Dr. techn. Olav Olsen

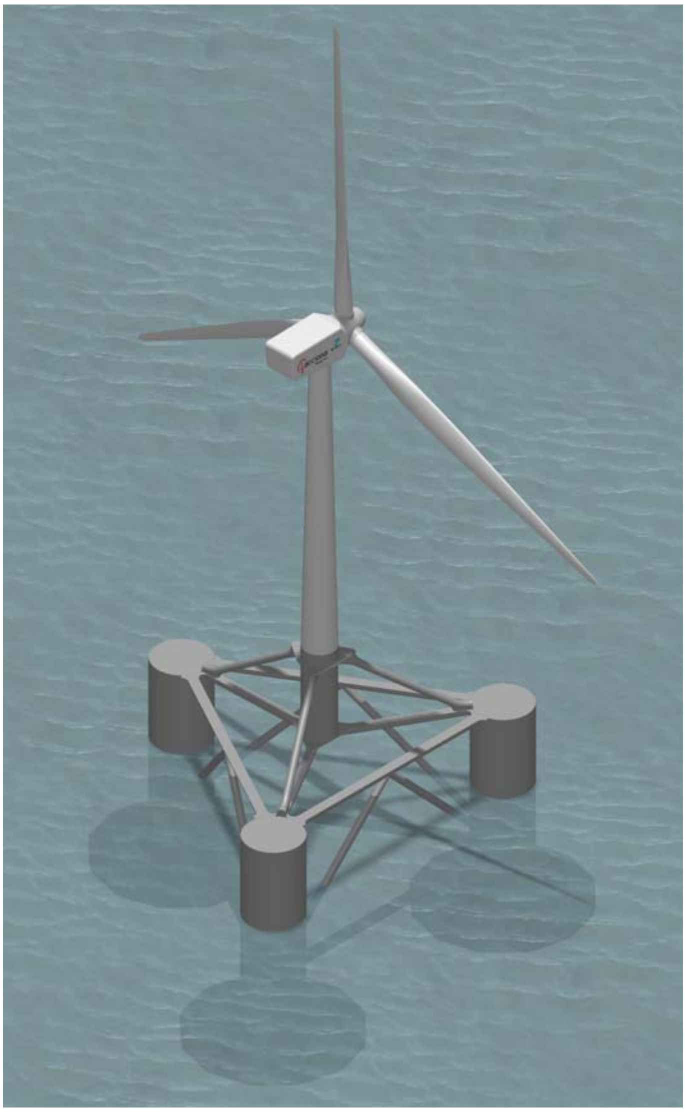
Structural and Marine Engineering Consultants
P.O. Box 139 - Dicks vei 10 - NO-1325 LYSAKER - NORWAY

Tel.: +47 67 82 80 00 - Fax.: +47 67 82 80 80
e-mail: firmapost@olavolsen.no - web: www.olavolsen.no

Project	Origi-nator	Doc. Type	Seq. No.	Rev. No.
11229	DSV	Report	01	01

Title:

HYPRWIND - DEVELOPMENT OF A COLUMN STABILIZED FLOATER FOR 1,5 MW TEST WIND TURBINE



Dr. techn. Olav Olsen

Structural and Marine Engineering Consultants P.O. Box 139 - Dicks vei 10 - NO-1325 LYSAKER - NORWAY			
Tel.: +47 67 82 80 00 - Fax.: +47 67 82 80 80 e-mail: firmapost@olavolsen.no - web: www.olavolsen.no			
01	30.10.2010	WP 1 D1.1 report	Dsv/HA
00	01.07.2010	Preliminary draft	HA/DSV
Rev.	**Date:**	**Description:**	**Prepared by:**

CONTENTS

FLOATER DESIGN

1.1 Introduction

The project HiPRwind, under the EU Frame Program FP7, aims at developing technology and cost effective solutions for large offshore floating Wind Turbines. The project has duration of 60 months (5 years) and started 01.10.2010. A part of the project includes the development, fabrication, installation and testing of a smaller scale floating Wind Turbine (1,5MW). One important assumption is that the floating structure shall be possible to fabricate and complete in limited water depth, typical for normal shipyards and harbors. The purpose is to establish general experience and knowledge of design and analysis tools for this type of concept and then utilize this for development of large full scale floating wind turbines (5-10MW).
Different concepts have been considered, including Spar (Hywind), TLP and Column Stabilized floaters. The conclusion from this study, based both on technical/cost aspects and draft limitations, was that the most promising concept is a 3 column stabilized floater in steel.

Scope of work for Dr.techn.Olav Olsen as
Based on our experience in developing floating and fixed wind turbine concepts, including Hywind, we were invited to participate in the project and take responsibility for concept development of the floating substructure. The work allocated to Olav Olsen is a part of Work Package 1 (WP 1) “Platform Concept” and WP4 “Advanced Floater and Mooring System”.
The project is defined and described in Annex I – “Description of work”
Task force leader for WP1 is ACCIONA Energia. Total resources allocated to this Work package is 130 Person Months (PM) and the duration is from month 1 to month 30 (first half part of the project time)
WP 1 includes the following tasks:

Task 1.1 Conceptual design of test platform
Task 1.2 Selection of dynamic simulation tools
Task 1.3 Dynamic analyses of the floating platform
Task 1.4 Mooring system and dynamic cables configuration
Task 1.5 Design, development and test of control system

Dr. techn Olav Olsen is responsible for Task 1.1 and partly for mooring in Task 1.4, with a total allocation of 16 Person Month. The duration of this work will be within the first 12 months of the project with major part of the work performed within the first 6 months. The work performed by Dr.techn.Olav Olsen a.s will be used as a basis for analyses models utilized by the other partners in the project and for detail engineering and fabrication of the floater structure. The scope of work for Olav Olsen is summarized as follows:

Task 1.1 Conceptual design of test platform
- Develop a parametric design spread sheet model - available for partners
- Establish Geometry and mass distribution model for advanced analyses by the partners
- Simplified dynamic response analyses for optimization of concept and mooring system
- Perform simplified global structural response analyses
- Perform conceptual structural design
- Establish a detailed 3D DAC model and illustrations

Task 1.4 Mooring system and dynamic cable configuration
- Propose and analyse a various mooring solutions

1.2 Concept description

The basic requirement to the HiPRWind floating Wind Turbine is that it can be fabricated in a normal shipyard and completed with tower and turbine in shallow water and floated out to an offshore site and moored. This assumes that the structure can float and be stable with shallow draft in transit conditions. Stability in shallow draft requires water plane stability. At the same time it is very important to minimize dynamic response in waves. The concept that best meets these requirements is the column stabilized floater and the selected design is the most simple and effective solution based on these principles.
The proposed concept consists of 3 columns and an optimal bracing system for structural integrity. The lower end of the columns has circular damping plates. These are required and dimensioned to obtain a specified heave period of around 20 sec. The turbine is located in the centre axis of the floater to minimize static and dynamic yaw moments and thereby obtain a simplest possible mooring system. The transit and service draft in completed condition is designed to 8m while the operation draft is 15.5m
The final concept is defined by the **Spread Sheet** parameters and the structure in detail by the 3D Microstation **CAD model**.

1.3 Projecting method

A parametric model of the concept has been established on a spread sheet. Based on a complete description of the geometry, experience data for steel weights, turbine specifications etc., basic characteristics as static stability and heave period are calculated.
The preliminary design requirements are that the static heel angle under maximum wind trust shall be around 5 degrees, and the heave period shall be approximately 20 seconds. The static heel angle is calculated based on the stability parameter (GM) and the mooring line arrangement and response.
There are infinite combinations of main parameters that will satisfy these requirements. However, an additional requirement is to minimize the steel weight. Based on experience and parametric variations a combination of main dimensions is sought to minimize structural weight.
The fundamental requirement, however, is to minimize response with respect to dynamic pitch angles and accelerations in the top of the tower. This requirement relates to dynamic loadings on the turbine and its functionality with respect to power production. Another important issue is fatigue of the tower which is directly related to the top mass acceleration and the dynamic heel angle. For a small turbine this is probably manageable within normal wall thicknesses in the lower part of the tower. For bigger turbines this may become a feasibility issue.
In our analyses at Dr.techn.Olav Olsen a.s we have established a direct creation in the spread sheet of the hydro dynamic models for WADAM and SIMO input such that parameter variations can be run very efficiently and quick. This is done by applying a “Python” script.
For other users there is established a geometry and mass distribution matrix that can be utilized for automatic and easy creation of their hydro dynamic models. Other users can then perform their own parametric studies in an efficient way. This matrix has the form shown in table 1.1 and figure 1.2

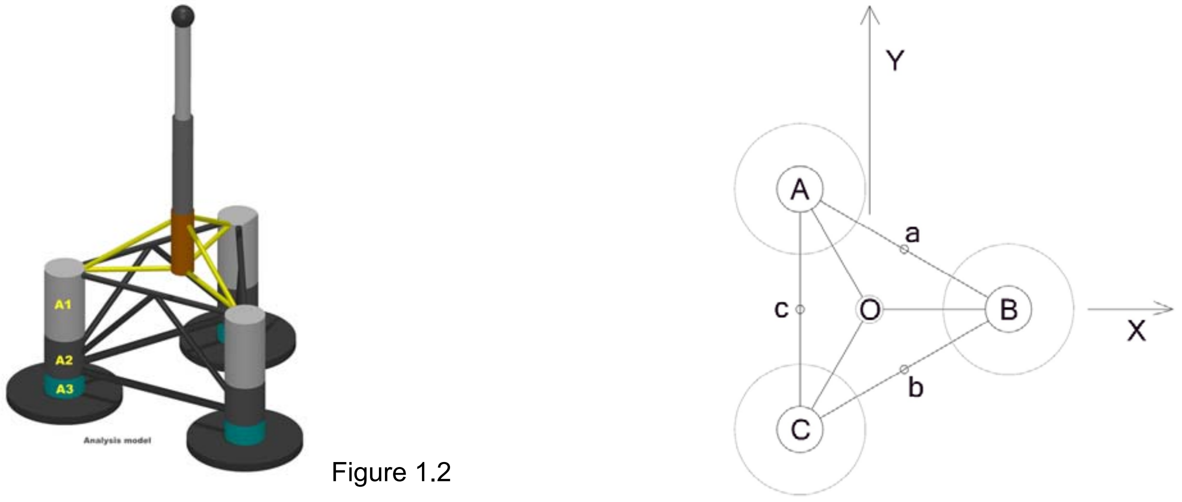


Figure 1.2

INPUT DATA FOR GEOMETRICAL AND MASS MODEL OF HiPRwind floater concept

Water plane level 15,50 m Origo at geometric center and bottom structure (keel line)

TOP MASS	X	Y	Z	Weight
TURBINE	0	0	**74,77**	**87400**

TOWER &	End coordinates 1			End coordinates 2			Diam.	Length	Weight/m
TOWER BASE	X	Y	Z	X	Y	Z	(m)	(m)	Kg/m
Upper tower	0	0	54,925	0	0	73,35	2,748	18,425	1357,72
Lower tower	0	0	36,500	0	0	54,925	3,583	18,425	3277,76
Tower base	0	0	24,600	0	0	36,50	4,00	11,900	3906,38

COLUMNS	End coordinates 1			End coordinates 2			Diam.	Length	Weight/m
	X	Y	Z	X	Y	Z	(m)	(m)	Kg/m
A1	-10,104	17,500	25,5	-10,104	17,500	12,750	7,00	12,750	7807,8
A2	-10,104	17,500	12,75	-10,104	17,500	6,538	7,00	6,212	5552,0
A3	-10,104	17,500	6,538	-10,104	17,500	1,540	7,00	4,998	43303
DAMP A	-10,104	17,500	1,53974	-10,104	17,500	0,000	19,54	1,540	354951
B1	20,207	0,000	25,5	20,207	0,000	12,750	7,00	12,750	7807,8
B2	20,207	0,000	12,75	20,207	0,000	6,538	7,00	6,212	5552,0
B3	20,207	0,000	6,538	20,207	0,000	1,540	7,00	4,998	43303
DAMP B	20,207	0,000	1,53974	20,207	0,000	0,000	19,54	1,540	354951
C1	-10,104	-17,500	25,50	-10,104	-17,500	12,750	7,00	12,750	7807,8
C2	-10,104	-17,500	12,75	-10,104	-17,500	6,538	7,00	6,212	5552,0
C3	-10,104	-17,500	6,538	-10,104	-17,500	1,540	7,00	4,998	43303
DAMP C	-10,104	-17,500	1,53974	-10,104	-17,500	0,000	19,54	1,540	354951

BRACES	End coordinates 1			End coordinates 2			Diam.	Length	Weight/m
	X	Y	Z	X	Y	Z	(m)	(m)	Kg/m
Ht AB	-7,073	15,75	25,05	17,176	1,75	25,05	0,900	28,000	435,224
Ht BC	17,176	-1,75	25,05	-7,073	-15,75	25,05	0,900	28,000	435,224
Ht CA	-10,104	-14	25,05	-10,104	14	25,05	0,900	28,000	435,224
Hb AB	-7,073	15,75	7,50	17,176	1,75	7,50	1,300	28,000	673,259
Hb BC	17,176	-1,75	7,50	-7,073	-15,75	7,50	1,300	28,000	673,259
Hb CA	-10,104	-14	7,50	-10,104	14	7,50	1,300	28,000	673,259
D Aa	-7,073	15,75	11,01	5,0518	8,75	25,05	0,900	19,827	349,577
D Ba	17,176	1,75	11,01	5,0518	8,75	25,05	0,900	19,827	349,577
D Bb	17,176	-1,75	11,01	5,0518	-8,75	25,05	0,900	19,827	349,577
D Cb	-7,073	-15,75	11,01	5,0518	-8,75	25,05	0,900	19,827	349,577
D Cc	-10,1	-14	11,01	-10,1	0	25,05	0,900	19,827	349,577
D Ac	-10,1	14	11,01	-10,1	0	25,05	0,900	19,827	349,577
Ht AO	-8,354	14,47	25,05	-1,000	1,732	25,05	0,900	14,707	507,247
Ht BO	16,707	0	25,05	2,000	0,000	25,05	0,900	14,707	507,247
Ht CO	-8,354	-14,47	25,05	-1,000	-1,732	25,05	0,900	14,707	507,247
D AO	-8,354	14,469	25,50	-1,000	1,7321	35,45	0,9	17,757	479,420
D BO	16,707	0,000	25,50	2,000	0,000	35,45	0,9	17,757	479,420
D CO	-8,354	-14,47	25,50	-1,000	-1,732	35,45	0,9	17,757	479,420

Mooring force	X	Y	Z	Weight
Point A	-11,85	20,53	25,50	**39551,48**
Point B	23,71	0	25,50	**39551,48**
Point C	-11,85	-20,53	25,50	**39551,48**

Table 1.1

1.4 Concept specification

The projecting of the concept has been through several developing steps that includes Initial parametric description based on experience data for structural weights, limitation of static heel, calibration of dynamic mass of the damper plates by WADAM analyses, parametric variation study for optimization of the structure, structural design and analyses for direct calculation of structural weight for updating of spread sheet model. The final results are presented in the following data sheet (table 1.2) for the floater.

HiPRWind TEST FLOATER	**Water Depth:**	
Dr. techn. Olav Olsen as - Dagfinn Sveen	**80**	**m**
Turbine & Tower data		
Turbine effect	**1,50**	**MW**
Turbine top weight	**87,40**	**t**
Rotor diam	**77**	**m**
Max static wind trust	**265**	**kN**
Tower weight	**85,41**	**t**
Tower bottom diameter	**4,00**	**m**
Tower top diameter	**2,33**	**m**
Tower height	**36,85**	**m**
Hub height above SWL	**60,00**	**m**
Floater dimensions		
Operating Draft	**15,5**	**m**
Freeboard to column top	**10,0**	**m**
Operating Displacement	**3212**	**t**
Ballast water in operation	**1533**	**t**
Column Center distance	**35,00**	**m**
Column Diameter	**7,00**	**m**
Column height	**25,50**	**m**
Damper plate diameter	**19,54**	**m**
Structural steel weight floater	**903**	t
Columns	431	t
Braces	183	t
Damper structure	243	t
Tower base	46	t
Secondary structure & equipment	**80**	t
Mooring system - No. of lines: 3 chain		
Mooring line length	**600**	**m**
Chain dimension	**92**	**mm**
Total dry weight of all 3 lines	**307**	**ton**
Heave period	**19,45**	**sec**
GM in operation	**3,75**	**m**
Tilt under static wind load	**5,17**	**deg**

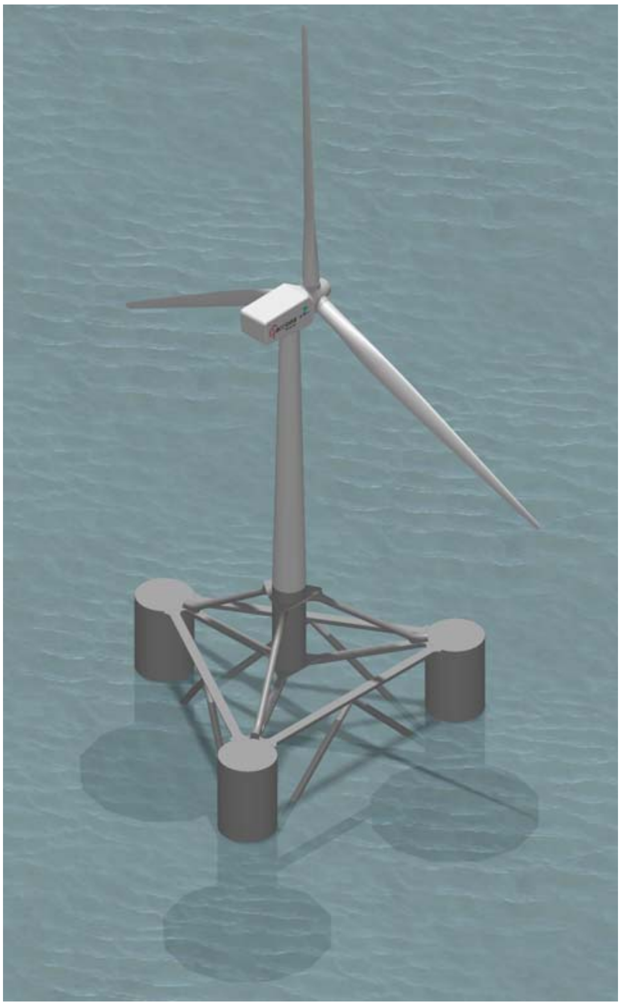


Table 1.2

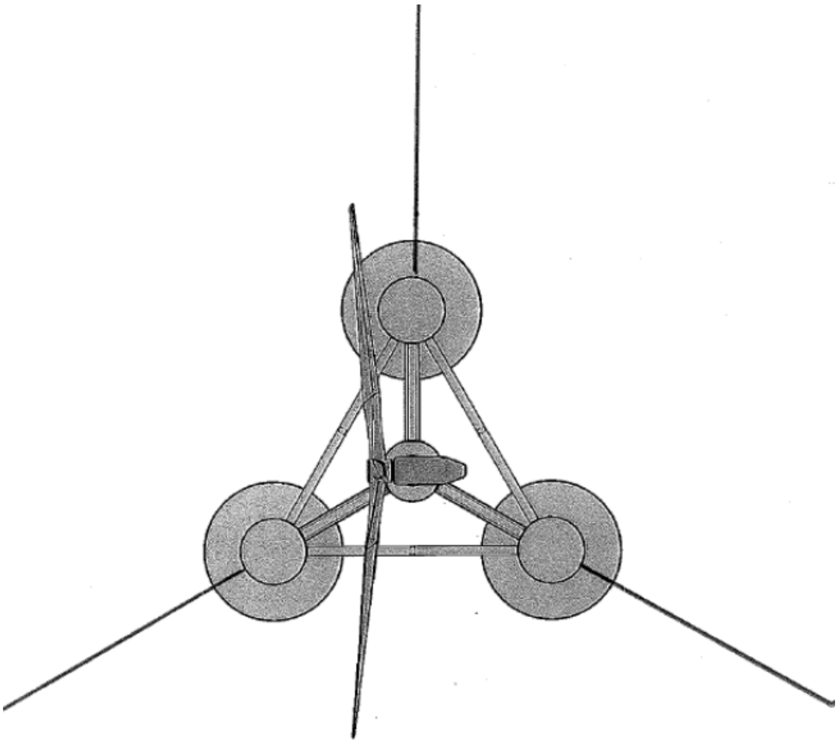
1.5 Mooring System

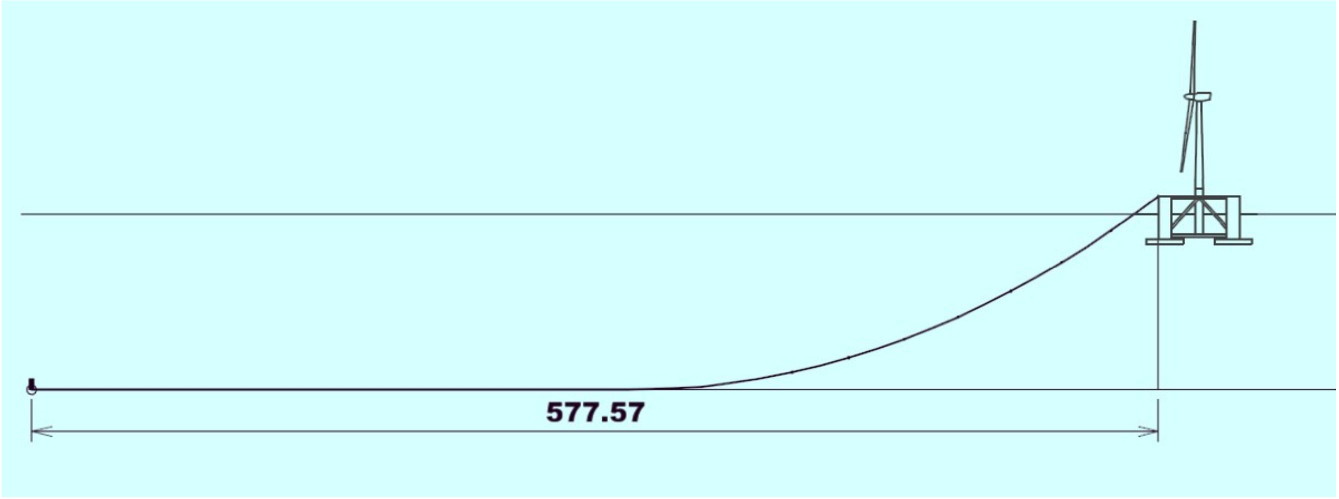
The mooring system may represent a major cost element both with respect to component cost and installation cost. One basic goal is accordingly to select the most cost effective solution.
One parameter in this context is the number of lines. Theoretically the smallest no of lines is 3.
The feasibility of only 3 lines is related to requirement for redundancy (application of rules) and the practical limitations in component size (as line diameter). The definition of redundancy may vary dependent on the consequences of line breakage. For an offshore oil/gas platform the redundancy of the mooring system is defined by its ability to stay on location after line breakage without breaking the risers and thereby polluting the environment.
For a wind turbine the consequence of a drift off from the initial position may lead to damage of the electric cable. However, this is an economical risk not an environmental pollution risk.
Another level of redundancy may then be defined to be that the installation stays in the area and not drifts into other wind turbine installations. This requirement can be satisfied by using only 3 lines. The applicability of this philosophy must be considered on the basis of a cost/risk evaluation.
If more than 3 lines have to be used, the number will naturally be 6 (assuming that the floater has 3 columns)
The question is then if 3 or 6 anchors shall be used.
Another requirement to a mooring system is the need to individually tension the lines. This requirement is especially related to the installation procedure as the lines have to be connected between the anchor and the floater in a slack condition. Normally the lines are tensioned from the floater by individual winches and the lines are guided via fairleads, however, this can be an expensive solution.
One important aspect of the mooring system is that it shall provide sufficient yaw stiffness when an upwind turbine is used. This may however, be a less problem with a semi submersible type of structure than a spar type due to the wide structure and thereby longer moment arms than for a narrow spar structure. Another aspect is, however, that a semi type structure will have yaw movements induced directly by the waves, something that is clearly documented in our analyses.

There are also several other design aspects related to the mooring system that may be cost driving and complex. Experience shows that installation and marine operations will have a crucial influence on the design of the system and component selection.
For information it may be mentioned that the mooring system for HYWIND is based on a 3 line system. The two first lines were installed easily in slack condition. The third line was installed under moderate tension and connected in the middle area above sea surface on a service vessel. When all lines were connected, the lines were tensioned by hooking on a clump weight on each of the lines. The yaw stiffness was obtained by a craw foot line arrangement at the connection to the floater.
For the Hiprwind concept we have proposed and used only 3 mooring lines in our analyses. It is assumed that the pretension can be adjusted with one line only, as a 3 point system will be in static balance. For simplicity we have also connected the line directly to the top of the columns to avoid fairleads and complicated arrangements. This “high” connection is also selected to minimize heeling due to wind trust on the turbine. By this arrangement the overturning moment is minimized and accordingly also the static tilt angle. In this context it must also be remembered that current and drift forces may create tilting moments in opposite direction. Also it could be expected that the high connection can cause somewhat higher dynamic load reactions in the line due to 1.order movements of the floater.
The effect of different connection levels for the mooring line was studied by running analyses with the lines connected at the water line level. However, no significant effect was found with respect to loads reduction compared with the line connected to the top of the column at + 10m.
One problem with a catenary mooring system on small water depth (80m) as for HyPRwind, is that the stiffness gets rather high and consequently the first order dynamics of the floater induce high dynamic loads in the line. This problem is enhanced by the non linear stiffness characteristic of the line. As seen from our analyses also the wind, drift and current forces influence the 1. order dynamic load response in the lines. This is because the static loads tighten the line and force the dynamic fluctuation into a steeper area on the mooring line characteristic. We can from our analyses also see that the mooring system in such cases has a significant influence on the dynamic response on the floater by increased accelerations and pitch angles. In our analyses we have used different chain

dimensions (82 and 92mm) and chain length (800 and 600m) The final concept is a 92mm stud-less chain of 600m length.

Principal mooring arrangement as used in analyses





1.6 Structural design

The conceptual structural design includes different areas as

- Tower support structure
- Substructure bracing system
- Damper plates and damper tanks
- Main columns

The design has been based on different methods and design standards as explained for each area. The design is based on preliminary but conservative assumptions with respect to loads and acceptance criteria. Different parts of the structure will be governed by fatigue, but it has not been possible to perform direct fatigue analyses at this stage of the design. Instead fatigue has been allowed for by keeping extreme dynamic stresses on a low level based on experience from offshore platforms.

The most fatigue exposed structure is probably the lower part of the turbine tower and the structure constituted by the support braces and their details. The fatigue loads are mainly due to inertia and the gravity forces of the turbine and the tower at dynamic behavior in wave and wind conditions. This is probably one of the most important technical feasibility issues for floating wind turbines, especially when the size is increased, as is the goal of this project.

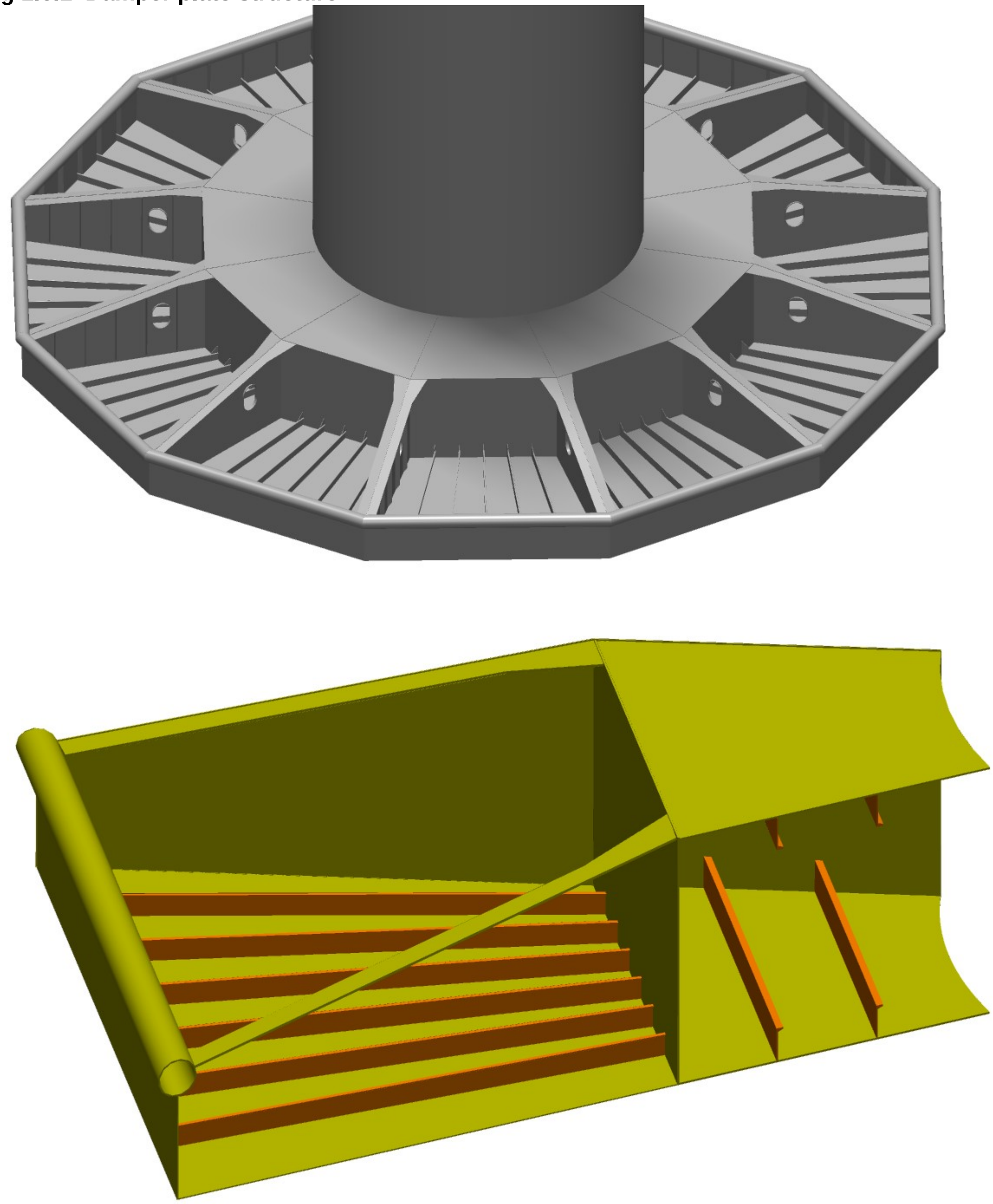
The substructure bracing system is governed by fatigue due to internal wave loads between the columns, so called split forces, and in addition bending effects from the waves acting transversally on the braces. In our case this will primarily affect the connection details to the column, represented by the internal structural solutions in the columns. This is probably not a great feasibility issue as this has been solved for a great number of rather big semi submersible platforms. However it means that the extreme stress level will be rather low.

The damper plates may also for certain elements and areas be governed by fatigue design. This is especially the outer open part as the loads are purely dynamic from the dynamic differential pressure acting vertically on the damper plate. The inner tank part must be designed for rather high hydro static pressures and will probably be rather robust with respect to fatigue.

1.6.1 Design of Damper structure

The damper is a circular disk structure at the lower end of each column. The main purpose of the damper is to create added mass in vertical direction to increase the heave and pitch periods. The damper plate will also give considerable viscous damping.
The Damper is formed as a circular disc structure outside the column and consists of an inner circumferential tank and an outer open plate structure in fig 1.6.1. For the purpose of design and fabrication the damper structure has been divided in 12 segments of 30 deg.
The circumferential tank is open into the column such that the bottom tank of the column and the ring tank constitute one tank unit.
The purpose of the tank is to add sufficient buoyancy to obtain a minimum draft of 8m in transit condition.
The damper is divided in 12 equal segments (30deg.).The tank part is designed for an outer pressure related to an extreme wave condition where the column is totally submerged with wave surface at the top of the column. It is then assumed that the tank is filled up to approximately 6-7m from the keel line.
The open area is designed for a differential pressure created by extreme design wave conditions.
The differential pressure is calculated in a separate analysis and the maximum pressure has been found to be approx. 2,3m pressure height.

Fig 1.6.1 Damper plate structure



HiPRWind FLOATER DAMPER PLATE DESIGN

General data		
Column diameter	**7,00**	m
Damper plate outer diameter	**20**	m
Nuber of segments	**12**	
Diameter of the tank section	**12**	m
tank height	**2**	m
Outer rim height	**1**	m
Segment angle	0,524	rad
Segment area (from center)	25	m2
Area of damper plate (incl. Column)	300	m2
Equivalent diameter of damper	**19,544**	0,97721
Inner diameter of Damper	**19,319**	0,9659

Tank segment (1/12 of total)	**30**	**deg**
Tank segment outside volume	**11,586**	m^3
Tank segment bottom and top		
Segment area	5,793	m2
Plate thickness	**15**	mm
Plate weight	**0,682**	t
No of radial stiffeners	**2**	
Stiffener angular distance	**10**	
Stiffener average distance	0,829	m
ST1 length	2,4240	m
ST 1 HP240x10	**25,4**	kg/m
ST2 length	2,424	m
ST 1 HP240x10	**25,4**	kg/m
Weight tank deck stiffeners	**0,123141**	t
Open side wall		
Wall length	2,50	m
Wall area	**5**	m2
Thickness	10	mm
Plate weight	**0,3925**	t
Vertical buckling stiff. No.	**3**	
Buckling stiff height	100	mm
Buckling stiff thickness	10	mm
Buckling stiff weight	7,85	kg/m
Buckling stiff weight	**0,0471**	**kg**

Outer Wall bulk head		
End wall area	6,211657	m2
End wall thickness	15	mm
End wall plate weight	**0,731423**	**t**
HP240x10 - Vertical	**25,4**	kg/m
Stiffenerdistance	0,776457	
Stiffeners weight	**0,1016**	t
Weight one tank segment	**2,883**	t
Volume occupied by steel	**0,367**	m3
Netto tank volume	**11,219**	m3
Netto tank volume one damper	**134,62**	m3
Netto tank volume for all dampers	**403,8709**	m3

Outer segment radial stiffeners no	**5**	
Inner distance	0,523599	m
Outer distance	0,872665	m
Stiffener length	3,863703	m
Plate area	16	m2
Plate thickness	10	mm
plate weight	**1,256**	t
Stiffener HP180x8	14,8	kg/m
Weight of stiffeners	**0,286**	t

Radial Girder web area	6	m2
Web plate thickness	10	mm
web plate weight	**0,471**	t
Flange width	300	mm
Flange thickness	20	mm
Flange weight	**0,392**	t
Vertical buckling stiffeners no	5	
Width	100	mm
Thickness	10,000	mm
Weight of buckling stiffeners	**0,059**	ton

Total length of board plate	5,176381	
board plate area	5,176381	m2
Thickness	10	mm
Board plate weight	**0,406**	ton
Buckling stiffeners no	5	
Buckling stiffeners width	150	mm
Thickness	10	mm
Weight of buckling stiffeners	**0,0589**	t
Edge Pipe diameter	**250**	mm
Pipe thickness	10	mm
Pipe weight 250mm x 10mm	**0,306**	t

Contingency factor weight	**1,1**	
Weight one segment	**6,118**	t
calculated weight of damper	**73,42087**	t
Estimated weight (incl. Cont. Factor)	**80,76296**	
Weight of all damper plates	**242,2889**	t

Calculation of equivalent damper data for hydrodynamic

Tank volume one damper	139,031	m3

Calculation of equiv. Volume outside ballast tank		
F	6,211657	m2
f	5,176381	m2
Entrapped volume in outer part	21,96963	m3

Equivalent volume one damper	402,6666	m3
Area plate	**261,5155**	m2
Equivalent height	**1,539743**	m
Volume of steel	**10,28828**	m3
Volume water	**392,3783**	m3

1.6.2 BRACES

The floater has two different types of bracing systems; substructure bracing and tower support bracing.

The substructure bracing system inter connects the 3 columns to be a rigid structure and the axial loads in these braces are mainly related to internal wave load response transferred between the columns. In addition there will be direct wave loads on the braces.

The tower bracing supports the tower and is exposed to static and dynamic loads. The dynamic loads are related to inertia and tilting due to global dynamic behavior from waves and wind. In addition also direct wind load on the turbine and tower contributes to the bending moments in the tower.

Dynamic stresses
From experience with semi submersible platforms in the oil business it is realized that the extreme dynamic stresses in the braces can be a scale for the fatigue capacity of the brace. A simplified method for fatigue assessment is to use a closed form method based on the extreme dynamic stress response and a Weibul long term distribution factor. For this type of structures the Weibul factor can typically be around 1.0
In addition to the average stress in a brace there will be stress concentrations that influence the fatigue life calculations. For the conceptual design before direct fatigue analyses can be performed, there is a good rule to look at the dynamic stresses in the braces and keep this on a low level (40-70MPa). Some braces will have low static loads as in our case for the substructure braces. This means that the extreme stresses in these braces must be far below the normal accepted stress level for structure exposed to static loads.
For the conceptual design of the HyPRwind concept we have tried to keep the dynamic stress level low as indicated above. When direct fatigue analyses are performed later in the detail engineering, there will be two important factors to consider; the nominal stress (undisturbed) and the stress concentrations. If a lightest possible structure is desired, one must keep the stress concentrations as low as possible. This may also increase the quality requirements to the fabrication, at least for the critical details.
Another possibility is also to reduce the nominal stress level by increasing the thickness locally. For the braces this will normally mean to increase the thickness at the brace connection to the columns.

Substructure bracing
The substructure bracing system has been analyzed by using a design wave approach in accordance with DNV Standard OS C 103
The design and analyses of the braces has been performed in two steps. In the first step, brace dimensions were based on conservative assumptions and the stresses checked with design wave method. Based on the results the braces were re-dimensioned and in general reduced in diameter and thicknesses due to the low stress results. In the latest analyses, the final concept design was used and the stress results are reported in table 2.7.9
As can be seen from the table the stress level is still rather low. One reason for this is that the braces are governed primarily by fatigue. This means that the extreme dynamic stress level should be rather low and in this context the obtained stress level is in line with experience data for semi submersible platforms applied in the oil business. A part of the detail engineering will be to perform local design for the brace to column connections and to do fatigue analyses. Another aspect with the bracing design is over all and local buckling which limits the L/D and D/t ratios in practical design.

1.6.3 Tower and tower support structure

The tower support braces have also been included in the design wave analyses and the structural response related to the internal system forces and static loads are reported. However, the response in these structures is also related to the global dynamic behavior which is not included in the design wave method.

Accordingly for the purpose of concept design, the extreme loads in the tower and support braces have been based on simplified spread sheet analyses by applying maximum dynamic pitch angle, maximum tower top accelerations and static wind loads.

	Tower data			Inertia forces				Gravity forces	
	Section No	Level m	Section Weight kg	Accel in VCG m/s2	Inertia force N	Mom.arm tower base m	Tower base moment	Transv.W Comp **0,292372**	Tower base moment
Turbine		74,77	87400	**3,00**	262200	38,27	10034394	250677,7	9593437
Top		**73,35**							
	19	71,41	2094	2,89	6049	35,86	216958,8	6006,712	215426,2
	18	69,47	2307	2,80	6456	33,93	219011,5	6615,703	224441,5
T	17	67,53	2529	2,71	6850	31,99	219101,2	7252,249	231976,4
O	16	65,59	2760	2,62	7229	30,05	217220,6	7916,351	237870,5
W	15	63,65	3001	2,53	7591	28,11	213382,2	8608,008	241963,6
E	14	61,71	3252	2,44	7933	26,17	207619	9327,221	244095,2
R	13	59,77	3512	2,35	8253	24,23	199983,9	10073,99	244105,1
	12	57,83	3782	2,26	8548	22,29	190549,9	10848,31	241832,9
	11	55,89	4062	2,17	8815	20,35	179410,1	11650,19	237118,2
	10	53,96	4351	2,08	9052	18,41	166677,6	12479,63	229800,9
	9	52,02	4650	1,99	9256	16,47	152485,7	13336,62	219720,5
	8	50,08	4958	1,90	9424	14,54	136987,8	14221,17	206716,7
	7	48,14	5276	1,81	9555	12,60	120357,3	15133,27	190629,1
	6	46,20	5604	1,72	9645	10,66	102787,6	16072,93	171297,5
	5	44,26	5941	1,63	9691	8,72	84492,49	17040,14	148561,5
	4	42,32	6288	1,54	9692	6,78	65705,57	18034,91	122260,8
	3	40,38	6644	1,45	9645	4,84	46680,62	19057,24	92235,07
	2	38,44	7010	1,36	9547	2,90	27691,52	20107,12	58323,93
Bottom	1	**36,50**	7386	1,27	9395	0,96	9032,222	21184,55	20367,08
BASE		24,60	46486		**424825**		**12810530**	**495644**	**13172180**

Wind force	
Wind Load N	Wind m. arm m
150000	39
	5850000

Based on the various results from the dynamic response analyses the extreme tower bending moments have been calculated for a tower top acceleration of 3m/sec and an extreme pitch angle of 17 deg. With a wall diameter of 4m and a thickness of 40mm t this gives a bending stress of 63,6 MPa in the tower transition to the base structure. This rather low extreme bending stress should be a good basis for fatigue capacity. In the detail design and documentation the tower has to be documented for fatigue based on an integrated wave and wind approach. However, it is assumed that the present dimensions should be sufficiently conservative. The axial stress in the tower base due to gravity is calculated to 34 MPa

Tower bending stress extreme condition		
Z-Level Centre of rotation	**10**	m
Tower top Acceleration	**3**	m/s
Pitch angle	**17**	deg
Moment at brace intesection	31989990	Nm
Shear force at brace inters	1070469	N
Diameter	4,00	m
wall thickness	40,00	mm
Section modulus	502654825	mm2
Bending stress top of base	**63,6**	**MPa**

Based on the extreme bending moment, the horizontal reaction forces at upper and lower supports of the tower have been calculated. Based on these forces the axial support forces in the braces have been calculated as presented in the table at right. However, the SESTRA analyses show that there is a ratio between the combined bending and axial stresses of approx 1,46. This means that the extreme dynamic stress can be approx 76 MPa in the diagonal brace. This is higher than the stress in the tower, however if there will be fatigue problems with the diagonal braces, the first measure is to increase the thickness which in the present case is 16 mm

Simplified dynamic axial stress in braces		
Distance Low to high supp	11,303	m
Total hor. Load	**1070**	kN
Hor. reaction force lower	**2830**	kN
Horiz. Reaction force upper	**3901**	kN
Diag. Brace angel with hor.	**0,5948**	rad
Diag. Brace angel with hor.	**34,0798**	
Upper brace axial load	2354,74	kN
Lower brace axial stress	**28,47**	Mpa
Diag. brace axial stress	**52,05**	Mpa

Based on the gravity of turbine and support structure, the static axial loads in the diagonal braces a have been calculated as shown in the table on right. From the SESTRA analyzes we get the same axial load in the diagonal braces, but it can be seen the von Mises stress is 52,9 MPa compared with the axial stress of 33,7MPa. This means that there is also a bending effect due to deflection of the structure. This is

Axial stress in diag brace due to gravity load		
WTG, Tower & base weight	219,29	t
Diag. Brace weight	25,54	t
Total weight taken by diag	244,83	
Angle of brace with horiz.	0,59480458	rad
Axial load in brace	1,429	MN
Sectio area brace	0,0424115	m2
Axial stress	**33,69**	Mpa

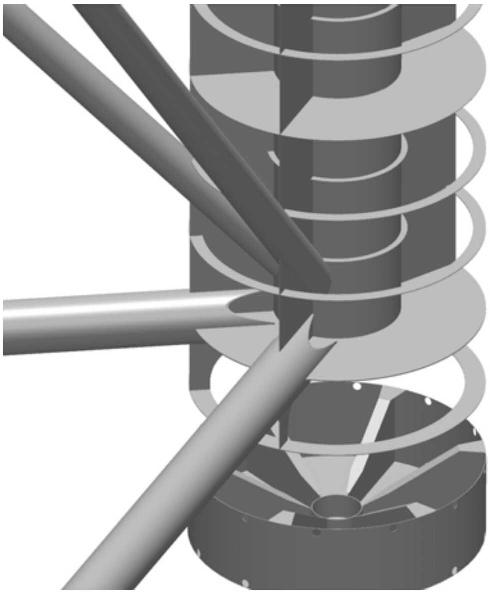
1.6.4 Columns design

The column which has a height of 25,5m and a diameter of 7m is divided from the bottom into a 7,5 m high ballast tank. Between the tank deck and the top deck there is an internal central shaft of 3m diameter up to the top deck. The column above the tank deck with height of 18 m is divided with two water tight decks into 3 sections of 6m each. In addition there are 3 vertical WT bulkheads, one in each of the two brace plans with 60 degree separation. The third bulkhead is in the middle plane of these separated 150 degrees from each.

The bulkheads in the brace planes have partly a function to transfer brace forces and to strengthen the column locally. The third bulkhead has a function to divide the column into sufficiently small compartments to survive a condition with two damaged compartments. This bulkhead will also be below the mooring connection which may have to take rather high loads and probably need local structural reinforcements.

The column external shell has ring stiffeners with 1m distance from bottom to top. The internal column has also ring stiffeners with 1m distance. The decks have radial stiffeners between outer and inner columns and the vertical bulkheads have horizontal stiffeners spaced 1m in line with the ring stiffeners on the outer and inner columns.

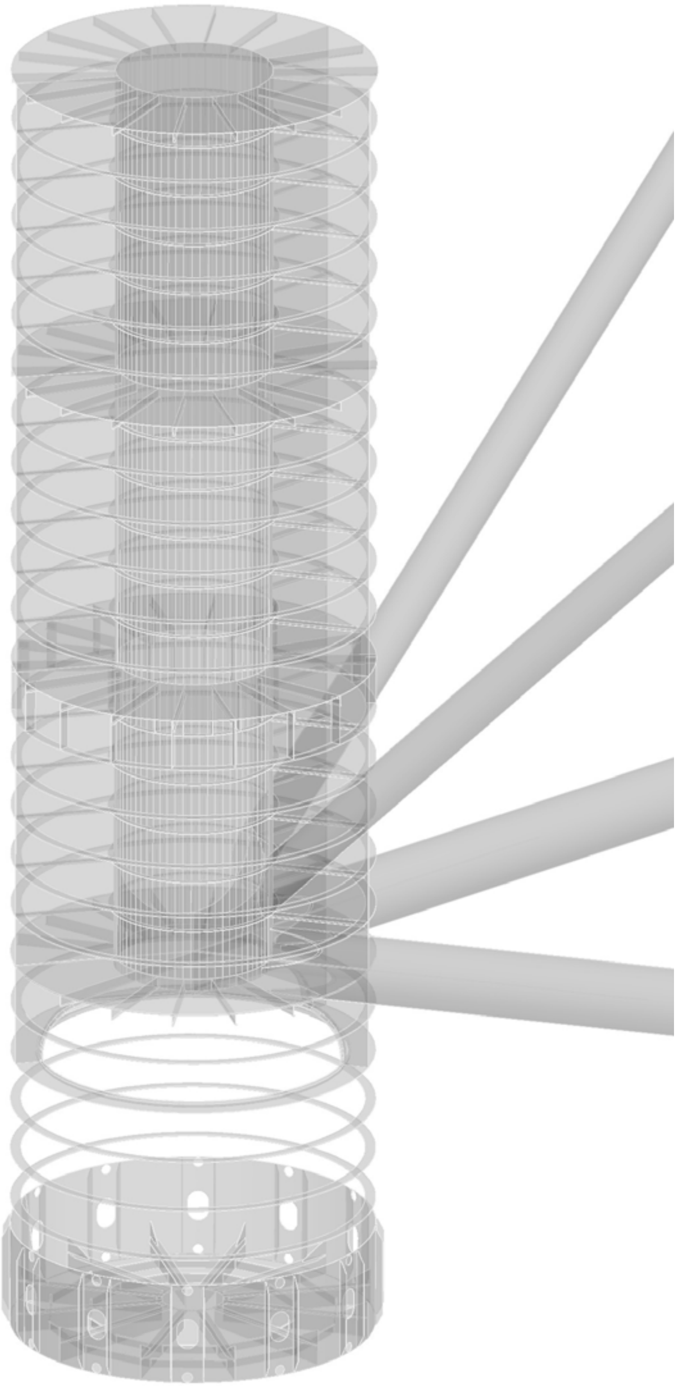
The horizontal brace which penetrates into the column are connected internally with its centre planes to the tank deck and to the vertical bulkhead which accordingly constitutes important structural elements. The required strength of these elements is controlled by the thickness. In the present design they have been given 30mm thickness which is consistent with the section area of the brace. This connection must be scrutinized with FE analyses in the detail engineering.



The dimensioning of the column structure is mainly based on simplified assumptions with respect to hydro static pressure. In intact condition it is considered that the column can be submerged to the top of the column giving an outside pressure head of 25,5 m at the bottom level. At the top of the column it is been decided to use a pressure head of 5m. The pressures at intermediate levels have been calculated by interpolation between the bottom an top level. In the condition with maximum pressure it has been considered that the ballast tank is filled with water, such that the differential design pressure in the ballast tank is approx. 20m Pressure head.

The internal decks and bulkheads in the dry compartments have also been calculated with the same pressure due to
consideration that the tanks can be damaged and exposed for external pressure.
The shell thickness and required ring stiffener dimensions are based on DNV RP- C202 " Buckling strength of shells"
Dimensioning of flat plates and stiffeners is based on DNV –OS-J101 "Offshore Wind"
The structure with all dimensions has bee modeled in 3D CAD. Structural weights and CoGs have been taken directly from the model and increased with a weight contingency of 5 %
The weight and resulting CoG have been transformed to weight factors for the upper and lower part of the columns and used for updating of the spread sheet model.
An important work in the detail engineering will be to perform detail dimensioning of the structure based on BV rules and regulation.
It will also be possible to optimize the overall an local design solutions based on the fabrication yards preferences and experience. The only requirement is that the global design and geometry is not changed and that the structure satisfies strength and fatigue requirements.

If it is considered to simplify the structure by increasing stiffener distances, it will probably lead to a heavier structure. However, this will have insignificant influence on the floaters dynamic behavior. The total floater weight has been reduced by the structural design compared with the earlier weight assumptions and it is mainly a matter of amount of ballast water.



Weight and CoG table from MS DAK model		Keel vertical level, $Z_K = 0$		In DAK model $Z_K =$ -128,381				
COLUMN STRUCTURE	Element name	Name of DAK level	Element Specification	Numb off	Element weight (t)	DAK vert. CoG	Weight (t)	Moment (tm)
Section 0	Top deck	TD PL	12mm	1	3,62524	-102,887	3,625	-372,9901
Z = 25,5	Deck stiffeners outer	TD STF	HP 180 X 8	16	0,0292871	-103,002	0,469	-48,26608
	Deck stiffeners inner	TD STF	HP 180 X 8	4	0,03	-103,002	0,120	-12,36024
Top deck area								
Mass (t)	Brace connection details			1	3	-103	3,000	-309
7,214								
CoG								
25,438						-102,9434	7,214	-742,616
SECTION 1	Bulkheads	VBH	12mm	3	1,12137	-105,884	3,364	-356,2054
Z = (19,5 - 25,5)m	Bulkhead hor stiff	VBH FR-STF	HP 180x8	15	0,0283852	-105,898	0,426	-45,08904
	Outer shell	OSH	11mm	1	11,353	-105,887	11,353	-1202,135
Dry compartments	Outer Ring frame	OSH RF	150 x 20	5	0,505166	-105,881	2,526	-267,4374
	Inner column shell	ISH	10mm	1	4,44644	-105,886	4,446	-470,8157
	Inner ring frame	ISH RF	120x12	5	0,111509	-105,881	0,558	-59,03342
								0
Mass (t)	Deck stiffeners	DK STF	HP 180 x 8	16	0,0292578	-108,982	0,468	-51,01718
26,071								
CoG	Deck	DK	12mm	1	2,92979	-108,881	2,930	-318,9985
22,103						-106,2779	26,071	-2770,732
SECTION 2	Bulkheads	VBH	12mm	3	1,12192	-111,881	3,366	-376,5646
Z = (13,5 - 19,5)m	Bulkhead hor stiff	VBH FR	HP 200x9	15	0,0393564	-111,901	0,590	-66,06031
	Outer shell	OSH	12mm	1	12,4081	-111,881	12,408	-1388,231
Dry compartments	Outer Ring frames	OSH RF	180 x 20	5	0,603358	-111,881	3,017	-337,5215
	Inner column shell	ISH	10mm	1	4,42427	-111,881	4,424	-494,9918
	Inner ring frames	ISH RF	120x12	5	0,111509	-111,881	0,558	-62,37869
								0
Mass (t)	Deck stiffeners	DK STF	HP 200x9	16	0,0406888	-115,016	0,651	-74,87781
29,254	Stiffener brackets	DK STF	15mm	16	0,0362604	-115,407	0,580	-66,95526
CoG	Deck	DK	15mm	1	3,66047	-114,881	3,660	-420,5185
15,984						-112,3965	29,254	-3288,099
SECTION 3	Bulkhead 1	VBH	12mm	1	1,11631	-117,881	1,116	-131,5917
Z = (7,5 - 13,5)m	Bulkhead 2&3	VBH	20mm	2	1,86052	-117,881	3,721	-438,6399
	Bulkhead hor frames	VBH FR	HP 220x10	15	0,0436449	-117,9	0,655	-77,18601
Dry compartments	Outer shell	OSH	13mm	1	13,4402	-117,881	13,440	-1584,344
	Outer Ring frames	OSH RF	200 x 20	5	0,668228	-117,881	3,341	-393,8569
	Inner column shell	ISH	12mm	1	5,34819	-117,881	5,348	-630,45
	Inner ring frames	ISH RF	120 x 12	5	0,111509	-117,881	0,558	-65,72396
	Diagonal brace stub	DBR	Cut inside shell	2	0,929201	-118,405	1,858	-220,0441
	Horizontal brace stub	HBR	Cut inside shell	2	0,792526	-120,881	1,585	-191,6027
Mass (t)				3				
41,083	Deck stiffeners		HP 220x10	10	0,0448952	-121,029	0,449	-54,33621
CoG	Deck plate (brace connection)		30mm	1	9,01139	-120,881	9,011	-1089,306
9,668						-118,7132	41,083	-4877,082
SECTION 4	Brace support web plates	BRS WP	20mm	3	0,560153	-121,491	1,680	-204,1606
Z = (2 - 7,5)m	Web fllanges	BRS WP	300 x 20mm	3	0,173435	-121,892	0,520	-63,42102
Ballast tank	Shell plate	OSH	15mm	1	14,2115	-123,631	14,212	-1756,982
Mass (t)	Ring frame 1	OSH RF	200x20	1	0,667833	-121,881	0,668	-81,39615
20,368	Ring frame 2	OSH RF	485x12, 120x20	1	1,28473	-122,881	1,285	-157,8689
CoG	Ring frame 3-4-5	OSH RF	200x20	3	0,667833	-124,881	2,003	-250,199
4,953						-123,4283	20,368	-2514,028
SECTION 5	Shell plate	OSH	15mm	1	4,7878	-127,381	4,788	-609,8748
Z = (0 - 2)m	Vertical buckling stiffeners	OSH STF	FB 150x12	24	0,0246992	-127,381	0,593	-75,50901
	Bottom girders	BTM GD		12	0,219751	-128,096	2,637	-337,7907
Ballast tank	outer brackets	BTM GD		12	0,0121341	-127,861	0,146	-18,61774
	Inner brackets	BTM GD		24	0,00190038	-127,976	0,046	-5,836873
	Central circular box	BTM GD		1	0,166576	-128,176	0,167	-21,35105
	Outer stiffener	BTM STF		12	0,021226	-128,257	0,255	-32,6686
Mass (t)	Inner stiffener	BTM STF		12	0,0151086	-128,257	0,181	-23,2534
13,482	Bottom plate	BTM PL		1	4,49283	-128,373	4,493	-576,7581

ANALYSES

1.7 General description

Input for the hydrodynamic software *HydroD*/*WADAM* (DNV software) and *SIMO* (Marintek/DNV software) is generated from the spreadsheet using a python script. This approach ensures that parameter variations are easy to conduct. The python script will generate input for the finite element pre-processor *PREFEM* (DNV software) and the pre-processor for assembling super elements *PRESEL* (DNV software) which together generates the finite element description of the geometry. These finite element models are then used as input to *HydroD/WADAM*.

Two coordinate systems have been used in this study. The global coordinate system (North-East) has been used to define the loading conditions experience onsite, while a local coordinate system has been used for the floater. The correlation between the two coordinate systems defines the orientation of the floater. A principal sketch of the coordinate system and angle of orientation is shown in Figure 1.7.1.

The damper plate will be modelled in a simplified geometry. The complex geometry of the damper is approximated by a simple short cylinder part. This cylinder part should have approximately the same added mass properties as the real configuration. The entrapped water in the outer parts of the damper plate is accounted for by including it in the volume of the damper.

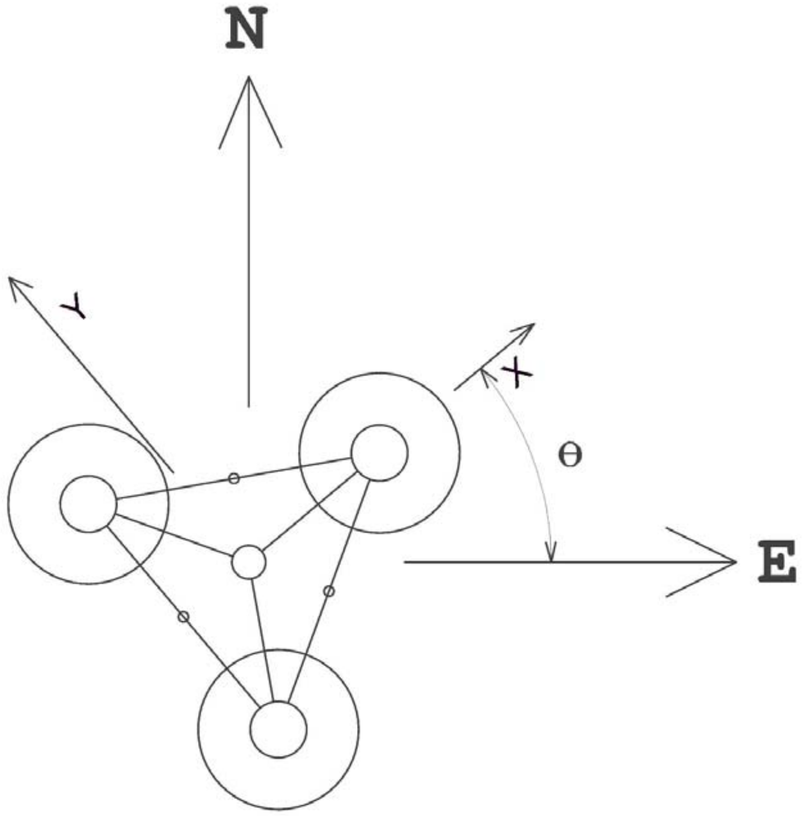


Figure 1.7.1 Global coordinate system (N-E) and local coordinate system (x-y) with floater orientation (θ) defined.

1.8 Global response analyses

The global response analysis presented here is based on calculations preformed in *SIMO* and *HydroD*/*WADAM*. *SIMO* is used to perform a coupled response analysis of floater and mooring system in the time domain. A Jonswap wave spectrum is used to approximate the relevant sea states. Non-linear catenary effects in the mooring lines are included in the global response analysis. The effect of drag forces on the mooring lines is, on the other hand, not included due to a limitation in *SIMO*, giving a quasi static mooring model. Morison theory beam elements (*SIMO*: “slender elements”) are included in the *SIMO* model to account for non-linear drag forces of columns and braces in addition to added mass of braces. *SIMO* will calculate the Morison forces based on the incident wave field (not accounting for the diffracted wave). Morison elements above still water level will be subjected to drag forces based on the wave velocity at still water level.

Potential theory properties of the columns are calculated by *WADAM*, based on the panel model in Figure 1.8.1(a), and imported into *SIMO*. These potential theory properties are global mass data, added mass (of the potential theory model), potential damping (radiation), excitation forces from potential theory (diffraction) and hydrostatic stiffness. Global mass data has been calculated in the spreadsheet and are given as direct input to *WADAM*. A more refined mass model will be used in the structural analysis of the braces.

The *WADAM* model used in the global response analysis includes a Morison beam model, see Figure 1.8.1(b), without drag contributions ($C_d = 0$). Drag forces for the braces are neglected in the *WADAM* model to avoid problems when importing data from *WADAM* to *SIMO*. The only contribution from the Morison beam elements will then be added mass. The effect of added mass is important when estimating the eigen periods. These eigen periods will be used, in *WADAM*, to calculate the applied damping (6% of critical damping).

The global response analysis will for each case include a 3 hour simulation with a time step of 0.1 seconds. Running a 3 hour simulation should be sufficient to estimate the actual behaviour of the floater in a given sea state.

1.8.1 Included hydrodynamic effects

The present model includes linear potential wave theory and the following non-linear effects:

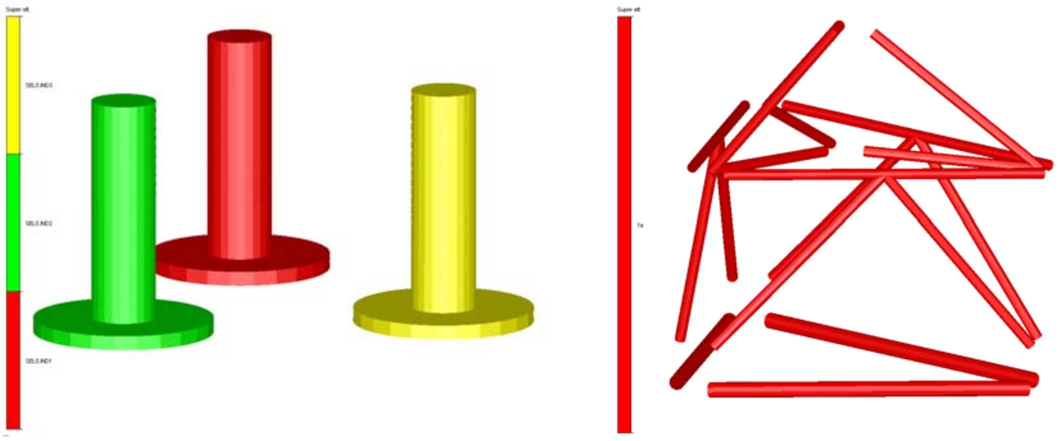
- Drag forces on Morison elements.
- Catenary effects in mooring lines.

The following non-linear effects are not included:

- Waves on deck.
- Slowly varying wave drift forces.

Slowly varying wave drift forces is not included since the final mooring configuration is uncertain. The effect of slowly varying wave drift forces is significantly dependent on the surge/sway eigen period of the system. The eigen period in surge/sway will be very dependent on the selected mooring configuration. A study of slowly varying wave drift forces should be conducted when the final mooring configuration is decided.

Damping is included to ensure reasonable results and numerical stability. A damping corresponding to 6% of the critical damping in each degree of freedom is used.



(a) Panel model *(b) Morison/beam model*

Figure 1.8.1 Shows the finite elements models included in the global response analysis. The Morison model (b) does only include contributions related to added mass and buoyancy while drag is neglected ($C_d = 0$).

1.8.2 Loading conditions

The preliminary Loading conditions proposed by Tecnalia for the BIMEP are, see Table 1.8.1, will be used in the current study. To reduce the number of load cases we have selected to analyse condition 1a, 1b, 1c, 2b, 3b and 4b. Wind thrust on the tower and turbine is based the calculations in the spreadsheet “110629-HiPRWind-WP1-Design Basic Data_Rev07-send.xls” prepared by Acciona. The operational conditions with maximum thrust will be considered in the detail engineering phase with more advanced analyses. A cos(2s) wave spreading with 2s=40 will be applied. A Jonswap spectrum with the peakedness parameter γ=1.7 will be used for all wave conditions considered, both sell and wind sea.

Case	**1a,b,c**	**2 a,b,c**	**3 a,b,c**	**4 a,b,c**	**5 a,b,c**	**6 a,b,c**
Objective	**Swell governed**	**Wind/current governed**	**Swell & wind/wave**	**Limit operation**	**Operation max thrust colin. (~DLC1.6a)**	**Operation max thrust 90°. (~DLC1.6a)**
Wave RP (year)	**50**	**0,1**	**1**	**1**	**50**	**50**
Hs	10,51	5	8.28	8.28	10,51	10,51
Dir (from)	WNW	WNW	WNW	WNW	WNW	WNW
Tp a (min)	12	10	10	10	12	12
Tp b (mean)	16	15	15	15	16	16
Tp c (max)	20	20	20	20	20	20
Wind Sea RP (year)	**~0,1**	**~50**	**~1**			
Cross wind sea	**(1m/6s/NE)**	**(4m/10s/NE)**	**(3m/8s/NE)**	**-**	-	-
Wind RP (year)	**5**	**50**	**1**	**-**	-	-
Wind1hr at 10m (m/s) *$V_{@60m}=V_{@10m}$ *1.218	27,9	32,4	24,3	25 at turbine level	11,4 at turbine level	11,4 at turbine level
Wind dir (from)	WNW	NE	NE	SSW	WNW	SSW
Currents RP (year)	**5**	**50**	**1**	**2**	-	-
Current speed	1,2	1,4	1,1	1,1	0,5	0,5
Current dir (towards)	E	W	W	NE	E	E

Table 1.8.1 List of loading conditions used in the current study (ref.: Tecnalia 29.06.2011). Load cases included in the current report is indicated with red font.

1.9 Structural analysis of brace forces

WADAM is used to calculate the expected brace forces used in the structural design. The element model used in the structural finite element analyses is shown in Figure 1.9.1. Brace forces for regular waves are scaled in accordance with the wave steepness factors given in DNV-RP-C103 (H_{100} = 25 m) to give design forces. Wave directions from 30 degrees to 150 degrees are studied, with a step of 15 degrees. All wave directions are related to the local coordinate system and the angle is positive in the counter-clockwise direction. It should be sufficient to study a direction range of 120 degrees considering the symmetric properties of the floater. Wave periods in the range from 4 to 20 seconds are studied with a step of 0.5 seconds. Linearized drag forces on the braces and added mass of the braces are included in this analysis. In addition the analysis includes linear potential theory calculations for the columns. The complex panel pressures and beam loads calculated by *WADAM* is then used as input in a quasi-static finite element analysis performed in *SESTRA* (DNV software). The elastic moduli of the columns are set to 1000 times the elastic modulus of steel to avoid local deformations of the columns. This approach should give reasonable boundary conditions for the braces. Results from the finite element analysis are then imported into a program called *FRAMEWORK* (DNV software) and the Von Mises stress is calculated for all beam elements at cross sectional predefined stress points (distributed along the outer diameter of the pipe sections).

The mass model is created by applying equivalent thicknesses, assuming the density of steel, to the panel parts of the model (cylinders and damper plates). The columns are divided into three different parts to account for the actual weight distribution. The three parts consists of an upper part (steel column mass and secondary structure mass), intermediate part (steel column mass) and a lower part (steel column mass and water ballast). This subdivision of the columns is described in more detail in the spreadsheet under “Mass & Geometry model”. The relevant water ballast will also be included when calculating the equivalent thickness of the damper plate. The pipe shaped braces are modelled with real dimensions and the weight contingency for each brace is included by scaling the density of the material for the relevant brace. The upper U-shaped braces are modelled as pipes with nominal thickness and the diameter specified in the description. Further studies, considering the actual geometry of the cross-section, must be conducted for the U-shaped braces in the detail design. Braces connected at the top of the column are modelled with the neutral axis in the same level as the column top. The nacelle mass and the equivalent mooring masses are included as point masses at the relevant locations. The horizontal eccentricity of the nacelle is, however not included. The tower is modelled with 19 beam elements accounting for the actual weight distribution.

The following basic properties have been assumed for the steel material:

- Density: 7.85 kN/m^3
- Elastic modulus: $2.10 \cdot 10^8$ kN/m^2

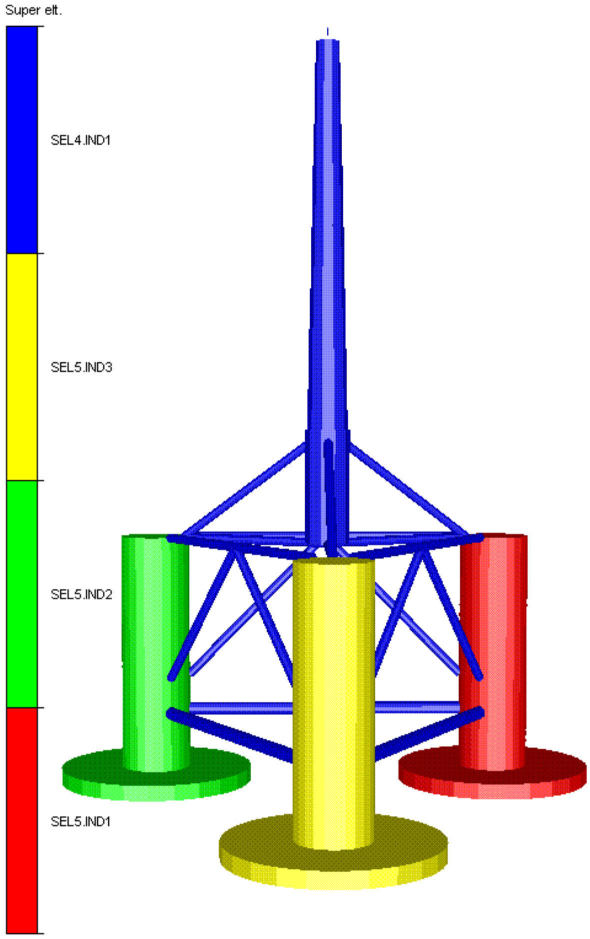


Figure 1.9.1 The structural element model with super element subdivision. The same model is used as mass model.

1.10 Analysis of dynamic loads on damper plate

1.10.1 Summary

This MEMO describes an approximate analysis of dynamic pressure on a thin rim connected to a column. The damper plates in the HiPRWind floater concept will contain an outer thin rim. It is important to consider the dynamic pressure on the outer rim part when performing the structural design of the damper plates.

1.10.2 Approximate model

The approximate analysis is carried out in WADAM (DNV software) which is based on potential theory formulations. Only one column, was included in the analysis due to limitations in WADAM. The column is assumed fixed to exclude the effect of non-physical motion. The analysis will then include the effect of a diffracted wave field, while the effect of radiation and fluctuating hydrostatic pressure is neglected. The floater will experience limited dynamic response, indicating that the effect of radiation should be relatively small. The effect of fluctuating hydrostatic pressure will be approximately the same on the top and bottom surface of the rim, giving no resultant pressure contribution.

A rim thickness of approximately 24 cm is used. In the final design of the outer rim this thickness will be significantly lower, but the current thickness is selected due to practical modelling limitations. It is assumed that the considered rim thickness will give a reasonable estimate of the resultant pressure on the thin rim parts. The mesh used for the top and bottom of the thin rim is shown in Figure 3.

A design wave approach, in accordance with DNV-RP-C103, is used to estimate the resulting pressure. Wave periods from 6 to 18 seconds have been studied. The maximum wave height for a wave with a 100 year return is set to 25 m and the wave steepness for each periode is calculated with respect to this value. A maximum wave height of 24.5 m are reported in the “ General description of the bimep” for waves with a 225 year return period, so the maximum wave height used should be conservative.

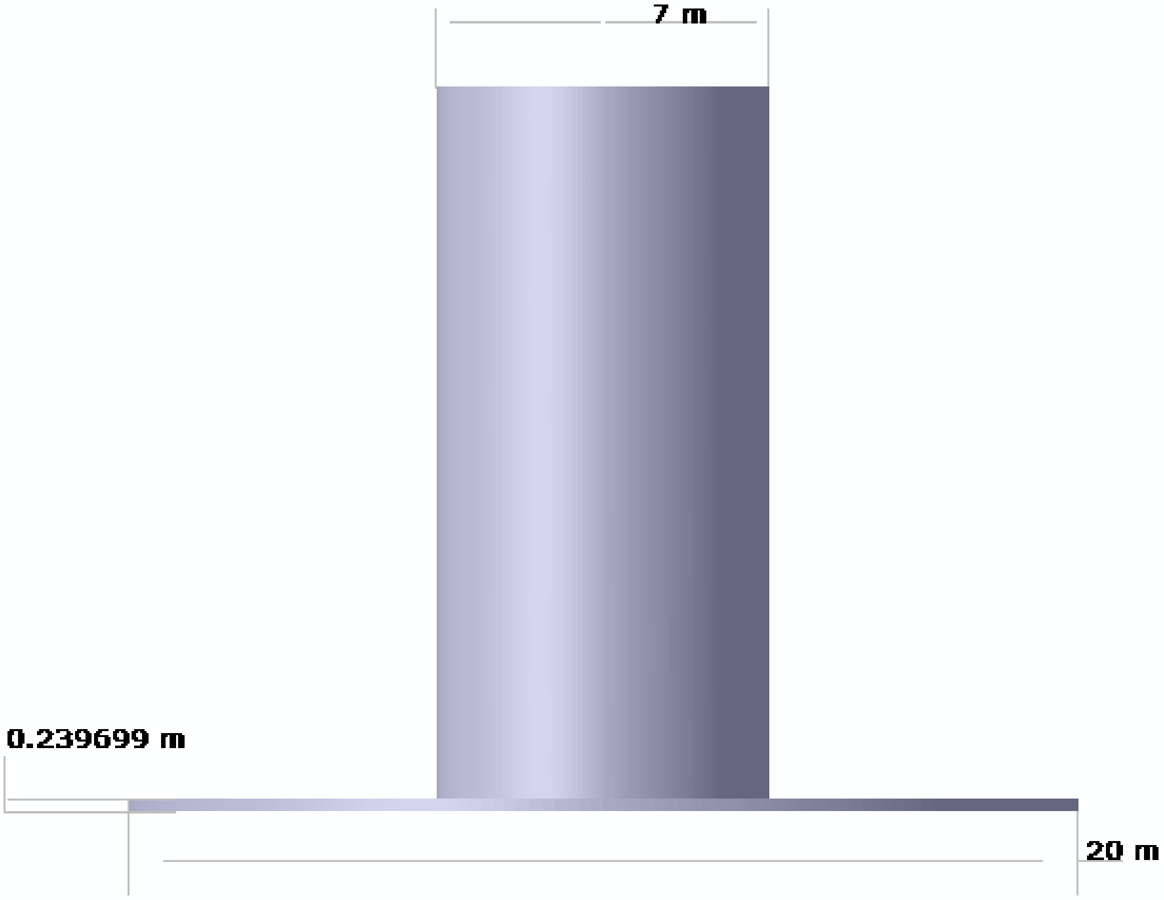


Figure 1.10.1 Geometry of simplified column.

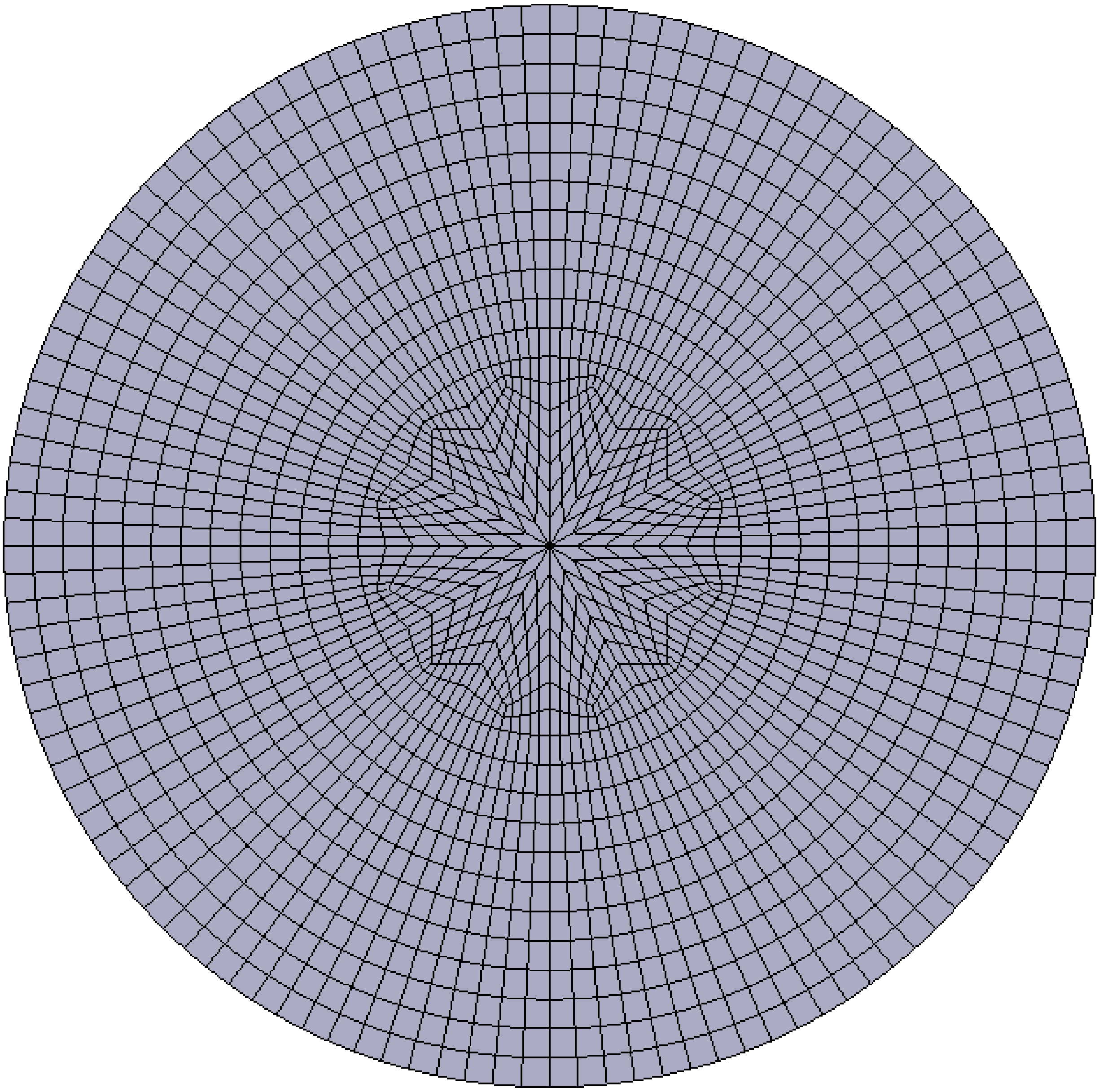


Figure 1.10.2 Finite element mesh of simplified plate.

1.11 Assumed hydrostatic properties

The following values are used for the Morsion elements (*SIMO*: “slender elements”):

- **Drag coefficient:**
  The drag coefficient, Cd, is assumed to be approximately equal to 1.0 for the pipe cross sections used. This value can be found in DNV-RP-C205 and is valid for a Reynolds number Re ~ 105. This drag coefficient is used to calculate both the drag effect due to wave motion and current in the global response analysis.
- **Drag linearization velocity:**
  A drag linearization velocity of 1 m/s is applied in the structural analysis of the braces.
- **Added mass coefficient:**
  The added mass coefficient, Ca, for the braces is set equal to 1.0 as given in DNV-RP-C205.

The global damping of the system is, as mentioned above, assumed to be 6% of the critical damping in each degree of freedom. The damping contribution is calculated directly by *WADAM* for heave, roll and pitch. Values for surge, sway and yaw is based on eigen period calculations for the coupled system performed in *SIMO*.

1.12 Uncertainties

1.12.1 General damping

The current degree of damping is selected to give a reasonable maximum RAO (response amplitude operator) as estimated by *WADAM* (mooring not included). Model tests should be carried out to estimate the actual degree of damping for the forthcoming detail design. Damping will in the current model be an effect of both drag contributions and the global applied damping matrix. This combined damping effect should be accounted for when the model is tuned with respect to model test results.

1.12.2 Effects of damper plates

The damper plates will contribute to the damping of the structure through considerable drag effects in the transverse direction. These effects are not directly included in the current model, but they might, to some extent, be included in the global damping matrix contributions for heave, roll and pitch movement. The drag effect of the damper plates will be a local effect for each column giving contributions in heave, roll and pitch movement for the entire floater. This effect can be modelled by including a drag coefficient in the vertical direction for each damper plate or as contributions in the global damping matrix. Drag effects in the horizontal plane are in the present model included for the damper plates (in the same manner as for the columns). Another insecurity with respect to the damper plate is the added mass since the damper plate is simplified in the analyses models. The size of the added mass terms from the analyses should be compared with model tests.

1.12.3 Drag effects

The effect of drag in correlation with currents and waves should be investigated during model tests to ensure reasonable results. The effect of shielding might reduce the drag forces significantly especially for constant current flow.

1.12.4 The mooring system

The present mooring system is most likely not the final one. Global response parameters will vary with changes in the mooring configuration. This indicates that new analyses should be carried out when the final mooring configuration is decided. These final coupled analyses should also include the effect of slowly varying wave drift forces (as discussed above). This final configuration should then fulfil the excursion requirement in addition to the relevant requirement for maximum line load. Both these requirements will be significantly dependent on the actual current and wind loads on the structure in addition to the dynamic wave loads.

1.13 Results

1.13.1 Global response analysis

Results for a number of floater orientations will be presented to show how the orientation influences the response of the coupled system.

Approximate eigen periods for the floater is given in Table 1.13.1. The eigen periods in surge, sway and yaw are dependent on the mooring system. Eigen periods in surge and sway will be in the range of 50-70 seconds while the yaw period seems to be in the range of 40-45 seconds for the present mooring system. Heave and pitch response amplitudes operator (RAOs) are shown in Figure 1.13.1 and Figure 1.13.2 respectively.

	Heave	***roll***	***pitch***
Eigen period (s)	19.4	32.5	32.5

Table 1.13.1 Approximate eigen periods from WADAM analyses.

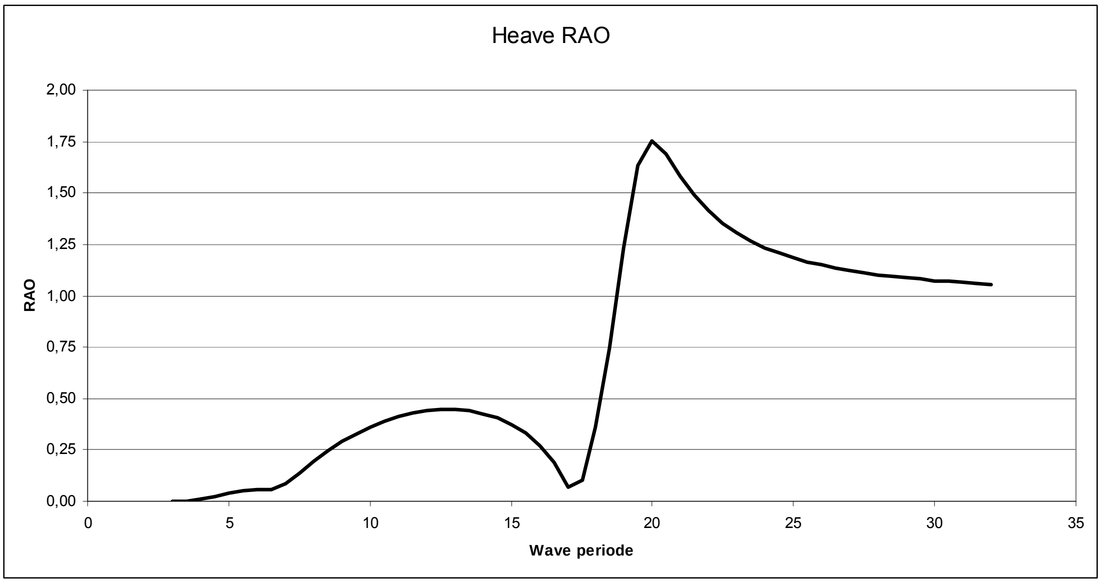


Figure 1.13.1 The heave RAO as a function of wave period for the floater. (Drag forces are neglected).

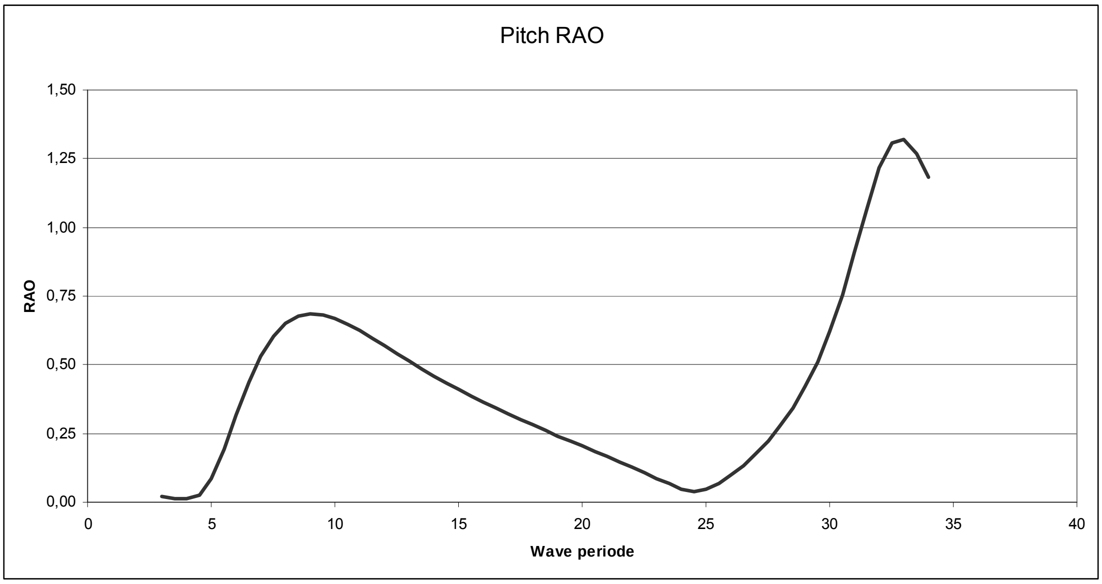


Figure 1.13.2 The pitch RAO as a function of wave period for the floater. (Drag forces are neglected).

The global response analyses are performed for a short-term sea state (3 hour real time simulation, with one random generator seed for the waves). The results for the selected load cases, are given in Table 1.13.2 to Table 1.13.7. The present mooring system is seen to fulfil the excursion criterion for all the considered load cases. The response for load case 1c is quite extreme with respect to heave movement and pitch angle. These extreme values should be expected since the peak period of the appropriate spectrum is approximately equal to the heave eigen period of the system, giving resonant behaviour. The damping applied in the present model will most likely be a conservative estimate of the actual damping. A higher degree of damping would reduce the resonant response significantly. This high peak period of 20 seconds should anyway represent an unlikely loading condition. Based on the results presented here, it is concluded that load case 1 seems to be the most critical one. An orientation of the floater in the range of 75°-120° seems to be the most reasonable orientation based on the global response characteristics presented here. The maximum mooring line loads should, for this orientation range, be within the acceptable range, but the relatively high mooring loads will imply significant strengthening of the structure at the mooring connection points.

Load case 1a:

Orientation	Excursion	Heave	Yaw	Line load	Nacelle Acceleration		Pitch
					Horizontal	***Vertical***	
(deg)	*(m)*	*(m)*	*(deg)*	*(kN)*	*(m/s2)*	*(m/s2)*	*(deg)*
0	18,45	4,16	4,96	2046	2,02	1,29	13,36
15	16,85	4,22	5,84	2263	1,83	1,27	12,15
30	15,37	4,29	3,57	2430	1,99	1,25	12,55
45	14,28	4,33	4,72	2469	2,08	1,27	12,71
60	14,60	4,29	5,73	2322	1,81	1,30	12,68
75	16,30	4,24	5,85	2084	2,03	1,33	12,88
90	17,76	4,18	4,30	1752	2,16	1,33	12,82
105	18,56	4,16	4,96	1655	2,31	1,33	13,57
120	18,43	4,16	5,12	2042	2,09	1,29	13,28
max:	18,56	4,33	5,85	2469	2,31	1,33	13,57

Table 1.13.2 Characteristic maximum response parameters for the load case 1a in Table 1.8.1.

Load case 1b:

Orientation	Excursion	Heave	Yaw	Line load	Nacelle Acceleration		Pitch
					Horizontal	***Vertical***	
(deg)	*(m)*	*(m)*	*(deg)*	*(kN)*	*(m/s2)*	*(m/s2)*	*(deg)*
0	17,49	5,64	4,82	2704	2,10	1,05	13,04
15	16,70	5,68	4,76	3583	2,64	1,07	12,80
30	15,45	5,78	3,09	3975	3,12	1,06	12,94
45	14,99	5,90	3,24	4185	3,28	1,06	13,27
60	15,03	5,84	4,70	3598	2,56	1,05	12,44
75	16,34	5,80	4,47	3077	2,38	1,05	12,96
90	17,22	5,73	3,59	2299	2,00	1,05	13,11
105	18,05	5,74	3,80	1930	2,00	1,05	12,97
120	17,49	5,64	4,47	2701	2,08	1,05	12,99
max:	18,05	5,90	4,82	4185	3,28	1,07	13,27

Table 1.13.3 Characteristic maximum response parameters for the load case 1b in Table 1.8.1.

Load case 1c:

Orientation	Excursion	Heave	Yaw	Line load	Nacelle Acceleration		Pitch
					Horizontal	***Vertical***	
(deg)	*(m)*	*(m)*	*(deg)*	*(kN)*	*(m/s2)*	*(m/s2)*	*(deg)*
0	16,51	10,11	4,92	2899	2,32	1,33	16,97
15	15,83	10,34	4,49	3757	2,91	1,44	19,08
30	14,52	10,41	3,56	4302	3,29	1,57	21,24
45	14,14	10,42	4,76	4488	3,45	1,68	20,37
60	13,83	10,27	4,52	3859	2,77	1,61	17,68
75	15,19	10,23	4,55	3195	2,46	1,49	17,42
90	16,01	10,07	4,68	2447	2,20	1,35	16,29
105	16,91	10,05	4,53	2291	2,15	1,31	15,54
120	16,52	10,12	4,84	2904	2,32	1,33	16,98
max:	16,91	10,42	4,92	4488	3,45	1,68	21,24

Table 1.13.4 Characteristic maximum response parameters for the load case 1c in Table 1.8.1.

Load case 2b:

Orientation *(deg)*	**Excursion** *(m)*	**Heave** *(m)*	**Yaw** *(deg)*	**Line load** *(kN)*	**Nacelle Acceleration**		**Pitch** *(deg)*
					Horizontal *(m/s2)*	***Vertical*** *(m/s2)*	
0	11,11	2,80	1,39	1837	1,24	0,75	9,56
15	11,34	2,80	1,65	1756	1,25	0,75	9,26
30	12,19	2,80	1,26	1608	1,30	0,76	8,78
45	13,35	2,77	1,36	1429	1,29	0,76	9,18
60	13,84	2,76	1,37	1263	1,09	0,76	9,10
75	13,87	2,76	1,66	1581	1,20	0,75	9,32
90	12,95	2,76	1,28	1790	1,22	0,75	9,63
105	11,96	2,78	1,56	1915	1,28	0,74	9,91
120	11,11	2,80	1,56	1833	1,24	0,75	9,50
max:	13,87	2,80	1,66	1915	1,30	0,76	9,91

Table 1.13.5 Characteristic maximum response parameters for the load case 2b in Table 1.8.1.

Load case 3b:

Orientation *(deg)*	**Excursion** *(m)*	**Heave** *(m)*	**Yaw** *(deg)*	**Line load** *(kN)*	**Nacelle Acceleration**		**Pitch** *(deg)*
					Horizontal *(m/s2)*	***Vertical*** *(m/s2)*	
0	11,57	3,55	1,72	1873	1,57	0,90	8,69
15	12,05	3,54	2,29	1704	1,62	0,90	8,23
30	12,11	3,49	2,11	1531	1,55	0,90	7,73
45	12,51	3,51	1,93	1364	1,56	0,90	7,80
60	12,33	3,51	1,88	1319	1,33	0,91	7,86
75	12,32	3,54	2,45	1631	1,52	0,93	8,84
90	11,79	3,52	1,64	1827	1,57	0,92	9,68
105	11,72	3,56	1,69	1965	1,66	0,91	9,71
120	11,57	3,54	1,69	1872	1,54	0,90	8,69
max:	12,51	3,56	2,45	1965	1,66	0,93	9,71

Table 1.13.6 Characteristic maximum response parameters for the load case 3b in Table 1.8.1.

Load case 4b:

Orientation *(deg)*	**Excursion** *(m)*	**Heave** *(m)*	**Yaw** *(deg)*	**Line load** *(kN)*	**Nacelle Acceleration**		**Pitch** *(deg)*
					Horizontal *(m/s2)*	***Vertical*** *(m/s2)*	
0	10,11	3,76	2,10	1569	1,45	0,77	9,44
15	10,32	3,78	2,55	1528	1,59	0,77	9,22
30	10,41	3,80	3,05	1375	1,64	0,78	9,13
45	10,50	3,84	3,06	1197	1,55	0,79	8,89
60	10,29	3,86	2,39	1038	1,43	0,80	8,59
75	10,21	3,87	2,27	1159	1,31	0,81	8,61
90	10,02	3,83	2,04	1348	1,33	0,81	9,06
105	10,04	3,79	2,03	1508	1,37	0,78	9,43
120	10,12	3,77	2,29	1570	1,44	0,77	9,45
max:	10,50	3,87	3,06	1570	1,64	0,81	9,45

Table 1.13.7 Characteristic maximum response parameters for the load case 4b in Table 1.8.1.

1.13.2 Structural analysis of brace forces

The structural analysis results presented here are the results for our proposed floater concept. Several structural analyses have been performed in the earlier phases of the conceptual design to find the suitable dimensions for the braces.

A basic comparison of the structural finite element model and the spreadsheet description is given in Table 1.13.8. The deformation of the braces under of the static loads (buoyancy and weight) is shown in Figure 1.13.3.

Maximum axial and bending stresses as a function of orientation is shown in Figure 1.13.4 for lower braces, Figure 1.13.5 for upper braces and Figure 1.13.6 for diagonal braces. The axial and bending stresses are calculated based on maximum axial force and bending moment in the braces, without accounting for location in the brace or wave period. The maximum axial stress and bending stress observed for the lower brace in Figure 1.13.4 will be related to the maximum splitting force.

The program *FRAMEWORK* has, as described above, been used to check Von Mises stresses at the outer circumference of the pipe cross section. The maximum Von Mises stresses and corresponding maximum forces are given in Table 1.13.9. Actual stresses in the tower to column braces should be calculated by a more refined analyses accounting for the effect of the nacelle and rotor. The nacelle and rotor is only included, in the current structural analysis, as a point mass in the correct elevation above the tower.

Property (relative to)	***Spreadsheet model***	***HydroD/WADAM***
Centre of gravity (keel)	9.14 m	9.20 m
Centre of buoyancy keel)	5.09 m	5.09 m
Metacentric height (GM)	3.75 m	3.69 m
Pitch/roll radius of gyration (still water level)	21.90 m	22.23 m
Yaw radius of gyration (still water level)	20.21 m	20.60 m

Table 1.13.8 Basic hydrostatic properties from the spreadsheet and analysis model.

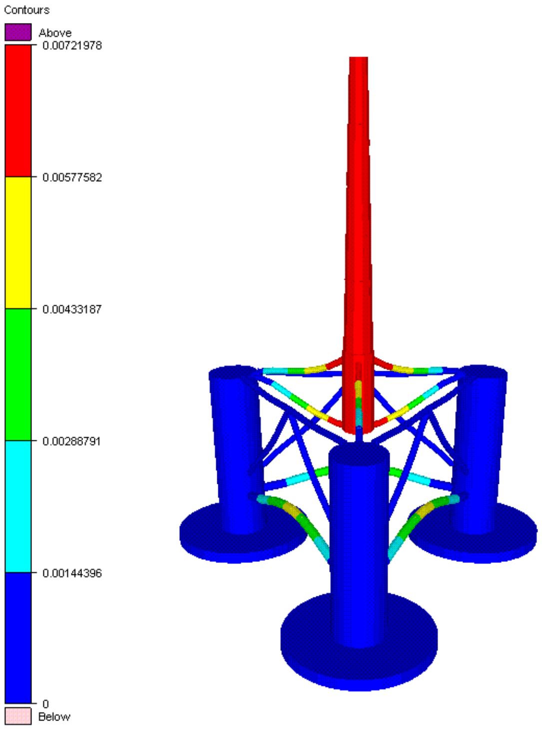


Figure 1.13.3 Deformation of braces under static loads. (Contours show total displacement in metre.)

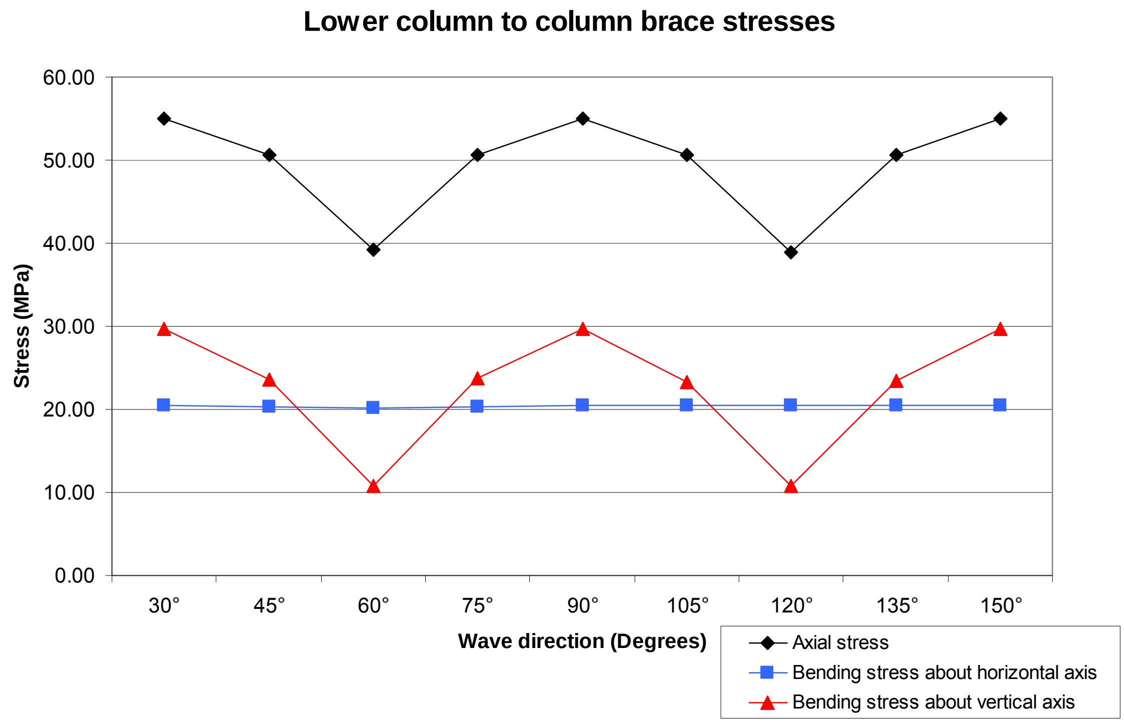


Figure 1.13.4 Stress distributions in lower column to column braces as function of wave direction.

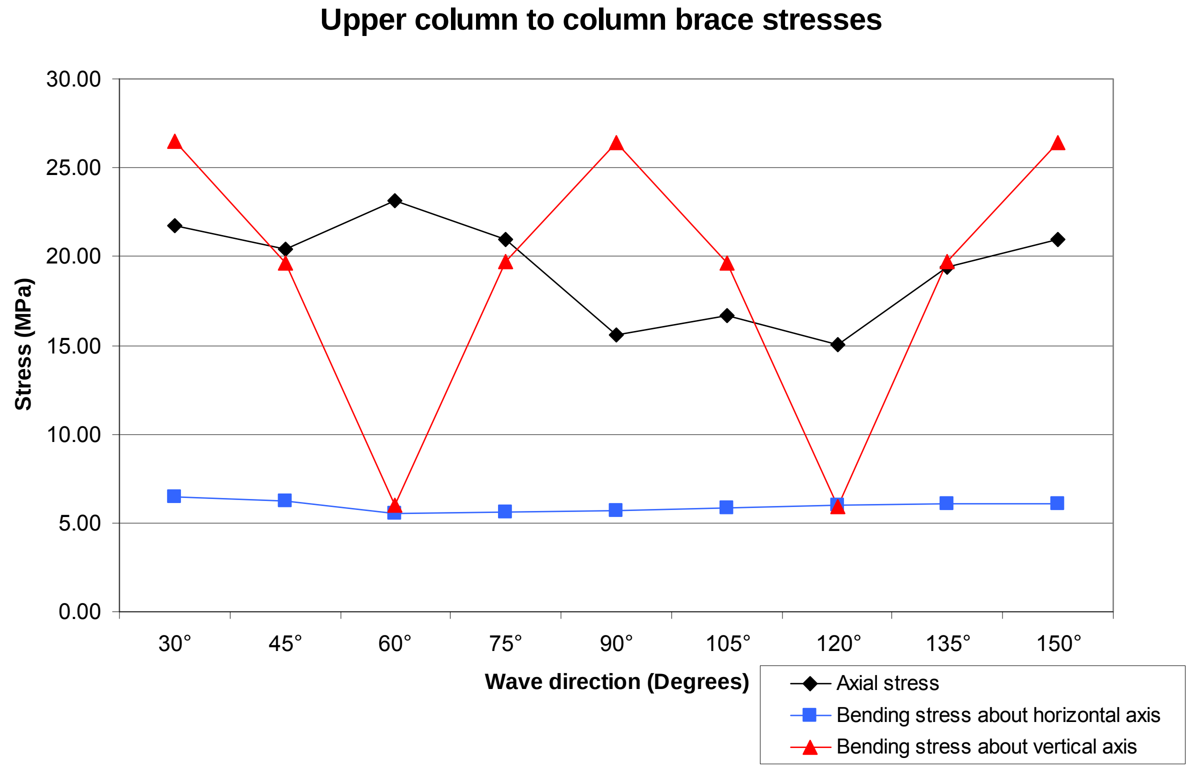


Figure 1.13.5 Stress distributions in upper column to column braces as function of wave direction.

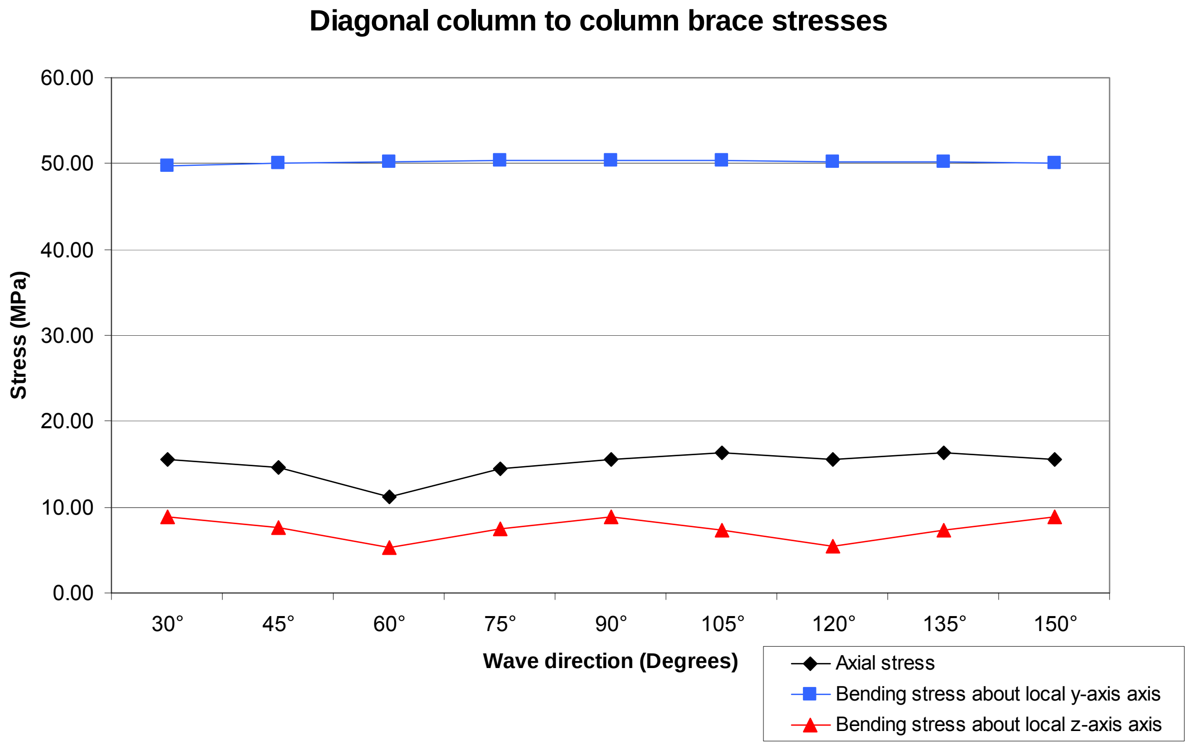


Figure 1.13.6 Stress distributions in diagonal column to column braces as function of wave direction.

	max. Von Mises stresses (MPa)				
	Lower (CC)	*Upper (CC)*	*Diagonal (CC)*	*Radial (TC)*	*Diagonal (TC)*
Static	17.4	18.1	7.78	32.6	52.8
Dynamic	65.5	36	53.8	28.7	29.1

(a)

	Corresponding Normal force (kN)				
	Lower (CC)	*Upper (CC)*	*Diagonal (CC)*	*Radial (TC)*	*Diagonal (TC)*
Static	-19.8	407	-51.6	394	-1420
Dynamic	4410	-838	-168	170	884

(b)

Table 1.13.9 (a) gives the maximum Von Mises stress for the different braces in the static and dynamic conditions. Normal beam forces corresponding to maximum Von Mises stresses are shown in (b). CC and TC indicates column to column and tower to column braces respectively.

1.13.3 Analysis of dynamic damper plate pressure

Maximum dynamic resultant pressures, on the thin rim, are found by combining panel pressure RAOs for corresponding panels in POSTRESP (DNV software). A summary of the pressure resultants found are given in Table 1.13.10. This table also contains the maximum pressures found at the top and bottom of the plate in addition to the relevant wave amplitude calculated in accordance with DNV-RP-C103. Based on the results presented in Table 1.13.10 it seems reasonable to consider a resulting pressure of 25 KPa when dimensioning the outer rim plate.

		Pressure calculated by wadam					
Periode	*Wave amplitude*	*Top of plate*		*Bottom of plate*		*Pressure difference*	
T (s)	a (m)	pd/a (Pa)	pd (Pa)	pd/a (Pa)	pd (Pa)	Δpd/a (Pa)	Δpd (Pa)
6,0	4,0	3913	15710	1974	7925	3025	12145
6,5	4,6	4413	20125	2469	11259	3150	14365
7,0	5,1	4876	24923	2929	14971	3199	16352
7,5	5,7	5313	30091	3369	19081	3196	18101
8,0	6,2	5723	35557	3795	23578	3153	19589
8,5	6,8	6102	41225	4206	28416	3080	20808
9,0	7,3	6445	46984	4598	33519	2984	21753
9,5	7,8	6755	52774	4968	38813	2874	22453
10,0	8,3	7031	58511	5314	44222	2753	22910
10,5	8,8	7277	64157	5635	49681	2627	23161
11,0	9,3	7496	69678	5932	55140	2500	23238
11,5	9,8	7691	75047	6205	60547	2373	23155
12,0	10,2	7866	80258	6457	65882	2249	22947
12,5	10,6	8023	85296	6689	71113	2129	22634
13,0	11,0	8163	90139	6902	76215	2014	22239
13,5	11,4	8290	94808	7097	81164	1905	21786
14,0	11,8	8405	99292	7278	85979	1802	21288
14,5	12,2	8509	103590	7443	90612	1706	20769
15,0	12,5	8604	107711	7596	95092	1615	20218
15,5	12,8	8691	111660	7736	99390	1531	19670
16,0	13,2	8770	115428	7866	103530	1452	19111
16,5	13,5	8843	119038	7986	107502	1378	18550
17,0	13,7	8910	122485	8096	111295	1309	17995
17,5	14,0	8971	125767	8199	114944	1245	17454
18,0	14,3	9028	128911	8294	118430	1186	16935
			128911		118430		23238

Table 1.13.10 Maximum pressure values for given periods. The yellow cells indicate maximum values for the relevant table column.