Design optimisation of an adjustable Pre-Piling Template

for wind-turbine installation

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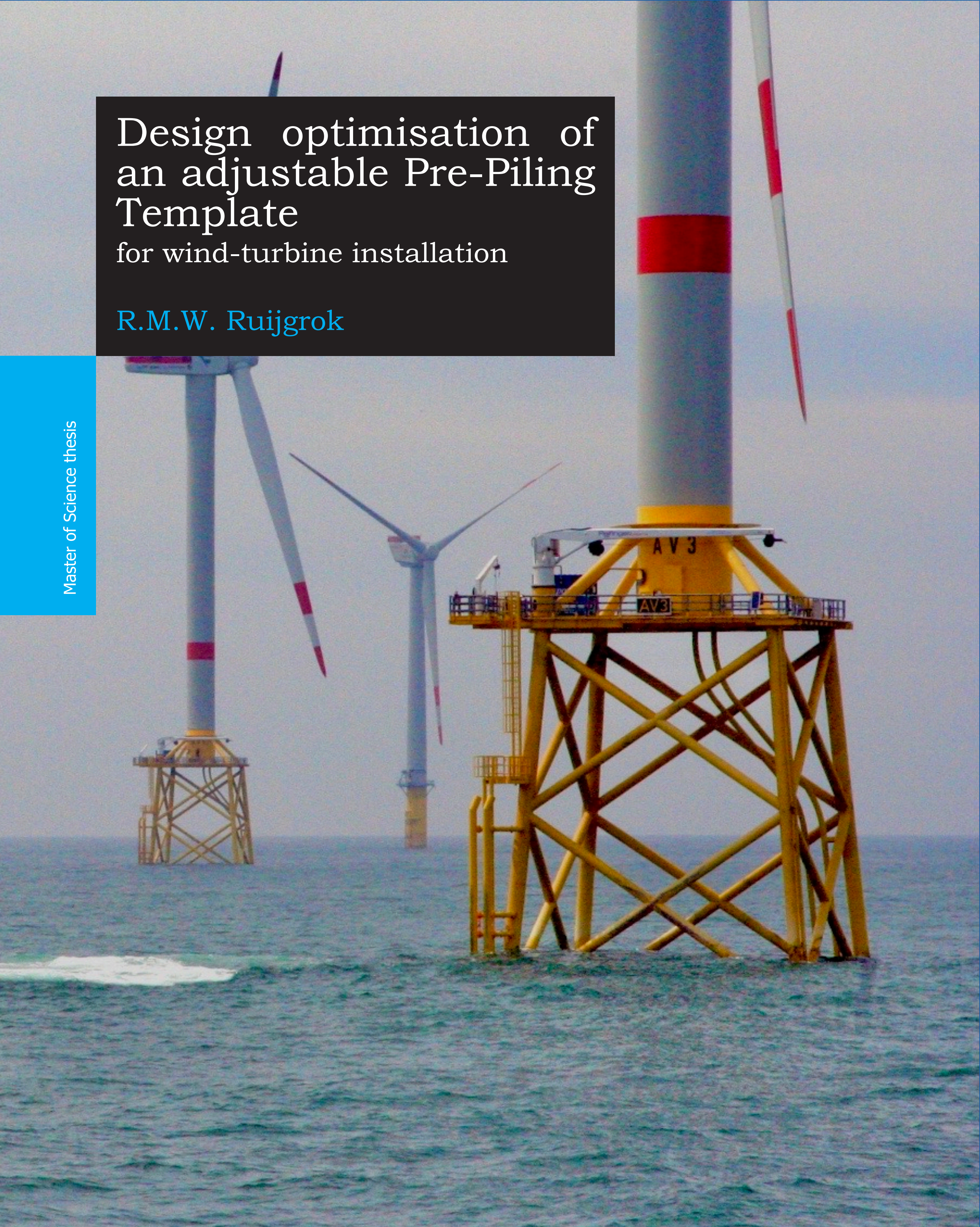
Master of Science thesis



Delft University of Technology

Challenge the future

Design optimisation of an adjustable Pre-Piling-Template for Wind-turbine installation

by

Reinier Ruijgrok

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TU Delft

Abstract

The demand for a substantial increase in renewable energy causes the need for more and bigger wind turbines. To counter the problem of available space, windfarms will move into deeper water. The challenge of deeper water in combination with higher turbines, require new developments in the wind industry. The often used monopiles make way for a new jacket-founded windturbine. Installation of these type of structures opens a market for a so called Pre-Piling Template.
This thesis aims to analyze the adjustability of the Pre-Piling Template for windturbine installation based on quasi-static calculations.

First a number of conceptual designs of a versatile adjustable Pre-Pilling Template are made. A wide variety of configurations is configured. The complicated part of the design is that the Pre-Piling Template must be viable for a three-legged *and* four-legged configurations with several centre-to-centre distances. Thereby, it should be possible to convert the entire system on deck of a vessel during given offshore conditions. From eleven concepts a selection of two alternatives has been made, based on listed criteria by the client: *Robustness, Adjustability, Financial costs and Safety*.

For two selected cross-centre alternatives a global structural analysis is performed under environmental loading. One cross-centre is a composed cross centre, with which a three- and four-legged configuration can be installed with the same cross-centre mid-frame of the PPT. The other alternative consist of two separate mid-frames, one for a three- and one for a four-legged configuration. To speed up the installation process, primarily all the piles to be installed will be stabbed into the Pre-Piling Template. After all piles have been stabbed into the frame, the hammering procedure will start. When all piles are stabbed significant forces arises from wind and especially hydrodynamic actions. The static deformations of the template induced during the multiple installation steps can cause overall displacements of the centre of each particular sleeve.

The added value of a Pre-Piling Template is the installation speed versus the required accuracy of the pile installation. A high installation speed only makes sense if piles can be installed within the required tolerances. Therefore the deformations of the frame and the corresponding displacements are governing. To determine the displacements, a 3D-model is constructed and a rotational and translational spring is implemented to model the soil-structure interaction. To consider this soil-structure interaction, a model by *A.B. Cammaert et al(2011)[1]* is used to determine the required stiffnesses. The model is modelled using *Matrix Frame software*, with which the final displacements, at the height of the mid-frame, have been determined.

A detailed analysis of the static internal forces is worked out based on a bolted flange-flange connection. Checks are done conform *Det Norske Veritas*[7] and based on a ULS-driven design. Two potential connection configurations are worked out; an alternative with less but more heavy bolts of M64, as well as an alternative with substantial more smaller bolts of M36.

Finally, several optimisations are identified to speed up the installation time of assembling and disassembling the adjustable Pre-Piling Template. Recommendations are made in cooperation with *Breman Machinery* and will result, in consultation with installation experts that are well known with the barge of the client, to a final design.

A clear conclusion, about the PPT-design, can not be made because the installation is site specific. If a project includes two different configurations, a three- and four-legged foundation design, a composed mid-frame that is viable for both configurations is recommended. For this composed mid-frame variant the operation to adjust the frame to another footprint can be done more efficient with a higher safety level on deck of the vessel.

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1
Introduction

1.1. Wind-turbine installation

The request for additional renewable energy causes the need for more wind turbines. To prevent an overcrowded coast, the wind industry moves into deeper water. To go into deeper water, the often applied single pile, also known as a monopile wind turbine, is replaced by a new configuration of a jacket structure. The installation of these jacket structures must be executed quickly, as it is done by costly massive vessels. A so called Pre-Piling Template [1] has to increase the installation speed of the foundation piles.

1.2. Pre-Piling Template (PPT)

A PPT provides a preset centre to centre distance of the foundation piles belonging to a particular jacket size. First, the template will be put in the right position, and all the foundation piles will be stabbed into the PPT. Subsequently, hammering of the piles can be started. When all piles are installed, the template can be removed to the deck of the vessel. The PPT is now ready to lay the next foundation of a wind turbine.

Furthermore, wind turbines in the deep water region have a wide span width and therefore need to be placed further apart. Thereby the size of the windfarm increases, causing a high potential of inequality of depth. Due to this range in depth, various sizes of jackets are needed.

The diversity of jacket size, generates the demand for an adjustable design which should be converted on deck of the vessel in offshore operations. Good structural performance and a high degree of workability of the frame and applied connections are required to limit the installation time.

1.3. Problem statement

The challenge of this new future windfarms is to design an adjustable pre-piling template which makes it viable to use several configurations without returning to the port. On top of that, the conversion time must be as short as possible.

1.4. Main objective and research questions

The main objective of this thesis is three-folded;

1. To investigate the feasible configurations of the template structure.
2. To investigate the loads *on* the PPT and *in* the connection of structural elements.

[1] PPT

3. To investigate the operating performance to achieve an efficient assembling and disassembling procedure.

The objectives will be investigated by analyzing the following research questions:

1) How to make a conceptual design of an adjustable Pre-Piling Template? Which can be subdivided into the following sub-questions:

- What are the project requirements of the PPT?
- What are the selection criteria for a versatile adjustable PPT?
- Which viable alternatives are applicable?
- Step by step installation process for positioning of the foundation piles.
- Concept selection based on the pros- and constraints of the alternatives against the selection criteria provided by the client.

2) What are the loads working on and as well as in the connection of the structural elements? Which can be divided into the following sub-questions as well:

- Which external forces will be acting on the PPT?
- Which deterministic supports should be applied to model the PPT in 3D to approach the real loadcase as accurate as possible?
- What is the maximum structural displacement at the mud-line, and at the height of the mid-frame due to quasi-static loading?
- Which internal forces will act in the plane of the connection between the structural elements of the PPT?
- Which design criteria will apply for a demountable connection?
- How to minimize the time-period required for assembling and disassembling during the conversion of the PPT?

The analyses of the above mentioned research questions will result in a basic feasible design for a versatile adjustable PPT. On top of that the study will conclude in some recommendations for further investigation.

1.4.1. Company objective

Breman Machinery intent to offer a PPT for wide variety of projects to the Offshore Wind-industry. The PPT-design should be applicable for a wide range of pile lengths, diameters, soil-conditions and as well as sea-conditions. The biggest challenge for the newly designed PPT is to create a structure which is safe and can be quickly and easily adjusted while on sea. Therefore simplicity of the design is key-item(and worth the price it comes with).

1.4.2. Thesis objective

The internal forces caused by the environmental loading on the PPT should be determined. When these static internal forces are determined, a preliminary design can be made and a recommendation will be formulated for the connection and as well as the conversion method.

2
State of the art

Current templates are worked out with use of a lattice steel structure. These proven designs are not adjustable and in addition not versatile. To create a versatile and adjustable design all the so far existing knowledge should be applied and combined.

A lattice structure can be made versatile, by simply adding the additional functions(like exchangeable mud-mats, levelling-system) to the structure. However these structures are not adjustable for several centre-to-centre distances. *Seaway* recently launched a construction viable for a four-legged foundation with a fixed footprint, but a variable pile diameter up to 2.2m. This design includes a roller system which ensures vertical installation of the foundation piles, with hydraulic cylinders in the sleeve.

Also the mud-mats, forming the basing of the PPT, has been subject to a strong development. By these mud-mats, stability of the PPT on the seafloor can be achieved during installation of the foundation piles. The design of the mud-mats is a study in itself. The suction force between soil, with its specific characteristics, and the PPT-structure is an important design criteria for the mud-mat. So a lot of research has to be done to design an optimal mud-mat configuration for each specific soil conditions.

By retrieving the PPT back to the deck of the vessel, the PPT is strongly sucked to the bottom. To lift the PPT from the sea bottom, specially designed mud-mats equipped with a water-jet system should be applied. For an optimal balance of size of the mud-mats further research should be done, see recommendations **??**.

A design of a lattice structure from *Seaway* is shown in figure 2.1, with a water-jet system in the mud-mats.

Figure 2.1: Alternative pre-piling template design

Three designs of a lattice structure from *IHC IQIP* are shown in figures 2.2, 2.3 and 2.4. Figure 2.2 shows a relatively straight design from 2015, with a flat mud-mat without waterjet-system. This system can only be applied in a situation with a relatively sandy flat sea-bottom.

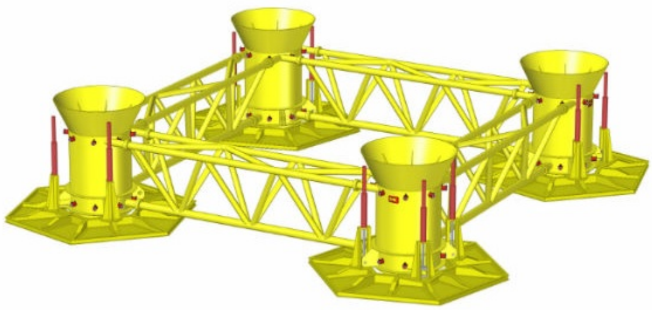

Figure 2.2: Alternative pre-piling template design

The figures 2.3 and 2.4 shows a four- and three-legged PPT with a more developed design including the additional levelling-system and waterjet-systems.

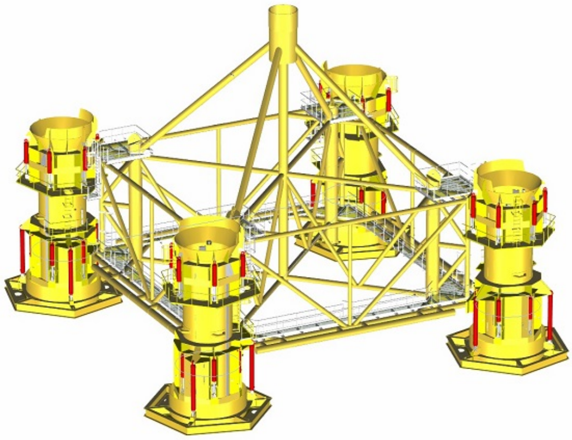

Figure 2.3: Alternative pre-piling template design

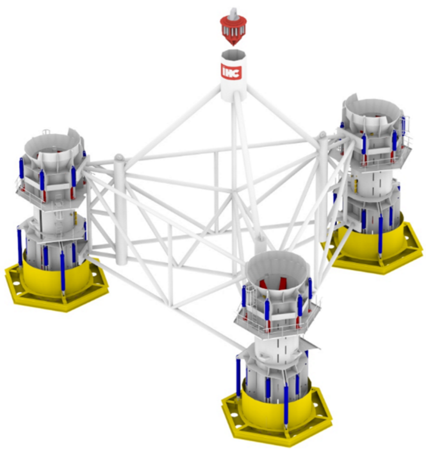

Figure 2.4: Alternative pre-piling template design

Since windturbines will be installed in deeper water areas, the windindustry has to re-design there equipment including versatile adjustable PPT designs. Another additional effect of deeper water regions is a more uneven seabed. This induces the problem of different jacket sizes for a particular windfarm. To create an uniform hub-height of the windfarm, the height of the jacket has to be adjusted to the waterdepth of each location. The different height of the jackets induces different footprint sizes which require an adjustable PPT.

The PPT should be adjustable to install all the foundations piles for a single project. Currently, the foundations for a singular project must have all the same footprint, because the available PPT design has a fixed centre-to-centre distance. This could be a limitation for the installation of newly developed windfarms.

The fixed dimensions of the currently used PPT-structures causing higher costs due to the site specificity. These PPT-structures are not adjustable and thereby not suitable for different jacket foundations. For each foundation dimension a tailor made PPT is required. For the deeper water areas the fixed footprint PPT structures will not be feasible.

The overall size and weight of the PPT structure is strongly limited by the size and lifting capacity of the installation vessel. For this study, the contractor will use the *Bokalift 1* to install the foundations. The size and lifting capacity are listed as design criteria, see section **??**.

Figure 2.5: Bokalift 1

For new upcoming projects an adjustable PPT design is required, viable for a predetermined range of footprint configurations. This design has to meet the following design criteria.

- Structural requirement for the PPT
- Functional requirements for the PPT
- Foundations design requirements for the PPT
- Operational requirements for the PPT
- Technical requirements for the PPT

This requirements will be more specified in the next chapter, chapter **??**.

3

Determination of the loads on the Pre-Piling Template

In the previous chapter two concepts are determined for the design of an adjustable PPT. In this chapter, the focus is put on the loads on both of these concepts. Firstly a review of the wind-loads will be made and subsequently of the hydrodynamic loads. At the end a review has been made and the governing loadcase is determined.

3.1. Wind Loads on the PPT

When the piles are stabbed into the PPT and the pile length is higher than the maximum depth, the piles subject to wind force. The basic wind pressure is defined following the guidelines from chapter 5 in the *DNV-RP-C205[7]*.

$$q = \frac{1}{2}\rho_a U_{T,z}^2 \tag{3.1}$$

To calculate the general windforce on a structural member the following formula can be applied:

$$F_W = CqS \tag{3.2}$$

where:
$U_{T,z}$ = mean wind speed [m/s]
C = shape coefficient [-]
q = basic wind pressure [N/m]
S = projected area of the member normal to the direction of the force [m^2]

To find the shape coefficient, firstly the Reynolds number must be defined, and according to Figure 6-6 in the *DNV-RP-C205[7]* the shape coefficient can be determined. Since the Reynolds number is high, $Re > 10^6$ the dependence of the drag-coefficient on roughness $\Delta = k/D$ must be taken at *0.65*.

$$R_e = \frac{DU_{T,z}}{\nu_a} \tag{3.3}$$

$$\Delta = \frac{k}{D} \tag{3.4}$$

where:
ν = kinematic viscosity of air
k = surface roughness [m]

3.2. Wave Loads on the PPT

In the first instant, the predominant horizontal components of the hydrodynamic actions are considered. Secondly, due to only considering the quasi-static structural response under given circumstances, the time variation due to the passage of waves has not been taken into account. The wave is 'frozen' at the moment that it generates the maximum total horizontal force and/or overturning moment on the structure. To determine this horizontal wave loads acting on the structure the *Morison* load formula is used, chapter 6 from *DNV-RP-C205[7]*. These formula is applicable when the following condition is fulfilled:

$$\lambda > 5D \tag{3.5}$$

The wave length, λ, can be determined with:

$$\lambda = T\sqrt{gd}\left(\frac{f(\omega)}{1+\omega f(\omega)}\right)^{0.5} \tag{3.6}$$

$$f(\omega) = 1 + \sum_{n=1}^{4} \alpha_n \omega^n \tag{3.7}$$

$$\omega = \frac{4\pi^2 d}{gT^2} \tag{3.8}$$

where
λ = wave length [m]
d = depth [m]
and α_1= 0.666, α_2= 0.445, α_3= -0.105, α_4= 0.272

Deep water approximation if:

$$d > 0.5\lambda \tag{3.9}$$

Shallow water approximation if:

$$d < 0.05\lambda \tag{3.10}$$

With the dispersion relation for the finite depth d, equation 3.8, the wavelength can be calculated. The wavelength in relation to the waterdepth, determines if the condition is defined as deep, intermediate or shallow water.

Following the given design requirements, it results in:W

$t_{min} = 4s, d = 50m, \lambda = 24.98m$ gives deep water approximation

$t_{max} = 9s,\ d = 50m,\ \lambda = 125.89m$ gives intermediate water approximation

The above mentioned equations show that the condition in equation 3.5 is valid, and therefore the *Morison* formula is applicable.

The drag component of the hydrodynamic force, is a result of the flow along(tangential) the cylindrical pile. The inertia force component is dependent on the mass coefficient and can be determined by integrating the force acting along the height of the structure. The PPT structure with piles stabbed into it, is modelled with use of a stickmodel following *Handbook of Bottom Founded Offshore Structures Part I and II, by Jan H.Vugts[4] [5]*

For the drag and the mass coefficients, respectively C_D and C_A, the following definitions apply:

$$C_D = \frac{f_{drag}}{\frac{1}{2}\rho D v^2} \tag{3.11}$$

$$C_A = \frac{m_a}{\rho A} \tag{3.12}$$

where
f_{drag} = sectional drag force [N/m]
v = flow velocity [m/s]
m_a = the added mass per unit length [kg/m]

In general the fluid velocity vector will act in a direction relative to the axis of the slender member. The drag force f_{drag} is decomposed in a normal force f_N and a tangential force f_T, see section 6.1 from *DNV-RP-C205[7]*. Because it is assumed that the structure is fixed during installation, the sectional force in a two-dimensional flow normal to the member axis is given by:

$$f_N(t) = \rho(1 + C_A)A\dot{v} + \frac{1}{2}\rho C_d D v\,|v| \tag{3.13}$$

where
v = fluid particle(waves and/or current) velocity [m/s]
$\dot{v}$ = fluid particle acceleration [m/s^2]
A = cross sectional area [m^2]
D = diameter or typical cross-sectional dimension [m]
ρ = mass density of fluid [kg/m^3]
C_A = added mass coefficient [-]
C_D = drag coefficient [-]

To use the Morison's load formula to determine the hydrodynamic loads on a structure the variation of C_D and C_A as a function of Reynolds number, the Keulegan-Carpenter number and the roughness should be taken into account. The parameters of these numbers are generally defined by:

$$R_e = \frac{vD}{\nu} \tag{3.14}$$

$$K_C = \frac{v_m T}{D} \tag{3.15}$$

where:
T = wave period [s]
k = roughness height [m]
v = total flow velocity [m/s]
ν = fluid kinematic viscosity [m^2/s]
v_m = maximum orbital particle velocity [m/s]

Since there is a harmonic flow, assuming $Airy$ potential wave theory[6] for shallow water regimes, the Keulegan-Carpenter number can also be written following section 6.6.1.5 from *DNV-RP-C205[7]*:

$$K_C = \frac{\pi H}{D} \tag{3.16}$$

where:
H = wave height [m]

Marine growth may be neglected since the pipes are newly fabricated.

The Reynolds number is high ($R_e > 10^6$), thus the dependence of the drag-coefficient must be taken according section 6.7.1.5 from the *DNV-RP-C205[7]*.

The effect of the Keulegan Carpenter number *KC* on the variation of the drag coefficient can be approximated according:

$$C_D = C_{DS}(\Delta)\psi(KC) \tag{3.17}$$

where:
ψ*(KC)* = wake amplification factor

For low Keulegan-Carpenter numbers *KC < 12* the wake amplification factor can be taken as:

$$\psi(KC) = C_\pi + 0.10(K_c - 12) \qquad 2 \le K_C < 12 \tag{3.18}$$

where:
C_π = 1.50-0.024(12/C_{DS}-10)

Due to finite length of the pile, the reduction factor must be determined to make an estimate of the total drag force on a slender member with a characteristic cross-sectional dimension *d* and finite length *l*. According to Table 6.2 from *DNV-RP-C205[7]*, the reduction factor κ is 0,9.

For the added mass coefficients C_A, the effect of the KC-number and roughness may be taken into account. Since KC<3, the added mass coefficient can be found from formula 3.19, from section 6.9.1.2 *DNV-RP-C205[7]*

$$C_A = max\begin{cases} 1.0 - 0.044(K_C - 3) \\ 0.6 - (C_D S - 0.65) \end{cases} \tag{3.19}$$

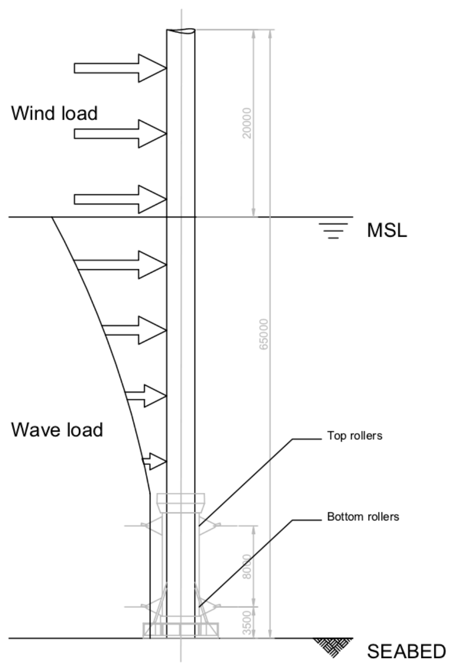

Figure 3.1: Wind and Wave load overview

When C_D and C_A are known, with the Morison's equation formula 3.13 the force per section can be determined. In this case, when considering only a single pile, the pile is divided into sections with $dz = 1m$. The velocity and acceleration can be determined belonging to the depth of each section which result in the corresponding inertia and drag force.

Using numerical integration over the total depth the $Simpson's - rule$ is used. Since the hydrodynamic profile is decreasing exponentially, the approximation to describe the profile with a second order polynomial is sufficiently accurate. With an upper-, centre- and lower boundary of each stick range the hydrodynamic force and the resulting moment can be determined with formula 3.20 and 3.21 respectively[1].

[1]Assumption made that each section does not influence the adjacent sections' flow

$$F_{Hydrodynamic} = \int_u^l f(z)dz = \frac{1}{6}((f(u) + 4f(c) + f(l)) \tag{3.20}$$

$$M_{Hydrodynamic} = \int_u^l f(z)dz = \frac{1}{6}(z_u(f(u) + 4z_c f(c) + z_l f(l)) \tag{3.21}$$

The maximum base shear needs to be determined as well. Since there is always a 90 degrees phase shift between the drag and inertia force, the two force vectors are perpendicular, and therefore the combined unit force can be obtained as follows:

$$f_{total} = \sqrt{f_d^2 + f_i^2} \tag{3.22}$$

The wave-direction influence the load on the structure. So the frame orientation relative to the wave direction is of importance. By use of a stickmodel the associated diameter of the structure is calculated based on the orientation of the structure. Orientation of the frame against the wave direction determines the corresponding diameter of each section for each particular height.

Furthermore the associated wave phase angle influences the total force, since inertia and drag are out of phase.

$$f(\varphi) = f_d(\varphi) + f_i(\varphi) = \frac{1}{2}\rho C_d A\,(v_w cos(\varphi) + v_c)\left|v_w cos(\varphi) + v_c\right| + \rho C_m V \dot{v} sin(\varphi) \tag{3.23}$$

where:
φ = wave phase angle

Normally, for relatively slender offshore structures, the drag is dominated over inertia. However it depends on the characteristic diameter of the structure, the wave height and the wavelength.

The stabbed piles can be considered as a small volume structure since the characteristic dimension D is smaller than the typical wave length.

$$D < \lambda/5 \tag{3.24}$$

For this structure, with a characteristic diameter of *d*=3m the inertia component is governing. The highest load forces arise with a wave phase angle of 89*degrees* (1.55rad), and a period of t_{max}=9*s* following section **??**, see figure 3.2. The drag force consists only of the current component, since the cosinus term is almost zero. For a schematization of the drag and inertia forces following the stickmodel, see figure **??**. The horizontal lines in the graph are not completely horizontal due to the section-height.

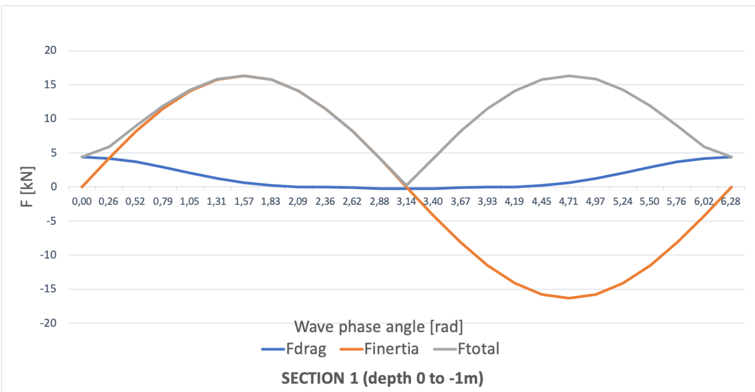

Figure 3.2: Wave phase angle

From figure 3.3, the structure can be classified under regime III. This means that the inertia force is large and the drag force is small. It depends on the characteristic dimension of the structure, wave-height and wave-length.

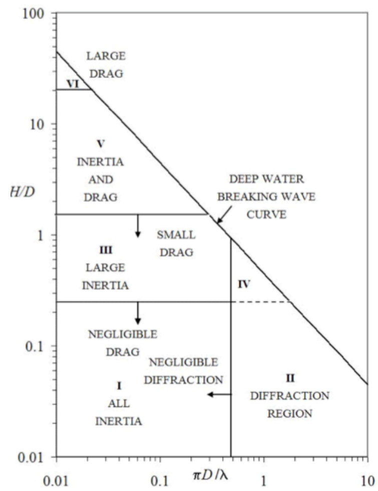

Figure 3.3: Different wave force regimes
D = characteristic dimension, H = waveheight, λ = wave length

Considering the situation of one single pile is stabbed into the frame, and belonging frame-section, the loads on the pile are determined using the stickmodel. The maximum force due to wind and wave at the situation where one pile is stabbed into the frame is shown in table 3.1. In the following chapter the total loads on the PPT will be considered depending on the orientation of the frame against the wave direction.

	Value	Unit
Waterdepth	50	m
Pile length	65	m
Diameter pile	3.0	m
Load on pile		
Water force on pile	2.6	kN
Wind force on pile	321.8	kN
Moment pile wind	8.58E+04	Nm
Moment pile water	2.16E+07	Nm
Moment pile total	2.16E+07	Nm

Table 3.1: Load on pile

3.2.1. Partial-load factors

In the calculations the general LRFD-method is used with the loadfactors shown in table 3.2, according to *chapter 4.4.1 from DNV-GL-C101[3]*.

Combination of design loads	G	Q	E	D
a)	1.3	1.3	0.7	1.0
b)	1.0	1.0	1.3	1.0

Table 3.2: Load factors γ_f for ULS

4

Global structural analysis

With use of the calculation software *Matrix frame*, the global structural analysis are worked out. *Matrix frame* is a flexible software package to calculate several loadcases in a 3D-steel framework by use of a finite element method, to determine the internal forces. First of all, the supports, knot-connections and constraints of the model are defined, to get a realistic simulation and reliable results. Subsequently the deformations of the PPT are checked within the given tolerances, after which the internal forces can be analysed in more detail.

> Model 3 and 4: Empirical models, with a hinge and clamped support respectively. These models are based on the effective depth and on the fixity depth, see *Bottom Founded Offshore Structures handbook*[5]. Commonly known, is that these models give results with a 5percent tolerance with respect to a full finite element model for fully installed piles

4.0.1. Empirical Model 3 and 4

Model 3 and 4 contain the supports like a hinge and clamped constraint at respectively the effective depth (d_e) and the effective fixity depth $2d_e$. Depending on the pile diameters and size of the structure the ratio of d_e should be determined. In table **??** the empirical values of these simplified models are shown. The fixity depth(point upon which the moment equals zero) for a jacket structure is lower than with a monopile. These values for jacket structures are based on a pile-trough-leg configuration, and diameters of the foundation piles up to 2.5m.
The foundation piles installed with the PPT, are up to 3m. Therefore both empirical models are checked with different values for the fixity depth. The values for d_e are empirically proven and the results are representing the reality within a range of 5 percent.

4.1. Overview of resulting displacements

In table 4.1 the displacements of the discussed models are shown. For model 1 and 2 the calculated displacement is based on the determined K_{rot} and K_{trans} depending on the soil layer.

Model	Configuration	Loadcase [kN/m]	Ktrans,sand,tot [kNm/rad]	Krot,sand,tot [kN/m]	Ktrans,clay,tot [kN/m] [kNm/rad]	Krot,clay,tot [m]	dx,mudline [m]	dx,midframe [m]	phi [rad]
1	4 piles stabbed	FC4	4.55E+09	7.99E+10	-	-	0	0.0279	0,00310
1	3 piles stabbed	FC4	4.55E+09	7.99E+10	-	-	0	0.0294	0,00327
1	1 pile soft clay	FC4	4.55E+09	7.99E+10	1.15E+06	3.70E+07	0	0.0284	0,00316
1	2 piles soft clay	FC4	4.55E+09	7.99E+10	1.15E+06	3.70E+07	0	0.0285	0,00317
2	4 piles stabbed	FC4	8.05E+09	2.59E+11	-	-	0	0.0232	0,00290
2	3 piles stabbed	FC4	8.05E+09	2.59E+11	-	-	0	0.0234	0,00291
2	1pile in soft clay	FC4	8.05E+09	2.59E+11	1.15E+06	3.70E+07	0.0013	0.0226	0,000288
2	2piles in soft clay	FC4	8.05E+09	2.59E+11	1.15E+06	3.70E+07	0.0013	0.0235	0,000292

Table 4.1: Maximum displacements

4.2. Conclusion

There are different steps in modelling the frame, to investigate the static deformation due to static environmental loading:

Following table 4.1 the highest displacements during installation of the piles with model 1 and 2, will act in the situation when 3 piles are stabbed in a four-legged configuration. The displacement during installation becomes $0,0294m$ which is acceptable and within the limit.

5

Detailed analysis of internal forces and connecting of adjustable elements

After analysing the structural global analysis there is looked into the detailed analysis. First the design criteria of a demountable connection are examined, and different concepts have been reviewed. For a simple flange-flange connection all corresponding checks are done, conform the guidelines listed in section **??**. To optimize this flange-flange connection various improvements are described, and a recommendation is given for a site-specific project.

In case of a bolted connection, a consideration should be made between the shear-force and the amount of pre-tension in the bolts. For a static ULS-based calculation the shear-force could be absorbed by the bolts, causing zero required pre-tension. Torsion forces between the frame elements are converted to shear forces in the bolts, therefore the shear resistance of the bolts should be checked. In this approach pre-tension is not required in a connection.

However, due to the vibrations of the dynamic parts of the loading, the pre-tension should be determined following a dynamic ULS-based and a FLS-based calculation (see section **??**) since fatigue is not checked in this thesis. Pre-tension, causing an extra slip resistance between the flanges results of a higher shear resistance of the connection.

Limiting the number of bolts is only possible if disassembling and assembling remains workable. Most activities involved should be carried out by human work force, so the total weight of bolts should be significantly under the legal maximum weight that may be lifted. On top of that, the deck of the installation vessel should be a safe working environment. For example when a large bolt falls down from an scaffold truck, serious damage may occur. Installation time of the bolted connection should therefore be not too long, to avoid unnecessary downtime of underlying installation areas.

A fast-fix system together with a fast alignment procedure should provide a short period of crane occupancy for each connection. If the elements can hold in place safely, the crane can be detached and be used for the next connection. With the use of a fast-fix system in combination with the effective aligners, the hanging element will be clamped and hold in place to the fixed structure due to its self-weight, and single support of a cradle at the other end of the element. There are several alternatives viable to create a fast-fix system, see section 5.1.5.

Thereby, an important design criteria is to create a connection where the separate elements behave like a continuous beam. A stiff connection protects the connection against damaged by vibrating of individual elements. The required pre-tension could be achieved by creating an extern moment, causing a large internal force F, see figure 5.1. The force F should be significantly higher than the tension force developed by M_{Ed}. Due to this kind of pre-tension the different individual members behave as one rigid continuous system, which is favorable against fatigue.

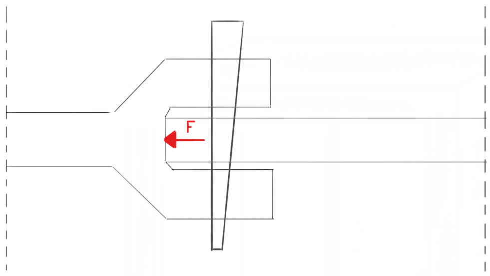

Figure 5.1: principle connection

5.1. Flange connection

Following the National Standard EN 1993-1-8[2] a 'simple' flange connection is checked for the horizontal members of the PPT. To design a bolted connection the factored design strength should be higher then the factored load. The design assumptions which are considered according *section 2.5* of the Standard[2]:

1. The internal forces and moments assumed in the analysis are in equilibrium with the forces and moments applied to the joints,
2. The assumed distribution of internal forces shall be realistic with regard to relative stiffnesses within the joint
3. The deformations implied the distribution of forces do not exceed the deformation capacity of the fasteners or welds of the connected part,

The bolts which are selected are pre-loaded of bolt class 10.9 with the requirements belonging to Group 7 from 1.2.7[2]. In this thesis the quasi-static situation is checked, so bolted connections subject to shear should be designed following Category C: *Slip-resistant at Ultimate Limit State*. Concerning the connection subject to tension it should be designed following Category E: *Preloaded*. The associated design checks for Category C: 5.1, 5.2, 5.3 and for Category E: 5.4, 5.5 are:

$$F_{v,Ed} \leq F_{s,Rd} \tag{5.1}$$

$$F_{v,Ed} \leq F_{b,Rd} \tag{5.2}$$

$$\sum F_{v,Ed} \leq \sum N_{net,Rd} \tag{5.3}$$

$$F_{t,Ed} \leq F_{t,Rd} \tag{5.4}$$

$$F_{t,Ed} \leq B_{p,Rd} \tag{5.5}$$

where:
$F_{v,Ed}$ = design shear force per bolt for the ultimate limit state
$F_{t,Ed}$ = design tensile force per bolt for the ultimate limit state
$F_{s,Rd}$ = design slip resistance per bolt
$F_{b,Rd}$ = design bearing resistance per bolt
$N_{net,Rd}$ = design axial force per bolt
$F_{t,Rd}$ = design tension resistance per bolt

The corresponding failure modes can be divided into;

1. Shear failure of the bolts
2. Failure of member being connected due to fracture shear (tension failure)

3. Plate fracture of the flange
4. Fracture of the connected plate between two bolt holes

To resist failure of the flange connection, the shear resistance, bearing resistance, tension resistance and the combined shear and tension resistance should be checked. For each mode a unity check must be done whether the factored design strength is higher than the factored load in each particular plane. Since there are different loading directions possible, a high range of loadcases could occur. This research only considers the highest loaded members, with respect to the calculated loadcase. For these components all the loads have been determined at the plane of the connection, using *Matrix Frame*. The governing situation for internal forces occurs when four piles are stabbed into the PPT, see figure **??**.

5.1.1. Shear resistance

In a bolted shear connection the bolts are subject to shear and the connecting plates are subject to bearing stresses. The shear resistance can be defined using equation 5.6.

$$F_{v,Rd} = \frac{\alpha_v f_{ub} A}{\gamma_{M2}} \tag{5.6}$$

For the design shear, the amount of bolts determines the total design shear strength. The shear force can be divided into a direct shear and a rotational shear. Due to in plane bending of the frame, caused by a torsion-moment, the rotational shear arises, see figure **??**. The rotation point is in the middle of the tubular member.

With the equations 5.7 to 5.10, the shear force in the bolts should can be calculated.

$$F_{V,y} = \frac{M_x z}{\sum\left(y_i^2 + z_i^2\right)} \tag{5.7}$$

$$F_{V,z} = \frac{M_x y}{\sum\left(y_i^2 + z_i^2\right)} \tag{5.8}$$

$$F_{V,Ed,1} = \sqrt{F_{V,z}^2 + +F_{V,y}^2} \tag{5.9}$$

$$F_{V,Ed,2} = \sqrt{\left(F_{V,y} + \frac{V_y}{bolts}\right)^2 + \left(F_{V,z} + \frac{V_z}{bolts}\right)^2} \tag{5.10}$$

5.1.2. Bearing resistance

For this design the holes are oversized to speed up the conversion time, to achieve a relatively quick alignment. Hereby the bearing resistance reduces to 0.8 times the bearing resistance of normal fitted holes.

$$F_{b,Rd} = \frac{k_1 \alpha_b f_u d t}{\gamma_{M2}} \tag{5.11}$$

5.1.3. Tension resistance

The maximum tension in a bolt is caused by multiple moments. Due to these moments, a tension force is developed to capture these moments. The tension resistance can be determined with the use of equation 5.12.

$$F_{t,Rd} = \frac{k_2 f_{ub} A_s}{\gamma_{M2}} \tag{5.12}$$

For the design tension strength, the amount and positioning of the bolts influence the total tension force for each particular bolt. The bolts should be designed within the elastic region, therefore a linear profile over the height of the flange is used to determine the corresponding tension force for each bolt row. The out of plane moments cause the required tension forces, see figure **??**.

Using equation 5.13 the tension forces due to the out of plane bending moments can be calculated.

$$F_{T,Ed} = \frac{M_y z_i}{\sum z_i^2} + \frac{M_z y_i}{\sum y_i^2} \tag{5.13}$$

5.1.4. Unity checks

Two different configurations of the flange-flange connection are checked. Firstly, a configuration with 24 $M64$ bolts in one single row outside the pipe is checked. Subsequently, a configuration with two bolt rows and 200 $M36$ bolts with one bolt row inside and one outside the pipe is checked, see figure **??**.

In table 5.1 and 5.2 the unity checks are shown regarding configuration 1 and 2 respectively. The governing loadcase for maximum internal static forces due to environmental loading is also shown in figure **??**. For this situation the in plane forces of at the connection has been checked for member IF and member IG, for a member overview see also figure **??**.

REMARK: These checks are done based on a quasi-static ULS driven design. A check should be done regarding the eigenfrequency of the structure against the dynamic part of the loads. Furthermore a fatigue assessment due to dynamic stress ranges of the waves and wind, and the fatigue stresses due to hammering should be investigated.

Checks	Member IF		Member IG	
	Point 1	Point 2	Point 1	Point 2
$F_{t,Ed}/F_{t,Rd}$	0.36	0.40	0.17	0.05
$F_{v,Ed}/F_{v,Rd}$	0.31	0.55	0.11	0,22
$F_{v,Ed}/F_{b,Rd}$	0.07	0.13	0.03	0.03
$F_{v,Ed}/F_{S,Rd}$	0.40	0.75	0.11	0.11
$Combined\ shear\ and\ tension$	0.56	0.84	0.23	0.15

Table 5.1: Amount of bolts: 24 times M64

Checks	Member IF		Member IG	
	Point 1	Point 2	Point 1	Point 2
$F_{t,Ed}/F_{t,Rd}$	0.40	0.44	0.18	0.05
$F_{v,Ed}/F_{v,Rd}$	0.57	0.57	0.08	0.08
$F_{v,Ed}/F_{b,Rd}$	0.08	0.08	0.01	0.01
$F_{v,Ed}/F_{S,Rd}$	0.89	0.94	0.36	0.34
$Combined\ shear\ and\ tension$	0.85	0.88	0.21	0.12

Table 5.2: Amount of bolts: 200 times M36

5.1.5. Flange check

For the design of the flanges the checks should comply with the Eurocode [2]. There are three failure modes regarding flange failure:

Mode 1: Complete yielding of the flange

Mode 2: Bolt failure with yielding of the flange

Mode 3: Bolt failure without yielding of the flange

The tension resistance of the flange belonging to failure mode 1 to 3 can be determined using equation 5.14 to 5.17 respectively.

$$F_{T,1,Rd} = \frac{4\ M_{pl,1,Rd}}{m} \tag{5.14}$$

$$F_{T,1,Rd} = \frac{(8n - 2e_w)M_{pl,1,Rd}}{2mn - e_w(m+n)} \tag{5.15}$$

$$F_{T,2,Rd} = \frac{M_{pl,2,Rd} \ + n\sum F_{t,Rd}}{m+n} \tag{5.16}$$

$$F_{T,3,Rd} = \sum F_{t,Rd} \tag{5.17}$$

where:
$M_{pl,1,Rd}$ = design moment resistance of the flange for mode 1
$M_{pl,2,Rd}$ = design moment resistance of the flange for mode 2
$F_{t,Rd}$ = design tension resistance of the flange

Deflection of the flanges is not acceptable since structure must be connected and disconnected multiple times. Deformation of the flanges cause a problem during conversion. By over-designing of the flange, the elastic deformations of the flange would be minimized and the tension resistance of mode 3 is governing.

The highest strength of the connection corresponding with the chosen bolts is given by failure mode 3. A thick flange, with high strength, causing a governing bolt strength. The flange with corresponding bolts is designed considering all three failure modes. The positioning of the bolts (the effective length), type of bolts and thickness of the plate is selected in such a way that mode 3 will be governing. Table 5.3 shows an overview of the flange characteristics. In table 5.4 and 5.5 an overview of the unity checks is shown for the above mentioned configurations.

	Symbol	Value	Unit
$thickness$	t_f	120	[mm]
$yieldstrength$	f_y	690	[N/mm^2]

Table 5.3: Flange dimensions

REMARK: These checks are done based on a quasi-static ULS driven design. A check should be done regarding the eigenfrequency of the structure against the dynamic part of the loads. Furthermore a fatigue assessment due to dynamic stress ranges of the waves and wind, and the fatigue stresses due to hammering should be investigated.

Checks	Member IF		Member IG	
	Point 1	Point 2	Point 1	Point 2
$F_{t,tot}/F_{T1,Rd}$	0.07	0.08	0.03	0.01
$F_{t,tot}/F_{T2,Rd}$	0.11	0.12	0.05	0.01
$F_{t,tot}/F_{T3,Rd}$	0.10	0.11	0.05	0.01

Table 5.4: Flange checks assuming 24 bolts of M64

Checks	Member IF		Member IG	
	Point 1	Point 2	Point 1	Point 2
$F_{t,tot}/F_{T1,Rd}$	0.04	0.05	0.02	0.01
$F_{t,tot}/F_{T2,Rd}$	0.06	0.06	0.03	0.01
$F_{t,tot}/F_{T3,Rd}$	0.20	0.22	0.09	0.03

Table 5.5: Flange checks assuming 200 bolts of M36

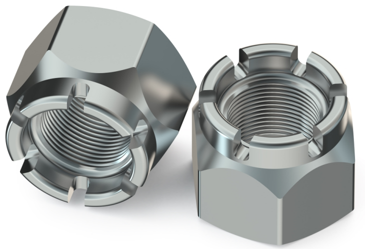

Figure 5.2: Castellated nuts

Due to the variety of configurations consisting of several elements the assembling and disassembling time should be reduced. A final design of these procedures should be made in closed consultation with the installation expert.

Fast-fix system

6
Conclusion

Design assumptions(section 2.5 EN1993-1-2005):

1. the internal forces and moments assumed in the analysis are in equilibrium with the forces and moment applied to the joints
2. each element in the joint is capable of resisting the internal forces and moments
3. the deformations implied by this distribution do not exceed the deformation
4. the assumed distribution of internal forces shall be realistic with regard to relative stiffnesses within the joint
5.

Joints loaded in shear subject to impact vibration and/or load reversal(section 2.6 EN1993-1-2005): where a joint loaded in shear is subject to impact or significant vibration one of the following jointing methods should be used.

1. welding
2. bolt with locking devices
3. preloaded bolts
4. injection bolts
5. other types of bolts which effectively prevent movement of the connected parts
6. rivets

6.1. Eccentricity

When there is eccentricity at intersections, the joints and members should be designed for the resulting moments and forces, except in the case of particular types of structures where it has been demonstrated that it is not necessary.

Bolted Connections (section 3.4) shear connections(3.4.1
Category A: No preloading and special provisions for contact surfaces are required. The ultimate shear load should not exceed the design shear resistance obtained from 3.6, nor the design bearing resistance obtained from 3.6 and 7

Category B: slip should not occur at the serviceability limit state. The design serviceability shear load should not exceed the design slip resistance.

Category C: In this category preloaded bolts in accordance with 3.1.2 should be used. slip should not occur at the ultimate limit state. The design ultimate shear load should not exceed the design slip resistance, obtained from 3.9 nor the design bearing resistacne, obtained from 3.6 and 3.7 In addition for a connection in tension the design plastic resistance of the net cross-section at bolt holes should be checked. (6.2 EN1993-1-1)

Tension conditions (section 3.4.2) Category D: No preloading is required, this category should not be used whre the connections are frequently subjected to variations of tensile loading. However, they may be used in connections designed to resist normal wind loads.

Category E: in this category preloaded 8.8 and 10.9 bolts with controlled tightening in conformity with

6.2. Welded connections

- welds subject tot fatigue shall also satisfy the principles given in EN 1993-1-9
- fillet welds may be used for connecting parts where the fusion faces form an angle of between 60 and 120
- angles smaller than 60 are also permitted. however in such cases the weld should be considered to be a partial penetration butt weld.
- fillet welds all round

6.2.1. Design resistance of a fillet weld

- the effective length of a fillet weld leff should be taken as the length which the fillet is full sisze. this mabye taken as the overall length of the weld reduced by twice the effective throat thickness a
- a fillet weld with an effective length less than 30 mm or less than 6times its throat thickness whichever is larger, should not be designed to carry load.

6.2.2. Classisfication of joints

- Joints may be classified by their stiffness (see 5.2.2) and by their strength (see 5.2.3)

Modelling of beam-to-colum joints (section 5.3)

- to model the deformational behavirou of a join, account should be taken of the shear deformation of the web panel and the rotaional deformation of the connections.
- joint configurations should be designed to resist the internal bending moments Mb1,ed and Mb2,ed, normal forces Nb1,Ed and Nb2,Ed and shear forces Vb1,Ed and Vb2,Ed applied tot het joint by the connected members.
- the resulting shear force V in the web panel should be obtained using equation 5.3
- see figure 5.6 and 5.7
- connection between sleeve and horizontal frame model like a single-sided joint configuration

6.3. Structural Properties Joint

- joint may be represented by a rotational spring connecting the centre lines of the connected members at the point of intersection, as indicated in figure 6.1(a) and (b) for a single sided beam to column joint configuration. The properties of the spring can be expressed in the form of a design moment-rotation characteristic that describes the relationship between the bending moment Mj,Ed applied to a joint and the corresponding rotation phi,Ed between the connected members. Generally the design moment-rotation characteristicis non-linear as indicated in Figure 6.1(c)

- a design moment-rotation characteristic see figure 6.1(c) in EN1990-1-8 should define the following three main structural properties: moment resistance, rotational siffness and rotation capacity.
- Design moment resistance the design moment resistance Mj,Rd which is equal to the maximum moment of the design moment-rotation characteristic, see Figure 6.1(c), should be taken as that given by 6.1.3(4) see also 6.2.7 and 6.2.8
- Rotational stiffness section 6.1.2.3 see 6.3.1
- Rotation capacityY 6.1.2.4 see 6.4
- Reinforcement: given in section 6.2.4.3 and 6.2.6

6.3.1. Design Resistance (section 6.2)

- Internal forces
- Shear forces
  In welded connections, and in bolted connections witch end-plates, the welds connecting the beam web should be designed to transfer the shear force from the connected beam to the joint, without any assistance from the welds connecting the beam flanges.
  In bolted connections with end-plates the design resistance of each bolt-row to combined the shear and tension should be verified using the criterion given in table 3.4, taking into account the total tensile force in the bolt including any force due to prying action
  BOLTED CONNECTIONS WITH ANGLE FLANGE CLEATS (SECTION 6.2.2.3)
- Bending moments

6.3.2. 6.2.7.2 Beam-to-column joints with bolted end plate connections

- for bolted end plate connections the centre of compression should be assumed to be in line with the centre of the compression flange of the connected member.
- the effective design tension resistance Ftr,Rd for each bolt-row should be determined in sequence, starting from bol-row 1, the bolt-row farthest from the centre of compression
- when determining the effective design tension resistance Ftr,Rd for bolt-row r the effective design tension resistance of all other bolt-rows closer to the centre of compression should be ignored
- The effective design tension resistance Ftr,Rd of bolt-row r should be taken as its design tension resistance Ft,Rd as an individual bolt-row determined from 6.2.7.2(6) reduced if necessary to satisfy the conditions specified in 6.2.7.2(7) (8) and (9)
- The method described in 6.2.7.2(1) to 6.2.7.2(9) may be applied to a bolted beam splice with welded end plates.

6.3.3. Rotational stiffness (section 6.3)

- for a bolted end-plate joint with more tan one row of bolts in tension, the stiffness coefficients ki for the related basic components should be combined. for beam to column joints an beam splices a method is given in 6.3.3

- in a bolted end plate joint with more than one bolt-row in tension, as a simplification the contribution of any bolt-row may be neglected, provided that the contributions of all other bolt rows closer to the centre of compression are also neglected. the number of bolt-rows retained need not necessarily be te same as for the determination of the design moment resistance.
- Provided that the axial force NEd in the connected member does not exceed 5 percent of the design resistance Np,Rd of its cross-section the rotaional stiffness Sj of a beam to columnm joint or beam splice, for a moment Mj,Ed less than the design moment resistance Mj,Rd of the joint may be obtained with sufficient accuracy from equation 6.27

Bolt Preload and static loading VIDEO LECTURE YOUTUBE
Torque range on pretension bolts, to determine the amount of preloading on the bolt.

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