear load displacement relationships of soil-pile interaction are conveniently modeled with piece wise linear springs in USFOS. In order to investigate the behavior of the structure with pile foundation at Ekofisk oil field, the pushover analysis of substructure with design pile foundation is performed.

5.5.1 Results

The dimension of the pile foundation for 70 meters water depth substructure has been discussed in Chapter 3. The length of the pile foundation connected with each leg ends is 45 meters with the constant thickness 25 millimeters. From bottom to the -2 meters soil depth, the diameter of the pile is 2 meters with the constant 1.8 meters diameters at other soil depth. We still adopt the same environmental condition and load cases as previous analysis. The results of the pushover analysis are tabulated as follow. Because soil-pile interaction, during the increasing the design load, the snap shot point will be generated in the load-displacement relationship diagram. Therefore, we document the load ratio first snap shot point generated. The critical element position is indicated in the figure a and figure c, APPENDIX B.

Result for operating condition:

Production 1	Initial Yield in Jacket	First Snap Shot	Ultimate collapse load
Failure place	1215 YIELD END2	9007 Soil Lay09	9005 Pile Lay02
Load Ratio	3.51762963	4.032592593	3.416888889

Table 5.18 70 meters water-depth pile foundation jacket critical load ratio for Production 1

Production 2	Initial Yield in Jacket	First Snap Shot	Ultimate collapse load
Failure place	1239 YIELD END2	9007 Soil Lay09	9008 Soil Lay08
Load Ratio	5.303111111	6.4	6.443259259

Table 5.19 70 meters water-depth pile foundation jacket critical load ratio for Production

Extreme Gust1	Initial Yield in Jacket	First Member Buckle	Ultimate collapse load
Failure place	867 YIELD MID	867 BUCKLE	9005 Soil Lay10
Load Ratio	2.360888889	3.192296296	3.278814815

Table 5.20 70 meters water-depth pile foundation jacket critical load ratio for Extreme Gust1

Extreme Gust2	Initial Yield in Jacket	First Snap Shot	Ultimate collapse load
Failure place	1240 YIELD END1	9008 Soil Lay09	9007 Soil Lay09
Load Ratio	2.304	2.608	2.608

Table 5.21 70 meters water-depth pile foundation jacket critical load ratio for Extreme Gust2

Result for shut down condition:

50years wind reduced wave	Initial Yield in Jacket	First Snap Shot	Ultimate collapse load
Failure place	1240YIELD END1	9008 Soil Lay09	1214 YIELD END2
Load Ratio	1.733333333	2.115555556	2.010666667

Table 5.22 70 meters water-depth pile foundation jacket critical load ratio for 50years wind reduced wave

50years gust reduced wave	Initial Yield in Jacket	First Snap Shot	Ultimate collapse load
Failure place	1240 YIELD END1	9005 Soil Lay10	1214 YIELD MI
Load Ratio	2.246518519	2.73362963	2.598518519

Table 5.23 70 meters water-depth pile foundation jacket critical load ratio for 50years gust reduced wave

50years wave reduced wind	Initial Yield in Jacket	First Snap Shot	Ultimate collapse load
Failure place	No Yield	9008 Soil Lay09	9005 Soil Lay08
Load Ratio	No Yield	2.465185185	2.645333333

Table 5.24 70 meters water-depth pile foundation jacket critical load ratio for 50years wave reduced wind

In the results, we observe that the smallest load ratio during the initial yielding in jacket is 1.733. For the ultimate capacity, the smallest load ratio the substructure could undertake is 2.01.That means for the rotor force needs to be almost twice as the design load when the initial yielding happens in jacket. It implies a safety factor of 1.7, and it is higher than the required 1. The jacket could undertake double of design environmental load and show certain conservativeness, the design of pile foundation is acceptable. Compared with pushover results from fixed bottom jacket, the load ratio at yielding and ultimate capacity all decrease slightly, such as for the critical load case, the load ratio decrease from 1.8 to 1.7 , 2.6 to 2.0 at first yielding and ultimate capacity respectively . That could indicate the pile foundation make the substructure system unstable. We also notice for load case Extreme gust 1, the snap shot point was not generated during pushover analysis. The load-deformation response looks exactly the same as jacket without piles. For the load case 50 years wave reduced wind, there is no failure occurring at the supporting jacket. The reason of the substructure collapse is the failure in the different soil layers. Therefore, considering the soil-pile interaction, the collapse mechanism of the supporting jacket is more complicated than the supporting jacket with fixed bottom. The detail discussion will be carried on in the next sections.

5.5.2 Critical Load Cases Sensitivity Study

In order to identify the process of the jacket collapse and find the difference in the failure position and stress distribution, the sensitive study of the critical design load case is performed. Based on the results of pushover analysis in previous section, we only focus on the load cases which could cause extreme base shear and overturning moment on the seabed in this section. Except the most critical load case from the previous analysis, the 50 years wind reduced wave, we also adopt the load cases Extreme Gust 1 and 50 years wave reduced wind into account. The former has wind load and wave in opposite direction. There is larger base shear and overturning moment from the wave and current for the latter load case compared with critical load case and the direction of the wind remains the same. First, we focus on the critical load case 50 years wind reduced wave. The load deformation relationship is plotting below. The failure mechanism is plotted following:

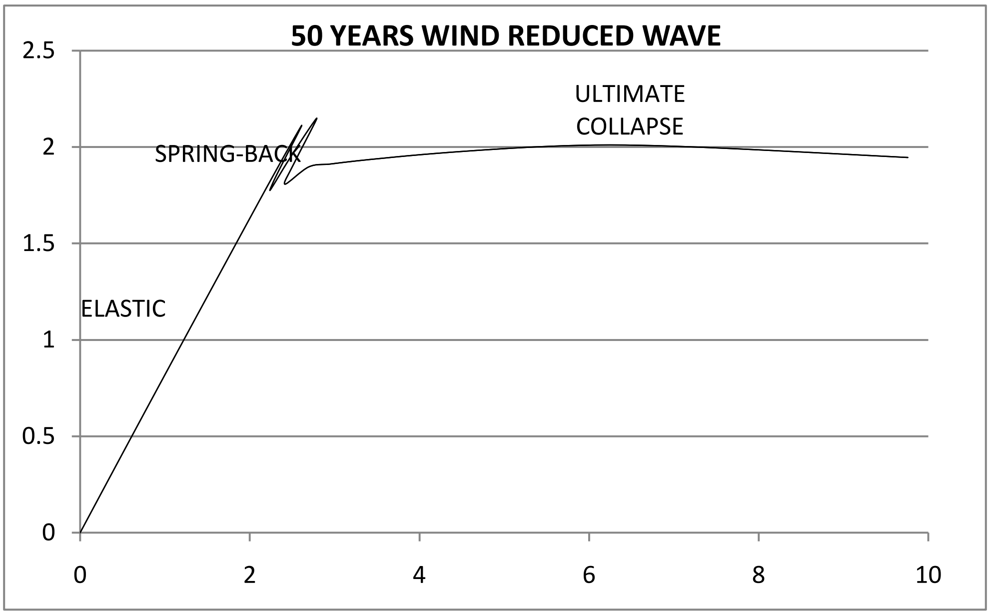

Figure 5.6 Load-deformation relationship of the 70 meters water-depth pile foundation jacket under 50 years wind

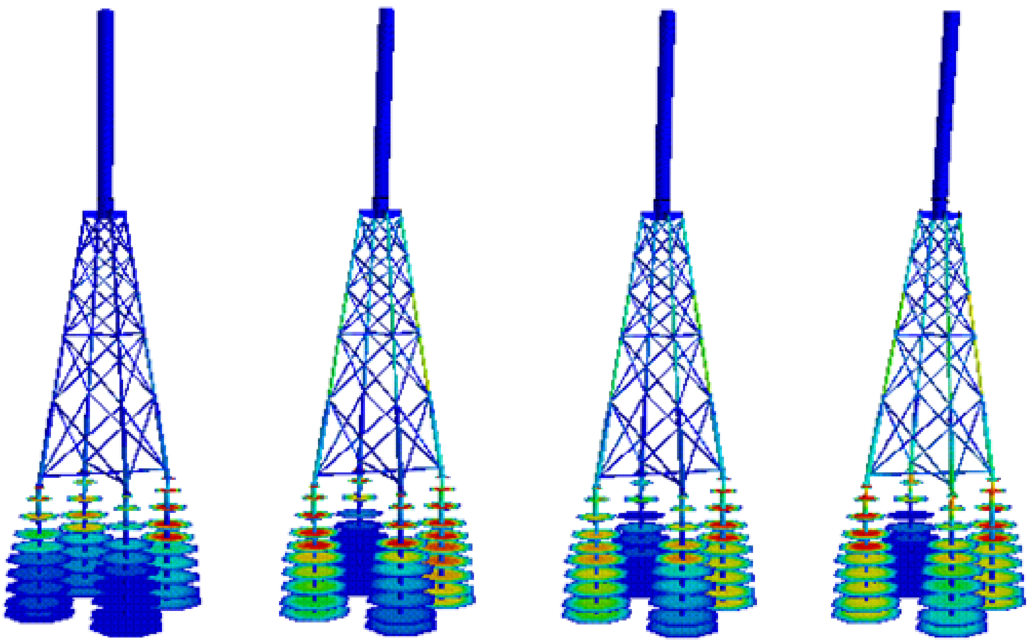

Figure 5.7Failure mechanism for70 meters water-depth pile foundation jacket under 50 years wind

The response of the supporting substructure is virtually linear until the yielding of soil layer 9 at -43 meters soil depth take place. The soil in the first and second layer starts to yield at pile connected to the compression legs during the beginning of the elastic increase of the load ratio. The failure of the soil from the upper to deeper layers keeps coming with the increase of the load ratio. Meanwhile, the first yielding of the jacket element happens at the element 1240, the compression legs at the second storey. But these responses don't affect the linear increase relationships. As shown in the plot, the snapshot first is generated at the load ratio 2.12 which is manifested by a reduction in global load carrying because of the yield in soil layer 9 at compression pile 9008. The drop is so dramatic that spring-back occurs. Subsequently, the forces are redistributed to adjacent soil layers at other pile foundation with a pronounced increase in the substructure stiffness. By further loading, the soil layer at the tensile pile yield at the deep depth. This is indentified by another snap shot in load deformation relationship graph. When all soil layers yield, global load carrying is predominately governed by jacket. When this mechanism has formed, the capacity decreases monotonously, but at a relatively slow rate until compression pile reach their maximum displacement. It is interesting to notice that the capacity still is larger than 2 times the design load. The response also indicates the piles have strong effect to the failure mechanism and ultimate capacity of substructure system. The critical failure position is drawing in the figure a and figure c, APPENDIX B. The load deformation responses for other load cases are tabulated in the APPENDIX C.

5.5.3 Compression with Other Critical Load Cases

The pile foundation is sensitivity to the wave load and direction since the corresponding base shear and overturning moment will be induced directly on the bottom of jacket. Compared with load case 50 years wind reduced wave, the load case 50 years wave reduced wind has larger base shear and overturning moment from the wave and current. Therefore, in the pushover analysis, we observe there is no yielding of the jacket components in this load cases. After virtually linear increase in the load deformation relationship, the first snapshot is caused by the yielding in the soil layer 9 at the compression pile. The forces are redistributed to adjacent soil layer and there is a pronounced increase in the stiffness. The yielding of soil at the opposite tension pile induce the second snap point with dramatic decrease of stiffness and second time redistribution force in other pile. After second group of soil reach the ultimate limit, the residual stress taken by the pile connected jacket. Continue increasing the load ratio the collapse occurs immediately at the compression pile 9008, the jacket start to overturn by wind load. The substructure could not take any more load and reach the maximum displacement, the analysis is terminated. The critical failure position is drawing in the figure a and c, APPENDIX B. The load displacement relationship is plotting as follow.

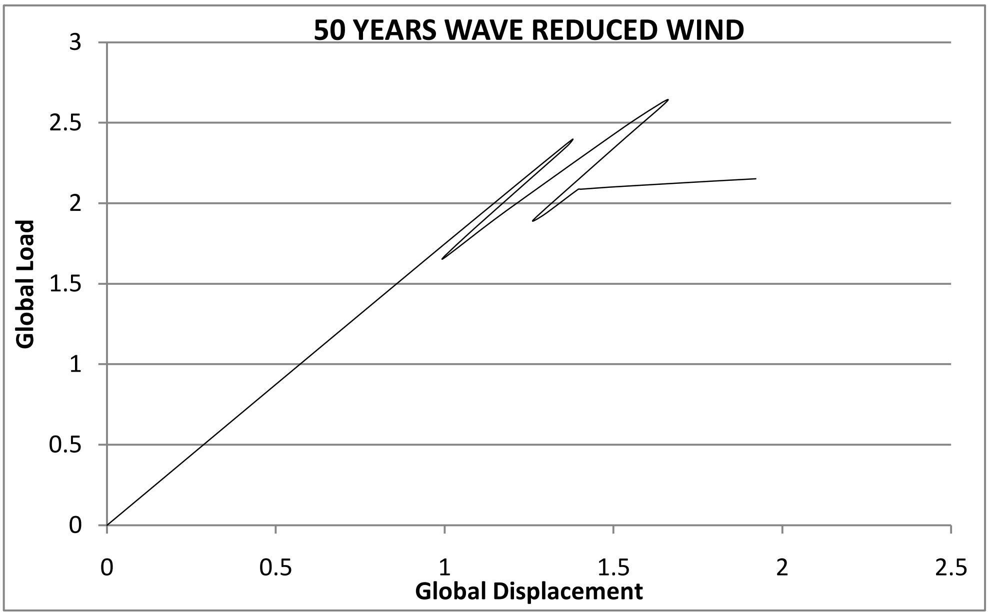

Figure 5.8 Load-deformation relationship of the 70 meters water-depth pile foundation Jacket at 50 years wave

Compared with the fixed end substructure, the pile foundation is very sensitive to the base shear and overturning moment on the seabed. For the load case Extreme Gust1. The load – deformation relationship is as follow.

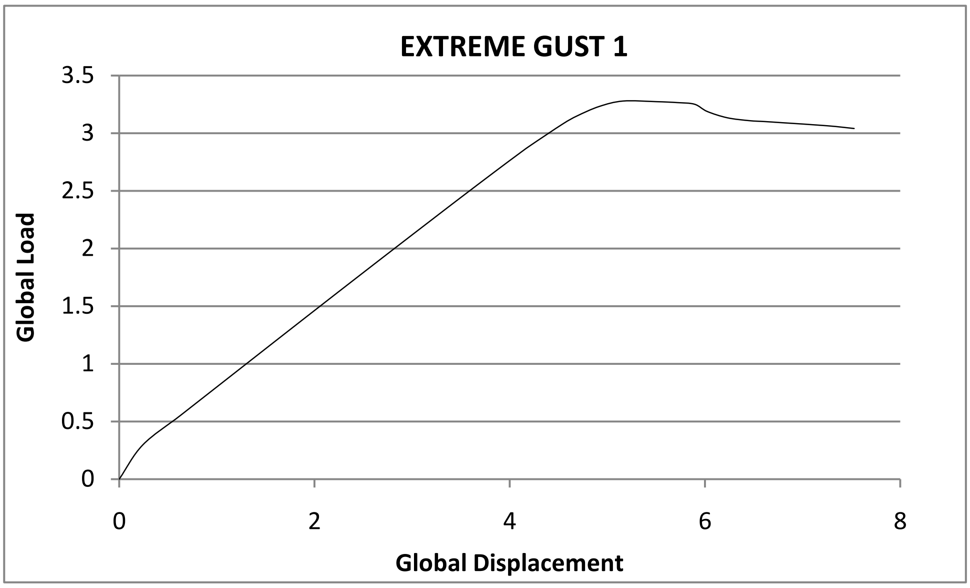

Figure 5.9 Load-deformation relationship of the 70 meters water-depth pile foundation jacket at extreme gust1

For the load case Extreme Gust 1, the direction of the wind load and wave load is opposite with each other. Therefore the base shear and overturning moment on seabed from the wave load will be offset with the increasing wind load. The canceled base shear and overturning moment reduced the compression and tension load at pile foundation. But the force in the leg element increase unaffectedly. In this way, the substructure could be simplified as fixed end supporting jacket. The response is virtually linear until buckling take place at the compression leg on the second storey. The buckling is manifested by a reduction in global load carrying and the substructure reaches its ultimate capacity. The redistribution load is carried by the compression pile, the soil layers therefore yield at the same load ratio. The load deformation relationship is plotted in figure 5.9.

5.5.4 Failure Position

The failure position in Jacket is indicated in the figure a and figure c, APPENDIX B. From the pushover analysis results, except load case extreme gust 1, we could also observe that the first snap shot point of the global stiffness mostly start from the failure of all soil layer at the compression leg. The yielding of the soil layer will induce the dramatic drop of the global loading and the forces will distribute in other layers of tensile leg afterwards. When soil layers of tensile leg reach their yield strength the spring- back phenomenon occurs again in the tensile soil layers. The ultimate capacity depends on load ratio jacket could sustain until soil connected to tension pile fail. In proof of this observation, we focus on the position of failure in the results of pushover analysis. The snap shot point is always induced when soil yield at the layer 9 or layer 10 which is the deepest soil layer pile could reach. Correspondingly, the second snap shot points is caused by the yielding in tensile layers. This could indicate that for the 70 meters water-depth jacket, the failure of substructure under extreme load is dominate by pile foundation. Compared with the pushover analysis results from fixed substructure, we also notice the failure in jacket remain the same position and process. In 7 load cases, the position of first yield of the jacket is always located on the compression legs on the second story, like element 1234, 1240 and 867. The difference is that the substructure reaches its ultimate capacity when all the soil layers fail and could not take more loads. Therefore, we define the greatest possible reason of the 70 meters water depth OFWT substructure collapse is the failure of the soil layers. The most likely failure modes of the foundation are pile pull-out in tension and punch through in compression due to insufficient later strength of the soil.

5.4.5 Discussion

The response of the substructure with pile foundation is more complicated than the fixed ends substructure. The results of the pushover analysis shows that the substructure including pile foundation could pass the ULS design check, and present some conservativeness. The sensitivity study shows the soil connected to the compression piles and compression leg on second storey are the critical place where the yielding easily happening. The upper soil layers from seabed to -8 meters soil layers are too soft to take the axial loads, but the yielding of the upper soil layer won't affect the global stiffness of the substructure because of the load redistribution to the other layers. The yielding of the deep soil layer will induce the dramatic drop of the global loading which will cause the snap shot of the global stiffness. The failure of the deep soil connected to the tension legs will cause the substructure reach its ultimate capacity because of piles pull-out in tension and punch through in compression. Therefore, we could identify the substructure is pile failure dominate considering the soil-pile interaction. It also indicated that the pile foundation is sensitivity to the wave load and direction since the corresponding base shear and overturning moment is induced directly on the bottom of jacket. Except the regime of the wind and variable wind direction during substructure design we also need to pay attention to the direction of the wave load. In addition, the soil property in different layers is also important to the ultimate and response of the substructure.

5.6 Pushover Analysis for 100 Meters Water-depth OFWT Jacket with Pile Foundation

In the similar way, we perform the pushover analysis for the 100 meters water-depth OFWT jacket considering the soil pile interaction. In order to support the larger functional loads and environmental load, the length of the pile foundation connected with each leg ends increases to 55 meters with constant thickness 25 millimeters. From bottom to the -2 meters soil depth, the diameter of the pile is 2 meters with the constant 1.8 meters diameters at other depth.

5.6.1 Results

Depends on the results from previous pushover analysis for 70 meters water depth substructure with piles, we pay much attention on critical load cases 50year wind reduced wave. The results of the pushover analysis are tabulated below:

Operating load cases:

Production 1	Initial Yield inJacket	First Buckle Member	Ultimate collapse load	Initial Yield in Pile
Failure place	1261 YIELD END2	9007 Pile Lay02	1261 BUCKLE	852 PLAST END2
Load Ratio	3.495111111	4.701037037	4.739555556	5.083259259

Table 5.25 100 meters water-depth pile foundation jacket critical load ratio for Extreme Gust1

Production 2	Initial Yield inJacket	First Buckle Member	Ultimate collapse load	Initial Yield in Pile
Failure place	1261 YIELD END2	1261 BUCKLE	9007 Pile Lay02	881 PLASTEND1
Load Ratio	5.194074074	7.062518519	7.415703704	7.508740741

Table 5.26 100 meters water-depth pile foundation jacket critical load ratio for Extreme Gust2

ExtremeGust1	Initial Yield inJacket	First Buckle Member	Ultimate collapse load	Initial Yield in Pile
Failure place	881 YIELD MID	881 BUCKLE	1232 UNLOD END1	No Yield
Load Ratio	2.363851852	3.08562963	3.276444444	No Yield

Table 5.27 100 meters water-depth pile foundation jacket critical load ratio for Extreme Gust1

ExtremeGust2	Initial Yield inJacket	First Buckle Member	Ultimate collapse load	Initial Yield in Pile
Failure place	1262 YIELD END2	9008 Pile Lay02	1262 BUCKLE	1232 BUCKLE
Load Ratio	2.447407407	2.928	3.178074074	3.434666667

Table 5.28 100 meters water-depth pile foundation jacket critical load ratio for Extreme Gust2

50years Wind	Initial Yield in Jacket	Initial Yield in Pile	First Buckle Member	Ultimate collapse load
Failure place	1262 YIELD END2	9008 Pile Lay02	1262 BUCKLE	1232 BUCKLE
Load Ratio	1.828740741	2.407703704	2.425481481	2.581925926

Table 5.29 100 meters water-depth pile foundation jacket critical load ratio for 50years wind reduced wave

50years Gust	Initial Yield inPile	Initial Yield in Jacket	First Buckle Member	Ultimate collapse load
Failure place	1262YIELDEND2	9008 Pile Lay02	1262 BUCKLE	880 YIELD END2
Load Ratio	2.369185185	3.112296296	3.112296296	3.389037037

Table 5.30 100 meters water-depth pile foundation jacket critical load ratio for 50years gust reduced wave

50years Wave	Initial Yield in Pile	Initial Yield inJacket	First Buckle Member	Ultimate collapse load
Failure place	9008 Pile Lay02	1262 YIELD END2	1262 BUCKLE	9005 Soil Lay08
Load Ratio	2.154666667	2.884740741	4.183111111	4.70162963

Table 5.31 100 meters water-depth pile foundation jacket critical load ratio for 50years gust reduced wave

In the results, we observe for the first member yielding and ultimate capacity load, the smallest load ratio is 1.8 and 2.4 respectively from the shut down design load case 50 years wind reduced wave (shown in red color). That means for the same safety factor, this load case gives worst bending moment and compression of the jacket element. It implies a safety factor of 2.4, and it is higher than the required one 1. The rotor force needs to be more than 1.8 times as the design load when the initial yielding happens in jacket. This could indicate that the design is acceptable. Compared with 100 meters water-depth fixed jacket, the load ratio remains almost same. That seems like the pile foundation won't affect the substructure response. However, we also notice except for the load case Extreme Gust 1, there is yielding of the pile in the process. For the load case 50 years wave reduced wind, the pile even fail before the initial yielding of the jacket. The substructure could take the increasing load until the jacket element buckling when they reach the ultimate capacity. Therefore, considering the soil pile interaction, the collapse mechanism of the supporting jacket is more complicated based on different load case. The detail discussion will be conducted in the next section.

5.6.2 Critical Load Cases Sensitivity Study

The sensitivity study of the critical design load case 50 years wind reduced wave is performed in the similar way as previous section to identify the failure mechanism of the substructure. The load deformation relationship is plotting below. The following is the failure mechanism. The response of the supporting substructure is virtually elastic linear, the upper soil layers fail at compression pile and yielding at compression leg affect little on the linear increase until first element buckling at the compression leg on third storey. The compression pile foundation at the upper layer also yield during the elastic increase. Subsequently, some of the loads redistributed in the x braces and deep soil layers and the global stiffness of the substructure decrease. The strength reserved beyond first member buckling is remarkably small. After slightly increases of the global load, the substructure reaches its ultimate capacity when the compression leg at fourth storey buckling. This could be indicated in the peak of curve in the figure 5.10. The critical failure position in Jacket is indicated in the figure b and figure cf, APPENDIX B.

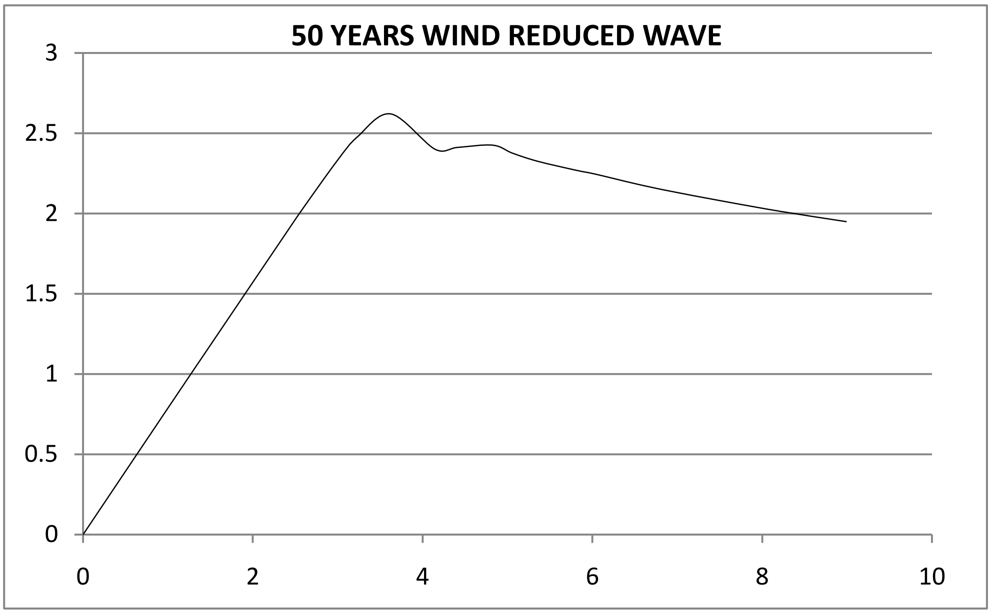

Figure 5.10 Load-deformation relationship of the 100 meters water-depth pile foundation jacket under 50 years wind

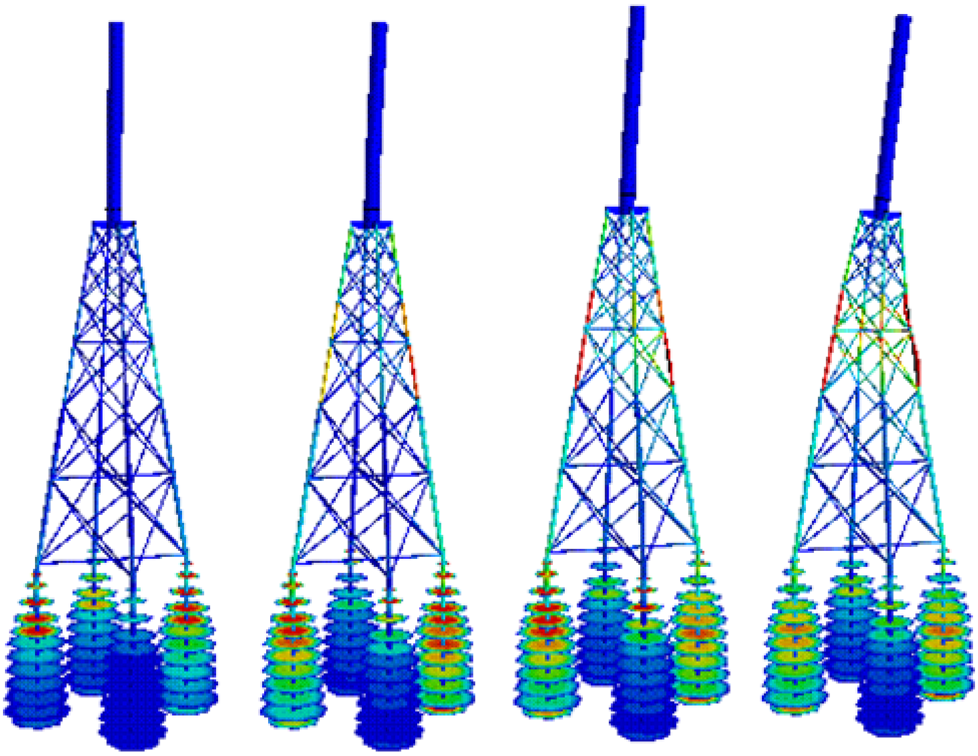

Figure 5.11 Failure mechanism for100 meters water-depth pile foundation jacket

After jacket reach their ultimate capacity, the capacity of the substructure decreases monotonously with slow rate. Because of the redistribution of load in the tension pile, the stiffness of the substructure increase slightly. But with the yielding of the deeper soil layers connected to compression pile, the load at the layer reach a steady state, the compression leg at fourth storey buckling with the increasing load. It is interesting to observe the deepest soil layer 11 and 12 didn't yield during pushover, which avoid the pull-out in tension and punch through in compression. We also notice that except the redistribution of load after peak, the load displacement relationship looks the same as substructure with fixed bottom. Although the pile at upper layers yield with soil layers yield with increasing load ratio, that is not the main reason of whole substructure collapse. The load deformation responses for other load cases are tabulated in the APPENDIX C.

5.6.3 Compression with Other Critical Load Cases

Except the most critical load case from the previous analysis, the 50 years wind reduced wave, we also adopt the load cases Extreme Gust 1 and 50 years wave reduced wind into account. The former has wind load and wave in opposite direction. There is larger base shear and overturning moment from the wave and current for the latter load case compared with critical load case and the direction of the wind remains the same.

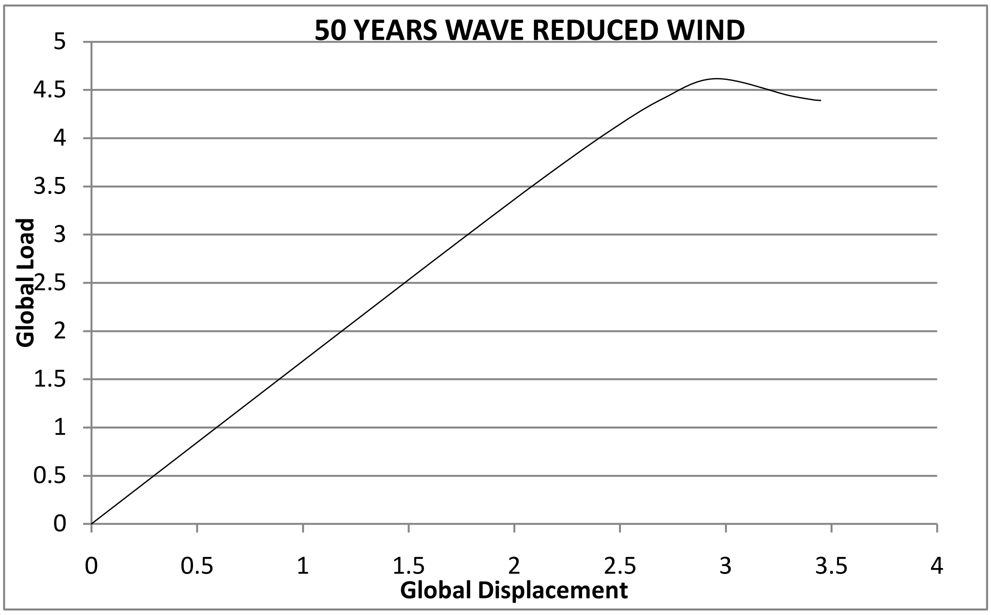

Figure 5.12 Load-deformation relationship of the 100 meters water-depth pile foundation jacket at 50 years wave

We observe the pile foundation yield before the failure of the jacket component in load case 50 years wave reduced wind. From the load displacement relationship graph, we also observe that the failure of the pile doesn't affect the linear increase in the load deformation relationship. The graph looks exactly the same as 100 meters water-depth jacket with fixed bottom. The reason is that the increasing length of piles in deeper soil layers 11 and 12 makes the pile foundation fast or secure which stop the piles pull-out in tension and punch through in compression When upper layer or pile element yielding, the load could redistribute fast to the adjacent soil layer and increase the stiffness of the pile foundation fast. In opposition to load case 50 years wave reduced wind, for the load case Extreme Gust 1, the direction of the wind load and wave load is opposite with each other. Therefore the base shear and overturning moment on seabed from the wave load will be offset with the increasing wind load. The canceled base shear and overturning moment reduced the compression and tension load at pile foundation. Therefore the pile doesn't fail in this load case. However, when tension jacket leg at fourth storey buckling, several failed members unload intermediately into the elastic range. The load redistribute consequently in the soil layer to make the substructure stable and regain considerable strength. This could be reflected from the load displacement relationship graph as follow. Anyway, the global response of the substructure system is dominated by the jacket although the piles yield at upper layers.

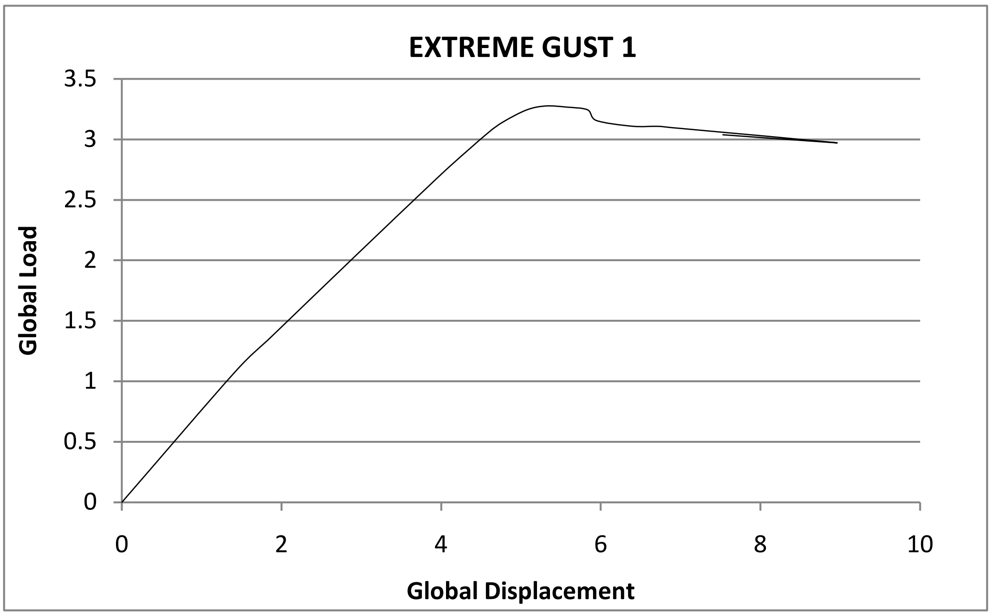

Figure 5.13 Load-deformation relationship of the 100 meters water-depth pile foundation jacket at extreme gust 1

5.6.4 Failure Position

The failure position is indicated in the figure b and figure c, APPENDIX B. From the pushover analysis results, except load case 50 years wave reduced wind, we could observe that the first failure mostly start from the compression leg at third storey. It is the same critical position as fixed bottom jacket. The yielding of the upper soil layer won't affect the global response since the forces will distribute in deeper layers afterwards. However, the pile yields first at upper soil depth from seabed to the -8 meters soil depth because the increasing wave height induced larger base shear and overturning moment. For other load cases the pile yielding in the pushover process after the compression legs fail. There is no failure of pile in the load case Extreme Gust1 because of the offset of the base shear and overturning moment from the wind and wave in opposite direction. The failure of the pile and soil layers affect little on the global response of the substructure because of the fast redistribution of the loads to deep soil layer, also the strong soli layer under -48 soil depth. The strong soil layers give the solid foundation of the substructure, therefore compared with the 100 meters water depth fixed ends substructure, the safety factor of pushover analysis and load displacement relationship alter little. So we could borrow the conclusion from pushover analysis for 70 meter water depth fixed ends substructure, the compression legs on both of the third and fourth storey, the piles at upper soil layers are hot spots.

5.6.5 Discussion

The results of the pushover analysis shows that the substructure including pile foundation could pass the ULS design check, and present some conservativeness. Compared with the 70 meters water depth substructure with pile foundation, because of the increasing length of the pile foundation to the solid soil layer, the 100 meters water depth substructure is insensitive to the yielding of the soil layer since deeper layer give capacity to avoid pull-out in tension and punch through in compression of piles. Instead of this, the piles at the upper soil layers are the hot spots where the compression piles yielding in the process of the pushover analysis. So, as designer, we should increase the diameter of the pile at upper layer to give more capacity to pile to undertake increasing base shear and overturning moment. However, the failure of the soil and pile doesn't affect the global response of substructure system. The strong deep soil layers give the solid foundation of the substructure, therefore compared with the 100 meters water depth fixed ends substructure, the safety factor of pushover analysis and load displacement relationship alter little. The compression legs on both of the third and fourth storey are real hot spots which dominating the substructure collapse.

6 API Design Code Checking

6.1 Objective

Through previous pushover analysis, we estimate the whole OFWT substructure ultimate capacity and design conservativeness for the characteristic load case. However, in the real process of the design, the relevant design codes and standards will be employed on all the elements of the structure to check the strength and stability requirements. Section 6.2 describes the theory of the API design code checking of steel cylindrical member. The API design code checking of the OFWT substructure will be performed in next few sections.

The API LRFD criterion and API WSD criterion are two main design standards issued by the API, American Petroleum Institute, in offshore design field. The Working Stress Design (WSD) is an implicit or explicit design criterion with a single usage factor accounting for all uncertainties. The result of WSD is based on stresses. The Load and Resistance Factor Design (LRFD) is a deterministic design criteria with partial safety factor, the result is based on the forces and moments capacity [17]. They are almost identical except for some material and load ratios in the linear analysis. Therefore, for the traditional design, the WSD should give same result as LRFD. For the characteristic load case, as we found in the pushover analysis, the results remain in the linear elastic section. And the pile foundation is designed by Dr Saha is based on LRFD method [13]. Therefore, we focus on LRFD code checking in this section because of slight difference for WSD and LRFD.

6.2 API LRFD Requirement

The structural strength and stability requirements for steel cylindrical members are specified in the API RP 2A-LRFD [13] Section D. According to this rule the following basic safety factor requirements shall be satisfied:

$$\sum_{i} \varepsilon_i S_i \leq R_c/\beta_c$$

Where$S_i$is characteristic load effect component i, ε_i is partial safety factor for the load effect component. R_c is the characteristic resistance with respect to the relevant failure mode and.β_c is the material factor. The code checking module return a utilization factor defined as follows:

$$\text{Utilization Factor} \quad = \frac{\sum_i \varepsilon_i S_i}{R_c/\beta_c}$$

If the utilization factor is larger than 1 that means the design is failure.

In detail, the utilization factor of cylindrical members under axial compression, bending, shear, or hydrostatic pressure have been designed to satisfy the strength and stability requirement specified in the API RP 2A-LRFD [13] respectively. According to the pushover analysis in previous paragraphs, for the 70 meters and 100 meters water depths OFWT substructure, , the compression leg element at certain story, is the first cylindrical member failure happen under combined axial compressive and bending loads. The failure of the tension leg is also significant for the global failure which will cause the pronounced decrease in the substructure stiffness. Therefore, during this part, we only focus on combined compressive and bending load condition and combined axial tension and bending respectively. The other utilization factor format for different load cases is attached in Appendix.

6.2.1 Utilization Ratio of the Cylindrical Members under Combined Axial Tension and Bending

The utilization factor under combined axial tension and bending moment should be designed to satisfy the following condition at all cross section along their length:

$$\text{Utilization Factor} \quad = 1 - \cos\left[\frac{\pi(f_t)}{2\emptyset_t F_y}\right] + \frac{\left[{f_{by}}^2 + {f_{bz}}^2\right]^{0.5}}{\emptyset_b F_{bn}} \leq 1.0$$

F_y=nominal yield strength, in stress unit

F_{bn}= nominal bending strength, in stress unit

$\emptyset_t$=resistance factor for axial tensile strength, 0.95

$\emptyset_b$= resistance factor for axial compressive strength, 0.95

f_t= axial tensile stress due to factored loads

f_{by}= bending stress about member y-axis due to factored loads

f_{bz}= bending stress about member z-axis due to factored loads

6.2.2 Utilization Ratio of the Cylindrical Members under Combined Axial Compression and Bending

The utilization factor under combined axial compressive and bending moment should be designed to satisfy the following condition at all cross section along their length:

For beam column buckling

$$\text{Utilization Factor} \quad = \frac{f_c}{\phi_c F_{cn}} + \frac{1}{\phi_b F_{bn}} + \left\{ \left[\left(\frac{C_{my} f_{by}}{1 - \frac{f_e}{\phi_c F_{ey}}} \right) \right]^2 + \left[\left(\frac{C_{mz} f_{bz}}{1 - \frac{f_e}{\phi_c F_{ez}}} \right) \right]^2 \right\}^{0.5} \leq 1$$

For local buckling due to bending

$$\text{Utilization Factor} \quad = 1 - \cos\left[\frac{\pi(f_c)}{2\phi_c F_{xc}}\right] + \frac{\left[{f_{by}}^2 + {f_{bz}}^2\right]^{0.5}}{\phi_b F_{bn}} \leq 1$$

For local buckling due to compression

$$F_{cn} < \phi_c F_{xc}$$

(D.3.2-1~D.3.2-3 from API RP 2A-LRFD)

Where

F_{xc}=nominal inelastic local buckling strength, in stress unit

F_{cn}=nominal axial compressive strength, in stress unit

F_{bn}= nominal bending strength, in stress unit

ϕ_c=resistance factor for axial compressive strength, 0.85

ϕ_b= resistance factor for axial compressive strength, 0.95

f_c= axial compressive stress due to factored loads

f_{by}= bending stress about member y-axis due to factored loads

f_{bz}= bending stress about member z-axis due to factored loads

C_{my}, C_{mz}=reduction factors corresponding to the member y and z axes, respectively

F_{ey}, F_{ez}=Euler buckling strengths corresponding to the member y and z axes, respectively

$$F_{ey} = {F_y}/{\lambda_y}^2 \quad F_{ez} = {F_z}/{\lambda_z}^2$$

λ_y, λ_z=column slenderness parameters for the member y and z axes defined by the equation $\frac{KL}{\pi r}\left[\frac{F_y}{E}\right]^{0.5}$, where the parameters K,L and r are chosen to corresponded to bending in y and z direction, respectively. In this way, if the compression or tension stress and bending of the cylindrical member is known, the utilization of this element could be calculated using the equation with the corresponding factors from the API RP 2A-LRFD.

6.3 API Design Code Check for 70 Meters and 100 Meters Water-depth OFWT Jakcet with Fixed Bottom

The API design code checking for OFWT fixed bottom substructure in 70 meters and 100 meters water-depth is performed in this section. The maximum utilization ratio is hand calculated for the each critical jacket cylindrical element using equalization mentioned in previous section. First, functional actions including permanent actions and live actions are incremented up to design value. Second, the design load and environmental condition are increased up to the substructure. The load factor 1.35 and 1.30 from the API 2R LRFD are applied respectively for environmental load and dead load to convert the characteristic load cases into design load cases during the beginning of the analysis. We adopt the same characteristic environmental condition and load cases as pushover analysis, so we could make the comparison with different methods results. The basic material factor is by default 1.15 and is automatically accounted for the in the design checks.

6.3.1 Results

With different load case, the utilization ratio is hand calculated for the heaviest stressed element with the corresponding element forces in six degree of freedom. For each load case, the utilization ratio for 4 heaviest stressed elements and element numbers are tabulated in following paragraph. The critical element position is identified in APPENDIX B.

70 Meters Water Depth Fixed end Substructure					
Production1	Element	1239	1240	1215	1214
	Utilization ratio	0.52	0.47	0.36	0.32
Production2	Element	1239	1240	1215	1214
	Utilization ratio	0.47	0.42	0.32	0.27
Extreme Gust1	Element	867	917	916	866
	Utilization ratio	0.36	0.35	0.33	0.31
Extreme Gust2	Element	1240	1239	1214	1215
	Utilization ratio	0.62	0.57	0.46	0.44
50 years wind	Element	1240	1214	1163	1188
	Utilization ratio	0.7	0.58	0.35	0.33
50 years gust	Element	1240	1214	1266	1163
	Utilization ratio	0.62	0.49	0.29	0.28
50 years wave	Element	1240	1214	842	1266
	Utilization ratio	0.51	0.39	0.25	0.24

Table 6.1 Utilization ratio for 4 heaviest stressed element in fixed bottom 70 meters water depth substructure

First, we focus on the utilization ratios for the 70 meter water depth substructure. We observe that the utilization ratios in the operating and shut down load cases are all smaller than 1 which could prove the design is feasible and show some conservativeness. For each load case, the largest utilization ratio is located on the element 1239 and 1240, the compression leg on the second story, where the initial yielding and buckling usually happen. This has been indicated in the pushover analysis with same result. The largest utilization ratio is 0.7 from shut down load case 50 years wind reduced wave. The interval of largest utilization ratio for all load cases is from 0.7 to 0.47.The compression leg element on the third story has the following large utilization ratio around 0.3 to 0.4 for the operating load condition. In contrary, because of the different incoming wave direction, the third and fourth largest utilization ratios are from compression leg on first of fourth storey. But these utilization ratios are all smaller than 0.5 which is not important. The table for 100 meters water depth substructure is below.

100 Meters Water Depth Fixed end Substructure					
Production1	Element	1261	1262	1233	1232
	Utilization ratio	0.51	0.46	0.39	0.35
Production2	Element	1261	1262	1233	1232
	Utilization ratio	0.44	0.4	0.34	0.29
Extreme Gust1	Element	881	933	932	930
	Utilization ratio	0.37	0.36	0.33	0.31
Extreme Gust2	Element	1262	1261	1232	1233
	Utilization ratio	0.6	0.58	0.49	0.43
50 years wind	Element	1262	1232	1179	1204
	Utilization ratio	0.69	0.61	0.36	0.35
50 years gust	Element	1262	1232	1314	1289
	Utilization ratio	0.6	0.51	0.31	0.27
50 years wave	Element	1262	1232	1314	1289
	Utilization ratio	0.48	0.38	0.26	0.24

Table 6.2 Utilization ratio for 4 heaviest stressed element in fixed bottom 100 meters water depth substructure

The largest utilization ratio is 0.69 which is obtained from the compression leg element on the third storey with the shut down load case 50 years wind reduced wave. Since the largest utilization ratio smaller than 1 and presents some conservativeness, the design is acceptable for the API LRFD design code. The compression leg on the third storey is also the critical position with the largest utilization ratio for other load cases. The compression leg elements on the fourth storey also have a large utilization ratio around 0.4 in the operating load cases. For shut down condition, the large utilization is located at the first and fifth storey. This distribution of the utilization ratio is similar as original 70 meters water depth substructure. We also notice that the utilization ratio remain in the same level as original 70 meters water depth substructure. This could prove the feasibility of the 100 meters water depth substructure design. Compared with the pushover analysis results, the position has largest utilization ratio is also the critical place the initial yielding and buckling usually happen. The detailed comparison between API code check and pushover analysis will be assessed in next section.

6.3.2 Comparison with Pushover Analysis

According to the API design code check in previous section and pushover analysis results, we have concluded that the critical position where the utilization ratio is large is also the critical place in pushover analysis where the yielding and buckling first happen. The results also indicate, for each cylindrical element, the size of the utilization ratio is relevant to the order of the failure in pushover process. In order to find the difference and relation between API design code check and pushover analysis, further analysis is performed in this section. We select the critical load case 50 years wind reduced wave for both of the substructure in different water depth. According to the pushover analysis, the first element yield at the load ratio 1.8015 times design load. The first yielding is located on the element 1240, the compression leg at second storey (*Table 5.6 Critical load factor for 50years wind reduced wave*). Therefore we apply this design load and increase it until element 1240 yielding and make the API design code check for the same critical element. The utilization ratio of the element 1240 is 1.08 at the first element failure, which is little bit larger than the allowable 1. As we have mentioned in chapter 5, for the pushover analysis, we identify load ratio at this moment as critical load with the safety factor 1.This indicate the API utilization ratio could be used to detect the failure of the element, the result is comparative conservative. In order to verify the validity of the results, we pick up first four failure elements and corresponding load ratio when they fail. Applying the design load and increasing it until the failure happened in first few elements, then the load ratios and the utilization ratios are obtained. Ideally, the utilization ratio should be equal to 1 when the element failure occurs.

Failure Element	Load Ratio	Utilization Ratio
1240	1.8015	1.08
1214	1.8519	1.02
866	2.2696	1.09
841	2.3330	1.01

Table 6.3Utilization ratio for first 4 failure element during yielding

We observe that the utilization ratio are all little larger than 1, when corresponding element yielding at the moment. That means we could predict and detect the failure using the utilization ratio. The pushover method and API design code check could produce same prediction to the ultimate capacity of the structure. The results of the API design code check are conservative compared with pushover analysis.

6.3.3 Discussion

Through the hand calculation of the utilization factor for heaviest stressed element under design load cases, we could find the largest utilization ratio smaller than 1 and presents some conservativeness, the design is acceptable for the API LRFD design code. Compared with the results from the pushover analysis, the critical load cases 50 years wind reduced wave remains the same with the largest utilization ratio at the compression legs, where is also the hot spot found in pushover analysis. The size of the utilization ratio is relevant to the order of the failure in pushover process. The pushover method and API design code check could produce same prediction to the ultimate capacity of the structure. For ULS design check, the results from API LRFD design code check are little conservative compared with static pushover analysis.

6.4 API Design Code Check for 70 Meters and 100 Meters Water-Depth OFWT Substructure with Pile Foundation

In the similar way, the API design code checking for OFWT substructure in 70 meters and 100 meters water-depth with pile foundation is performed in this section. The maximum utilization ratio is hand calculated for the each critical jacket cylindrical element using equalization mentioned in previous section. We still adopt the load factor 1.35 and 1.30 from the API 2R LRFD to convert the characteristic load cases into design load cases during the beginning of the analysis. We adopt the same characteristic environmental condition and load cases as previous analysis.

6.4.1 Results

The utilization ratios for the heaviest stressed elements in different load cases are tabulated below. The critical element position is identified in APPENDIX B.

70 Meters Water Depth Fixed end Substructure					
Production1	Element	1239	9005	9007	1240
	Utilization ratio	0.56	0.56	0.55	0.52
Production2	Element	1239	9005	9007	1214
	Utilization ratio	0.52	0.56	0.49	1240
Extreme Gust1	Element	867	917	916	866
	Utilization ratio	0.38	0.37	0.35	0.34
Extreme Gust2	Element	1240	9007	1239	9005
	Utilization ratio	0.65	0.62	0.59	0.58
50 years wind	Element	1240	9007	1214	1266
	Utilization ratio	0.75	0.66	0.62	0.42
50 years gust	Element	1240	9007	1214	1266
	Utilization ratio	0.66	0.62	0.53	0.35
50 years wave	Element	9007	1240	9008	1214
	Utilization ratio	0.56	0.54	0.42	0.41

Table 6.4 Utilization ratio for 4 heaviest stressed element in pile foundation 100 meters water depth substructure

We first consider the utilization ratio for the 70 meter water depth substructure. We observe that the utilization ratios in the operating and shut down load cases are all smaller than 1, the design of the pile foundation is feasible and show some conservativeness. For supporting jacket, the largest utilization ratio is still located on the element 1239 and 1240, the compression leg on the second story, where the initial yielding and buckling usually happen. This has been indicated in the previous API code check for substructure with fixed ends. The difference between the fixed ends substructure is that there are several utilization ratios from the compression pile foundation element between the second and third soil layer where the soil depth is -2 meters to -8 meters. According to the pushover analysis results from previous chapter, there is no yielding in pile even failure of pile-soil interaction dominate the response of jacket. The largest utilization ratio is still located on the element 1240 during shut down load case 50 years wind reduced wave. This shows that for the substructure with pile foundation, the stress distribution is more complicated. The pile at upper layer is also the critical position during the operation of the OFWT, even it doesn’t yield. The interval of largest utilization ratio for all load cases is from 0.75 to 0.52 which is larger than the utilization ratio of the same element from fixed end substructure. That means the 70 meters water-depth jacket for same load case, the element from pile foundation jacket has larger stress. The pile foundation makes the supporting jacket unstable. The table for 100 meters water depth substructure is below.

100 Meters Water Depth Fixed end Substructure					
Production1	Element	9005	9007	1261	1262
	Utilization ratio	0.58	0.56	0.51	0.47
Production2	Element	9005	9007	1261	1262
	Utilization ratio	0.53	0.52	0.47	0.44
Extreme Gust1	Element	881	933	932	880
	Utilization ratio	0.39	0.38	0.35	0.34
Extreme Gust2	Element	9007	9005	1262	1261
	Utilization ratio	0.64	0.63	0.6	0.57
50 years wind	Element	9007	1262	1232	1314

	Utilization ratio	0.68	0.67	0.56	0.42
50 years gust	Element	9007	1262	1232	9008
	Utilization ratio	0.62	0.59	0.47	0.39
50 years wave	Element	9007	1262	9008	1232
	Utilization ratio	0.58	0.49	0.43	0.37

Table 6.5Utilization ratio for 4 heaviest stressed element in pile foundation 100 meters water depth substructure

It's obvious that for the 100 meters water depth substructure with pile foundation, except load case extreme gust1, the largest utilization ratios are all from the piles at upper soil layers. The largest utilization ratio is 0.68 which is obtained from the compression pile element on the soil depth -2 meters to -8 meters when the shut down load case 50 years wind reduced wave is on the jacket. Since the largest utilization ratio smaller than 1 and presents some conservativeness, the design is acceptable for the API LRFD design code check. This also has been indicated in the previous pushover analysis that the pile foundation at upper layer is one of the hot spots, although the pile failure doesn't affect the response of substructure. The reason of the particularity of the load case Extreme Gust 1 is that the opposite direction of the wind and wave loads reduces the base shear and overturning moment. Except the utilization ratio of piles, the compression leg on the fourth storey is also the critical position with the largest utilization ratio for other load cases. For shut down condition, the large utilization is located at the first and fifth storey. Compared with the pushover analysis results, the position has largest utilization ratio is also the critical place the initial yielding and buckling usually happen.

6.4.2 Disscussion

In the pushover analysis with same substructure with pile foundation, we found the compression legs on both of the third and fourth storey, the piles at upper soil layers are hot spots, where is also the largest utilization we got in API design code check. The largest utilization ratios are 0.75 and 0.68 for 70 meters and 100 meters water depth respectively. Since the largest utilization ratio smaller than 1 and presents some conservativeness, the design is acceptable for the API LRFD design code check. Compared with pushover failure mechanism of same substructure, it is interesting to notice that the API utilization ratio could indicate the critical position which is unobvious in the pushover analysis. For the pushover analysis of 70 meters water-depth substructure, there is no yielding at piles. However, the failure of the pile pull-out in tension and punch through in compression is the dominate reason substructure collapse. Therefore, the utilization result could indicate us pay more attention on the pile-soil system. In contrary, for the 100 meters water-depth jacket, even the utilization ratio of pile at upper layers is always largest, there is only one pile yield early than jacket element in 7 load cases. Therefore, for utilization ratio from API code check, we need to focus on first few highest stressed elements, not only on the largest one. It is also interesting to notice that for 70 meters jacket, the utilization ratio of jacket with pile foundation is larger than without pile foundation, which could prove the pushover analysis results that pile foundation makes substructure system unstable. In similar, the utilization ratio for 100 meters jacket varies a little. Typically, the size of the utilization ratio is relevant to the order of the failure in pushover process. The hot spots in pushover analysis are also the positions with largest utilization ratio. The pushover method and API design code check could produce good prediction to the ultimate capacity of the structure and work complementary.

7 Discussion and Conclusion

The OFWT substructure models for both of the Water depth 70 meters and 100 meters with fixed bottom or pile foundation, have been respectively designed and imported into USFOS.

Eigen value analysis of different models has been performed. The first six eigen periods and eigen modes, including global and local modes, have been investigated.

Design environmental condition and load cases were indentified from characteristic load cases given by Aker Solution's report. The load and resistance factors were defined by DNV and API design codes.

Pushover analysis was performed with design load cases for each model. The result shows the 70 meters water depth OFWT fixed bottom substructure designed by Aker solution , as well as new design 100 meters substructure, could undertake the extreme design load cases with certain conservativeness. The compression leg element at second storey is the critical position where the yielding and buckling easily happen. The reserved strength after fist leg element failure is extremely small. It also indicated that jacket is sensitive to the direction of the wind load, therefore during substructure design we need to consider the regime of the wind and variable wind direction to avoid the annual most frequent wind direction on the 45 degree of the jacket. For, new design 100 meters water depth OFWT fixed bottom jacket. The load factors which represent how many times of the design load substructure could undertake remain at same level as original design demonstrated the feasibility of the design.

The response of the substructure with pile foundation is more complicated than the fixed ends substructure. The upper soil layers from seabed to -8 meters soil layers are too soft to take the axial loads, but the yielding won't affect the global stiffness of the substructure because of the load redistribution to the other layers. For the 70 meters water-depth jacket with pile foundation, the yielding of the deep soil layer will induce the dramatic drop of the global loading which will cause the snap shot of the global stiffness. The failure of the deeper soil connected with tension piles will cause the piles pull-out in tension and punch through in compression which induce collapse of the substructure. Therefore, for this substructure, considering the soil-pile as pile foundation system, the response under extreme load is pile dominate. For the pile longer than 50 meters, the 100 meters water-depth jacket, since the deep soil is soil, the effects from soil to the global stiffness reduce. The response of substructure system is jacket dominate, although the pile at upper layer is yielding due to larger axial load. The designed pile foundation could pass the ULS pushover analysis with conservativeness. The soil-pile interaction is sensitive to the wave load and direction. The compression leg elements in the middle storey, the piles at upper layers were hot spots.

API design code check was introduced through hand calculation of the utilization factor for heaviest stressed element under design load cases for each model. In different cases, we observed that the largest utilization ratio is smaller than 1 and present some conservativeness, all designs were acceptable for the API LRFD design code check. For substructure with pile foundation, the utilization result could indicate us pay more attention on the pile-soil system and we need to focus on first few highest stressed elements, not only on the largest one. But typically, compared with the results from pushover analysis, the conclusion from the pushover analysis could be confirmed by API design code check. The hot stops in pushover analysis were also the position with largest utilization ratio. The size of the utilization ratio is relevant to the order of the failure in pushover process. The pushover method and API design code check method could produce same estimation of the ultimate capacity of the structure and work complementary.

8 FutureWork

The design of the OFWT jacekt substructure is conservative according to the ULS pushover aanalysis and API design code check. That is possibly because the design is mainly against FLS of the jacket. Since the Design constraints for offshore wind turbine structures fall into either extreme load or fatigue categories, the fatigue analysis of wind turbine jacket should be performed.

Improvement to soil data and design of pile foundation should be further investigated. The upper soil layer is too soft to undertake the axial load, the yielding of the soil always happen before failure of substructure component. Even the failure of the soil at upper layers doesn't affect ultimate capacity of the substructure, the potential danger under long time operation needs to be considered.

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10 Appendix

A 3D perspective of the OFWT for 70 meters water depth

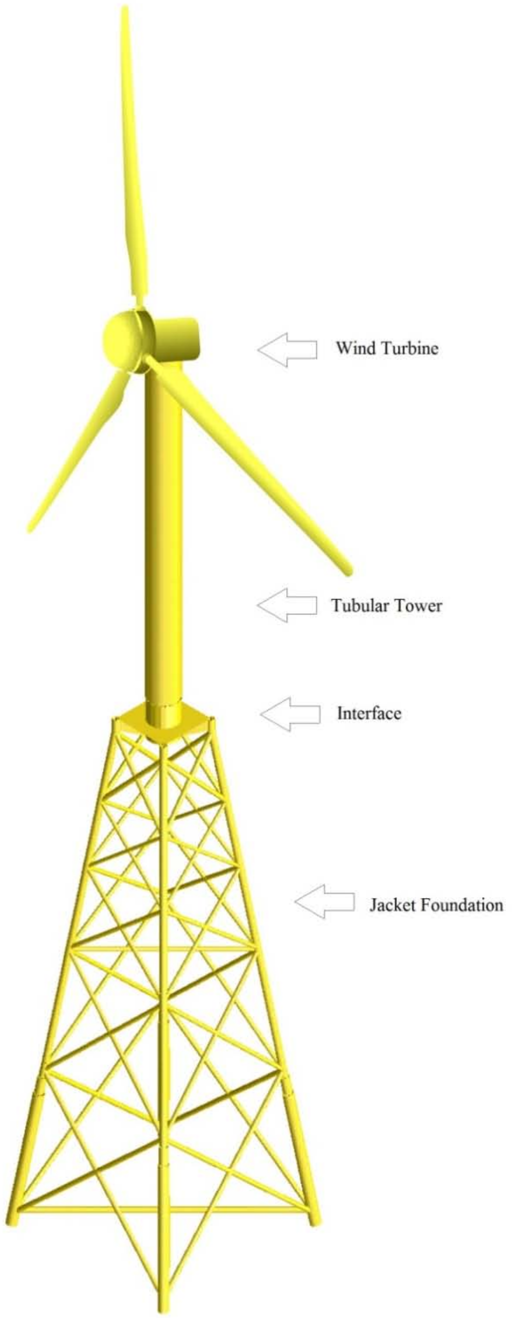


Figure A 3D perspective of the OFWT for 70 meters water depth

B Critical Element Number and Position

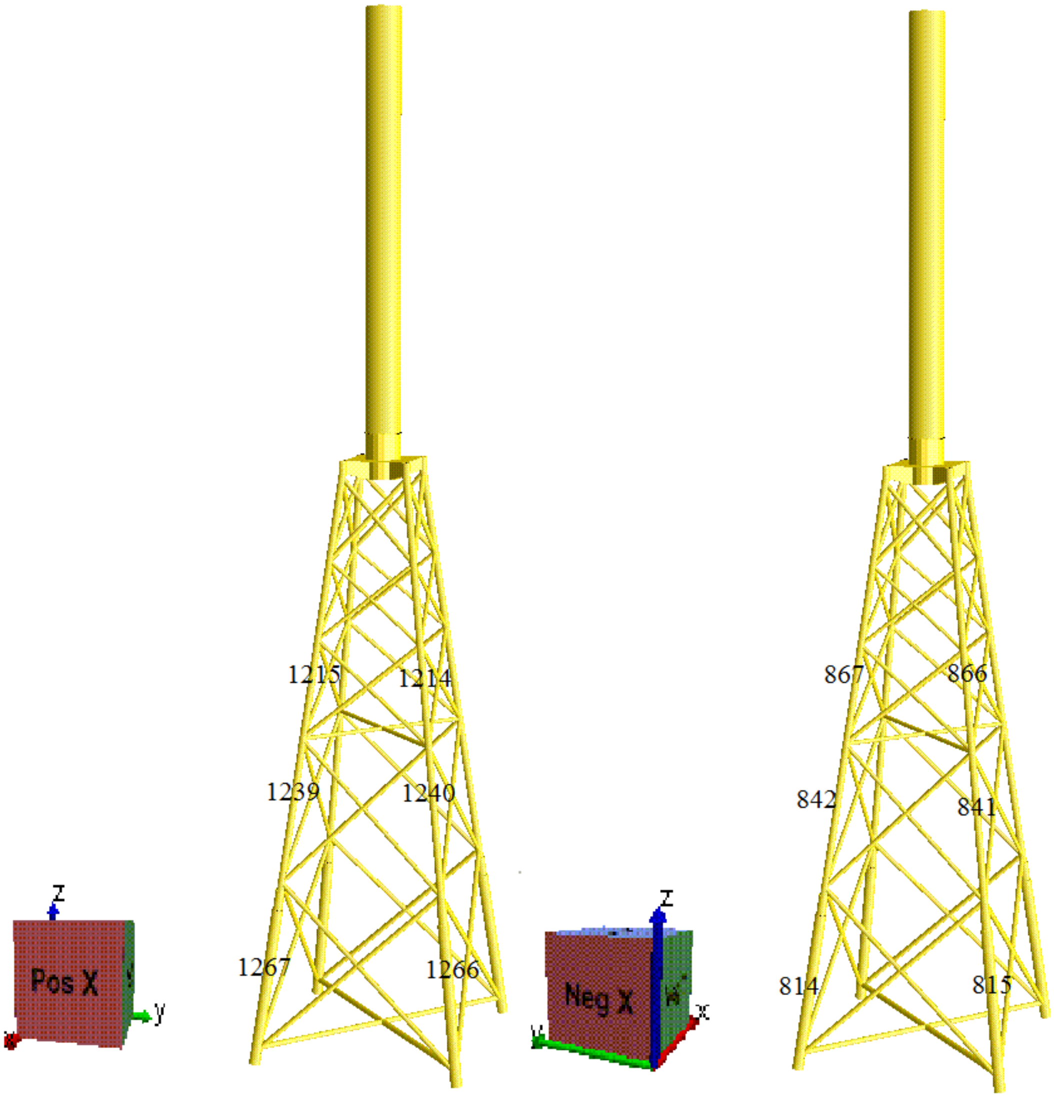


Figure a Critical element and position of 70 meters water-depth jacket

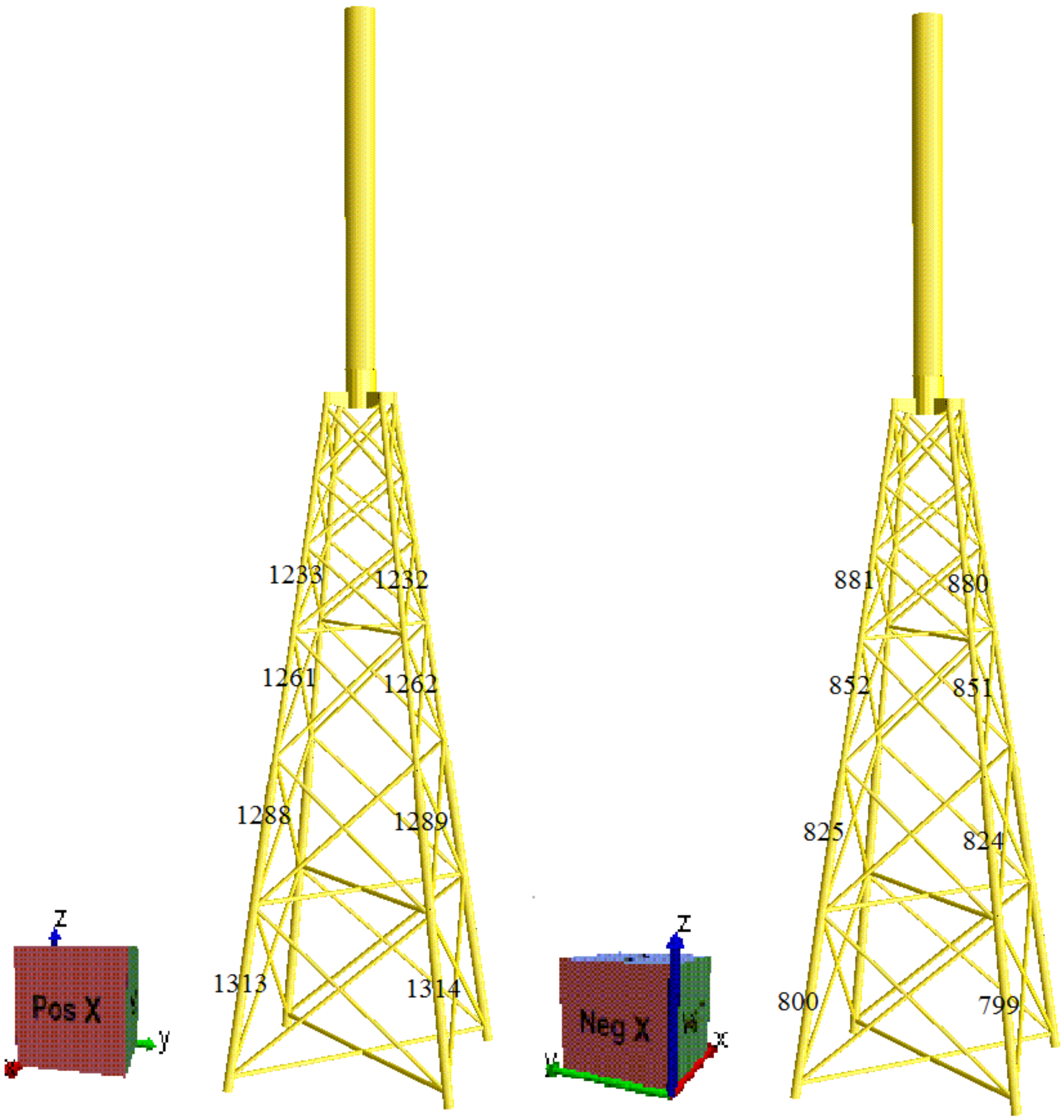


Figure b Critical element and position of 100 meters water-depth jacket

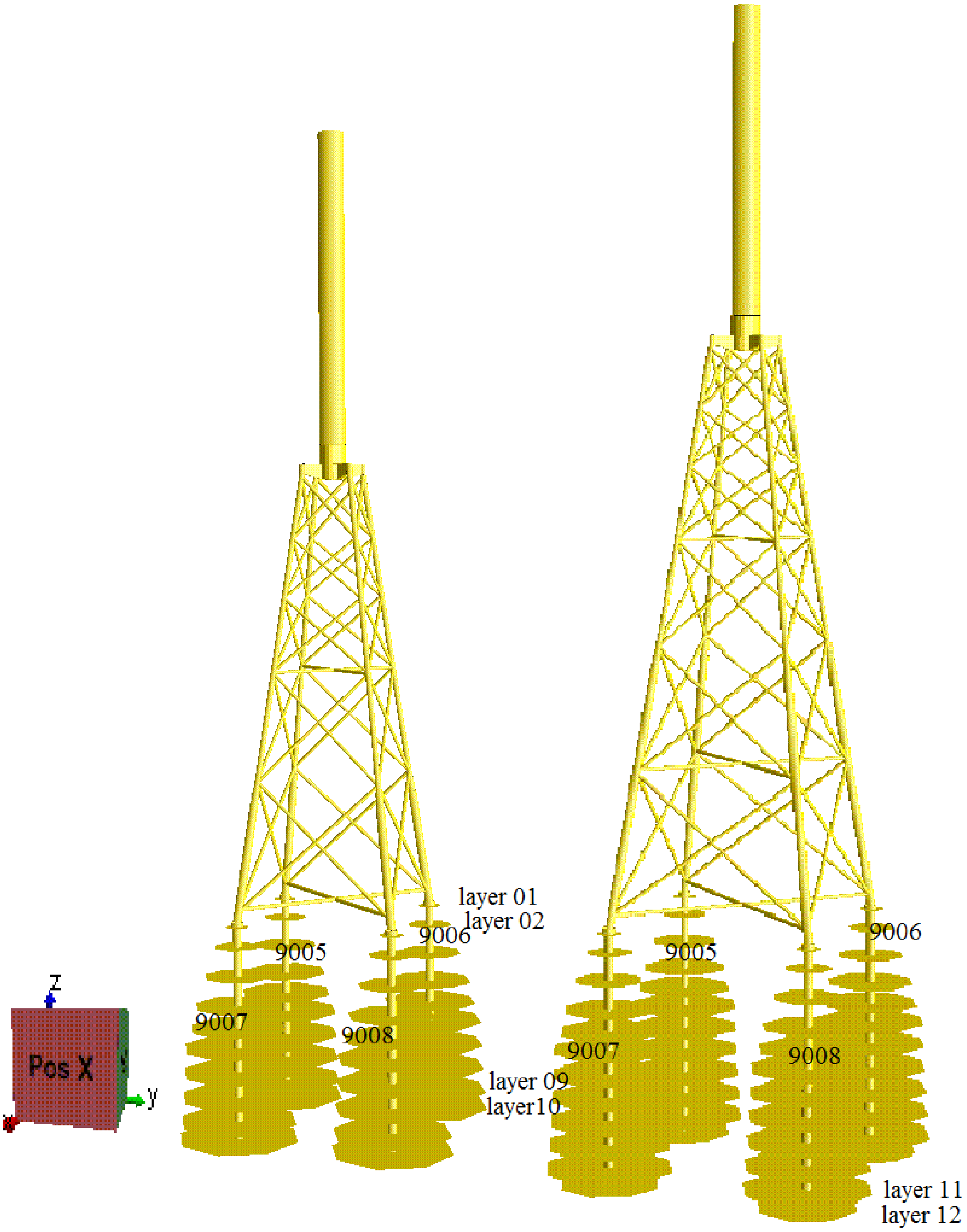
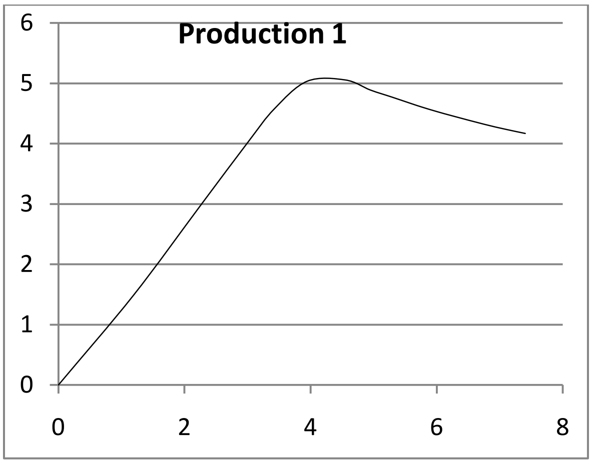
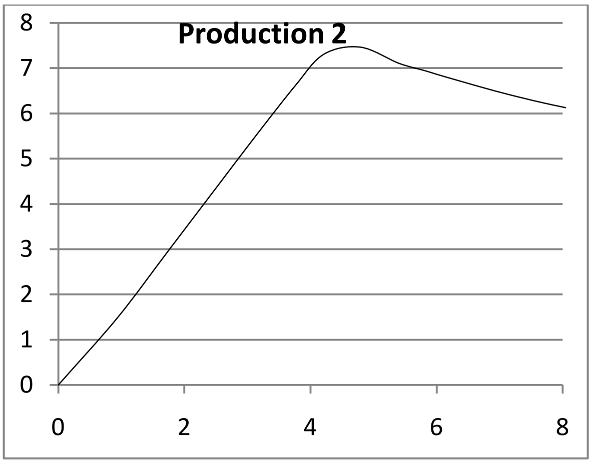


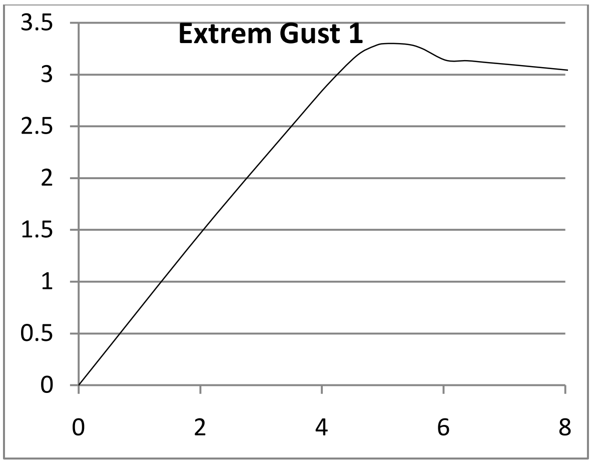
Figure c Critical element and position of 70 and 100 meters water-depth jacket pile foundation

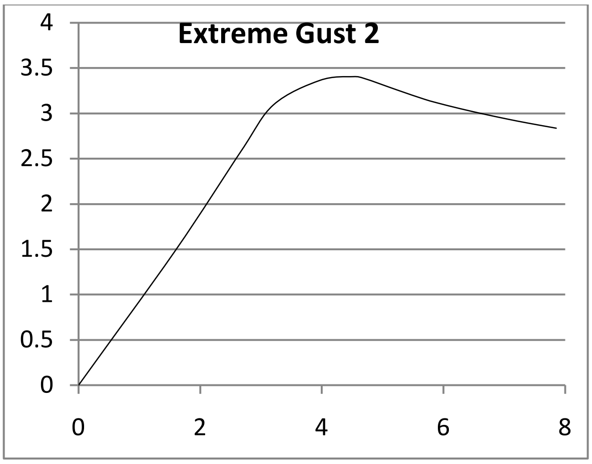
C Response of Substructure

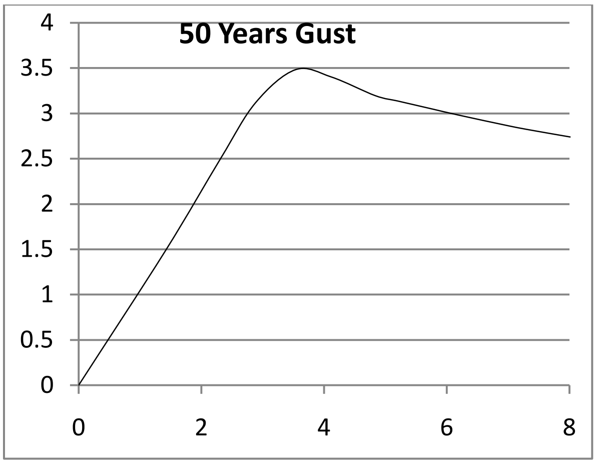
1. Response for 70 meters water depth OFWT substructure

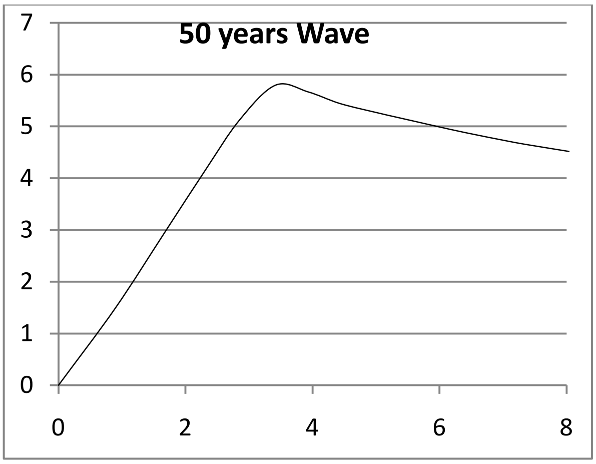




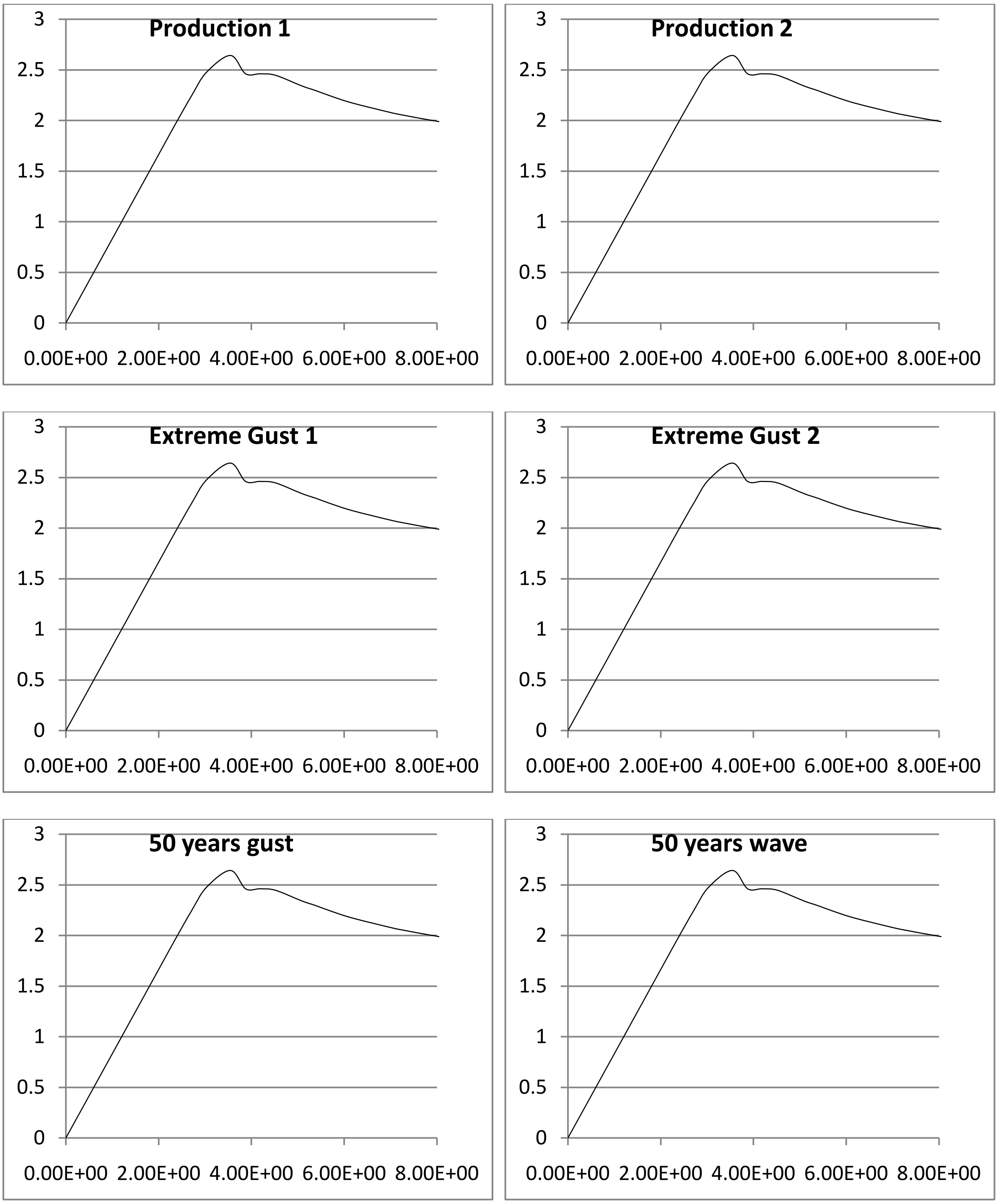




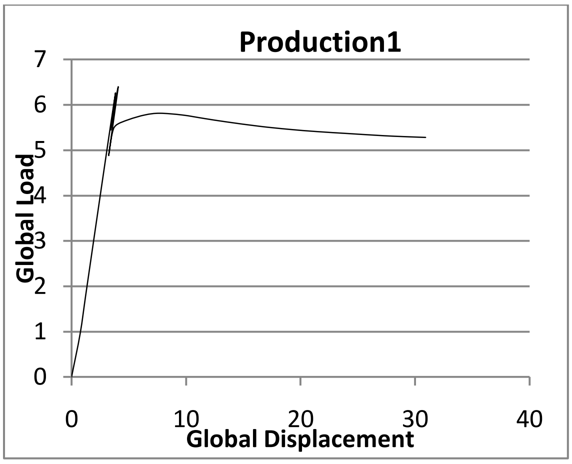


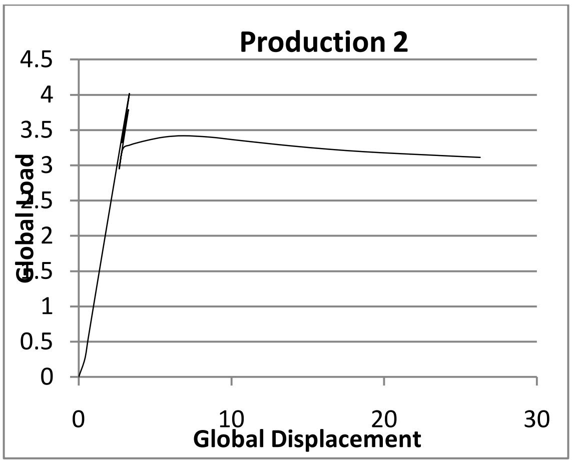


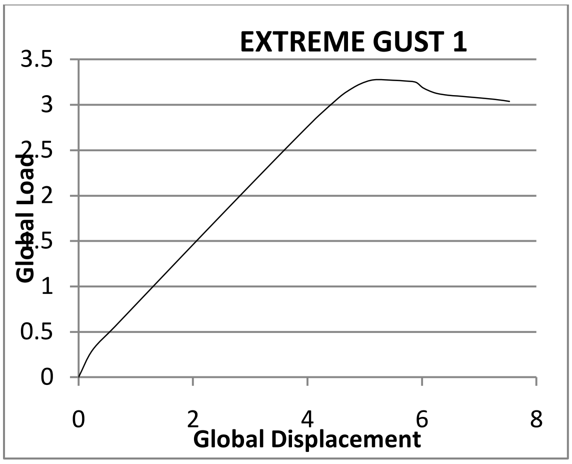
2. Response for 100 meters water depth OFWT substructure

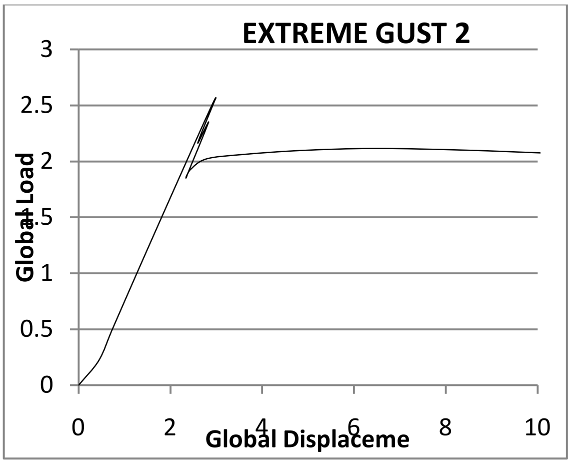


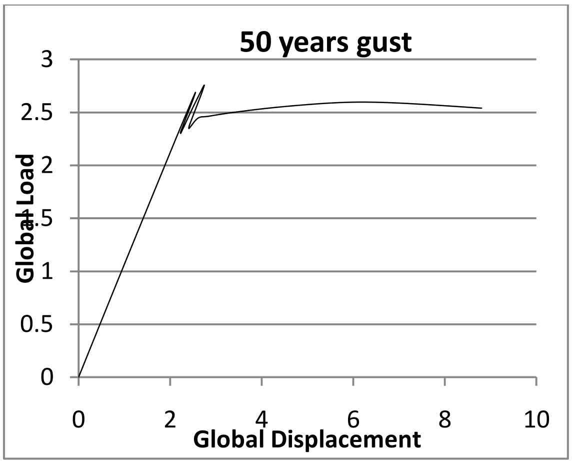
3. Response for 70 meters water depth OFWT substructure with pile foundation

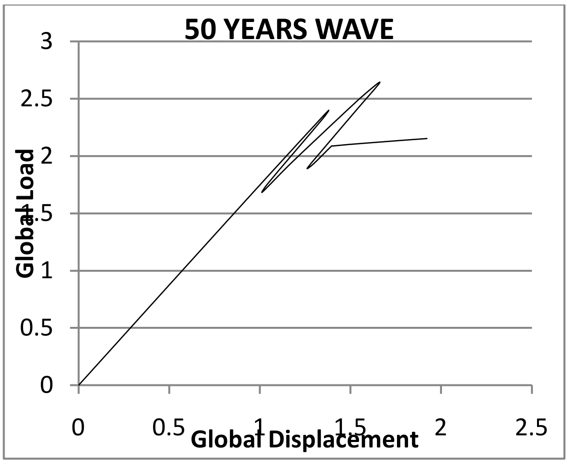




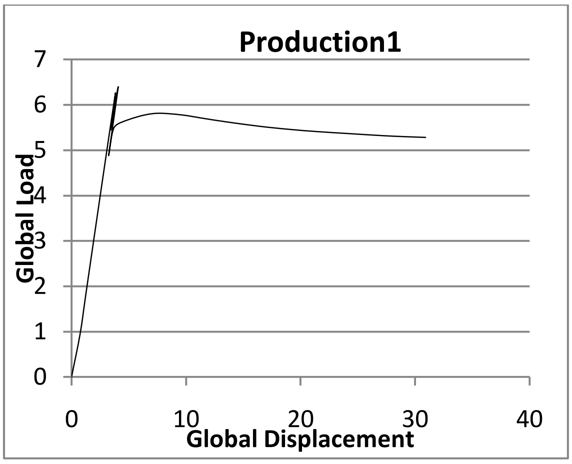


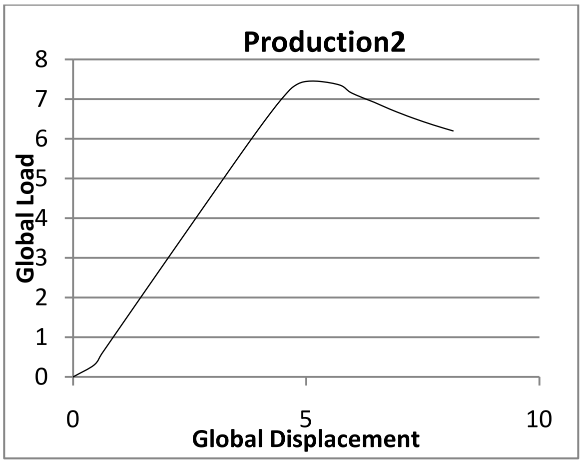


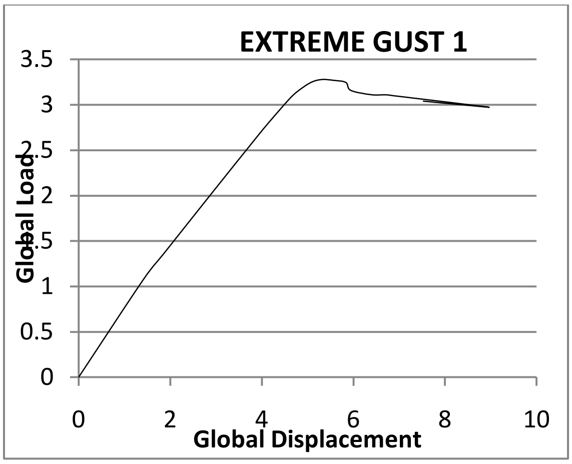


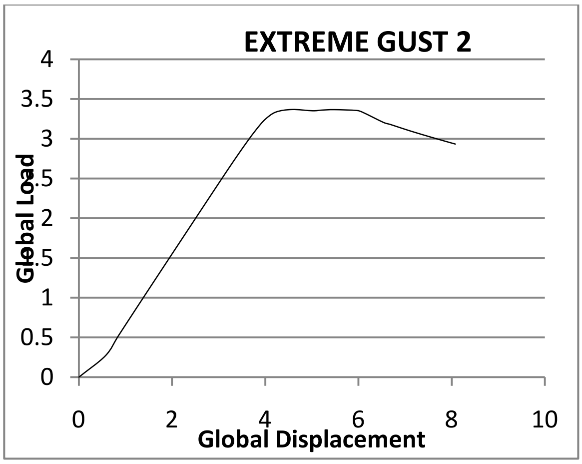


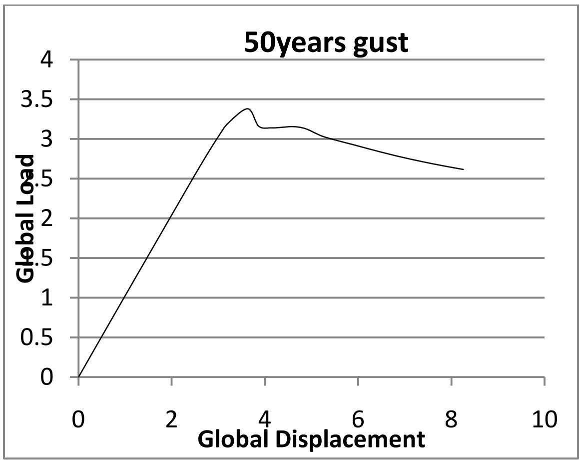
4. Response for 100 meters water depth OFWT substructure with pile foundation

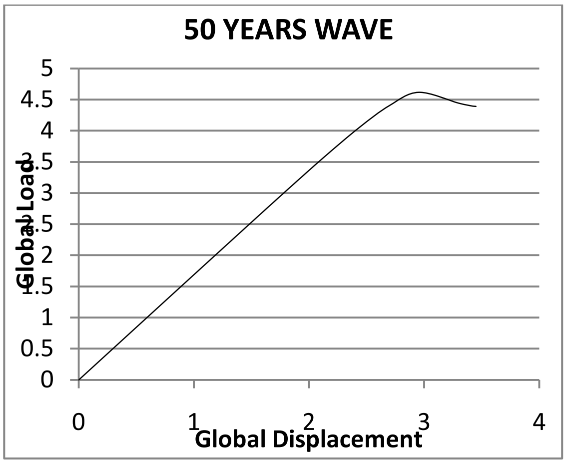












DVD

The model generated and input file for analysis herein are included in a DVD. The contents of the DVD are as follow:

Model
70 meters-water depth fixed bottom jacket
100 meters-water depth fixed bottom jacket
70 meters-water depth pile foundation jacket
100 meters-water depth pile foundation jacket

USFOS Input
Pushover analysis for 70 meters-water depth fixed bottom jacket
Pushover analysis for 100 meters-water depth fixed bottom jacket
Pushover analysis for70 meters-water depth pile foundation jacket
Pushover analysis for 100 meters-water depth pile foundation jacket
Eigen value analysis
Dynamic analysis

Master Thesis
Word text and Pdf text