Deliverable D4.36

Design Solution for a Support Structure Concept for future 20MW

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1 INTRODUCTION

The Deliverable D4.36 defines requirements and shows the challenges for designing future 20MW wind turbines and illustrates a possible design solution for such very large turbines. A 20MW wind turbine model and loads are required as a prerequisite for this design task. An extrapolated wind turbine and tower model is developed in INNWIND.EU Work Package 1 and basic aeroelastic load calculations are performed. At this stage a so called land-version of the wind turbine is applicable [5]. Consequently, the jacket design is carried out in a preliminary level using static extreme loads and simplified load cases for the fatigue analysis that are based on non-correlated wind and wave fatigue calculations. More sophisticated conceptual design studies require a more accurate offshore wind turbine model and integrated load simulation under due consideration of loads and responses, for which the preliminary jacket is a requirement. Results of the more accurate conceptual analysis are compared with the results of the preliminary analysis of the jacket.

The 20MW wind turbine model is derived from upscaling the 10MW reference wind turbine [4]. Tower root ultimate and fatigue loads are calculated for a few relevant IEC-61400 DLCs. These tower bottom loads are used for the jacket predesign. The subsequent conceptual analyses of the full offshore wind turbine model, which consists of the upscaled wind turbine, the initial jacket from the predesign and soil-pile interaction is modelled and analysed using aero-hydro-elastic calculations according to governing DLCs (i.e. design driving DLC for the foundation). These load calculations are carried out with DNVGL BLADED. The resulting tower bottom interface load time series are used to iterate the jacket concept design. The load iteration procedure of the models is following state-of-the-art methods used by the industry.

The design basis of the 10MW reference jacket design is applied also for the 20MW design. Hence, the water depth of the structure is still 50m and identical assumptions for soil and met-ocean conditions are assumed in order to perform a fair cost comparison between 10MW and 20MW jacket solutions.

The design considers ultimate limit state analysis for steel and soil bearing capacities, fatigue limit state analysis and natural frequency analysis. For the primary structure of the jacket foundation, the analyses are performed with the Ramboll Offshore Structural Analysis Program named ROSAP.

Even if the turbine is an upscaled land version and probably not an optimized solution in terms of size and mass, the analysis of a 20 MW offshore wind turbine illustrates how a support structure would look like by using state-of-the-art design processes and tools. The result show critical design challenges and enables discussions regarding necessary improvements of methods and models to design such large structures.

2 DESIGN ASSUMPTIONS AND LIMITATIONS

2.1 Site Description

The chosen location for the INNWIND.EU project is a 50m deep offshore site in the North Sea, which is the same location as used in the UpWind project. The design basis shows the data for a K13 deep offshore site in the Dutch North Sea. The coordinates of K13 are 53°13'04" north and 3°13'13" east, and the K13 site has an original water depth of 25m but for studies related to deep water sites, the site data is correlated to a 50m site. Detailed information about the site conditions is given in [7].

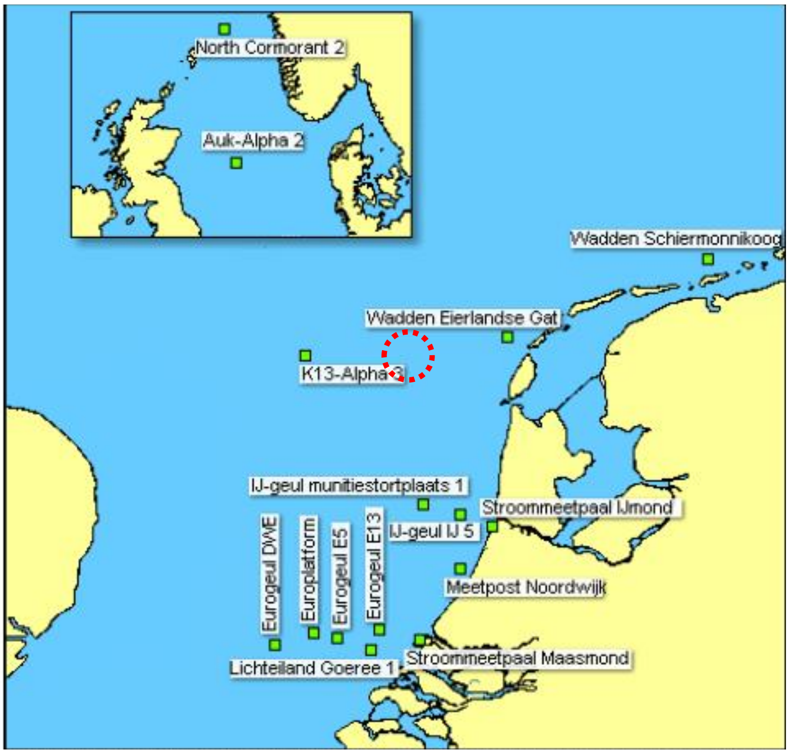


Figure 2-1: Locations for which Rijkswaterstaat measures wind and wave data [7]

2.2 Jacket Support Structure Concept

The general arrangement and used terms of the overall design layout is given in Figure 2-2. A pre-installed pile solution using a piling template is considered for the jacket design. The annulus between the jacket leg and the pile is filled with grout after the jacket is installed on the piles. Shear keys will be used for the improvement of axial load transfer.

The jacket legs are designed to be flooded, which means that a hole will be included in each leg at the bottom of the jacket to allow sea water to flow in during installation. Similarly, an air release hole will be designed at the top of each leg. These holes are sealed after installation, which will prohibit exchange of fresh air and seawater inside the legs and thereby internal corrosion in the legs is avoided. Braces are non-flooded and assumed to be sealed after welding to prohibit internal corrosion of the braces.

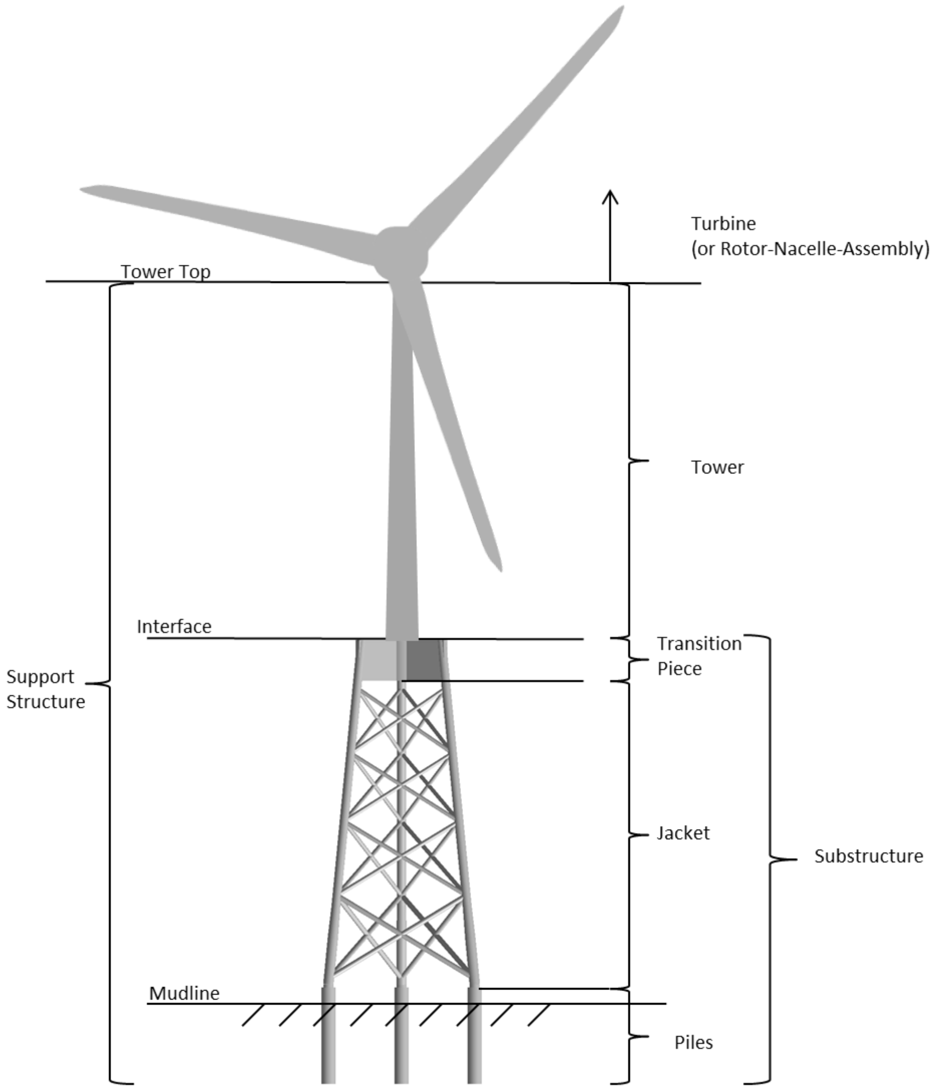


Figure 2-2: Definition of offshore wind turbine and the jacket

Secondary steel is included in the analyses of the primary steel by applying appropriate wave area, volume and masse, i.e. the additional loading from these structures are included. However, the detailed design of the secondary steel parts is not taken into account and the properties are based only on estimations. The secondary steel parts include items that are part of the access arrangement, external platform, internal platforms, cable protection systems and corrosion protection system.

The hydrodynamic loads on the secondary structural members that will increase the hydrodynamic loading in general are influenced by their orientation relatively to the direction of the flow (wave and current) which is also considered in the wave load analysis accordingly.

The design of the boat landing shall ensure safe and easy access and egress and provide at least one safe rescue facility in case of emergency situations. The boat landing shall consist of two inner steel tubes with the ladder in between and two outer fender steel tubes. The lowest impact level shall be determined from the transfer vessel properties of the smallest boat and the corresponding sea state that is acceptable for access operations. The outer fender tubes shall be extended sufficiently far below the lowest water level for boat access, considering the level of the lowest point of the boat mounted fender and the worst wave conditions that are acceptable for access operations in order to ensure safe access and egress and to provide a safety margin against the boat becoming trapped under the bottom of the boat landing fenders. The highest impact level and the lowest acceptable location of the transfer platform / lowest resting platform shall be determined from the transfer vessel properties of the largest boat and the corresponding sea state that is acceptable for access operations. The outer fender tubes shall be extended sufficiently far above the highest water level for boat access, considering the highest point of the boat mounted fender and the worst wave conditions that are acceptable for access operations in order to ensure safe access and egress.

All external access ladders shall have two side stringers. The external ladders below the access platform shall be designed for wave loads and slamming. In order to ease the access to and egress from the access or resting platforms, ladders shall be extended by at least 1.5 m above the platform or alternatively the sidebars of the ladder shall be extended by handrails of at least 1.5 m. On the access platform the extended part of the ladder shall be removable in order to ease service operations on the access platform and in order to ease the tower installation. The external ladders shall be designed in such way that they can be replaced in case they are damaged or corroded to an extent that the safety is not guaranteed any more. External ladders above the boat-landing ladder shall be designed with a fall arrest system or as shifted caged ladders. Safety hoops and fall arrest systems below the access platform shall be designed for wave loads and slamming.

2.3 Design Standards

The foundation structures are designed in accordance with DNVGL-ST-0126 [4], DNVGL-ST-0437 [10] and IEC 61400-3 [2].

2.4 Units and Coordinate Systems

The ISO International system of units is used in all calculations accordingly. In case of exchange of loads between the different project participants the loads should be specified in global coordinate system with consisted units. Otherwise the coordinate systems and units must be clearly documented.

The definition of the direction of wind and wave directions is depicted in Figure 2-3: A wind direction of 90° (or East) means for example, that wind is blowing from East to West. In contrast, the currents direction is defined as going towards the given direction.

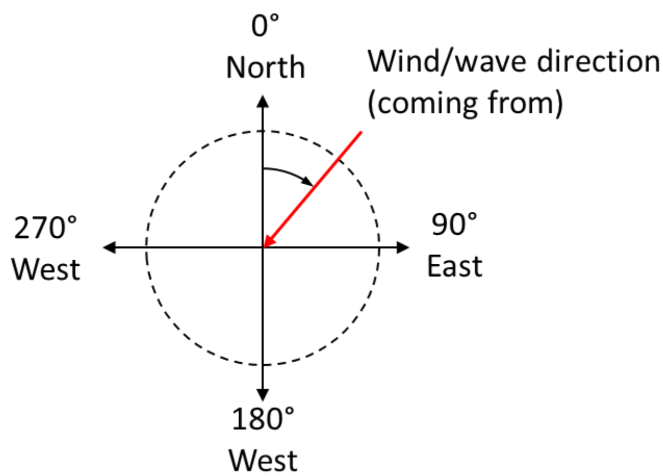


Figure 2-3: Global coordinate system of the sub-structures

The global coordinate system of the substructure and wind turbine models applied by the different analysis tools (i.e. Gast.mb, BLADED and ROSA) are shown in Figure 2-4. Global loads and displacements, which are given in these coordinate systems, need to be transformed accordingly. The vertical elevation is given in meters above mean sea level (offshore condition) and meters above ground (onshore conditions).

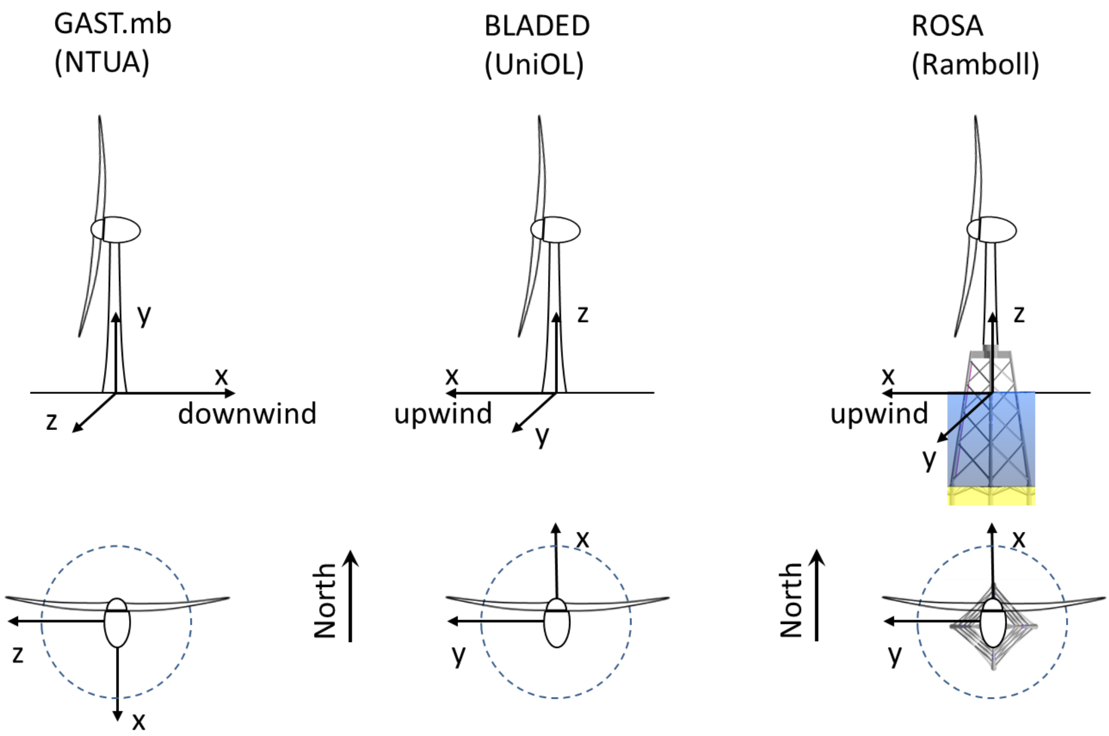


Figure 2-4: GAST.mb, Bladed and ROSA coordinate system

2.5 Assumptions

2.5.1 Global Dimensions

The general layout of the jacket has been maintained, which means that the jacket has four legs and four levels of X-bracings. The water depth is 50m MSL. The jacket legs, braces and joints are mainly driven by wind turbine fatigue loads and less by the wave action. Only the legs in the splash zone are also influenced by wave action but the governing loads are from the wind turbine.

The bottom elevation of the transition piece is determined by the maximum wave crest, sea level rise and an additional safety air gap. According to the reference design the lower elevation above mean sea level is defined to 18m. The transition piece design height is 8m, which is considered to be the minimum height. The interface elevation between WTG tower and transition piece is therefore 26m. Figure 2-5 shows the comparative summary between the 10MW and 20MW concept.

The height of the transition piece with only 8m is a given requirement in order to avoid additional stiffening of the entire model. It is an outcome of the natural frequency study of the 20MW reference wind turbine [5], which indicated the risk of developing generally a too stiff structure with 3p excitation risks. The jacket design of the 10MW reference wind turbine required this dimension [1], which indicates that a larger transition piece or a different transition piece concept is required to transfer the tower loads of the 20MW turbine safely to the jacket. Finally, a box girder concept is selected for the current jacket design. However the dimensions of the box girder are estimated only and not part of the design assessment yet.

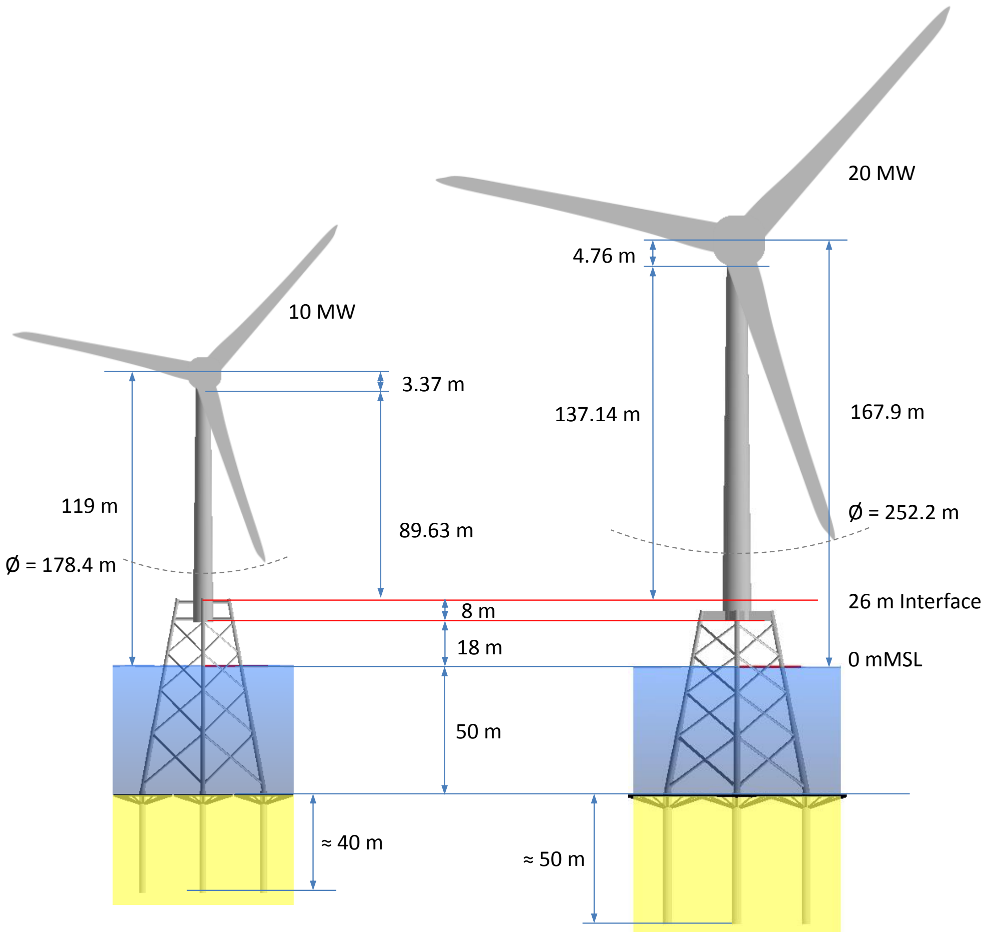


Figure 2-5: Comparison of the INNWIND.EU 10MW and 20MW jacket concept

The tubular members of the jacket are based on standardised diameter and wall thickness, where possible, to ensure a cost-efficient design. Usually steps of full inches are used for diameters of the piles, jacket leg and braces. Wall thickness increments of ¼ inch are mainly considered. In advance of the design implementation an estimation based on experience was made, the summary is given in Table 2-1. The design will results in an estimation of the overall layout, dimensions and masses of the jacket, pile and transition piece with the following details:

- Dimensions of legs, braces and piles
- Node coordinates
- Wall thickness distribution
- Masses of jacket, piles and transition piece

Table 2-1: Estimated jacket dimensions for 20MW offshore wind turbines

Parameter	Unit	Reference jacket for 10MW	Estimated jacket for 20MW
RNA mass	[t]	676[1]	1730
Interface level	[mLAT]	26	
Water depth	[mLAT]	50	
Width at mudline[2]	[m]	33	38
Width at top[2]	[m]	16	20
Total height[3] of assembled jacket	[m]	~82	~82
Lifting mass	[t]	900-1100	1600-1700

[1] according to INNWIND.EU reference wind turbine [4]
[2] with respect to center of leg, neglecting appurtenances
[3] including leg extension below mudline

2.5.2 Design Lifetime

The lifetime of the foundation structures shall be 25 years of operation and it is assumed that no inspection of the primary steel is required. Therefore the design fatigue factor of 3.0 is required according to DNV-ST-0126 [8] which results in a target design lifetime of 75 years.

2.5.3 Corrosion Protection

The corrosion control of the substructure is combination of corrosion allowance, protective coating and cathodic protection. The structure is split into various protective zones. The 3 zones relevant for the jacket design are:

- Atmospheric Zone
- Splash Zone
- Submerged zone

For a visual breakdown of the structure into the various zones see Figure 2-6.

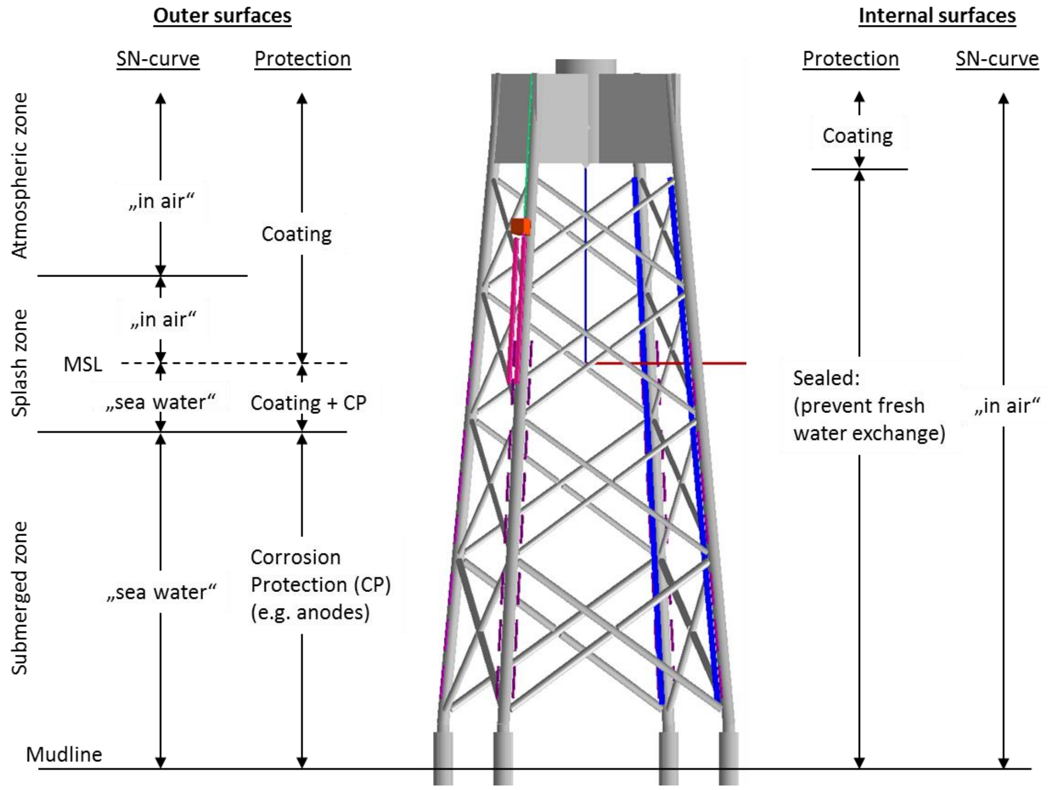


Figure 2-6: Overview of the Corrosion Control Strategy for primary jacket

Corrosion allowance is taken into account in the splash zone, where it cannot be assumed that the cathodic protection has an effect here nor that the coating can be maintained. Due to lower lifetime of the coating compared to the lifetime of the structure the corrosion allowance within the splash zone is defined as follows:

Corrosion rate according to DNV standard [8]:	0.3mm/year
Applicable corrosion period:	25 years (reference period)
Commissioning/decommissioning:	2 years
Coating	15 years
Applicable corrosion allowance:	12 x 0.3mm =3.6mm

The legs are assumed to be flooded and sealed after installation. Therefore only external corrosion allowance is considered in the splash zone for the legs.

All members that are fully submerged below the splash zone are protected by a cathodic protection system to prevent the structure from corrosion in this zone.

For fatigue calculations, half of the corrosion allowance has to been taken into account as an average over the lifetime. For extreme load calculations, the full corrosion allowance is applied.

2.5.4 Pile Driving Assessment

The piles should be driven by means of a hammer installed at the pile top. The lowest pile section can feature a larger wall thickness in order to account for higher strains at this part during driving and to enlarge the stiffness of the pile if a pile driving shoe is needed.

Driving modes shall be selected considering the stratigraphy in order to minimize the reduction in service-life. The damage due to pile driving can be determined in the pile driving analysis and added to the predicted damage during operation.

Pile driving analysis may be carried out on the basis of wave equation analysis of pile driving. The three components of the driving system (piling hammer incl. pile cap, pile and soil) shall be modelled by mass points, springs and damping elements.

The ultimate bearing capacities of the pile shaft and the tip (SRD = Soil Resistance during Driving) shall be calculated to determine the number of blows and pile stresses over the embedded length. This shall be done in order to verify the driveability of the piles and to choose the appropriate pile driving equipment.

Fatigue due to pile driving can be calculated on the basis of the number of blows and the associated stress range in the pile for each 1m-increment of the embedded length. The damages are added up by means of the Palmgren-Miner-Rule. The damage due to the pile driving shall be considered as a reduction of the remaining service lifetime of the piles.

The pile driving assessment described above is not analysed for this 20 MW jacket design in the current design phase. Conservatively the design lifetime of the piles is increased by 1 year to consider the additional damage in a simplified manner and it is assumed that driveability is not an issue for this site.

2.6 Limitations

The manufacturing of a 20MW structure is technically feasible with technologies currently applied in the offshore and marine market, for instance when comparing jacket components (legs and braces) with large monopiles. The main manufacturing cost contributors (material, welding, coating and assembly coasts) are independent of the size of the structure and the general layout. But the overall size of the structure becomes challenging for transport and installation and the current vessel fleet for installing large structure is small and expensive see ref. [3].

The total height of jacket is approximately between 80-90m. The footprint of the jacket is about 38 x 38m whereas the top area is about 20 x 20m.The overall height and width of the jacket structure can lead to additional coasts for transportation, installation and a logistical issue for fabrication.

To transport and install jacket foundations specific types of vessels are required which can travel to the wind park location under certain maximum conditions. The limiting environmental factors are wave height, wind speed and currents and they lead to a weather window with sufficient length (with a maximum sea state) to carry out the transportation and installation process. Due to the large dimensions of the jacket there are only a few installation vessels, which can handle a 88m high jacket and install rotor and nacelle in 170m above LAT.

For these large installation vessels there are additionally port and harbour requirements. The berth at quayside needs to have a sufficient water depth for the (in some cases) deep drafted installation vessels at any time of tide. Additionally, also the berth needs to be of sufficient length to reach the complete deck area by the harbour cranes. Those cranes need to have sufficient loading capacity (mass and hook height) to shift the jackets on the vessel. The storage and handling area is another critical factor of the harbour infrastructure see ref. [3].

Installation times per foundation vary between four and eight days. In general vessel types (jack-up vessels and heavy crane vessels) that can carry two jacket foundations need less time per foundation. The time for installation cycles is of similar magnitude for all vessel types, and the duration of offshore lifting operations is comparable as the jacket needs to be slowly lowered to the ground in every case. The overall influence of mobilization and demobilization on the total time is negligible and will be even further reduced if the amount of WTG in the wind park increases. The same applies to the transfer to and from the base harbour. As the vessels are situated in Europe the distance to the base harbour will be roughly of equal length. Exceptions are sheerlegs from Asia transferred to Europe for specific installation purposes. This is only economically feasible because sheerlegs in Asia are readily available for low daily rates ref. [3].

Heavy crane vessels and sheerlegs represent the highest installation costs per foundation. Heavy crane vessels have a high occupancy rate which raises the charter rate. Even though the installation time per foundation is the lowest, the high costs per foundation can be explained with charter rates. The assumed maximum sea state for heavy crane vessels results in very low downtime costs. Sheerlegs can be chartered for lower rates, the installation and feeder cycle require more time so that the costs per foundation are nearly equal to heavy lift vessels. Jack-up vessels (3rd/4th generation) and the U-barge prototype represent the lowest installation costs. Jack-up vessels have a fairly low charter rate and a satisfying behaviour in a seaway ref. [3].

3 DESIGN PROCEDURE

3.1 General

The design procedure is in accordance to the standards and guidelines for offshore support structures see ref. [8]. The design of substructures involves a number of different analyses in order to verify the overall structural integrity of the system for all loading conditions that may be experienced during the construction, installation, operation and decommissioning phases. The following analysis are considered:

- Natural frequency analysis (NFA)
- Extreme event analysis (ULS)
- Fatigue analysis (FLS)
- Serviceability analysis (SLS)
- Accidental limit state analysis (ALS)
- Ship impact
- Corrosion protection analysis
- Finite element analysis of local details and grouted connections
- Transportation and Installation analysis

The present jacket design is carried out in a conceptual design level considering the main design driving limit states, which are NFA, FLS and ULS. All primary steel of the jacket and piles is evaluated.

3.2 Natural Frequency Analysis

The natural frequency analysis (NFA) is carried out to determine the natural frequencies of the integrated foundation and wind turbine structure. The purpose of this analysis is to demonstrate that the natural frequency of the entire structure falls inside the allowable frequency band specified by the turbine vendor.

The first natural frequency of the integrated system is determined based on a soft, fatigue and stiff configuration. The parameters for marine growth thickness, corrosion allowance, water levels, soil profiles and scour are adapted to account for these conditions. Characteristically, for jacket support structure the bandwidth between these two configurations is rather low.

The obtained natural frequencies of the system will be utilized in other analyses, such as input for the damping model applied in the load calculation and load expansion. The natural frequency analyses (NFA) will be based on characteristic conditions, i.e. partial safety factors will be set to unity.

Given the global stiffness matrix $[K]$ and the global mass matrix $[M]$, the angular frequencies corresponding to the natural periods and the mode shapes are determined by solution of the following eigenvalue problem:

$$([K] - \omega^2[M])\{v\} = \{0\}$$

In the above expression, the eigenvalues, ω, are used to determine the natural frequencies of the structural vibration, and the eigenvectors $\{v\}$ determine the shapes of these vibrational modes. The corresponding natural period is given by:

$$T = \frac{2\pi}{\omega} \Rightarrow f = \frac{1}{T} = \frac{\omega}{2\pi}$$

3.3 Ultimate Limit State

The purpose of the extreme event analysis is to ensure that the jacket structure is capable of supporting the WTG for the least favourable combination of permanent loads, variable functional loads, wind turbine loads and environmental load conditions. Normal (N) and abnormal (A) design situations are considered in ULS. The purpose of the ULS analysis is to verify the capacity of all the elements and joints in the jacket. Full corrosion allowance, maximum marine growth, varying water levels, extreme waves, extreme currents, extreme wind loads and appropriate load safety factors are considered. The extreme load analysis is conducted with characteristic soil conditions, i.e. without soil safety factors.

The ULS design criterion for steel members is to keep the maximum member utilization ratio equal to or below 1.00, with this value being the ratio of the actual design stress in the member divided by the design material strength. From this analysis the pile design loads are extracted.

Table 3-1: Partial safety factors for loads

ULS partial load safety factor, γ_f			SLS, γ_f
Type of design situation		Favourable permanent loads[1)]	
N Normal	A Abnormal	All design situations	
1.35	1.10	0.90	1.00

[1] Favourable gravity or buoyancy loads if significantly relieve the total response

The pile-soil utilization check to determine the minimum required pile penetration, gravity and buoyancy loads are considered unfavourable for the piles under maximum compression and favourable for the piles under maximum tension. The pile design is based on plastic soil conditions including soil safety factors. In this geotechnical analysis the pile design loads from ULS are taken into account.

Unless otherwise specified, the following material safety factors γ_M are applied to the characteristic soil strength parameters to determine the design soil resistance:

Table 3-2: Partial material safety factors for pile foundations [8]

Type of geotechnical analysis	*Limit state*	
	ULS	*SLS*
	γ_M	γ_M
Effective stress analysis	1.15	1.0
Total stress analysis	1.25	1.0

3.4 Fatigue Limit State

The structural design shall ensure that the fatigue lives of all members and details fulfil the required service life. The foundation is designed according to the recommended practice DNV-RP-C203 [9]. The fatigue analysis is performed characteristically (i.e. no safety factors on loads and material) to account accurately for the bi-linear SN-curves and subsequently the resulting damages are multiplied with the Design Fatigue Factor (DFF) of three (3.0). This allows the design to survive without any service inspection. Thereby, if the target lifetime is 25 years, a design life time of 75 years is required see also section 2.5.2.

The fatigue damage is determined using an S-N curve approach combined with appropriate stress concentration factors (SCF), which consider the change in thickness and transitions between cylindrical and conical sections and tubular joints by joint classification according to Efthymiou [11]. An overview of the applied SN-curves is given in Table 3-3. Welds are considered as full penetration welds. A simplified approach is chosen in the upper splash zone region where free corrosion can occur after the coating reaches its lifetime. SN-curves specified as "in sea water" are applied over the entire lifetime instead of combining SN-curves “in air”, where the coating is in place and “free corrosion” where the coating is eroded.

Table 3-3: Applied SN-curves for FLS

	Description	S-N Curve		Valid for
		In and below splash zone	Above splash zone	
1	Element Fatigue[1)]	DNV-D-W	DNV-D-A	Circumferential welds made from both sides
		DNV-F-W	DNV-F-A	Circumferential welds made from one side with a backing bar
		DNV-F3-W	DNV-F3-A	Circumferential welds made from one side without a backing bar
2	Tubular Joints[2)]	DNV-T-W	DNV-T-A	Circumferential welds welded from both sides
3	Attachment fatigue with SCF[3)]	DNV-D-W	DNV-D-A	Internally and externally

[1)] Welds at the legs are made double-sided, welds at braces are made single-sided.
[2)] SCFs calculated according to Efthymiou [11]
[3)] With appropriate SCF based on experience

The fatigue analysis methodology is based on the hot spot stress approach. This means that the geometrical stresses created by the considered details are calculated, while the notch stress induced by the local weld geometry is excluded from the stress calculation. The notch effect is accounted for in the corresponding hot spot S-N curve.

The damage is calculated by using rain-flow counting on the stresses obtained from the dynamic analysis of the support structure. The fatigue damage D is calculated using the Palmgren-Miner rules

$$D = \sum_{i=1}^{n} N_i \,/ A \cdot {\sigma_i}^{-m} \leq 1$$

The various fatigue contributions from in-place analysis (i.e. operation), transport and installation are calculated separately and the individual damages are subsequently combined. The damage from in-place operation is calculated as a yearly damage, whereas damage from construction, transport and installation will be calculated as isolated one-time damages. In the current conceptual design only the damage from in-place analysis is considered. Therefore the inverse of the yearly damage is equal the resulting lifetime.

3.5 Load Simulation Approach

3.5.1 Methodology

The land version of the 20 MW Reference Wind Turbine [5] derived from the upscaling of the 10MW DTU RWT and is intended to be a Class IC (the 10 MW DTU RWT is an IEC-61400 Class IA design). This is mainly expected to affect the fatigue loads which are usually the design drivers for jackets. Classical upscaling techniques are applied in the beginning to derive the parameters. In a second step the up-scaled data are adjusted to consider learning curve expectations and technical innovations of future wind turbines in terms of reduced components masses. Most challenging is the definition of the system’s first natural frequency in connection to the variable speed schedule of the (onshore) turbine in order to account for the dynamics of the offshore configuration [5]. A number of different tower configurations are considered to calculate different fatigue load scenarios for different first natural frequencies.

The land version WTG loads (refer to section 5.1) are taken at interface height and are combined conservatively with random sea states for the foundation predesign. At this stage this means that the wind turbine loads do not consider the true stiffness of the structure and thus quite high uncertainty of the results must be expected.

The calculation method for subsequent load iterations for the concept design is different. The design loads are based on an integrated wind turbine and foundation analyses with exchange of foundation superelements and interface loads (refer to section 5.2). Thus the jacket design calculations are based on load time series that accurately consider wind and wave excitation and structural response. In a first step a superelement of the initial jacket from the pre-design is calculated and implemented in the wind turbine load simulation. The design load cases are performed and the interface forces at tower bottom are saved for the subsequent jacket design. These interface forces are combined with the wave loads from wind-correlated sea states. The stresses in the jacket members are calculated and further processed in the post-processing for ULS and FLS design checks. Figure 3-1 illustrates the load simulation method in three iterative steps.

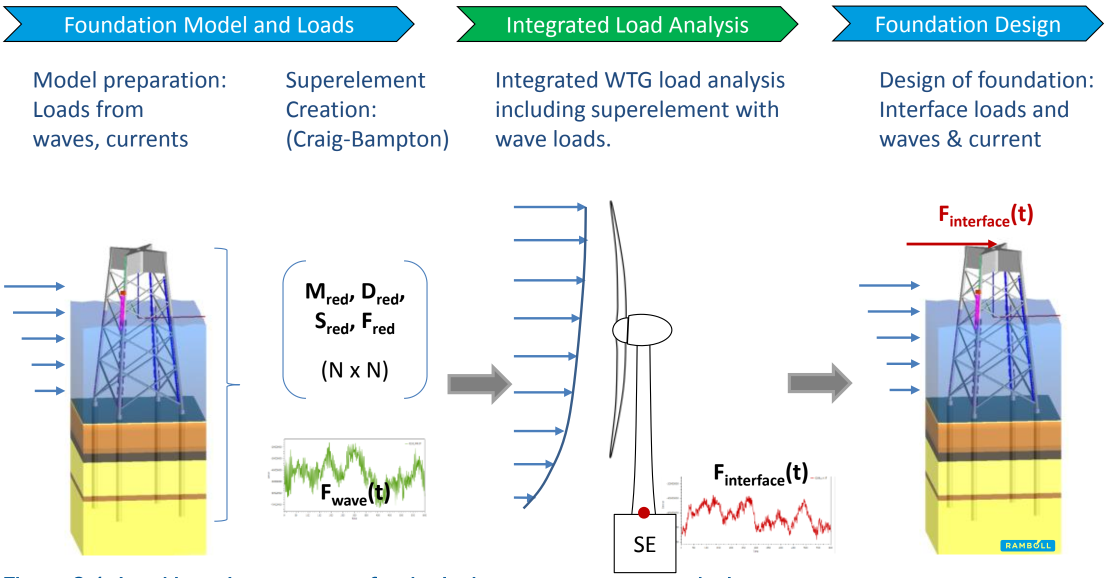


Figure 3-1: Load iteration sequence for the jacket support structure design

The integrated models used for the foundation analysis and WTG analysis have to be compared by means of modal properties of the models. This means that the NFA results of the full system need to agree. Differences will be present as the foundation design tools use a simplified approximation of the rotor-nacelle-assembly in the natural frequency analysis. However, the difference of the first global bending frequency is very small and is sufficient for the model validation.

3.5.2 Damping Model

Damping is used comprehensive term that shall have the meaning of all damping contributions other than aerodynamic damping, i.e. material damping of steel, soil damping, damping due to friction and wave damping (radiation).

The finite element model of the jacket substructure model (ROSA) applies Rayleigh damping, which basically is a mass and stiffness matrix proportional damping. The damping parameters are calculated from the assumed critical damping values of the 1st and 2nd fore-aft bending frequencies of the entire model (i.e. including simplified WTG and foundation). In this way the damping matrix of the jacket superelement model accurately accounts for the entire system. The considered critical damping value of this model is 1% for both modes. Figure 3-2 shows a representative example of Rayleigh damping and how the different frequencies are damped.

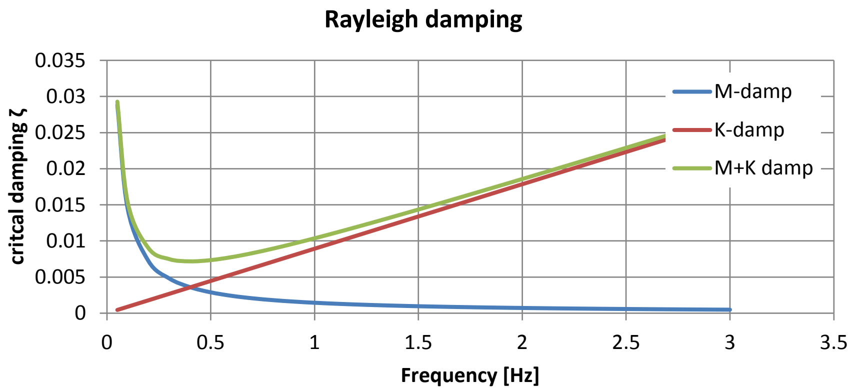


Figure 3-2: Example of Rayleigh damping using 1% critical damping at 0.17Hz and 0.95Hz.

The damping matrix C is a linear combination of stiffness and mass proportional damping with the following relation:

$$\boldsymbol{C} = \alpha \boldsymbol{M} + \beta \boldsymbol{K}$$

The Rayleigh coefficients α and β are calculated from the critical damping values $\zeta_{1/2}$ defined for the first and second global bending natural frequencies $\omega_{1/2}$. The following calculations are performed:

$$\alpha = \frac{2\omega_1\omega_2}{\omega_2^2 - \omega_1^2} * (\zeta_1\omega_2 - \zeta_2\omega_1) \quad with \;\; \omega = \frac{2\pi}{T}$$

$$\beta = \frac{2(\zeta_2\omega_2 - \zeta_1\omega_1)}{\omega_2^2 - \omega_1^2} \qquad\qquad with \;\; \omega = \frac{2\pi}{T}$$

The critical damping $\zeta_{1/2}$ for the jacket foundation is assumed to be 1.0% (i.e. 4.4% logarithmic decrement). The corresponding frequencies of the offshore model are given in the NFA results.

The wind turbine model (Bladed) applies a different damping model. Here, the damping is defined as the ratio of the critical damping for each mode individually, i.e. modal damping. It is possible to assign different damping ratios for each mode of the support structure and rotor blades. Once the foundation superelement is implemented in the wind turbine model the entire damping of the full model deviates from the specified system damping because the model now consists of two damping contributions in a row. The foundation superelement should not be adjusted as this would conflict the model consistency in the third step of the load iteration. Hence, only the modal damping of the tower can be adjusted (increased). The full system damping can be evaluated from a free decays analysis for example using an emergency shut down load case.

4 BASIS FOR DESIGN

The load cases reflect the different operating conditions of the wind turbine considering the wind and wave climate as well as other functional loads of the turbine. The results of these investigations with focus on the wind turbine are applied on the model of the jacket. In this section further details about loads and met-ocean parameters are given.

4.1 Permanent Loads

Permanent loads are loads that do not vary in magnitude, position or direction during the service life considered. The weight of the structure and all appurtenances are denoted as dead weight. These loads are considered constant and are included in all design load conditions.

Dead weight of the jacket structure includes typically the following items:

- All structural members
- Rotor-Nacelle-Assembly (RNA)
- Equipment inside the tower (ladders, platforms)
- Other electrical components
- Power units
- Boat landing
- Access ladder
- Internal power cables
- Internal platforms
- External platforms
- Anodes (GACP)
- Concrete/grout filling material for suction buckets

4.2 Variable Functional Loads

Variable loads are loads which may vary in magnitude, position or direction during the period under consideration, and which are related to operations and normal use of the installation.

For an offshore wind turbine structure, these loads are only relevant for local design purposes, e.g. walkways and platforms. For the global primary steel analyses, variable functional loads on platform areas are comparable small and neglected.

The variable functional loads on an offshore wind turbine typically include the following:

- Start-up loads
- Loads applied on access ways and internal structures, such as ladders and platforms
- Crane operational loads
- Stored material and equipment
- Loads associated with installation operations

4.3 Environmental Conditions

Environmental loads usually vary in magnitude, position and direction during the lifetime of the foundation structure. Typically, these loads come from or are influenced by:

- Wind
- Waves
- Currents
- Tidal effects
- Spray and precipitation ice (if relevant for the site)
- Marine growth
- Scour (if relevant for the site)
- Hydrostatic pressures according to varying water surface elevation

The parameters describing environmental conditions are based on observations close to the wind farm site, as well as on general knowledge on environmental conditions in the area. Simultaneous occurrence of waves, wind and current has to be considered.

The environmental loads shall be determined with the required probability of exceedance. The statistical analysis of measured or simulated data should make use of the different statistical methods to evaluate the accuracy of results. Environmental loads shall be determined in accordance with load cases according to IEC 61400-3 [2]:

- Dynamic loads from correlated wind and waves shall be considered in time domain simulations during different operational states of the wind turbine.
- Hydrodynamic loads induced by additional structures and attachments (anodes, access systems, mooring, platforms, etc.) shall be accounted for.
- The assumption of loads induced by water level shall account for different water levels whichever is more unfavourable.

4.3.1 Wave Load Conditions

4.3.1.1 Sea water

For the sea water, the following values are assumed, see ref. [7].

Table 4-1: Sea water data

Water density	1025 kg/m³
Water salinity	3.5 %
Water temperature (min/max)	0°C / 22°C

4.3.1.2 Water Depth and Levels

The water depth is 50m. In Figure 4-1: an overview of time offsets and tidal range in the German Bight is given.

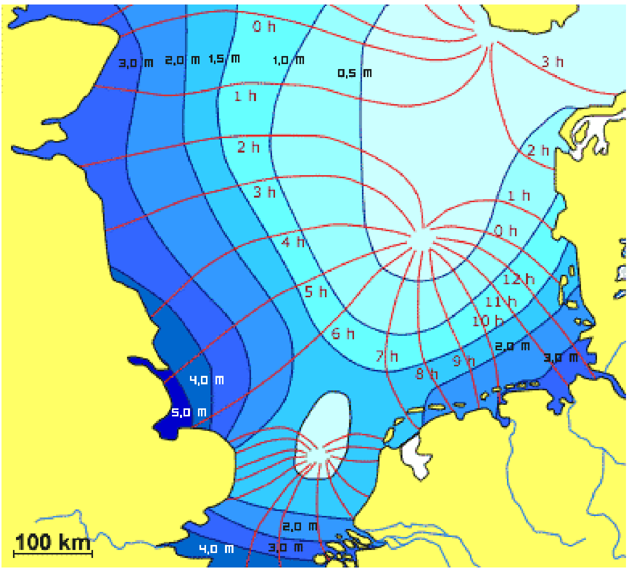


Figure 4-1: Overview of time offsets and tidal range in the German Bight

The following Figure 4-2: depicts the different water levels, which are design relevant.
A typical resulting total water level elevation consists of a superposition of water level elevations caused by astronomical tide and wind and storm surge.

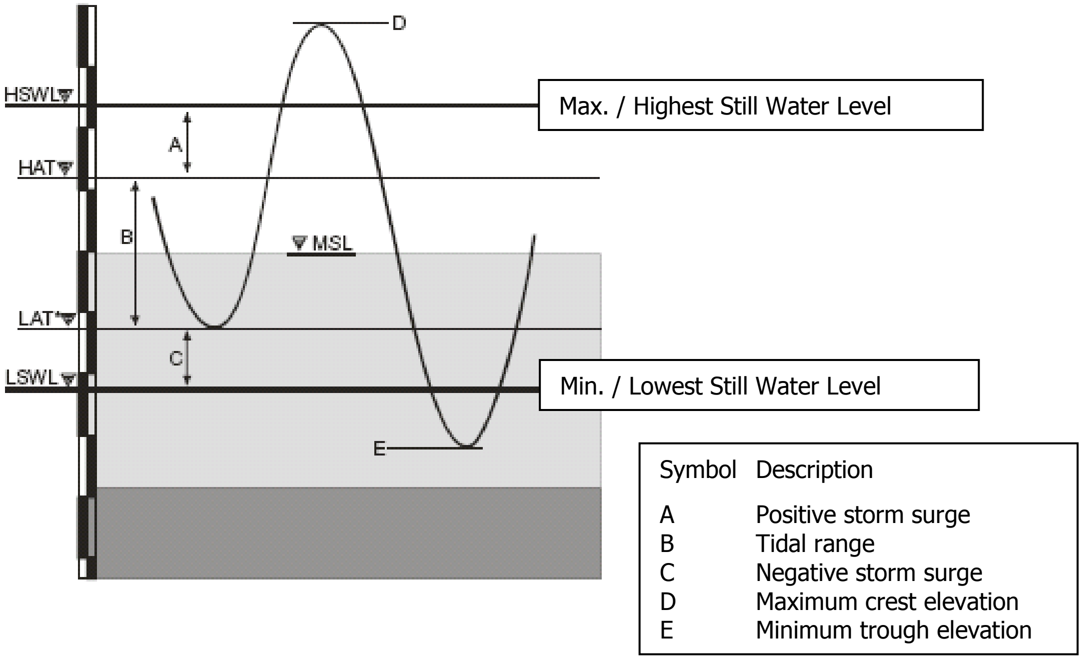


Figure 4-2: Definition of water levels

The measured water level and surge data is available for the K13 site. Figure 4-2 shows the water level values. The 50 year positive storm surge is 2.13 m, while the 50 year negative storm surge is -1.31 m.

Table 4-2: Measured water levels at the location [7]

HSWL	+ 3.29 m MSL
HAT	+ 1.16 m MSL
MSL	0 m

LAT (CD)	- 1.06 m MSL
LSWL	- 2.37 m MSL
A	+ 2.13 m MSL
B	2.22 m
C	- 1.31 m MSL

4.3.1.3 Splash Zone

According to the Upwind design basis ref. [7], the splash zone is determined as

Upper limit: $SZ_U = HAT + 0.6 \cdot (1/3) \cdot H_{s,\max}(100 years) = +4.61 m \quad MSL$

Lower limit: $SZ_L = LAT - 0.4 \cdot (1/3) \cdot H_{s,\max}(100 years) = -3.50 m \quad MSL$

with $H_{s,max}$(100years) = 16.05 m, HAT = 1.4m and LAT = -1.1m.

4.3.1.4 Marine Growth

The presence of plants, animals and bacteria leads to the marine fouling of submerged structures and structures in the splash zone. The presence of marine growth on the structural members can be taken into account by increasing their outer diameter.

For design purposes, marine growth has to be assumed. The density has to be taken as 1100 kg/m³. Table 4-3 shows the thickness as determined according to [7].

Table 4-3: Assumptions for marine growths

Level [m]	Thickness [mm]
MSL -2 to -40	100

4.3.1.5 Wave Parameters

The significant wave heights for different return periods (1, 10, 50, 100 years) can be found in the met-ocean report of the specific location.

In Table 4-4 different significant wave height values for different periods of occurrence are given. To obtain the maximum wave height the following relationship is used:

$$H_{max} = 1.86\, H_s$$

Table 4-4: Extreme wave heights as a function the return period [7]

Return period [yr]	Hs [m]	Tp [s]	Hmax [m]
1	6.05	10.12	11.25
5	6.95	10.54	12.93
10	7.34	10.69	13.65
50	8.24	10.97	15.33
100	8.63	11.05	16.05

The wave and wind correlation, i.e. turbulence intensity, wave period and height, are taken from the UPWIND design basis for the deep water site K13. The values are shown in Table 4-5.

Table 4-5: Wind and wave correlation

V [m/s]	2	4	6	8	10	12	14	16	18	20	22	24
TI [%]	29.2	20.4	17.5	16	15.2	14.6	14.2	13.9	13.6	13.4	13.3	13.1
Hs [m]	1.07	1.1	1.18	1.31	1.48	1.7	1.91	2.19	2.47	2.76	3.09	3.42
Tp [m]	6.03	5.88	5.76	5.67	5.74	5.88	6.07	6.37	6.71	6.99	7.4	7.8

4.3.1.6 Wave Load Generation and Wave Spectrum

The loading is automatically generated by ROSA, using Morison's equation

$$F = C_d \rho \frac{D}{2} |U| U + C_M \rho \pi \frac{D^2}{4} \frac{dU}{dt}$$

where

C_d	Drag coefficient
ρ	fluid density
D	reference diameter of the structural member
U	Water particle velocity
C_M	inertia coefficient

Waves kinematics are modelled representing a JONSWAP-spectrum with governing parameters H_s, T_p (or f_p) and the peak enhancement factor γ.

$$S_J(\omega) = A_\gamma S_{PM}(\omega) \gamma^{\exp\left(-0.5\left(\frac{\omega-\omega_p}{\sigma\omega_p}\right)^2\right)}$$

where

$S_{PM}(\omega)$	Pierson-Moskowitz spectrum
γ	non-dimensional peak shape parameter
σ	spectral width parameter

$\sigma = \sigma_a$ for $\omega \leq \omega_p$

$\sigma = \sigma_b$ for $\omega > \omega_p$

A_γ 1-0.287ln(γ) is a normalizing factor

$$S_{PM}(\omega) = \frac{5}{16} H_S^2 \omega_p^4 \omega^{-5} exp\left(-\frac{5}{4}\left(\frac{\omega}{\omega_p}\right)^4\right)$$

where:

- $\omega_p = 2\pi/T_p$ is the angular spectral peak frequency
- γ is the peak enhancement factor given by:

$$\gamma = 5 \; for \; \frac{T_p}{\sqrt{H_s}} \leq 3.6$$

$$\gamma = \exp\left(5.75 - 1.15 \frac{T_p}{\sqrt{H_s}}\right) for \; 3.6 < \frac{T_p}{\sqrt{H_s}} \leq 5$$

$$\gamma = 1 \; for \; \frac{T_p}{\sqrt{H_s}} > 5$$

Accordingly, the zero-upcrossing period T_z is calculated depending on the T_p and γ as follows:

$$T_z = T_p \sqrt{\frac{5+\gamma}{11+\gamma}}$$

For extreme load analyses based on extreme sea states (ESS) the maximum irregular wave in the time series is replaced by "blending in" a site specific maximum non-linear stream function wave. This allows for consideration of the dynamic behaviour of the structure introduced from the hydrodynamics prior to occurrence of the embedded, maximum wave.

The calculations are based on the assumption of non-breaking waves. If the replacement wave as described above cannot exist at positions with small water depths, the wave height is decreased to the maximum wave height that can theoretically exist.

The contribution to the water particle velocities from the steady-state current is added by specifying the current profile. The current is also taken into account when determining the wave kinematics as a Doppler shift. Hydrostatic pressure and buoyancy from the actual water level (i.e. wave surface) is included in the calculations.

4.3.1.7 Current

In general currents consist of

– currents induced by tidal movement and
– wind and wave induced currents (residual currents).

The current profile is chosen as:

$$CS(z) = CS_{surface} \cdot \left(\frac{z+d}{d}\right)^{\frac{1}{7}} = \frac{8}{7} CS_{DA} \left(\frac{z+d}{d}\right)^{\frac{1}{7}}$$

where,

d	is the water depth to SWL,
z	is the vertical distance above SWL,
CS(z)	is the current speed at vertical height z,
$CS_{surface}$	is the current speed at SWL,
CS_{DA}	is the depth-averaged current speed.

4.3.1.8 Scour

Scour effects can occur in two scales, local scour and global scour. Global scour describes changes of the seabed level over larger areas. Local scour describes the variations of the seabed due to hydraulic effects of the sub-structures. The jacket legs can be equipped with a scour protection in order to prevent any local changes in the seabed.

In this jacket design no scour protection is intended, an additional depth in relation to scour effects has been assumed in accordance to the outer diameter of the water piercing members, D, to be (1.3·D) according to [10].

4.3.2 Wind Load Conditions

Wind loads acting on the wind turbine and tower are included in the interface loads.

In Figure 4-3: the wind speed distribution of the K13 site at hub height can be seen. The measured wind data was converted from the reference height of 10 m to the hub height. The wind speed at elevation z above LAT is according to [7]:

$$V(z) = V(z_{ref}) \frac{\ln\left(\frac{z}{z_0}\right)}{\ln\left(\frac{z_{ref}}{z_0}\right)}$$

with:

V(z) = wind speed at elevation z

V(z_{ref}) = wind speed at elevation zref

z_{ref} = elevation for which wind speed is given

z_0 = roughness length, 0.002 m for offshore conditions

The relevant Weibull parameters are A= 11.68 m/s and k= 2.04, which leads to an annual mean wind speed of 10.05 m/s.

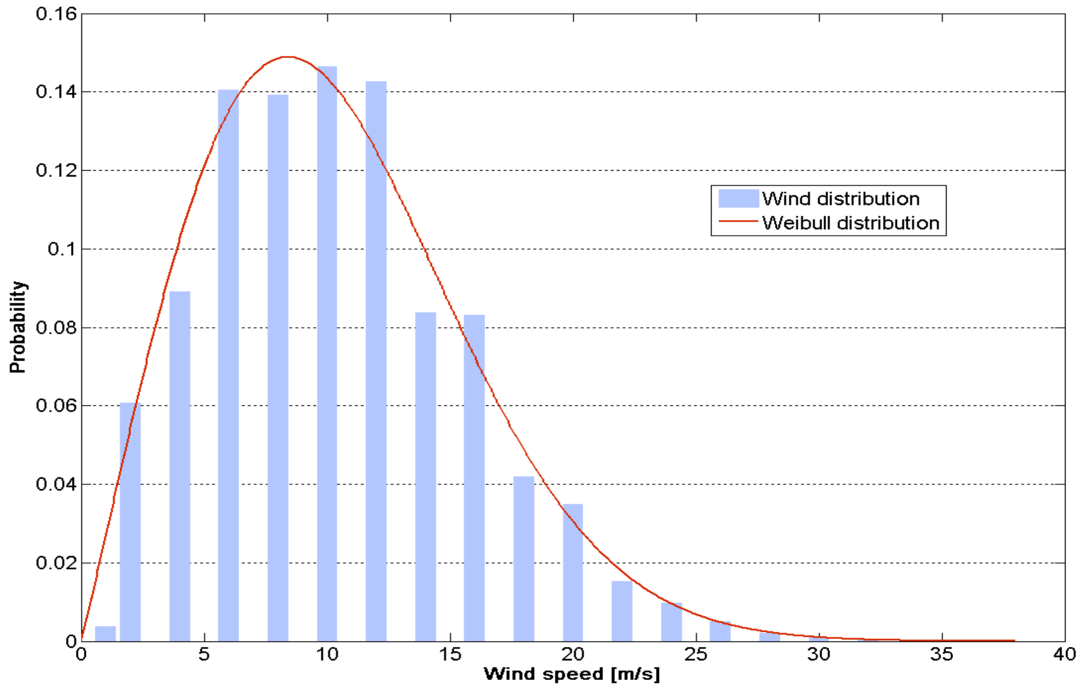


Figure 4-3: Wind speed distribution for the measurement location

Table 4-6 shows the maximum wind speed at hub height as a function of the return period. The values averaged 10-min wind speeds, where the original 3-hrs stationary situations were converted with a factor 0.9 according to IEC.

Table 4-6: Extreme wind speeds as a function of the return period

T_{return} [yr]	V_w (10min) [m/s]
1	32.74
5	36.85
10	38.62
50	42.73
100	44.50

4.3.3 Soil Conditions

The soil is modelled using P-y, T-z and Q-w curves. The curves are derived from a defined soil profile, which is the same as for the reference jacket design. The profile is given in Appendix B – Soil Profile.

4.4 Design Load Cases

A 100 % availability of the turbine is considered conservatively. Fatigue loads of jackets are driven usually by normal operation and not from idling. Wind and wave are assumed aligned. The wind rose will be the reference in order to determine the directional probabilities.

For FLS the DLC 1.2 and 6.4 are taken into account under consideration of the directional wind rose that is discretized in 12 sectors each with 30° width.

For DLC 1.2 the following applies in the conceptual design:
- 12 load directions
- 11 wind speeds between cut in and cut out wind speed
- 2 yaw errors (+/- 8 degree)
- Varying turbulence seeds

For DLC 6.4 the following is needed:
- 12 load directions
- wind speeds below cut in and above cut out
- 2 yaw errors (+/- 8 degree)
- Varying turbulence seeds

Table 4-7: Load Case Table

Load case table

Design situation	Design Load Case acc. to IEC 61400-3	Wind con-dition	Wind Speeds v_{hub} [m/s]	Yaw error [°]	Sea state	Water level	Wave period [s]	Direction [°]	Initiali-zation length [s]	Simulation length [s]	Total length [s]	Type of ana-lysis	Partial safety factor [-]
Power production	1.2	NTM	4, 6, 8,,..., 24	-8 , + 8	NSS	≥MSL	$T_p(H_s)$	0,30,60,...,330	50	600	650	F / U	1.00 / 1.35
Parked (standing still or idling)	6.4	NTM	< 4 / >24	-8 , + 8	NSS	≥MSL	$T_p(H_s)$	0,30,60,...,330	50	600	650	F / U	1.00 / 1.35

Wave definitions	
NSS	Normal Sea State
Wind definitions	
NTM	Normal Turbulence Model

General definitions	
F	Fatigue Limit State
U	Ultimate Limit State

5 WIND TURBINE

5.1 Preliminary Loads

An extrapolated 20 MW wind turbine and tower model is developed by WP1 and basic aeroelastic load calculations are performed [5]. At this stage a so called land-version of the wind turbine is available. The main data of this wind turbine is summarized in Table 5-1. Since the wind turbine is not certified according to standards and not optimised for jacket support structure designs the data and results shown are preliminary and probably have a high uncertainty in accuracy. The given tower data is based on a modified version of the provided onshore tower, which is cut at an elevation 26m to agree with the jacket interface elevation. A simple drawing of the tower is given in Appendix C – Tower Geometry. In further design studies of the wind turbine it is recommended to improve the tower geometry and hub height, currently the resulting clearance between lowest blade tip elevation and interface elevation is rather large and could be reduced if possible regarding aerodynamic requirements.

Table 5-1: Main data of the wind turbine ref.[5]

Wind turbine data		
Wind turbine model		Upscaled 20MW
Rated electrical capacity	MW	20.0
Number of blades	-	3
Hub height	m LAT	+167.9
Rotor diameter	m	252.2
Blade Length	M	122.14
Design Extreme Thrust Value	kN	9600
Rated wind speed	m/s	11.4
Minimum rotor speed	rpm	4.45
Maximum rotor speed	rpm	7.13
Weight of rotor (hub and 3 blades)	t	632
Weight of nacelle without hub and blades	t	1098
Weight of support–tower incl. internals (onshore tower)	t	1600-1780
Distance from the tower upper flange plane to the hub	m	4.76
Tower outer diameter at top of tower (preliminary tower)	m	7.78
Tower outer diameter at interface level (+26 m LAT) (preliminary tower)	m	11.74
1^{st} natural frequency (onshore)	Hz	0.18 - 0.21Hz

The 20MW reference wind turbine is designed for IEC Class IC. The design of a 20 MW offshore turbine includes further challenges regarding the proper selection of systems first global frequency in connection to the variable speed schedule of the turbine which is essential for its high performance. Further deviations from classical upscaling are thus effected to avoid the cross-cutting of the rotor 3P frequency with the 1^{st} global frequency at wind speeds that are critical for the turbine performance and loading [5].

The aeroelastic data of the onshore version [5] makes a first evaluation of the 20MW RWT (onshore version) in terms of its dynamics (natural frequencies of the system) and loads. Blade and tower ultimate and fatigue loads are presented for two relevant IEC-61400 DLCs. The tower bottom loads at 26m height are given for the designing the jacket.

According to [5] it is preferred to achieve a 1st global frequency of the offshore turbine close to 0.20 Hz. Higher values would move tower-rotor resonance at higher wind speeds while lower values might increase wave excitation loads. To do that the land version of the turbine should have an even lower 1st global frequency. This can be done either by increasing the tower height or increasing the tower top mass. Finally, an increase of the tower height has been chosen for the wind turbine design, which results in a blade-sea water clearance of approximately 42m. Although a longer tower increases the ultimate and fatigue moments on the support structure this will be counterbalanced through the suppression of the dynamic loads (rotor-support structure interaction) and the reduction of turbulence induced loads (both ultimate and fatigue) since the design class is now IC compared to the IA of the 10MW RWT ref [5].

The 20MW RWT preliminary loads (onshore) are calculated in a first step for two critical load cases, DLC 1.2 (fatigue) and DLC 6.2 (ultimate). DLC 1.2 is power production design situation with a normal turbulence model wind condition, normal sea state, normal current model and normal water level range for fatigue analysis. DLC 6.2 is a parked or still standing situation with an extreme wind model (wind speed 50m/s for Class I), extreme sea state or reduced wave height, an extreme current model and an extreme water level range for ultimate analysis.

5.2 Wind Turbine for Conceptual Jacket Design

In a second design phase (concept design) an offshore configuration of the 20MW wind turbine is taken into account to appropriately consider dynamic wind loads, wave loads and the structural response of the offshore support structure. The model is a modification of the provided land version of the 20MW wind turbine and the resulting preliminary 20MW jacket design. A large modern offshore wind turbine is a complex structure. Therefore, sophisticated methods are required to predict the detailed performance and loading of a large offshore wind turbine. These methods should take into account:

- The aerodynamics of the rotating blade, including induced flows (i.e. the modification of the flow field caused by the turbine itself), three-dimensional flow effects and dynamic stall effects when appropriate;
- structural analysis of the blades, drive train and tower, allowing their vibrational dynamics to be modelled;
- aeroelastic feedback, i.e. the modification of the aerodynamic forces due to the vibrational velocities of the structure;
- dynamic response of subsystems such as the generator, yaw system and blade pitch control system;
- control algorithms used during normal operation, start-up and shut-down of the turbine;
- Temporal and spatial variations of the wind field impinging on the turbine, including the three-dimensional structure of the turbulence itself.
- Hydrodynamic forces on the submerged structure; and
- Hydro elastic feedback, i.e. the modification of the hydrodynamic forces due to the vibrational velocities of the structure.

The input data for the Bladed software require detailed information about the rotor blades (blade geometry, aerofoil sections, mass and stiffness), aerofoil profiles (lift, drag and pitching moment characteristics of the blade), rotor (the turbine configuration data about turbine and rotor, and about the hub), the tower, the drive train (transmission, mounting and electrical or mechanical losses), the nacelle, the control system (power production control and supervisory control), wind and aerodynamic characteristics (upwind turbine wake, turbulence, time varying wind, wind shear and tower shadow) and sea state (currents, waves, tide). The different modules for the wind turbine modelling are described below.

5.2.1 Blades and Aerofoil

Input data for the blade and aerofoils are identical to the initial upscaled data from 10MW wind turbine data [5].

5.2.2 Rotor and Hub

The geometrical information of rotor and hub as well as the mass of hub are applied. Detailed information on the hub and rotor parameters is provided in [5].

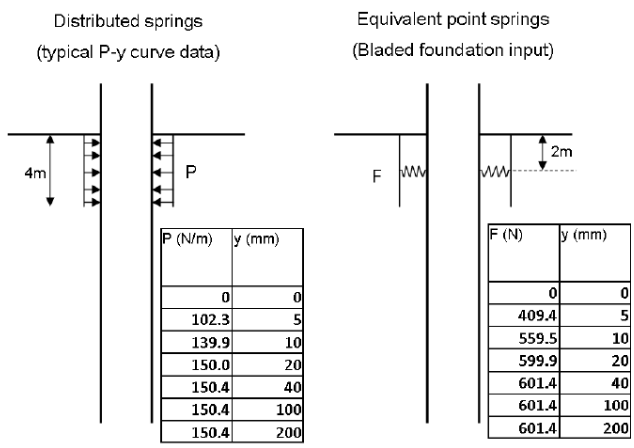
5.2.3 Support Structure

The structural model in Bladed is based on the multibody formulation. Each body has its own coordinate system and is connected via constrains. The aerodynamic and hydrodynamic loadings are calculated at each time step and the resultant forces are distributed on the nodes along the blade and support structure. The support structure can be designed as a simple tubular tower or as multi member lattice structure. In this project, the multi member feature is considered for the offshore tower since the foundation is a jacket.

For defining the tower in bladed, first step is to define all the connection nodes. The second step is to define all members as cylindrical or asymmetric members. The members are defined with two ending nodes, the geometrical properties as diameter of each end, thickness of the element and material properties.

5.2.4 Foundation and Soil

The jacket foundation is supported by four piles penetrated from the mudline to the soil with an approximately penetration length of 50m. To model the soil-pile interaction, the soil behaviour is modelled using the lateral, vertical and torsional springs represented respectively with the P-y, T-z and Q-w curves for different soil layers. The P-y, T-z and Q-w curves are calculated from the API standards using the site specific soil parameters. Considering the spring like behaviour of the soil, a stiffness matrix of soil is defined in bladed using P-y curve values as a look up table with values of force and deflection shown in Figure 5-1.



P (N/m)	y (mm)
0	0
102.3	5
139.9	10
150.0	20
150.4	40
150.4	100
150.4	200

F (N)	y (mm)
0	0
409.4	5
559.5	10
599.9	20
601.4	40
601.4	100
601.4	200

Figure 5-1: Typical p-y curves defined as a lookup table at different soil layers

The boundary conditions for the piles below the mudline were defined for different layers of soil. In this way the pile was defined by specific nodes representing each layer of soil and then the stiffness was defined for each node in a look up table containing the data of the lateral displacement values (m) and resistance values (N).

5.2.5 Drive Train and Nacelle

The drive train and generator parameters, e.g. masses, dimensions, the transmission system, mounting and electrical or mechanical losses are considered according to the upscaled wind turbine data [5].

5.2.6 Control System

The external controller model is similar to the one used for 10MW wind turbine. All controller parameters are tuned to give a proper behaviour of the 20MW wind turbine at the whole operational range.

5.2.7 Wind

The wind model is generated as a rectangular cube with a large number of discrete points forming the grid. The accuracy of the wind field increases with refining the grid. The Kaimal wind model is chosen which simulates the atmospheric turbulence of stochastic wind, including sudden accelerations, gust events etc. The turbulence grid is large enough to include the entire rotor.

The wind shear with a power law profile is chosen using a wind shear exponent of 0.14. In addition, the tower shadow which accounts the velocity deficit due to the presence of the tower is considered. The upwind turbine wake is not considered in simulations.

5.2.8 Waves

The hydrodynamic loading is calculated from the Morison's equation. The wave characteristics are modelled with an irregular wave model with a Jonswap spectrum.

The wind turbine model from Bladed including the rotor nacelle assembly (RNA), tower, jacket structure and piles are depicted in Figure 5-2.

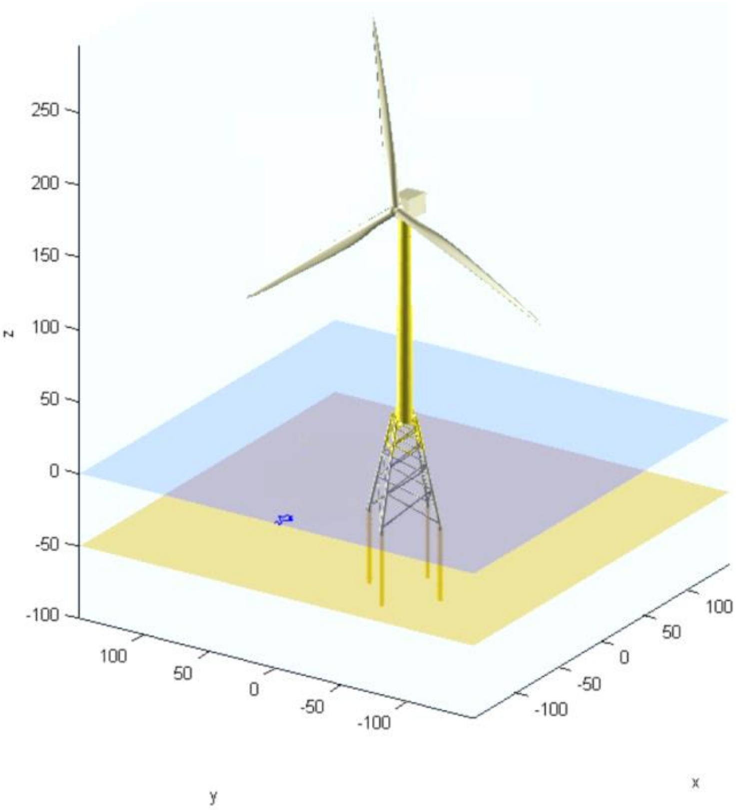


Figure 5-2: 20MW INNWIND.EU offshore wind turbine model in Bladed

6 DESIGN RESULTS 20MW JACKET

Two design phases are considered for the jacket as described on chapter 3. For both phases the structure is analysed regarding natural frequencies, ultimate limit state and fatigue limit state.
First is a preliminary design which is based on the provided onshore loads, as described in section 5.1, which are superimposed with separated wave loads afterwards in order get a first estimation of the 20MW jacket geometry and dimensions. This model is very basic because the given wind turbine loads neglect any interaction with the jacket.
The second design phase called the concept design phase considers a load iteration approach. Dynamic loads from combined wind and waves and the structural response are considered very accurately which results in a more accurately design results. Starting points for this second phase is jacket geometry from the preliminary design which is further optimized afterwards. A number of studies are taken into account to show the important design aspects.

6.1 Preliminary Design

The substructure of the 20 MW wind turbine is a four-legged jacket with 4 levels of x-braces. A subsea template is used to ensure the correct position of the pre-installed piles. When the piles are inserted into the seabed, the jacket substructure is lowered and fitted into the piles. The piles and jacket are connected by a grout connection. The transition piece is a box girder, which has a small height as required in the design assumptions. In Figure 6-1 the design concept and main dimensions are depicted.

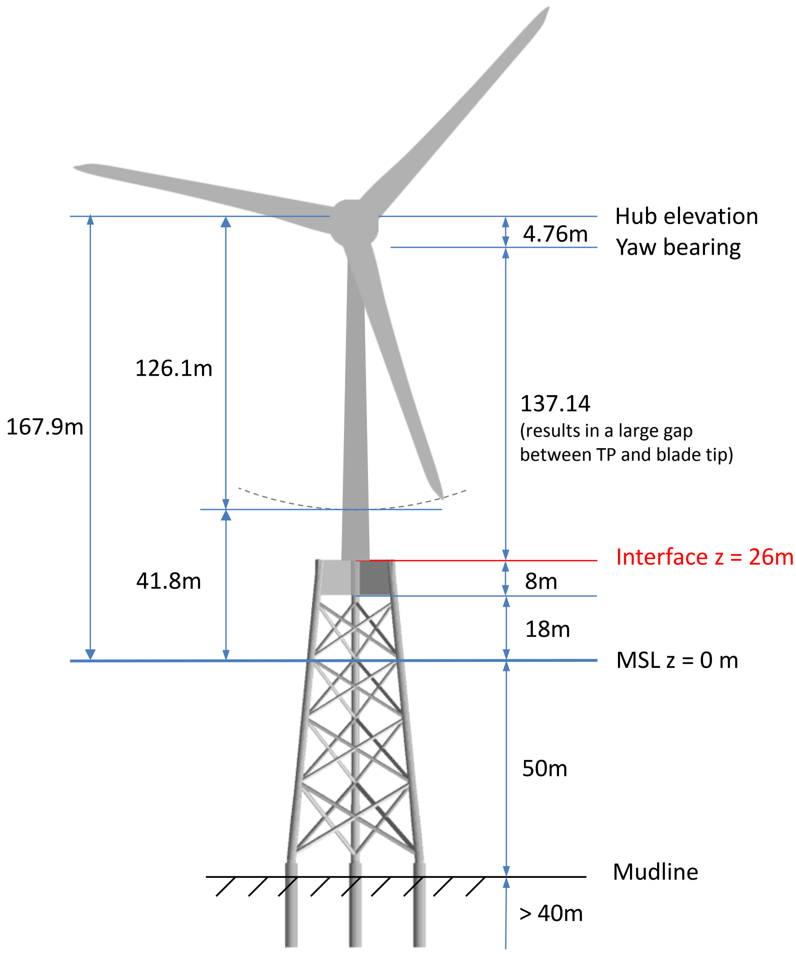


Figure 6-1: Estimated dimensions of the 20 MW jacket

The preliminary design of the jacket substructure has been based on the conditions provided in the 20MW Reference Wind Turbine calculated with GAST.mb, see ref. [5] and Table 5-1 for a brief summary of the main parameters.

The provided wind-only preliminary loads for the 20MW wind turbine consider DLC 6.2 for the extreme event analysis and DLC 1.2 for fatigue. DLC 1.2 considers all operational wind speeds with a 2m/s binning, assuming normal turbulence IC conditions, using 6 turbulence generation seeds. Calculations are performed for 0, ±8° yaw misalignment angles. DLC 6.2 is run for the reference wind speed 50m/s (Class I) assuming turbulent wind conditions using 3 seeds. The yaw misalignment range considered is [-180o, 180o] discretized in steps of 15°. No safety factors have been applied to the calculated ultimate and fatigue loads, see ref. [5].

The simplified model of tower, transition piece, jacket and piles in ROSA model is given in Figure 6-2. The rotor and nacelle are included as point masses.

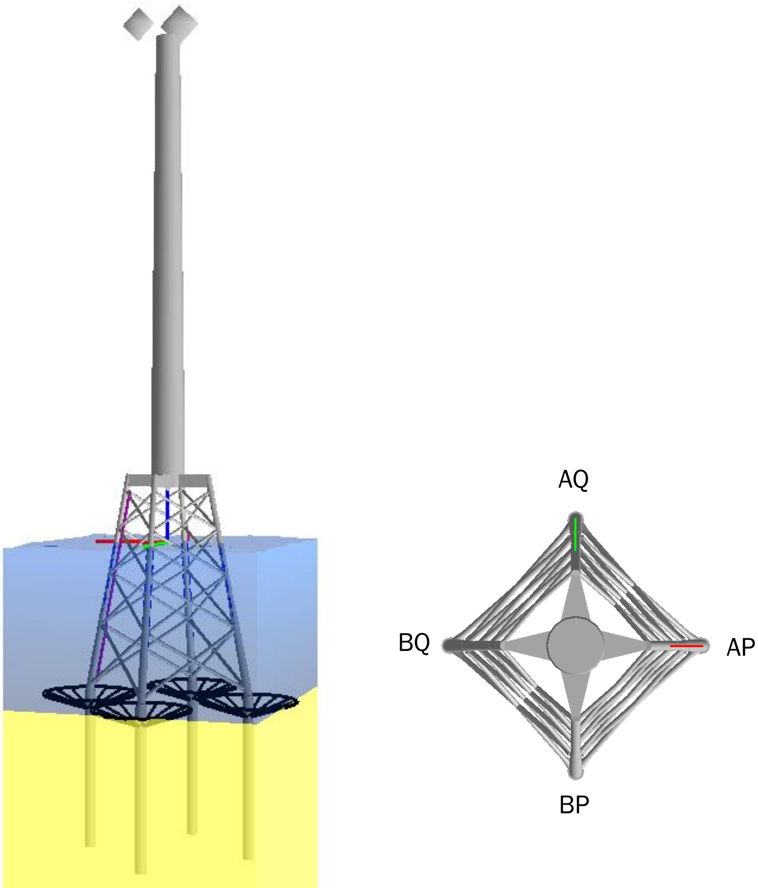


Figure 6-2: Simplified model with RNA, tower, jacket, piles and soil

6.1.1 Natural Frequency Results

The natural frequency analysis (NFA) is carried out to determine the bandwidth of the natural frequencies of the integrated foundation and wind turbine structure based on a soft, fatigue and stiff configuration. The natural frequency analysis is based on characteristic conditions, i.e. partial safety factors of the soil will be set to unity.

The purpose of this analysis is to demonstrate that the natural frequency of the entire structure falls inside the allowable frequency band specified by the turbine vendor of the preliminary design.

The obtained natural frequencies of the fatigue configuration are the input for the damping model applied in the dynamic load calculation.

6.1.1.1 Influence of Stiffness Configurations

There are three different stiffness configurations for the integrated foundation:

- Softest configuration: max. corrosion, marine growth included, local scour, water level HWL
- Fatigue configuration: 50% corrosion, marine growth included, local scour water level MWL
- Stiffest configuration: no corrosion, no marine growth, no local scour, water level LAT

Table 6-1: Natural frequency of the first 10 modes

Combin. name (1/s)	Freq. 1 (1/s)	Freq. 2 (1/s)	Freq. 3 (1/s)	Freq. 4 (1/s)	Freq. 5 (1/s)	Freq. 6 (1/s)	Freq. 7 (1/s)	Freq. 8 (1/s)	Freq. 9 (1/s)	Freq. 10 (1/s)
4X4LSOFT	0.1627	0.1635	0.816	0.9026	0.9576	1.2947	1.3344	1.8819	2.0014	2.2745
4X4LFATI	0.1628	0.1635	0.8169	0.9048	0.961	1.3084	1.3471	1.8989	2.0066	2.2772
4X4LSTIF	0.1632	0.164	0.8254	0.9303	1.0005	1.5978	1.6352	2.4639	2.4673	2.4796

Table 6-2: Periods of the first 10 modes

Combin. name (s)	Period 1 (s)	Period 2 (s)	Period 3 (s)	Period 4 (s)	Period 5 (s)	Period 6 (s)	Period 7 (s)	Period 8 (s)	Period 9 (s)	Period 10 (s)
4X4LSOFT	6.1447	6.1167	1.2255	1.1079	1.0443	0.7724	0.7494	0.5314	0.4997	0.4397
4X4LFATI	6.1425	6.1146	1.2242	1.1053	1.0406	0.7643	0.7424	0.5266	0.4984	0.4391
4X4LSTIF	6.1274	6.0993	1.2115	1.0749	0.9995	0.6259	0.6115	0.4059	0.4053	0.4033

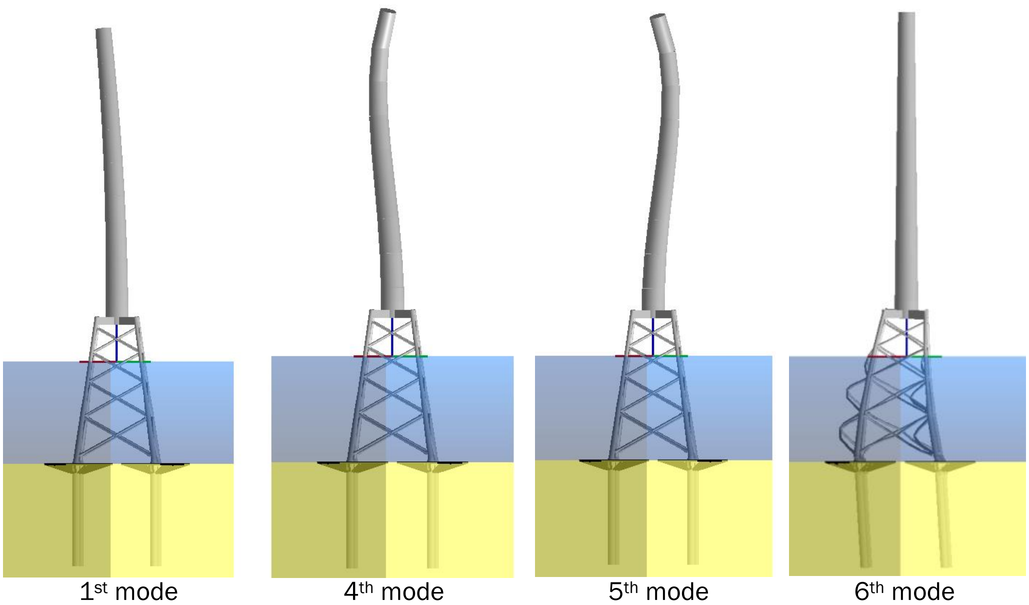


Figure 6-3: First eight eigenmodes of the jacket substructure

In addition, a modal analysis is performed for the full model of the turbine in Bladed. It is possible to set the number of the blade and support structure modes in Bladed. In this study, the first 6 modes of the tower and blades are considered. The natural frequencies of the full model in Bladed are calculated and compared with the simplified model in ROSA. The results are summarised in Table 6-3.

Table 6-3: Comparison of the natural frequencies of the first two modes

<table>
<tr><th rowspan="3">Global bending modes</th><th>ROSA</th><th colspan="4">BLADED</th></tr>
<tr><th>Simplified RNA + Substructure</th><th colspan="2">Superelement model</th><th colspan="2">Full model</th></tr>
<tr><th>Modal Freq. (Hz)</th><th>Modal freq. (Hz)</th><th>Error (%)</th><th>Modal freq. (Hz)</th><th>Error (%)</th></tr>
<tr><td>1st side-side mode</td><td>0.1628</td><td>0.165</td><td>+1.35%</td><td>0.1675</td><td>+2.88%</td></tr>
<tr><td>1st fore-aft mode</td><td>0.1635</td><td>0.167</td><td>+2.14%</td><td>0.1686</td><td>+3.20%</td></tr>
<tr><td>2nd fore-aft mode</td><td>0.8169</td><td>N.A.</td><td>--</td><td>0.6518</td><td>-20.2%</td></tr>
<tr><td>2nd side-side mode</td><td>0.9048</td><td>N.A.</td><td>--</td><td>0.8919</td><td>-1.42%</td></tr>
</table>

6.1.1.2 *Influence of Jacket Geometry*

In order to check the influence of the jacket geometry to the natural frequencies a study with nine different bottom width from 25m to 45m and ten different top width from 14m to 24m has been carried out. The results of all possible combinations of top width and bottom width are shown in Figure 6-4.

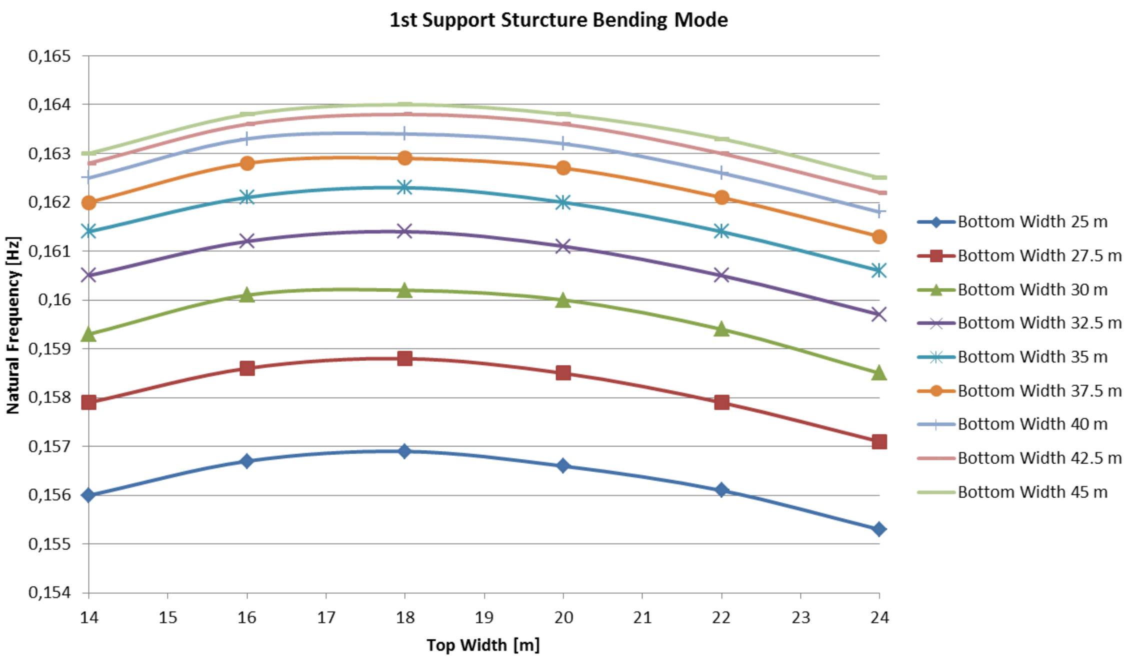


Figure 6-4: 1st Support bending mode for different bottom and top width

Although there is a big range of 10m top width range and 20m bottom width range, the maximum difference in the 1st natural frequency for the bottom width range is only 0.007 Hz, the overall maximum range of all possible bottom and top width combinations is 0.0087 Hz. That corresponds to a deviation of approximately 5% in the natural frequency.

6.1.1.3 Influence of RNA mass

Another possibility to influence the natural frequency is to modify the RNA mass. In this study the RNA masses are varied up to ±15 %. The results of this study are shown in Figure 6-5.

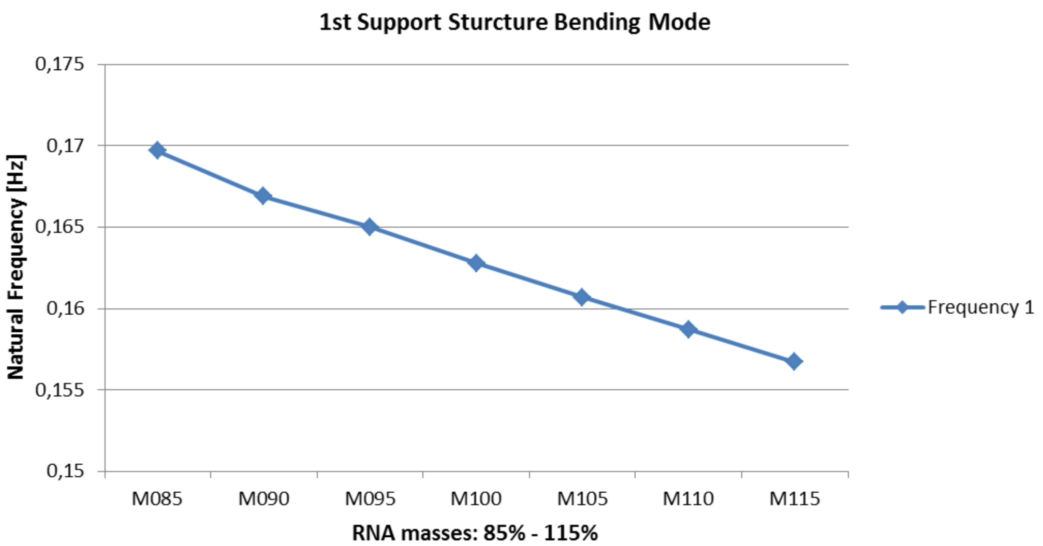


Figure 6-5: 1st Support bending mode for different RNA mass

The variation of the RNA mass results in a maximum difference of the 1st natural frequency of 0.013Hz. That corresponds to a deviation of approximately 8 % in the natural frequency with a linear trend, whereby the RNA mass range is 30 %.

The influence of variation the RNA masses are larger than the geometry of the jacket for the 1st natural frequency of the structure.

6.1.1.4 Influence of the Tower Length

The influence of the tower length for the natural frequency of the entire system was also examined. The tower length is varied with +/- 6m length. The results of the tower length influence can be seen in Figure 6-6.

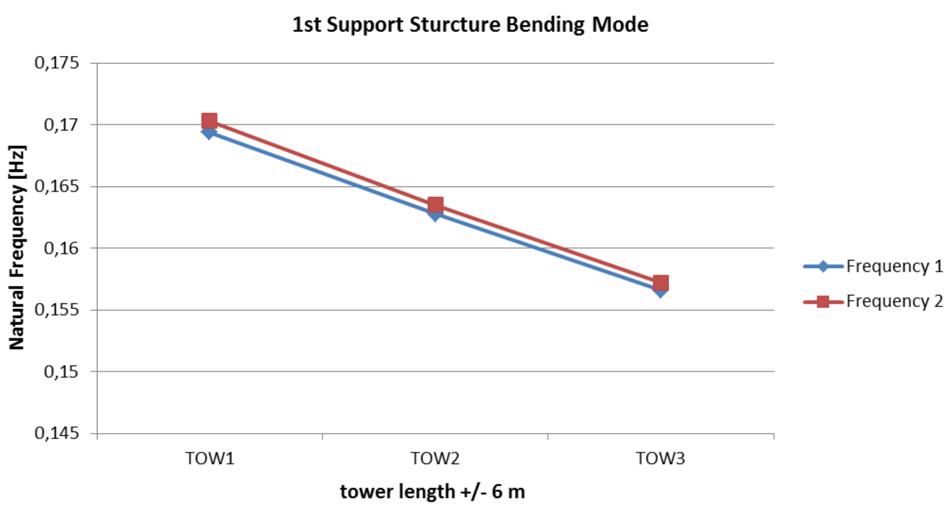


Figure 6-6: 1st support bending mode for different tower length

The variation of the tower length inducted a maximum difference in the 1st natural frequency of 0.013Hz. That corresponds to a deviation of 8 % in the natural frequency, whereby the tower length range is 8.7%.

It clearly shows that the influence of the tower length is significantly larger than major changes of the jacket geometry for the 1st natural frequency of the structure.

6.1.2 Ultimate Strength (ULS)

The extreme environmental loads on the sub-structure due to wind and waves are calculated with ROSA by superposition of the design wind loads and superimposed deterministic extreme wave loads according to section 3.3. The results for the most critical member utilization is shown in Table 6-4 and for the most critical joints in Table 6-5

Table 6-4: Maximum stress utilisations for beam elements – ULS

Load Condition	Side	Beam	Section	Load Case	max. Stress Utilisation
ULS	A	AQ30L	3	91	0.85
ULS	B	BQ40L	3	88	0.75
ULS	P	AP40L	3	89	0.77
ULS	Q	AQ30L	3	91	0.85

Table 6-5: Maximum stress utilisations for joints – ULS

Load Condition	Side	Node	Punching Shear Check	Load Case	max. Stress Utilisation
ULS	A	30A2V	Push	34	0.54
ULS	B	35B1V	Push	46	0.71
ULS	P	13P2V	Push	47	0.65
ULS	Q	45Q1V	Push	95	0.57

6.1.3 Fatigue Strength (FLS)

The cumulative (annual) damage of welded joints, piles, legs, braces and attachment are calculated in the fatigue analysis. The loads for the check of the fatigue strength are combined loads from the wave loads, which are computed with ROSA, and the loads given by the turbine manufacturer, here the onshore wind load time series. The probability of every time series is determined from multiplication of the wind direction probability and Weibull distribution of the mean wind speed divided by the number of considered random seeds.
The structural stresses are determined using stress concentration factors and the resulting damage is obtained from rainflow counting and comparison with the appropriate SN curve according to the method described in section 3.4. The resulting lifetime is the inverse of the annual damage. The results given below are design lifetimes including the DFF.

The detail category that is assigned depends on the geometry of the welded components and the type of weld that is used (i.e. single-sided or double-sided full penetration weld). All attachment welds are positioned in such a way that the distance to the next attachment or the next circumferential weld is sufficient to ensure that the stress concentrations are not influencing each other.

The jacket legs are designed for SN-curve D, the braces conservatively for SN-curve F3 and the joints for SN-curve T according to DNV [9]. A higher detail category can be used if grinding the welds, for example the jacket legs have the detail category D (transverse slices in plates flats and rolled sections), this can be increased to category C1 when high quality welding is achieved and the weld is proven free from significant defects by non-destructive examination, all welds are ground flush to plate surface.

6.1.3.1 Jacket Legs

In Table 6-6 the minimum fatigue life of the legs without grinding are summarised.

Table 6-6: Minimum fatigue life for legs

Load Condition	Side	Beam	Section	SN Curve	min. Fatigue Life
FLS	A	AQ30L	3	D	56
FLS	B	BP30L	3	D	55
FLS	P	BP30L	3	D	55
FLS	Q	AQ30L	3	D	56

In Figure 6-7 the life time of the main legs for SN-curve D are depicted. Lifetimes of the braces should be ignored, which are given in subsection 6.1.3.2 for the correct SN-curve. The given values in the image are the worst lifetimes of a component and can be from any stress points in the element, at sub-element ends inner stress points, inside and outside of the element.

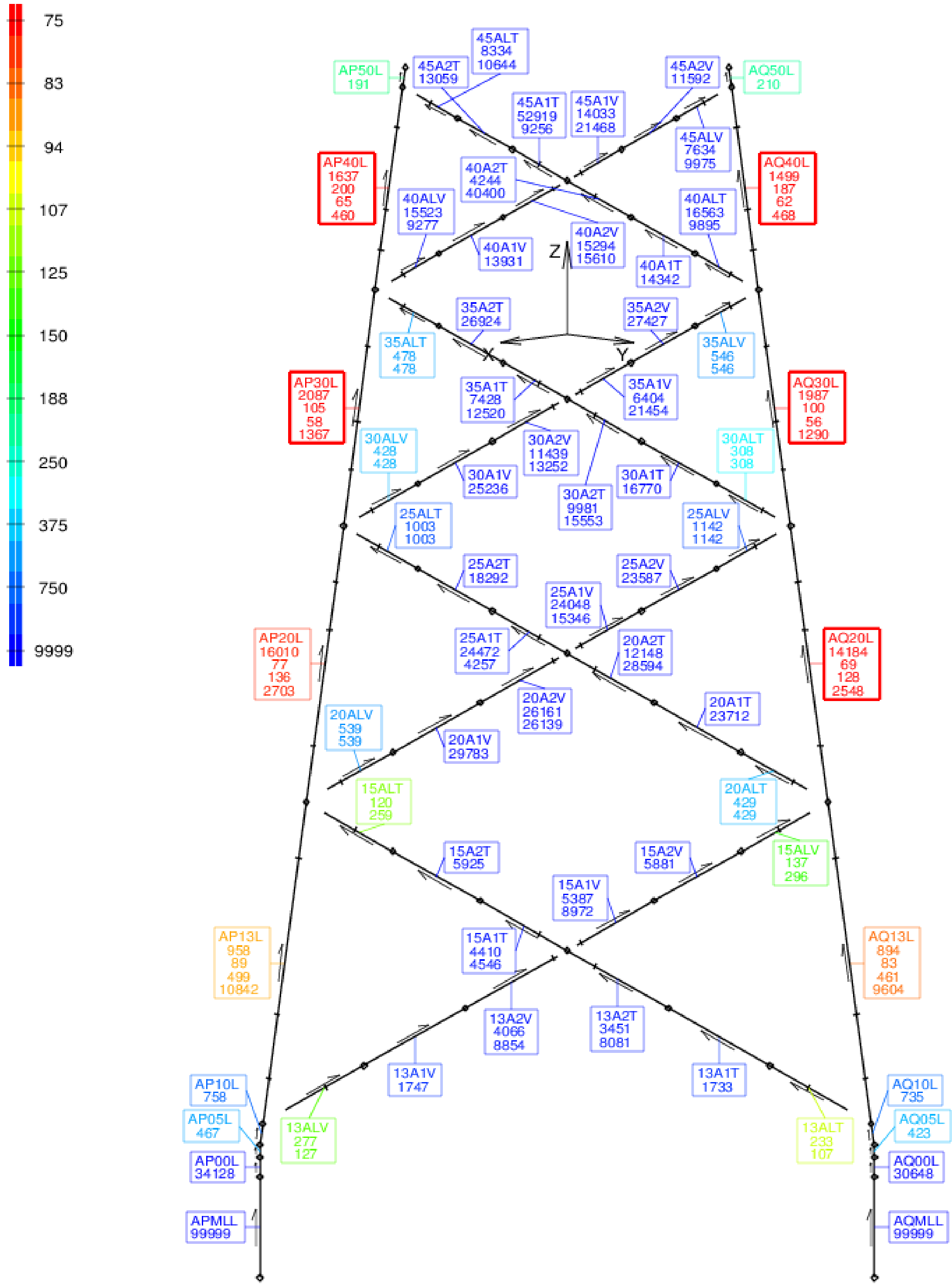


Figure 6-7: Fatigue Live for SN-curve D to be considered for the jacket legs, Jacket Side A

In both upper levels of the jacket the fourth weld between section 3 and 4 of the leg has to be grinded to have a sufficient design life time of 75 years (Design Fatigue Factor DFF= 3 with a target lifetime is 25 years). In level 2 of the jacket the third weld between section 2 and 3 has to be grinded. Figure 6-8 shows the locations.

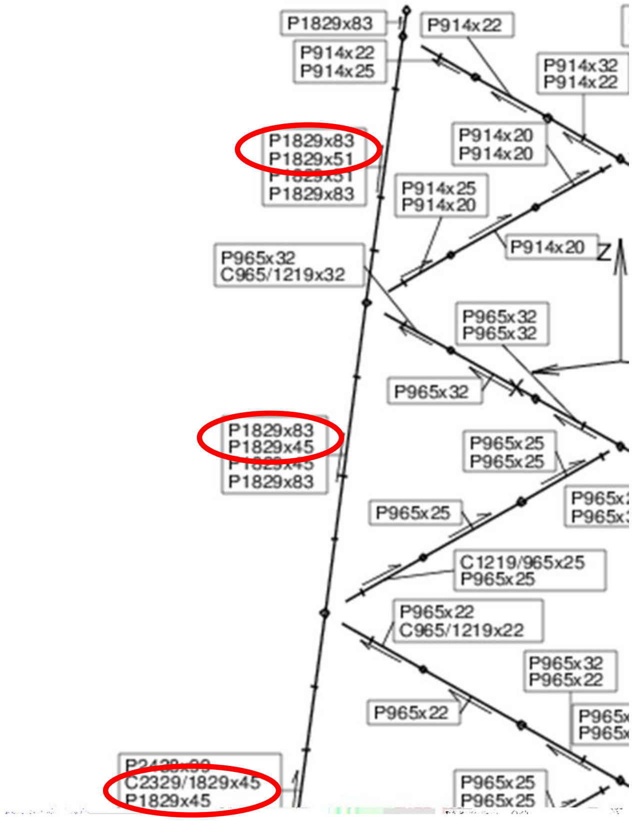


Figure 6-8: Grinded Welds of the Jacket Leg

6.1.3.2 Jacket Braces

In Table 6-7 the minimum fatigue life of the braces with and without grinding are summarised.

Table 6-7: Minimum fatigue life for braces

Load Condition	Side	Beam	Section	SN Curve	min. Fatigue Life without grinding	min. Fatigue Life with grinding
FLS	A	13ALT	2	F3	8	107
FLS	B	13BLT	2	F3	8	110
FLS	P	13PLT	2	F3	7	93
FLS	Q	13QLT	2	F3	7	90
FLS	A	25ALT	1	F3	86	
FLS	B	25BLT	1	F3	91	
FLS	P	25PLT	1	F3	83	
FLS	Q	25QLT	1	F3	87	

In Figure 6-9 the life times of the braces for SN-curve F3 are depicted. Lifetimes of the leg should be ignored, which are given in subsection 6.1.3.1 for the correct SN-curve. The given values in the image are the worst lifetimes of a component and can be from any stress points in the element, at sub-element ends inner stress points, inside and outside of the element.

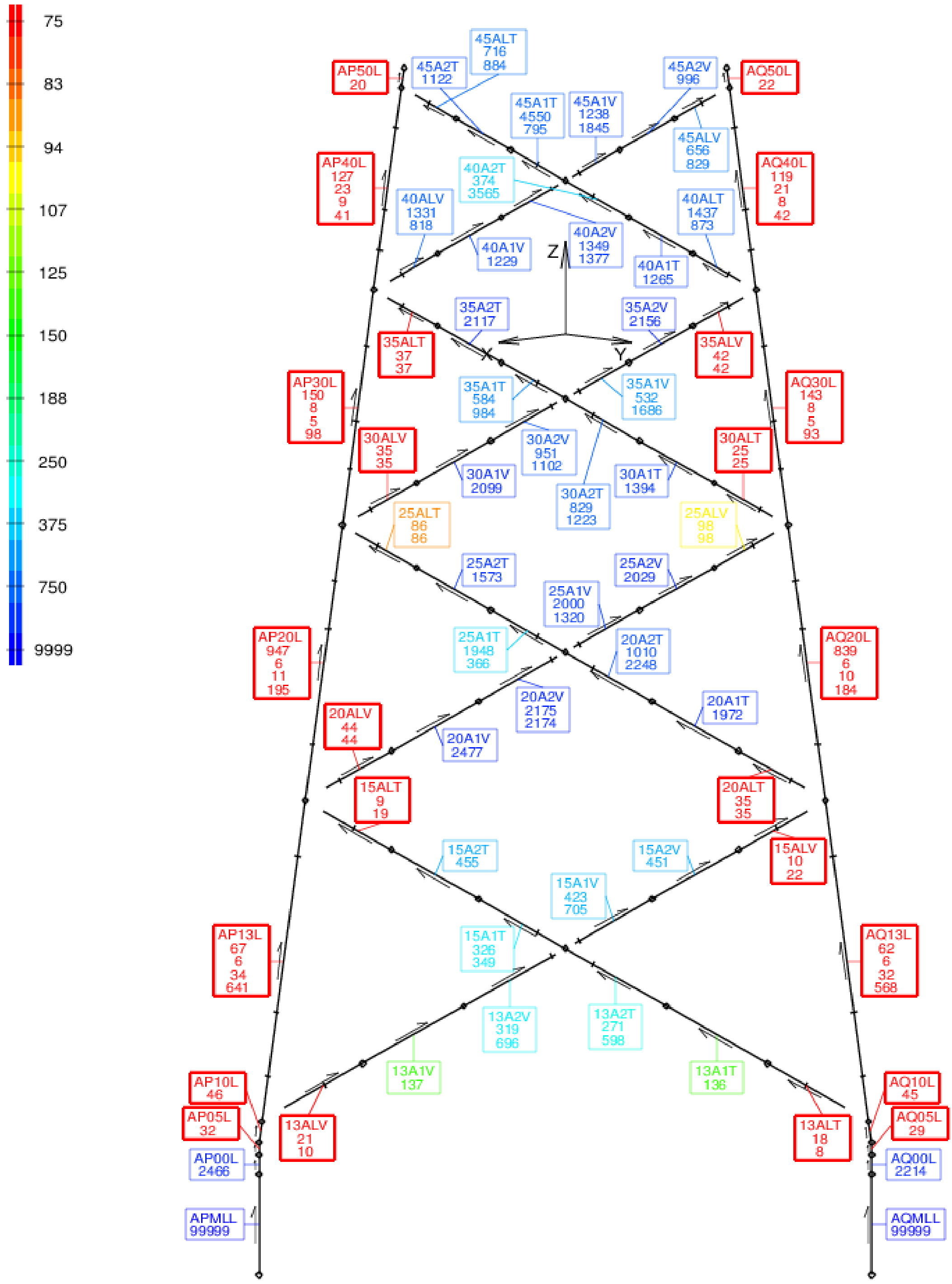


Figure 6-9: Fatigue lives for SN-curve F3 to be considered for the braces, jacket side A

The welds in the x-brace ends in level A (lowest) and C as well as the lower end of level 2 have to be grinded to have a sufficient design life time of 75 years (Design Fatigue Factor DFF= 3 with a target lifetime is 25 years). The locations are shown in Figure 6-10.

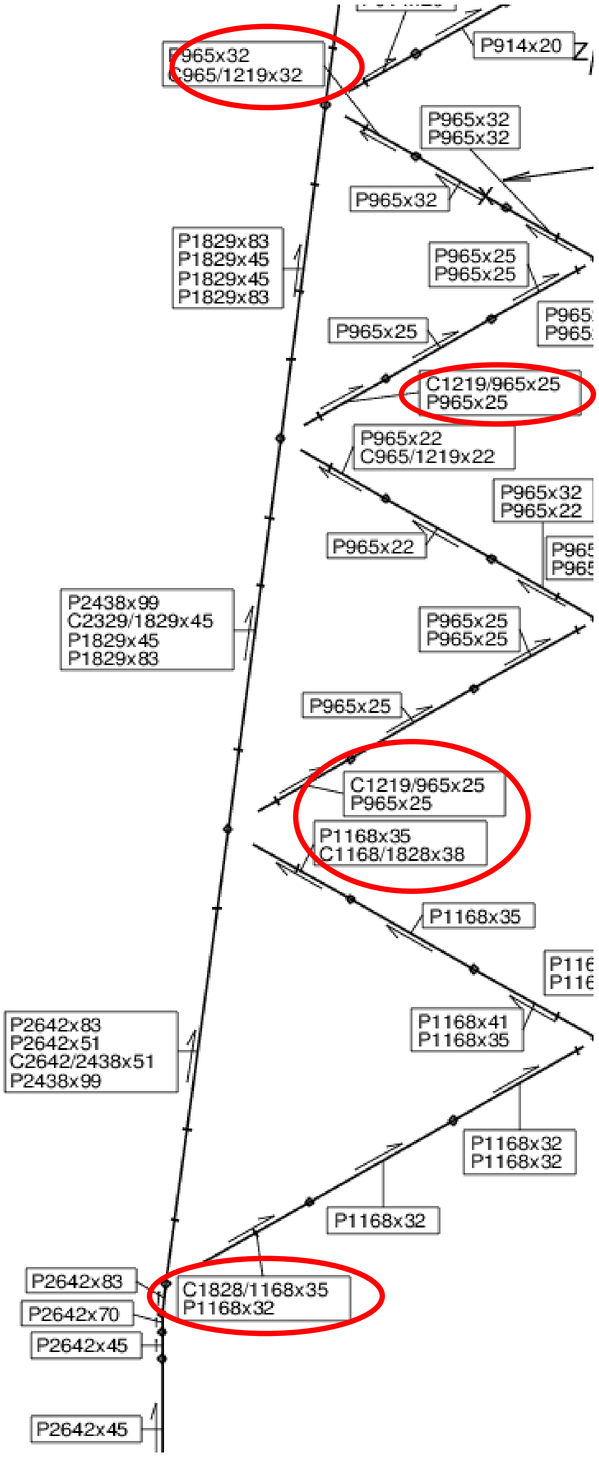


Figure 6-10: Grinded welds of the jacket braces

6.1.3.3 *Jacket Joints*

In Table 6-8 the minimum fatigue life of the tubular joints without grinding are summarised. The joints in the first 4 rows have a fatigue live lower than 75 years and have to be grinded; the other joints of this jacket side have a sufficient fatigue life for the joints without grinding (see Figure 6-11).

Table 6-8: Minimum fatigue life for tubular joints

Row	Load Condition	Side	Joint	SN Curve	has to be grinded	min. Fatigue Life without grinding
1	FLS	A	30A2T	T	x	27
2	FLS	B	35B1T	T	x	27
3	FLS	P	35P1T	T	x	27
4	FLS	Q	30Q1V	T	x	28
5	FLS	A	45ALV	T	-	83
6	FLS	B	45BLV	T	-	84
7	FLS	P	45PLT	T	-	83
8	FLS	Q	45QLV	T	-	83
9	FLS	all	other	T	-	>75

In Figure 6-11 the lifetimes of the tubular joints for SN-curve T are depicted. The given values in the image are the worst lifetimes of a joint and can be from any stress points along the welding curvature at the inside and outside of the brace or chord side of the joint.

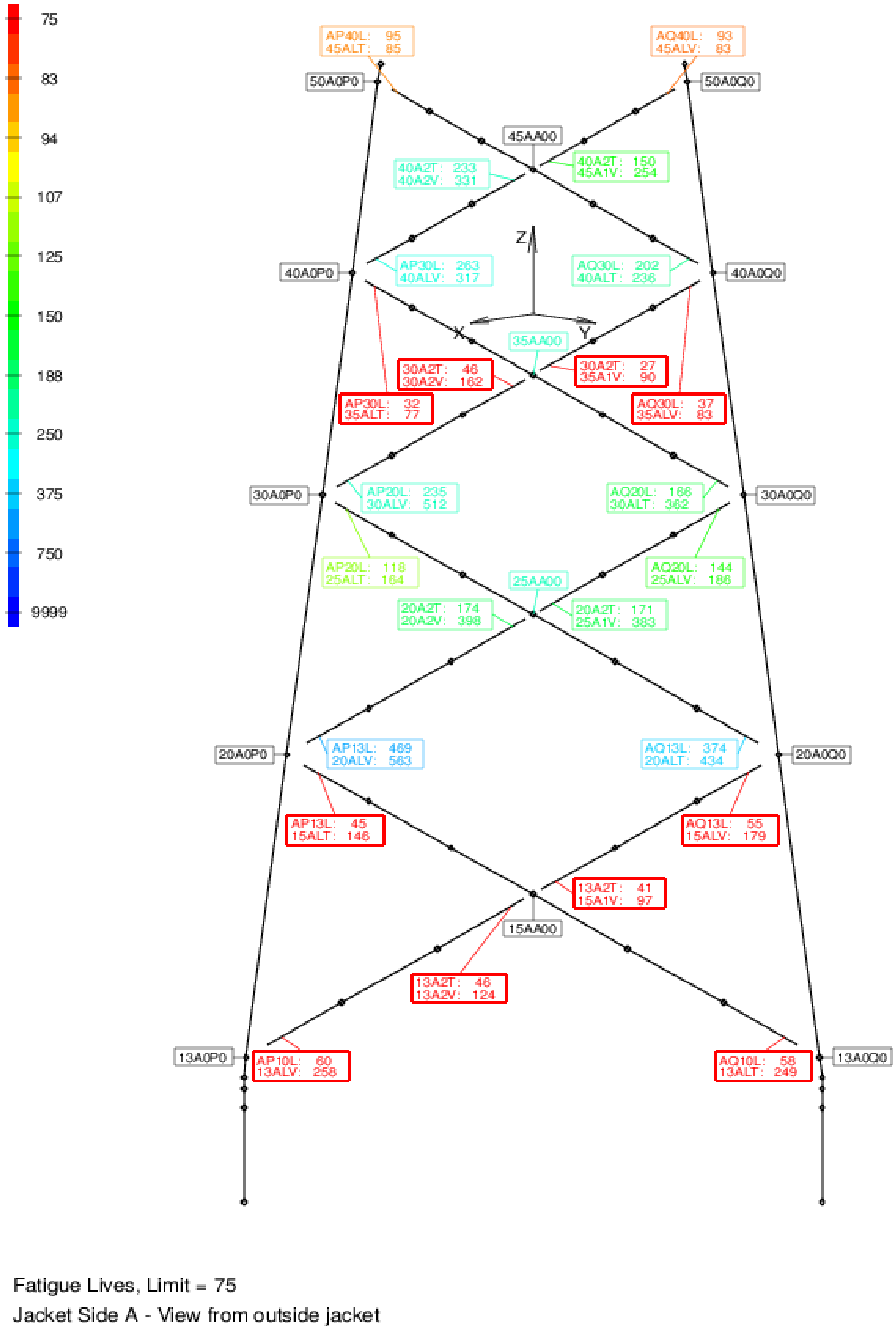


Figure 6-11: Fatigue lives for SN-curve T to be considered for the joints, jacket side A

The K-joints in lowest level A, the K-joints of level C and corresponding X-joints have to be grinded to reach a sufficient design life time of 75 years (Design Fatigue Factor DFF= 3 with a target lifetime is 25 years). The locations are shown in Figure 6-12.

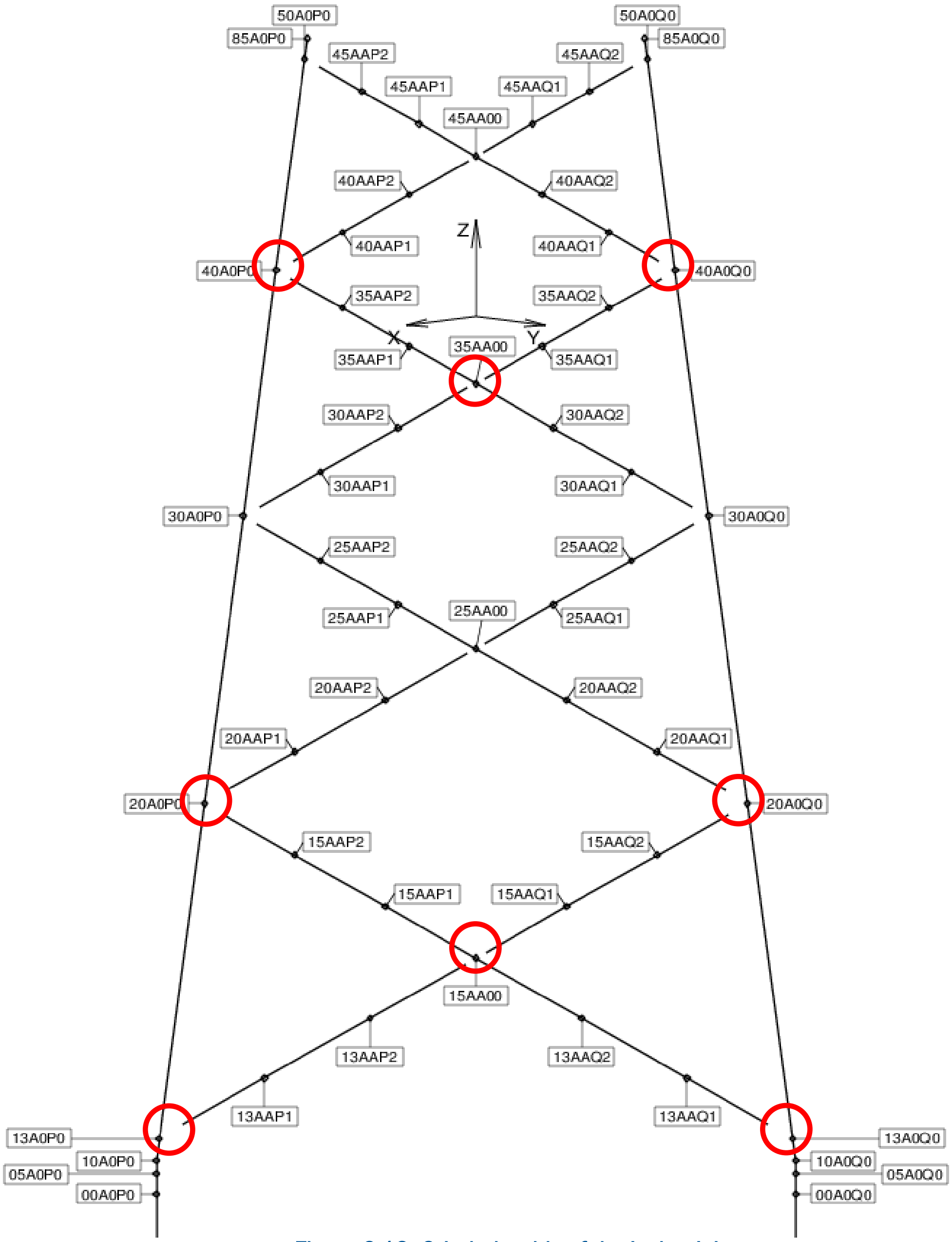


Figure 6-12: Grinded welds of the jacket joints

6.2 Conceptual Design

6.2.1 Operational Characteristics of the 20 MW RWT

As it was shown at the preliminary analysis of the onshore version of the 20MW RWT [5], the Campbell diagram showed a resonance between the first fundamental model of the wind turbine and the blade passing frequency around wind speed 8m/s. For the full model with a jacket foundation, lower natural frequencies of the entire system are expected. The natural frequencies of the full system with the preliminary jacket are shown in Table 6-3. Currently, the focus is on the first natural frequency of the turbine with the jacket. Figure 6-13 illustrates the revised Campbell diagram of the 20MW RWT with the preliminary jacket design.

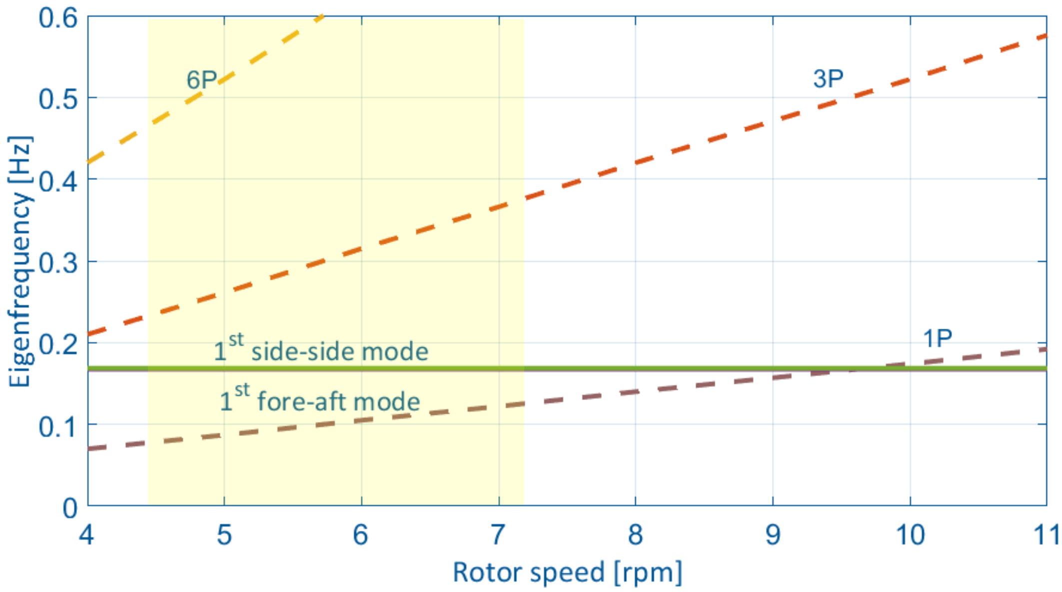


Figure 6-13: Campbell diagram of the 20MW RWT with preliminary jacket design.

As it can be seen from the figure, the resonance between the first natural frequency of the structure and the rotor 1P and 3P frequencies occurs at rotor speed of 9.6 rpm. However, the rotor speed range is between 4.45 rpm and 7.13 rpm (yellow highlighted) during the cut-in and cut-out speeds. A problem with 3P excitation is also avoided due to the significant lower natural frequency of the full offshore model.

6.2.2 Steady Operational Loads

The first analysis of the wind turbine is the steady operational loads which are determined from a uniform steady wind filed. This calculation generates the quasi-static wind loads caused by a uniform wind field as a function of wind speed. The steady loads calculation is an early analysis in the design of the wind turbine to ensure that both the structural model and the control system are working properly. The tower base steady moments at both fore-aft and side-side directions are shown in Figure 6-14.

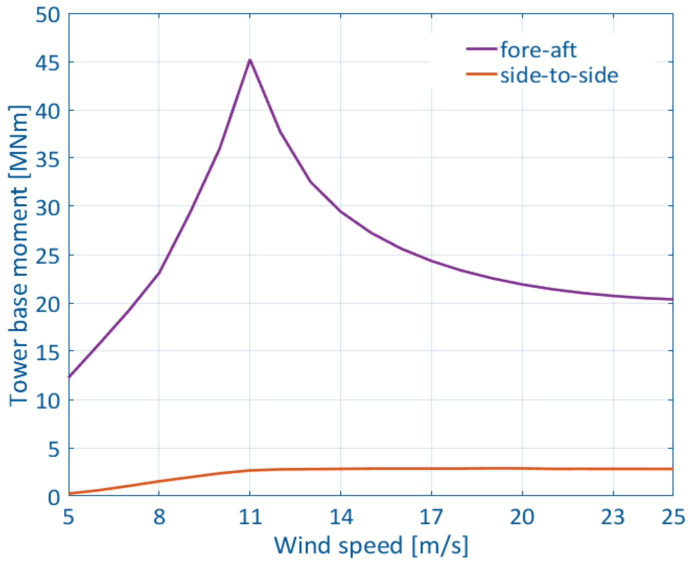


Figure 6-14: Average tower base moments in fore-aft and side-side directions with respect to the wind speed

In addition the rotor speed curve, power curve and mean blade pitch curve varying with wind speed are depicted in Figure 6-15.

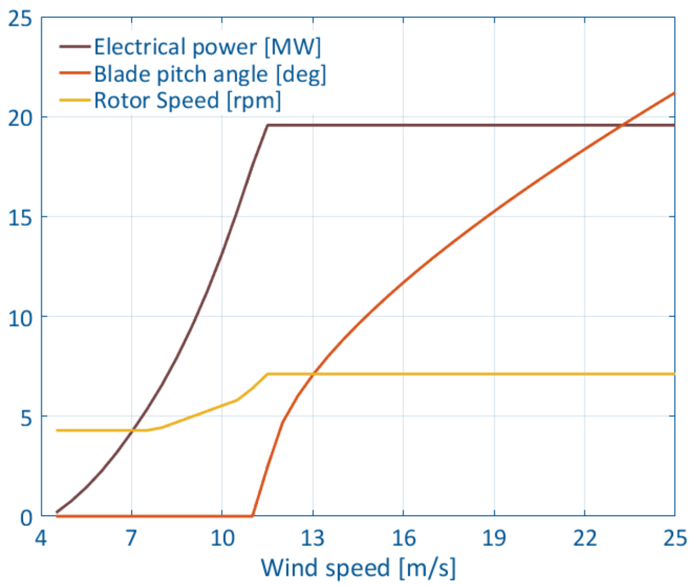


Figure 6-15: The power curve, blades pitch angle and rotor speed as a function of wind speed calculated by the steady analysis

6.2.3 Fatigue Strength (FLS)

The conceptual design is focused on the fatigue strength, because the preliminary design has shown that the fatigue analysis is the main driver for the 20 MW jacket design.

The fatigue assessment is performed very similar to the preliminary design, but with a more accurate and integrated load simulation model to determine the hot spot stresses. Assumptions for the applied SN curves and type of weld are identical. Stress concentration factors are geometry specific calculated. The jacket legs are designed for SN-curve D, the braces conservatively for SN-curve F3 and the joints for SN-curve T according to DNV [9]. A higher detail category can be used by grounding the welds (see chapter 6.1.3.).

The member stresses for the check of the fatigue strength are obtained from the integrated loads analysis between Bladed and ROSA. Thus accurate loads from wind and waves under due consideration of the dynamic response of the support structure is taken into account.

For the assessment of the influence of several design variations a reduced mini setup of the WTG loads with only 10 m/s and 12 m/s mean wind speed and correlating waves and probabilities is used as a representative set of load cases. The final resulting fatigue analyse has been done with the complete load case setup.

Analysed conditions are for example the wave loads. It has been shown that the influence of the wave loads is marginal and that the damage of the jacket element is due to the WTG loads only. Furthermore investigations concerning fatigue strength increase due to massive wall thickness changes, different bottom width and a general stiffer transition piece are investigated.

The analysis of a strengthened jacket with significant wall thickness increase shows that the lifetimes of the jacket are increased only little. Due to the increase of the wall thicknesses and the resulting wall thickness leaps the SCF factor increased very unfavourable. Hence a better lifetime due to a more massive jacket structure is compensated due to the worse SCF values. Similar results have been found for the jacket with changes of the bottom width or TP stiffness increase. Both solutions show only marginal improvements of some FLS results, but do not results in a sufficient overall design.

The tower length was extended for the onshore wind turbine extrapolation for the preliminary onshore conditions, because they expect a first global frequency of 0.18 Hz, which would move towards 0.20 Hz if a shorter tower was used and combined with a jacket (see [5]). The assumption that the system stiffness of jacket and WTG will be towards stiff and therefore the system has to be softer has not been confirmed. The natural frequency analysis (see 6.1.1) has shown that the 1st natural frequency is approximately 0.163 Hz and an extension of the tower length is not necessary. It is even negative for the support structure design resulting in larger overturning moments.

The present WTG loads are too conservative. For example the tower length can be reduced. The distance between the bottom edge of the tower and the blade tip is approximately 16m, a clearance of about 6-8m could be sufficient. Therefore the tower length can be shortened by around 8-10m, a reduction of 6-7% of the tower length. Due to that the dynamic moments of the WTG at interface will be reduced.

The fatigue analysis using integrated load simulations has shown that the life time of the jacket at this stage of the design process is not sufficient for a design life of 25 years (including DFF of 3 =75 years). Differences in the behaviour of these structural elements due to the preliminary or the conceptual loads are demonstrated in chapter 6.2.4.

The fatigue life in the lowest level A is too low, the fatigue life in level B is critical, but with post-weld improvement such as grinding of the welds the legs will have a sufficient life time for level B. Figure 6-16 shows the designated parts of the jacket. Most of the joints of the lowest three levels have too low lifetime in general.

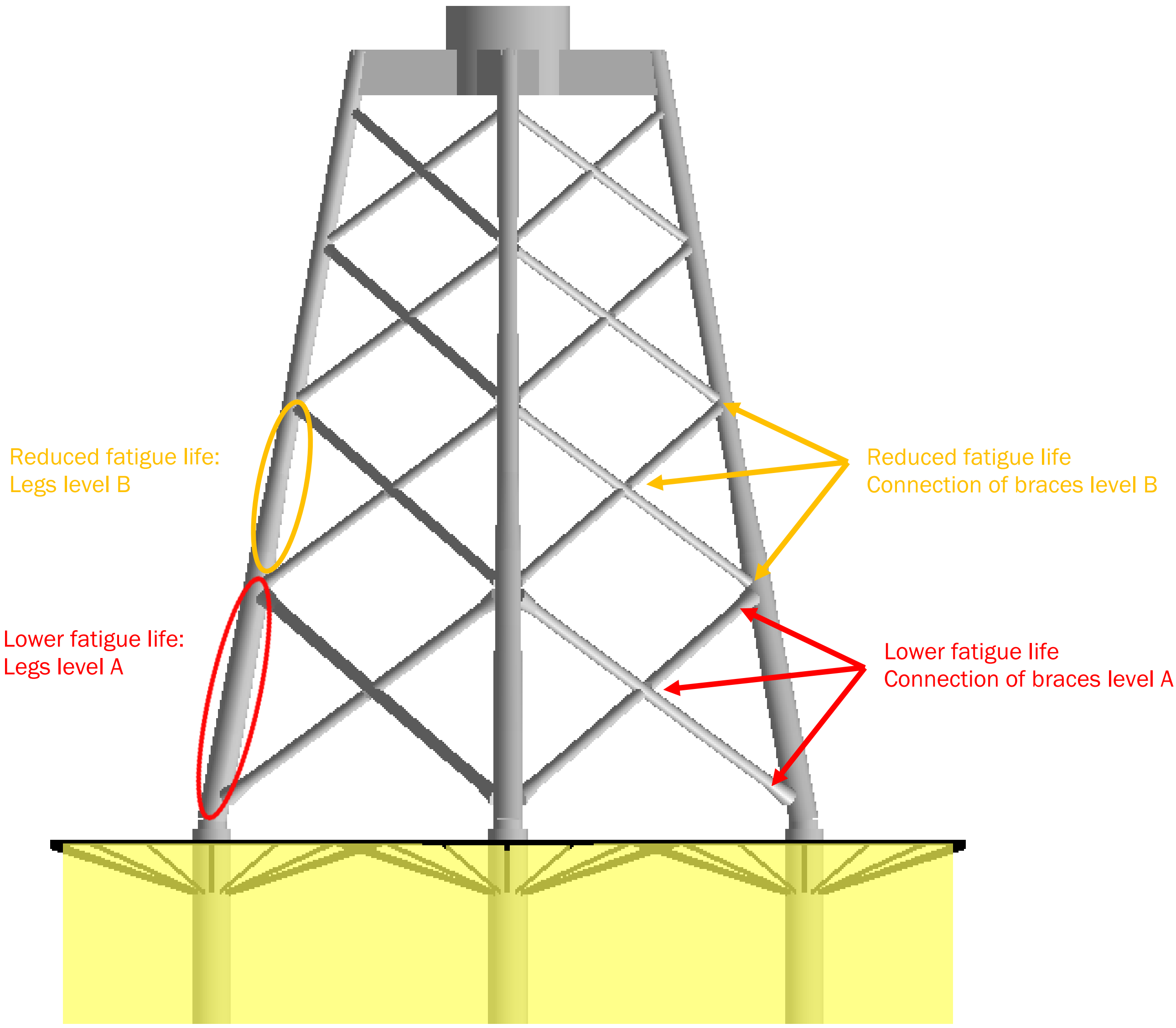


Figure 6-16: Fatigue lives of the jacket beams with less than 25 years

Table 6-9: Damage equivalent moments at interface (tower bottom)

Description		FLS Equivalent Moments	Fatgiue Life [a] Curve D - Legs			Fatgiue Life [a] Curve F3 - Braces			Fatgiue Life [a] Curve T - Joints		
		[MNm]	AP13L	AP20L	AP40L	13ALV	25A1T	35ALT	15ALT	30A2T	45ALV
Predesign		165	88	77	65	10	365	44	146	37	86
Conceptual Design	TFF 12 m/S 24 mini setup, predesign	175	4	52	62	0	4	13	10	13	52
	mini setup without waves, predesign	175	4	52	62	0	4	24	10	16	52
	TFF 12 m/S 24 mini setup, reinforced jacket	175	12	580	240	3	19	351	1	2	56
	mini setup without waves, reinforced jacket	173	12	572	288	3	19	218	1	7	55
	Iteration 02, complete setup	173	1	31	54	0	1	4	2	3	20

The damage equivalent bending moments at interface (Table 6-9) show, that the loads of the offshore WTG is only 6% higher compared to the preliminary loads from the land version WTG, but the amplification of leg and brace elements is significant higher, there is a resonance between the

WTG and the jacket, which is not excited using the preliminary loads, for further details see chapter 6.2.4.

6.2.4 Excitation of the Jacket using Preliminary and Conceptual Loads

To analyse the discrepancies between the preliminary onshore loads (NTUA) and the conceptual offshore loads of (Uni Oldenburg) the amplitude spectra of one main leg (AQ13L) and one brace element (15ALV) in the lowest level A of the jacket structure are compared. The time series with mean wind speed of 10m/s and 12m/s are taken into account.

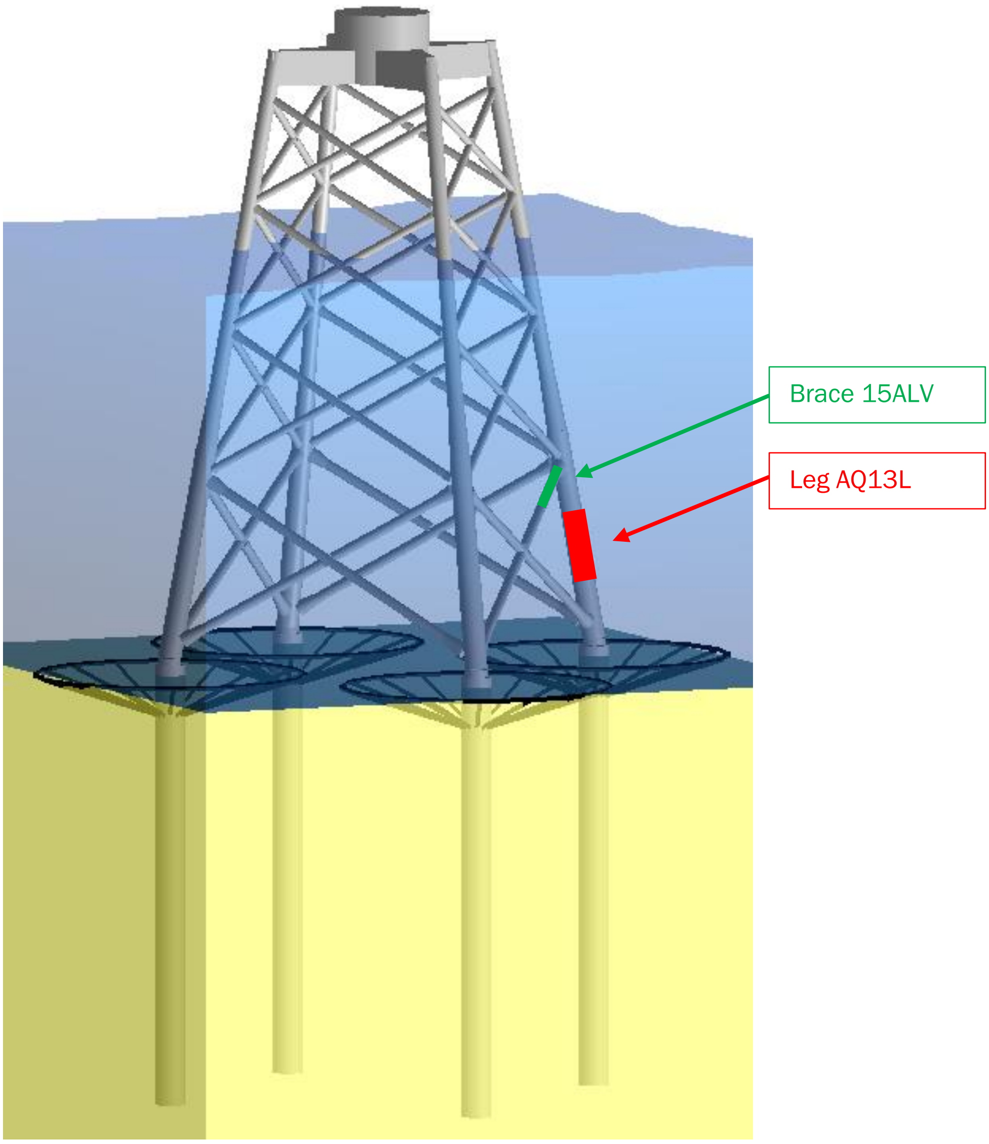


Figure 6-17: Positon of analysed main leg and brace member

6.2.4.1 Main Leg AQ13L

Figure 6-18 shows the spectrum from the preliminary loads for the out of plane bending moment V [kNm]. In Figure 6-19 the spectrum of the in plane bending moment W [kNm] is shown. The maximum amplitudes are approximately 200 kNm for both moments.

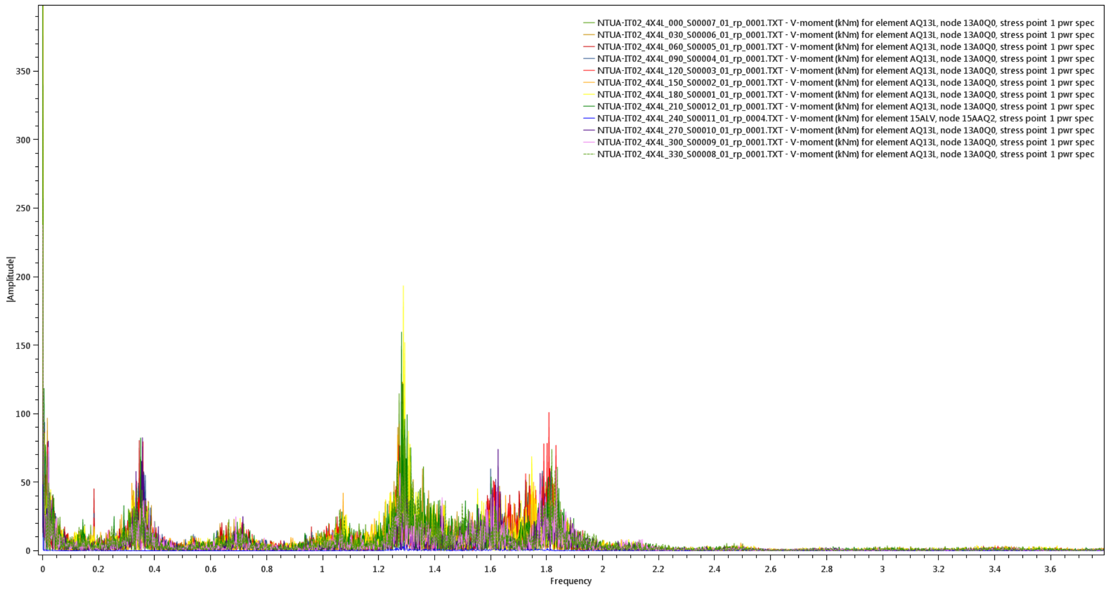


Figure 6-18: Amplitude spectrum leg – preliminary onshore loads – out of plane bending moment

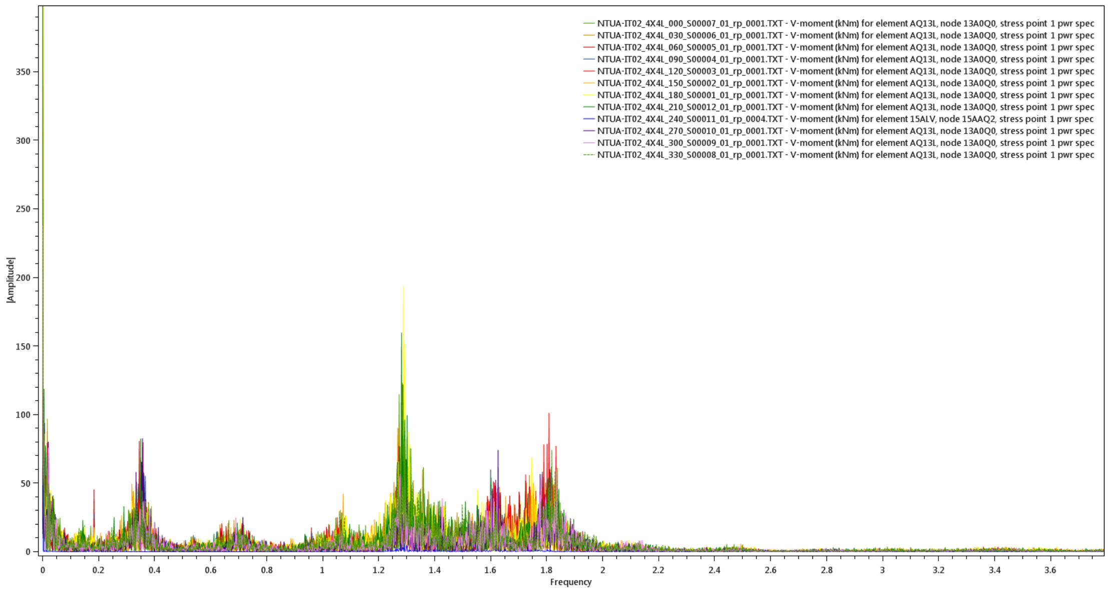


Figure 6-19: Amplitude spectrum leg - preliminary onshore loads – in plane bending moment

Figure 6-20 shows the spectrum from the conceptual loads for the out of plane bending moment V [kNm]. In Figure 6-21 the spectrum of the in plane bending moment W [kNm] is shown. The maximum amplitudes are approximately 240 kNm for the out of plane bending moment and 620 kNm for the in plane bending moment.

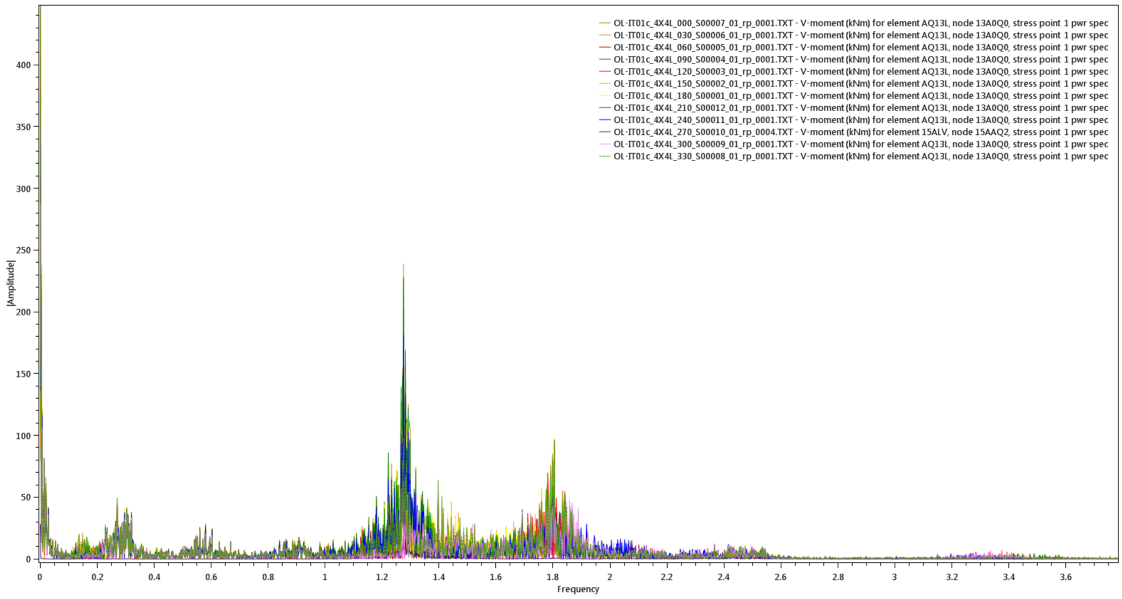


Figure 6-20: Amplitude spectrum leg - conceptual offshore loads – out of plane bending moment

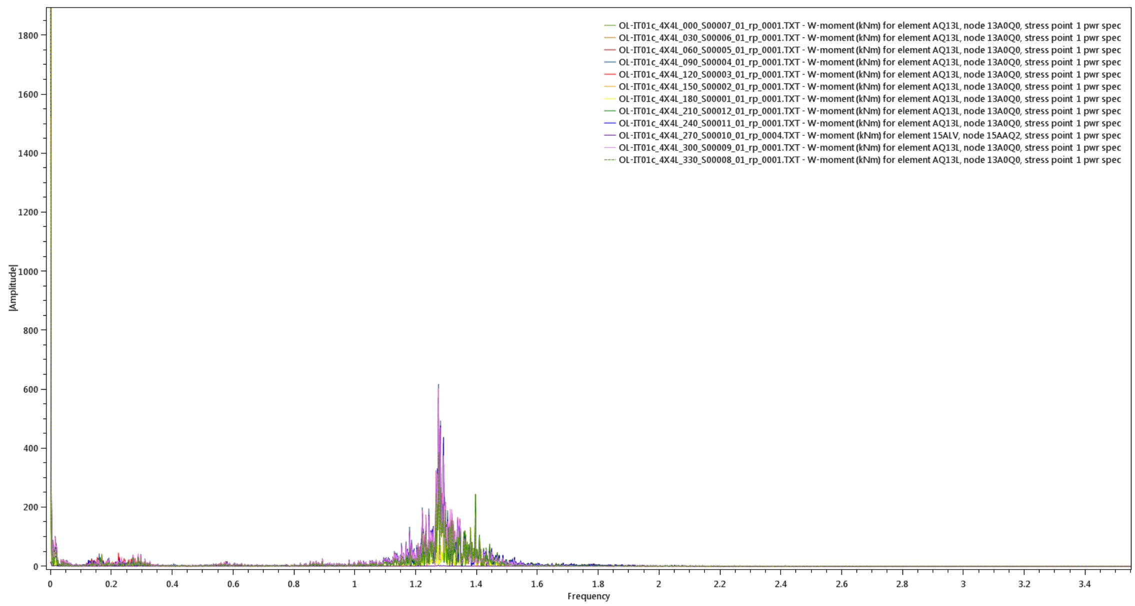


Figure 6-21: Amplitude spectrum leg - conceptual offshore loads – in plane bending moment

The spectra show that there are quiet high amplifications of the in plane and out of plane bending moments at 1.28 Hz, which clearly dominate in conceptual design using the integrated offshore model. For the frequency of 1.28 Hz there is a resonance between the 12p excitation and a coupled blade and support structure mode (12p at a rotor speed of 6.3 rpm) and another resonance occurs near the rated rotor speed (7.13 rpm) and 6p. The Campbell Diagram is given in Figure 6-22.

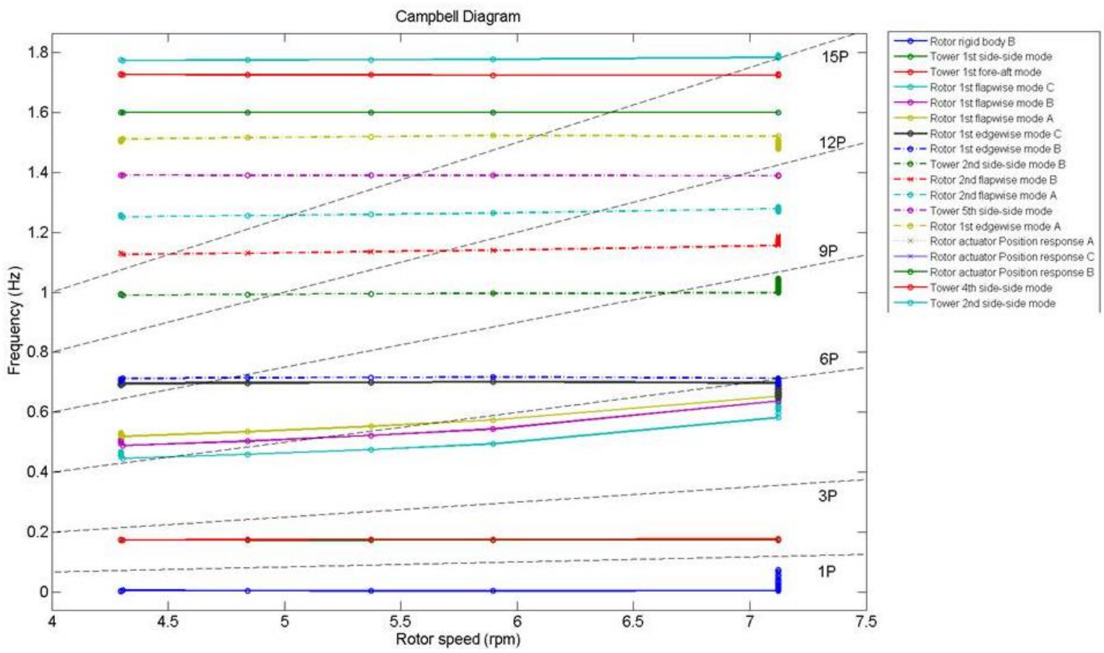


Figure 6-22: Campbell diagram

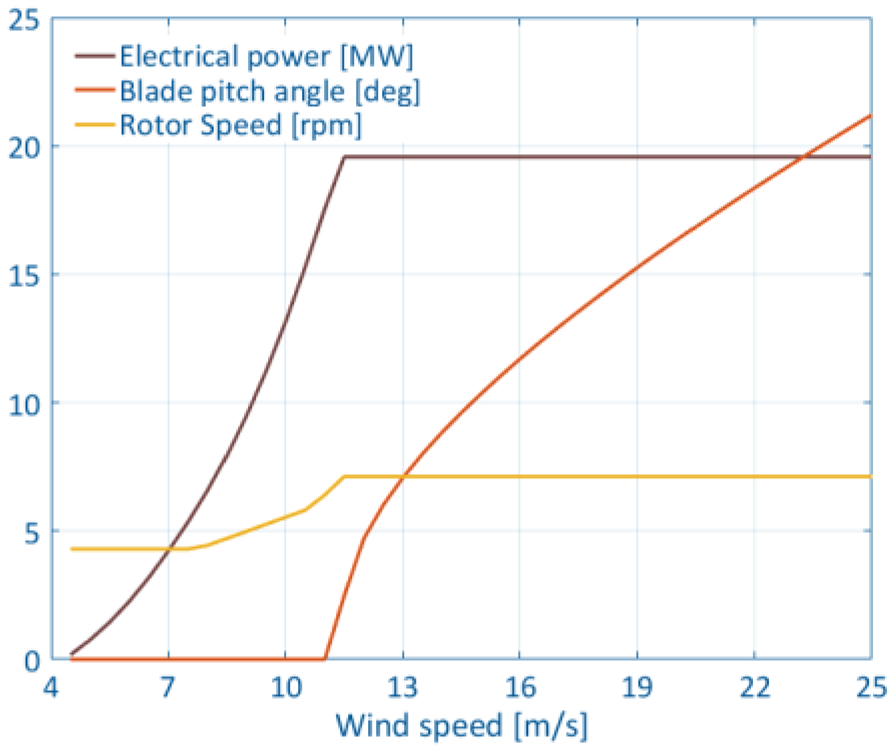


Figure 6-23: Corresponding rotor speed at different wind speeds

In Figure 6-24, Figure 6-25 and Figure 6-26 the amplitude spectra of time series showing largest differences are compared. For the out of plane bending moment the maximum amplitudes are:

- preliminary design: 1.2649 Hz Amplitude: 110 kNm
- Conceptual design : 1.2685 Hz Amplitude: 492 kNm

The amplitudes of the in plane bending moments are:

- preliminary design: 1.2649 Hz Amplitude: 45 kNm
- Conceptual design: 1.2685 Hz Amplitude: 582 kNm

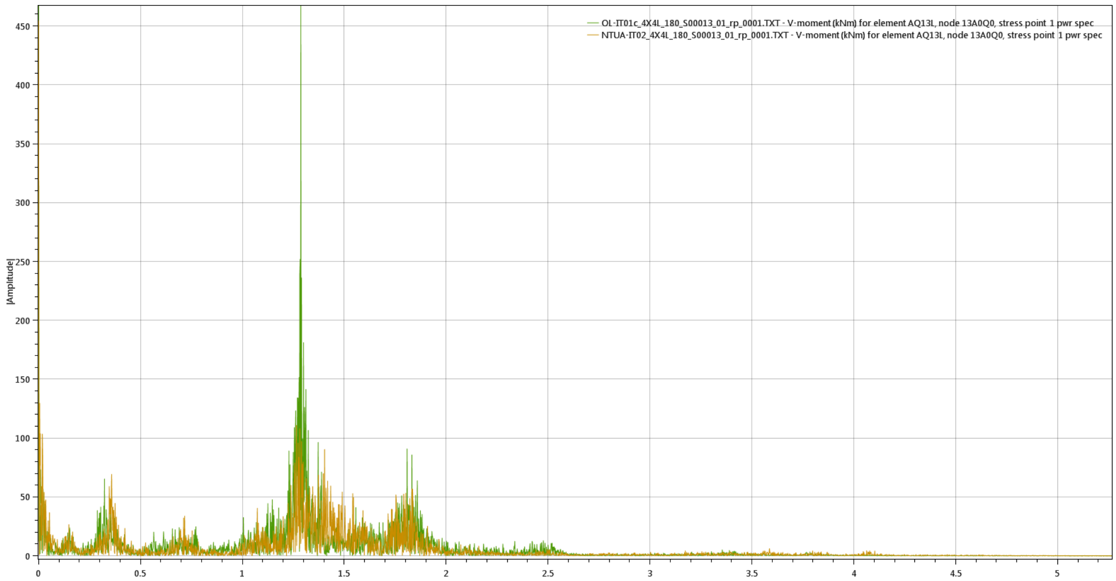


Figure 6-24: Comparison out of plane bending moment

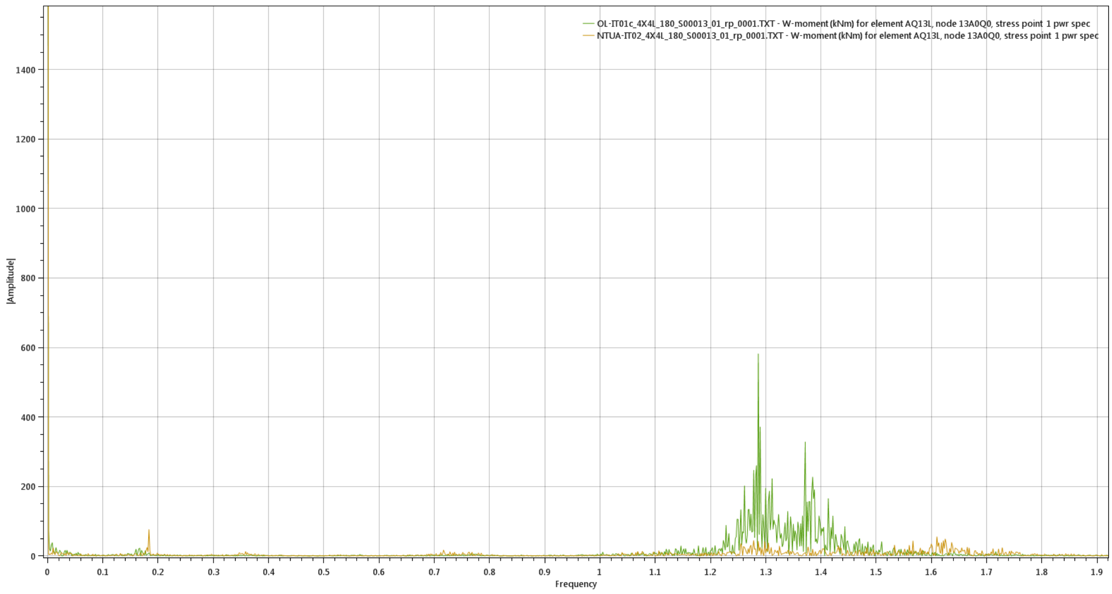


Figure 6-25: Comparison in plane bending moment

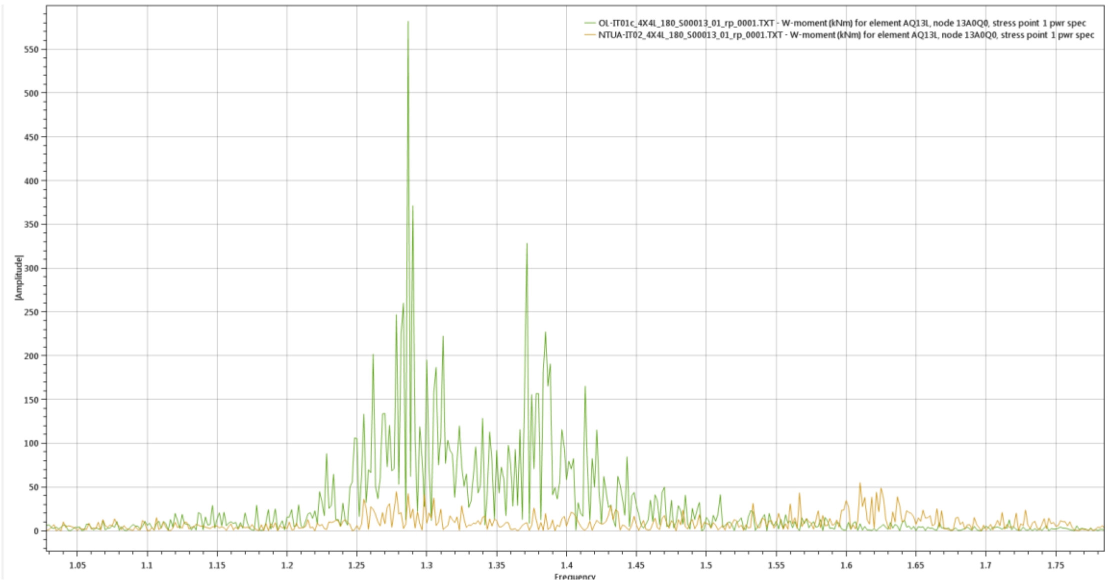


Figure 6-26: Detail in plane bending moment

The amplitudes of the main leg element AQ13L have a large discrepancy with a factor between 4 kNm and 10 kNm for the bending moments and due to the higher loads of the elements the lifetime of these elements is significant reduced for the conceptual design.

6.2.4.2 Brace 15ALV

Figure 6-27 shows the spectrum from the preliminary loads for the out of plane bending moment V [kNm]. In Figure 6-28 the spectrum of the in plane bending moment W [kNm] is shown.

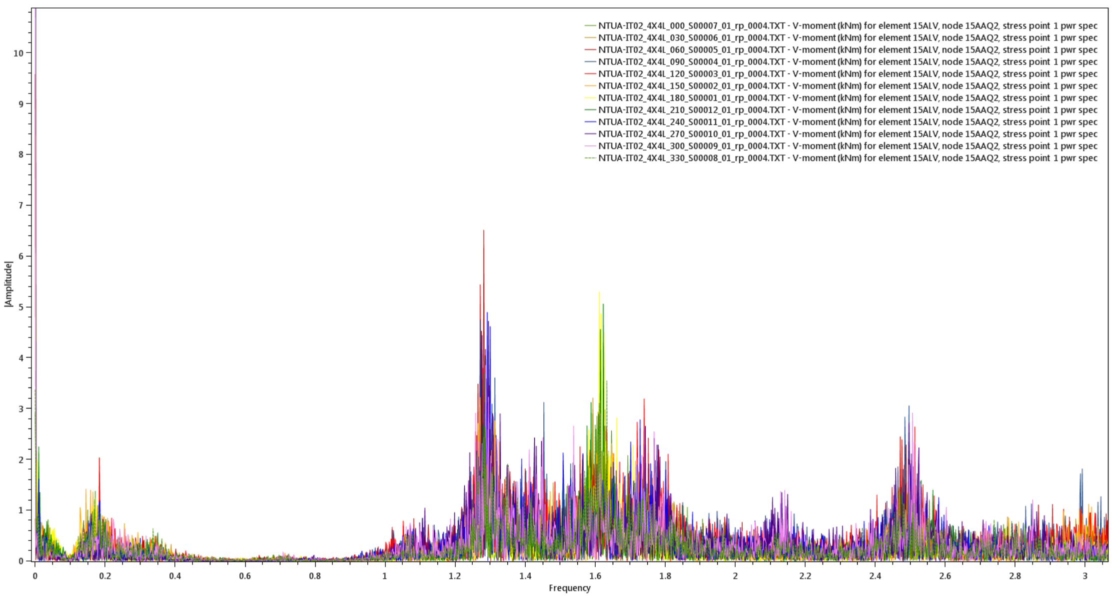


Figure 6-27: Amplitude spectrum brace - preliminary onshore loads – out of plane bending moment

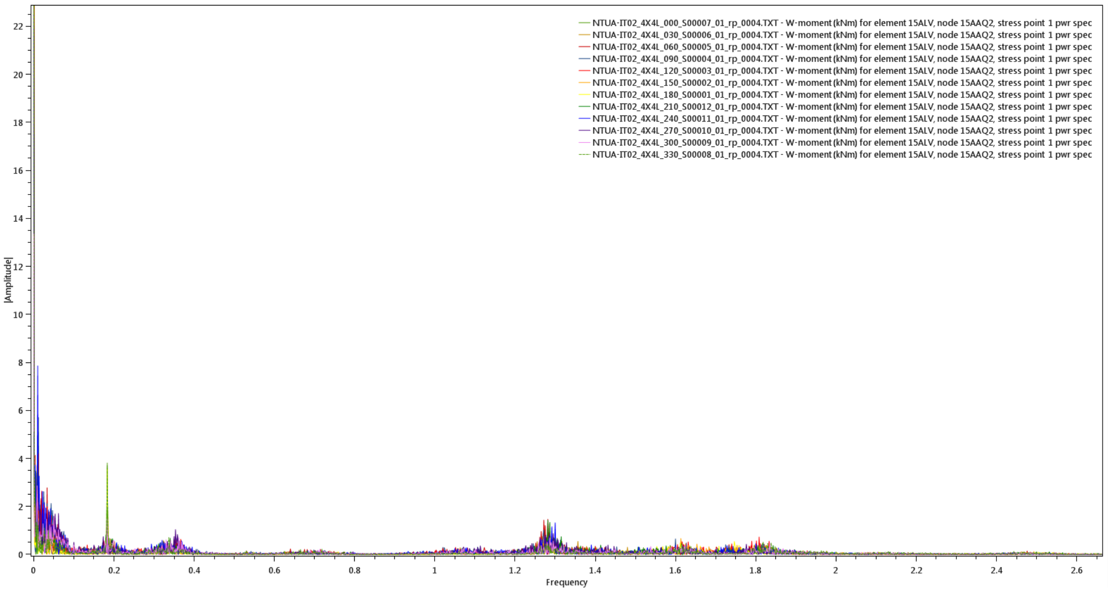


Figure 6-28: Amplitude spectrum brace - preliminary onshore loads – in plane bending moment

Figure 6-29 shows the spectrum from the conceptual loads for the out of plane bending moment V [kNm]. In Figure 6-30 the spectrum of the in plane bending moment W [kNm] for the conceptual loads is shown.

The maximum amplitudes of the preliminary loads are approximately 8 kNm and for the conceptual loads the maximum amplitude for the in plane bending moment is 4 kNm and for the out of plane bending moment is 20 kNm.

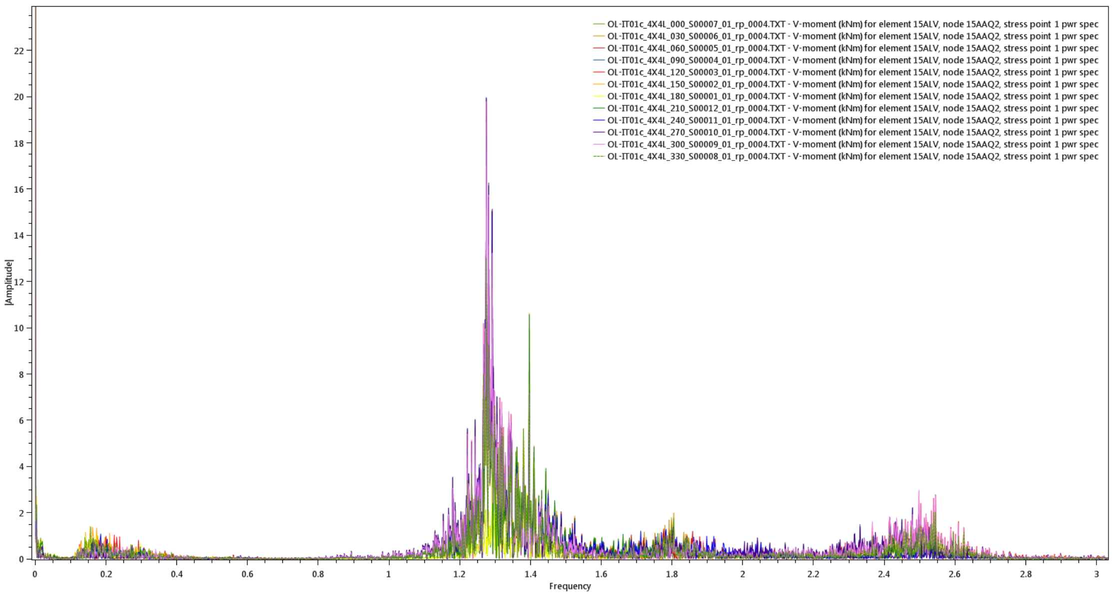


Figure 6-29: Amplitude spectrum brace - conceptual offshore loads – out of plane bending moment

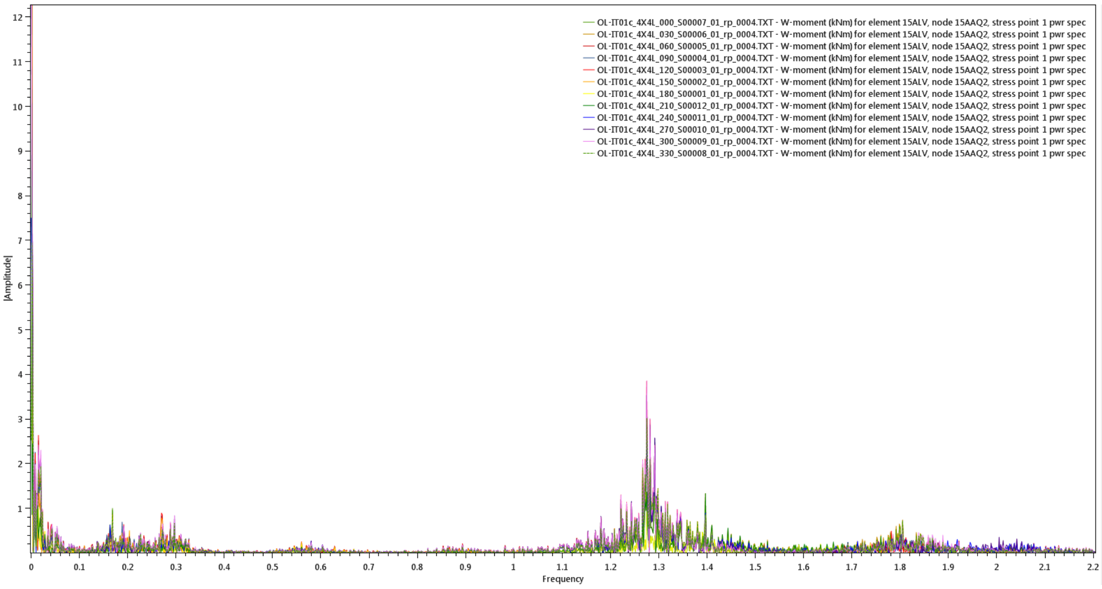


Figure 6-30: Amplitude spectrum brace - conceptual offshore loads – in plane bending moment

The results for the brace show also the different excitations between the preliminary loads and the conceptual loads. These spectra of the conceptual results show a much larger amplitude peak at approximately 1.28 Hz, which correlates with a support structure eigenmode and the 12p excitation of the rotor.

In the following figures the amplitudes of the maximum time series for the brace 15ALV are compared.

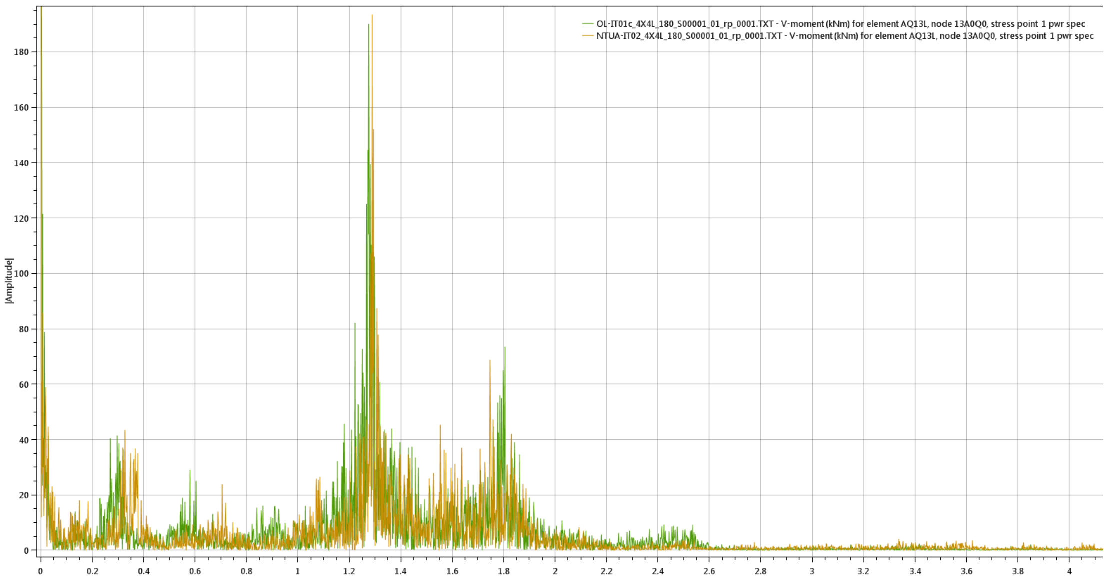


Figure 6-31: Comparison out of plane bending moment

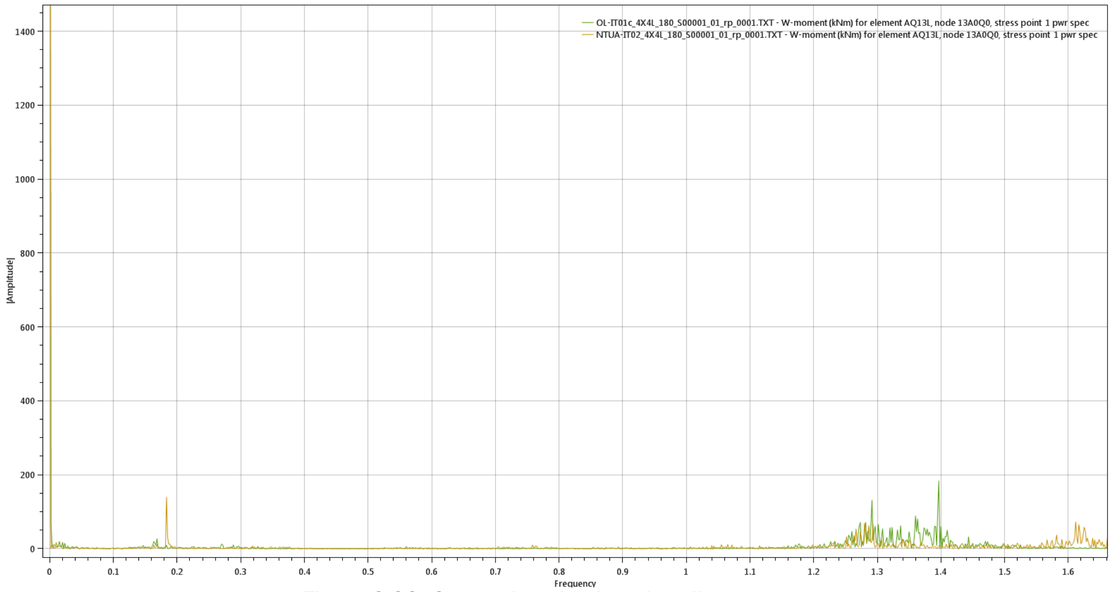


Figure 6-32: Comparison in plane bending moment

The amplitudes of the brace element 15ALV have a smaller difference (factor 2) as the legs. For the out of plane bending moments the amplification is even in the same range.

6.2.5 Mitigation of Local Jacket Vibrations

In chapter 6.2.4 local jacket vibrations due to resonance are evinced. The particular resonance peak at 1.28Hz compared with other frequencies is very large for the braces and lower amplified for the legs. Therefore the jacket has been stiffened with additionally horizontal and diamond braces in the lower levels as highlighted in Figure 6-33. Several configurations have been investigated and the shown version has the largest influence on the resulting fatigue lives. It should be noted that these additional braces mostly reduce the out-of-plane vibration of the braces and also increase the complexity of the X-joints, which in principle requires finite element analysis to derive the SCF more accurately. In plane excitation are not mitigated significantly with these braces.

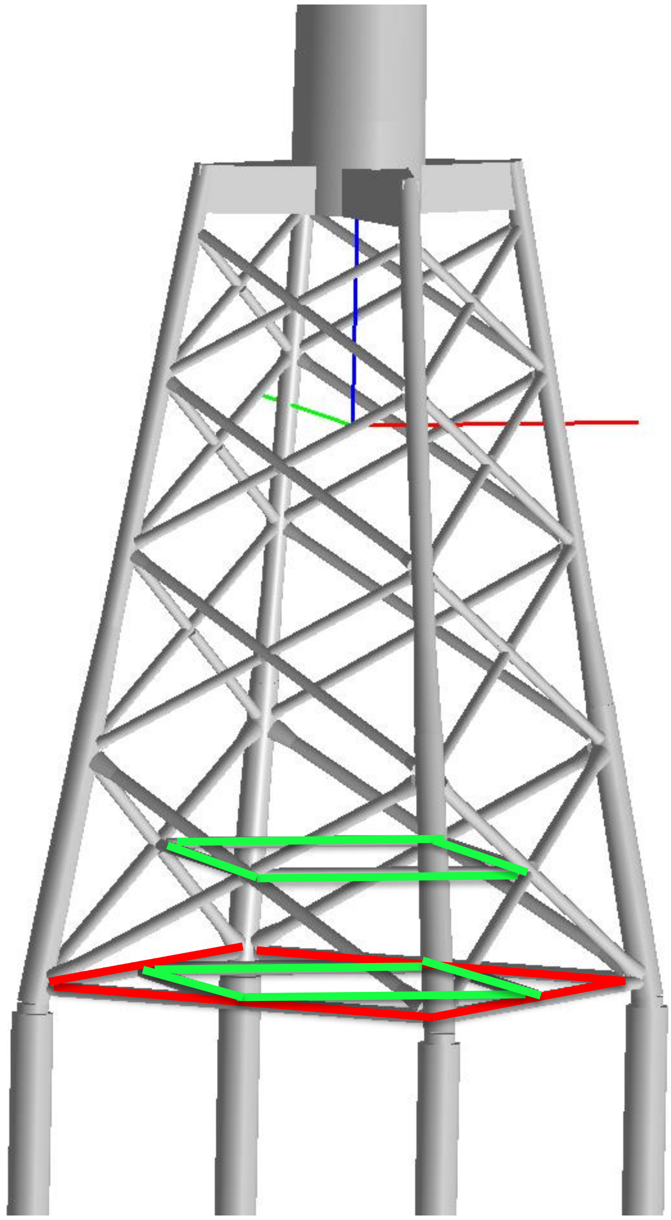


Figure 6-33: Jacket with horizontals and diamond braces

The design study is performed with the conceptual loads initially derived from the predesign jacket geometry without the horizontal and diamond braces. Hence, the wind turbine loads do not reflect the different jacket geometry. More accurately results demand load iterations according to Figure 3-1, but in the current scope of the project these additional load iterations have not been possible. Therefore loads and responses are not consistent for this study and the obtained fatigue lives are inaccurate.

Consistency of loads and responses is in general important for accurate results. Especially since the wind turbine and support structures are becoming very large and natural frequencies become rather low. According to the NFA results of the integrated model most natural frequencies are below 2 Hz and are prone to 1p to 12p excitations from the rotor. Therefore close cooperation between the wind turbine design and foundation design should be intended.

The different dynamics of the jacket structure can be demonstrated from the natural frequency analysis. The local bending modes of the braces (6^{th} to 8^{th} mode of the structure) are compared for the preliminary jacket and the stiffened conceptual jacket in Figure 6-34 and Figure 6-35.

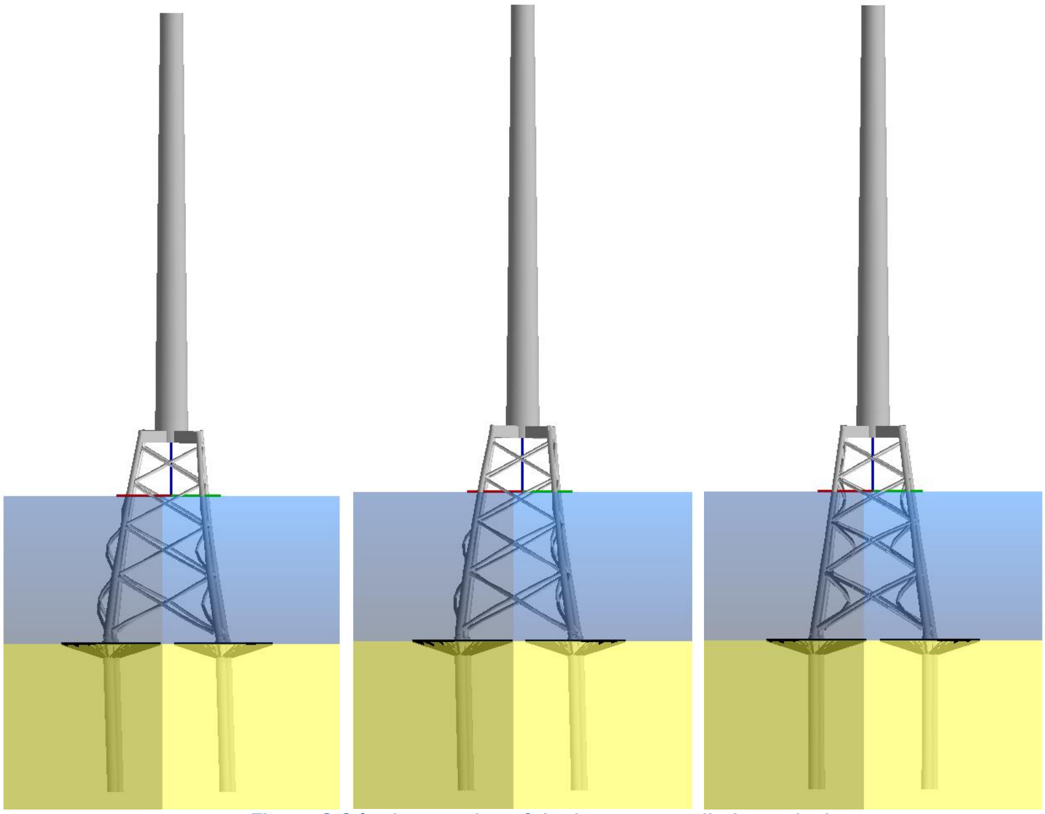


Figure 6-34: eigenmodes of the braces – preliminary design

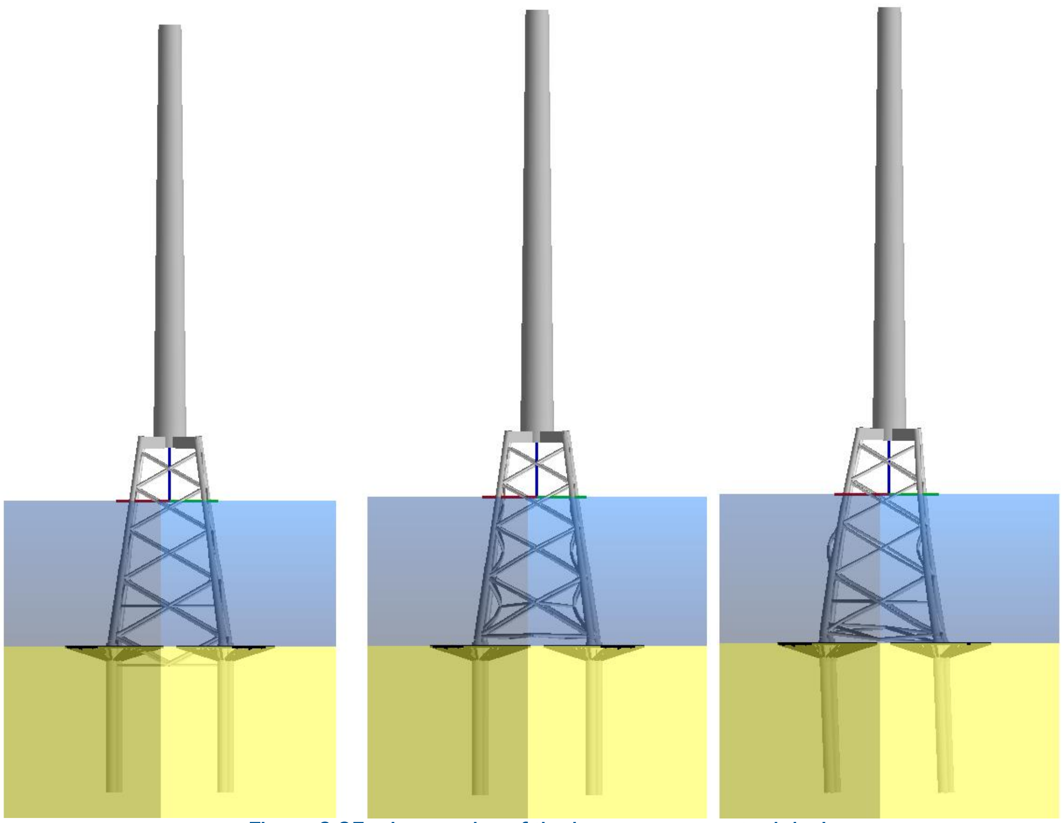


Figure 6-35: eigenmodes of the braces – conceptual design

The fatigue analysis was conducted with the predesign jacket and with the strengthened jacket. The lifetime of the original jacket shows a minimum lifetime for the upper legs (level C – D) of 51 years, in level B of 20 years and in the lowest level A the lifetime is below 10 years. The legs of the reinforced jacket have a nearly doubled lifetime, but in the lowest level A the calculated lifetime of is still not sufficient and below the design lifetime.
The same occurs in the braces, the fatigue results are increased in the parts where the additional braces are installed. Some braces in the upper level C and D, where not additional braces are located, the fatigue results are becoming even worse. Overall FLS results are improved from this design study, but are still insufficient for many details (i.e. legs, braces and joints).

6.3 Design Summary

It is assumed not realistic to design the 20MW jacket without further improvements of the wind turbine model, i.e. updated model and assumptions. A land based extrapolated wind turbine model is not an adequate starting point for the design. The dynamic influence of very large wind turbines and jackets is significant and requires closer collaboration between the wind turbine and foundation designs. Therefore it is sensible to improve the wind turbine performance based on the findings of the preliminary designed jacket before continuing the jacket design (see Table 6-10, Appendix A – Preliminary Jacket Dimensions and ref. [15]). For example modifications of the height of the TP and tower, load reduction (height of the tower, damping) and refining the controller performance according to the natural frequency results should be considered.

Table 6-10: Overview of the jacket geometry and masses from the preliminary design

Structural member	Dimensions	Value
Jacket		
Base Width	[m]	38
Top Width	[m]	20
Interface elevation	[m] wrt MSL	26
Transition Piece height	[m]	8
Number of legs	[-]	4
Jacket legs diameter (outer)	[mm]	1829-2642
Jacket legs maximum wall thickness	[mm]	101.6
Jacket legs minimum wall thickness	[mm]	44.5
Number of x-braces levels	[-]	4
Diameter upper x-braces diameters (outer)	[mm]	914
Diameter middle upper x-braces diameters (outer)	[mm]	965 / 1219
Diameter middle lower x-braces diameters (outer)	[mm]	965 / 1219
Diameter lower x-braces diameters (outer)	[mm]	1168 / 1828
Braces wall thicknesses	[mm]	20 - 40.5
Number of Piles	[-]	4
Pile penetration	[m]	50
Pile diameter	[mm]	3500
Pile wall thicknesses	[mm]	34.9 – 73
Pile top elevation above mudline (Stick-up length)	[m]	1.8
Overlap length (grout length)	[m]	7.5
Masses		
Jacket structure	[t]	1670
Transition Piece (estimation)	[t]	450
Steel Appurtenances (estimation)	[t]	50
Piles	[t]	4x 230
Total lifting mass (no piles)	[t]	2170
Natural frequency overall structure		
1st eigenfrequency (1st bending mode)	[Hz]	0.165

7 COST ANALYSIS

Costs associated with the fabrication of jackets are evaluated using two different cost models. Firstly, a lumped price model is used to calculate the total costs of the 20MW jacket. This model was applied for other support structures designed for the 10MW wind turbine [1], [12], [13] and therefore can be used to compare the costs. The assumed unit price rates are derived as of 2012, the beginning of the INNWIND.EU project. The costs are proportional to the mass of the components (i.e. transition piece, jacket and piles) and thus different aspects of steel works and assembly costs cannot be considered.
Secondly, relative cost differences between the 10MW and 20MW jacket designs are calculated using a more detailed fabrication cost model (Figure 7-1), which allows a comparison of important cost contributors for the jacket, namely material costs, welding costs and additional costs. The latter consists of costs for scaffold, crane, vessels, assembly aids, coating and logistics. However, the model is confidential and further details about the calculation and parameters cannot be explained.

The resulting levelized costs of energy (LCOE) for the 20MW wind turbine including the jacket are in the order of 93€/MWh according to [14]. The influence of the jacket fabrication costs on the LCOE is rather small. It is estimated that an increase about 10% of the jacket costs transfers into and LCOE increase of 1.44% (approximately 7:1).

It should be noted that the considered jacket design for the cost analyses are based on the same design procedures regarding NFA, FLS and ULS, which is a necessary requirement for a fair comparison. But different models and tools for the wind turbine load analyses have been applied throughout the INNWIND.EU project, which have not been compared in detail and therefore the developed jacket designs and the costs can be inaccurate. The load analyses for the 10MW jacket designs are performed using an integrated model (superelement approach) in LACFLEX. The 20MW jacket is designed using the preliminary (onshore) loads calculated with GAST.mb and superimposed with separate wave loads in ROSA. The integrated model from BLADED has been applied in the subsequent load and design iterations of the 20MW jacket (which is part of the conceptual design in this report).

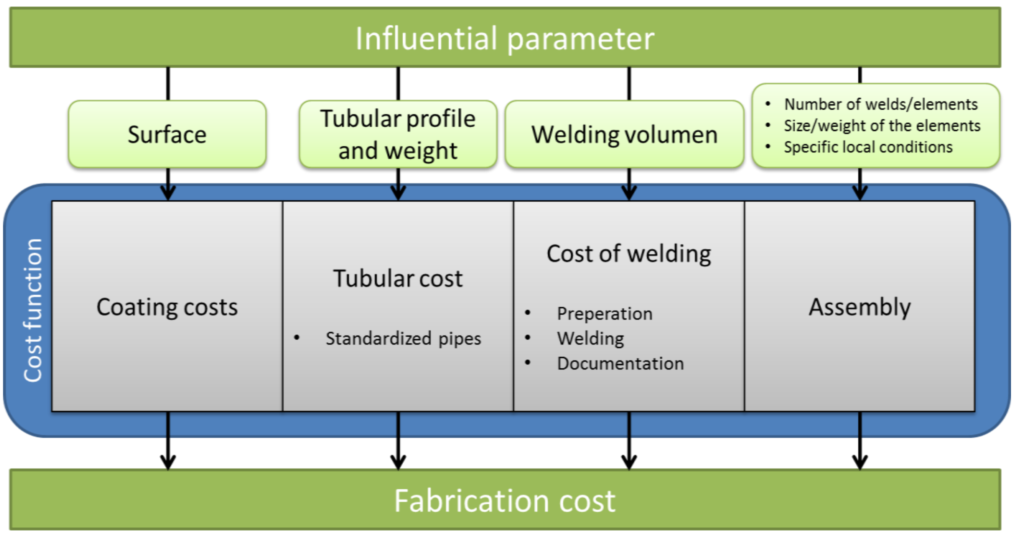


Figure 7-1: Fabrication cost model for jackets

The comparison of the fabrication costs with the lumped price model is shown in Table 7-1. The costs for the jacket increase between 44% - 61% although the nominal power of the wind turbine is doubled. The jacket study with additional bracings mitigates local vibrations and improves the fatigue results, but increases the fabrication costs additionally. It should be investigated if other possibilities exist to mitigate the identified load excitation from the wind turbine (see section

6.2.4) instead of increasing the complexity of the jacket structure. However, the maturity of the wind turbine models, especially the 20MW wind turbine, is rather low and the present turbine model is not optimized for the considered offshore site and thus might result in overly conservative results.

Table 7-1: Fabrication costs for 10MW and 20MW jackets using the lumped price model.

Unit Price		Reference Jacket 10MW		Innovative Jacket 10MW		Jacket 20MW		Jacket 20MW + diamond braces	
[€/ton]$_{2012}$		Mass [t]	Costs	Mass [t]	Costs	Mass [t]	Costs	Mass [t]	Costs
TP	5000	330	1,650,000 €	258	1,290,000 €	450	2,250,000 €	450	2,250,000 €
Jacket	4800	1210	5,808,000 €	1093	5,246,400 €	1670	8,016,000 €	1961	9,412,800 €
Piles	1200	380	456,000 €	342	410,400 €	920	1,104,000 €	920	1,104,000 €
Sum			**7,914,000 €**		**6,946,800 €**		**11,370,000 €**		**12,766,800 €**
			Base Case		Change -12 %		Change 44 %		Change 61 %

The results of the more detailed fabrication cost model show a similar outcome, but the predicted differences of the resulting overall cost between the designs are smaller compared with the lumped price model, e.g. only +33% cost increase of the 20MW jacket instead of 44%. Figure 7-2 shows the results of the four jackets normalized with the total costs of the reference jacket for 10MW.

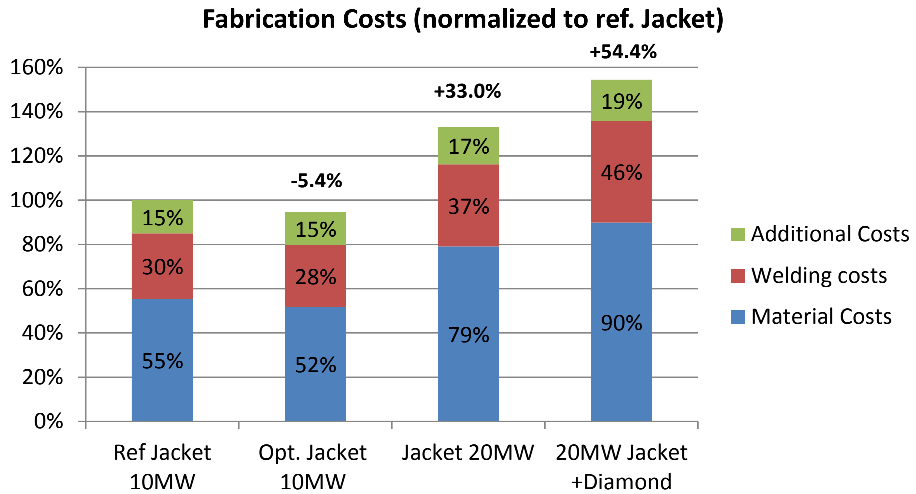


Figure 7-2: Relative cost comparison of the 10MW and 20MW jackets using the fabrication cost model

In general the cost composition of jacket fabrication costs is very similar between the 10MW and 20MW jacket designs. The main cost contributors are from material costs, which contribute between 55% and 60% to the overall costs. Welding costs of tubular joints and costs for butt welds of straight pipes contribute between 28% and 30%. The composition of costs for the preliminary 20MW jacket, the conceptual 20MW jacket with the additional bracings and the 10MW reference jacket are shown in Figure 7-3.

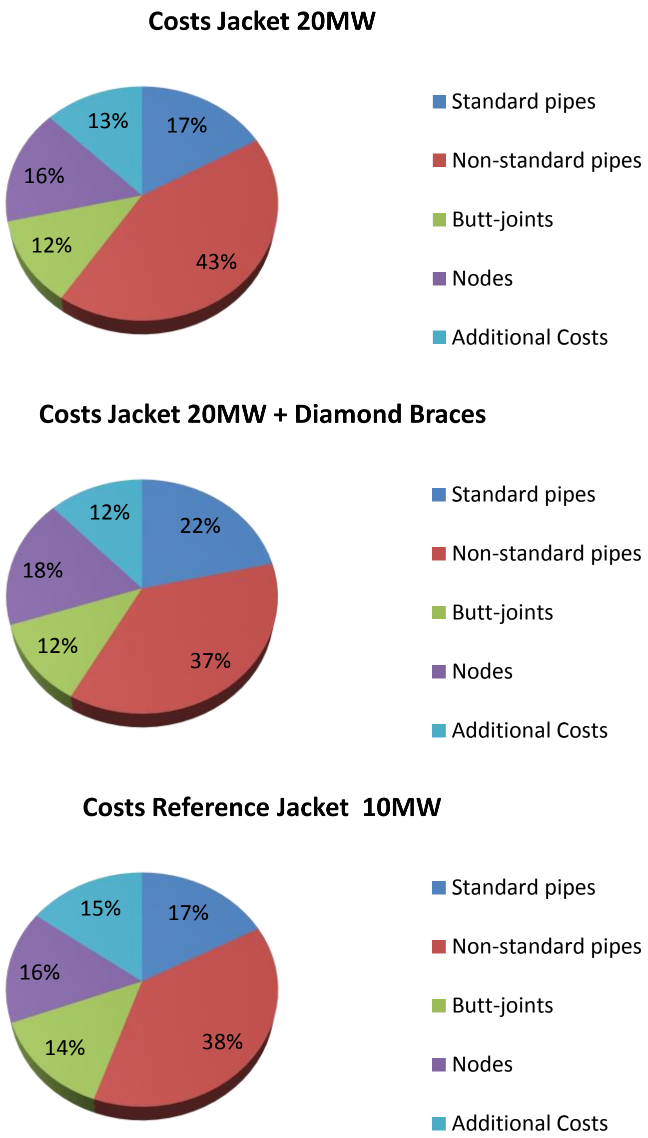


Figure 7-3: Cost composition of jacket fabrication costs

REFERENCES

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[2] IEC-61400-3, Wind turbines – Part 3: Design requirements for offshore wind turbines, 02-2009

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[4] INNWIND.EU project, 10MW Reference Turbine Data Sheet. excel-sheet, version 1.04, Revision 17, 15.01.2016

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[6] 20 MW_10DE2015_Tower2_fin.xls, Version 1.04 Revision 17

[7] Fischer, T., UpWind Design Basis (WP4: Offshore Foundation Support Structures), 2010

[8] DNV GL AS, Standard DNVGL-ST-0126, Support structures for wind turbines, April 2016

[9] DNV GL AS, Standard DNVGL-RP-C203, Fatigue design of offshore steel structures, April 2016

[10] DNV GL AS, Standard DNVGL-ST-0437, Loads and site conditions for wind turbines, November 2016

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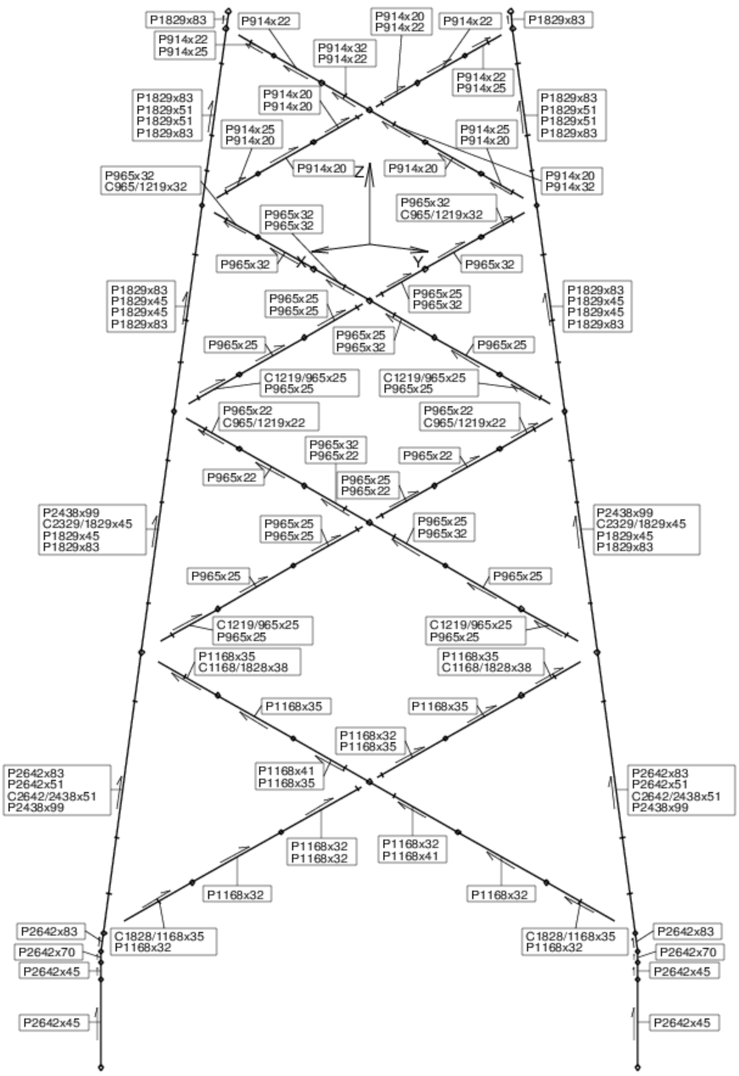
[13] INNWIND.EU project Deliverable D4.33 – Innovative Concepts for Floating Structures, 14.8.2015

[14] INNWIND.EU project Deliverable D1.25 – PI-based Assessment of 20MW Wind Turbines, September 2017

[15] INNWIND.EU project, Steel Jacket 20MW Ramboll-Rev01, September 2017

APPENDIX A – PRELIMINARY JACKET DIMENSIONS

Elements



Jacket Side A - View from outside jacket - InWi20

Ramboll Wind

Direction:	X:-0.701	Y:-0.701	Z:-0.135
Limits:	X(0.000, 26.870)	Y(0.000, 26.870)	Z(-56.300, 18.000)
Plot 1:	Date: 2017-09-25	Time: 09:49:17	Job NMJUVR (STPLOT 5.1)

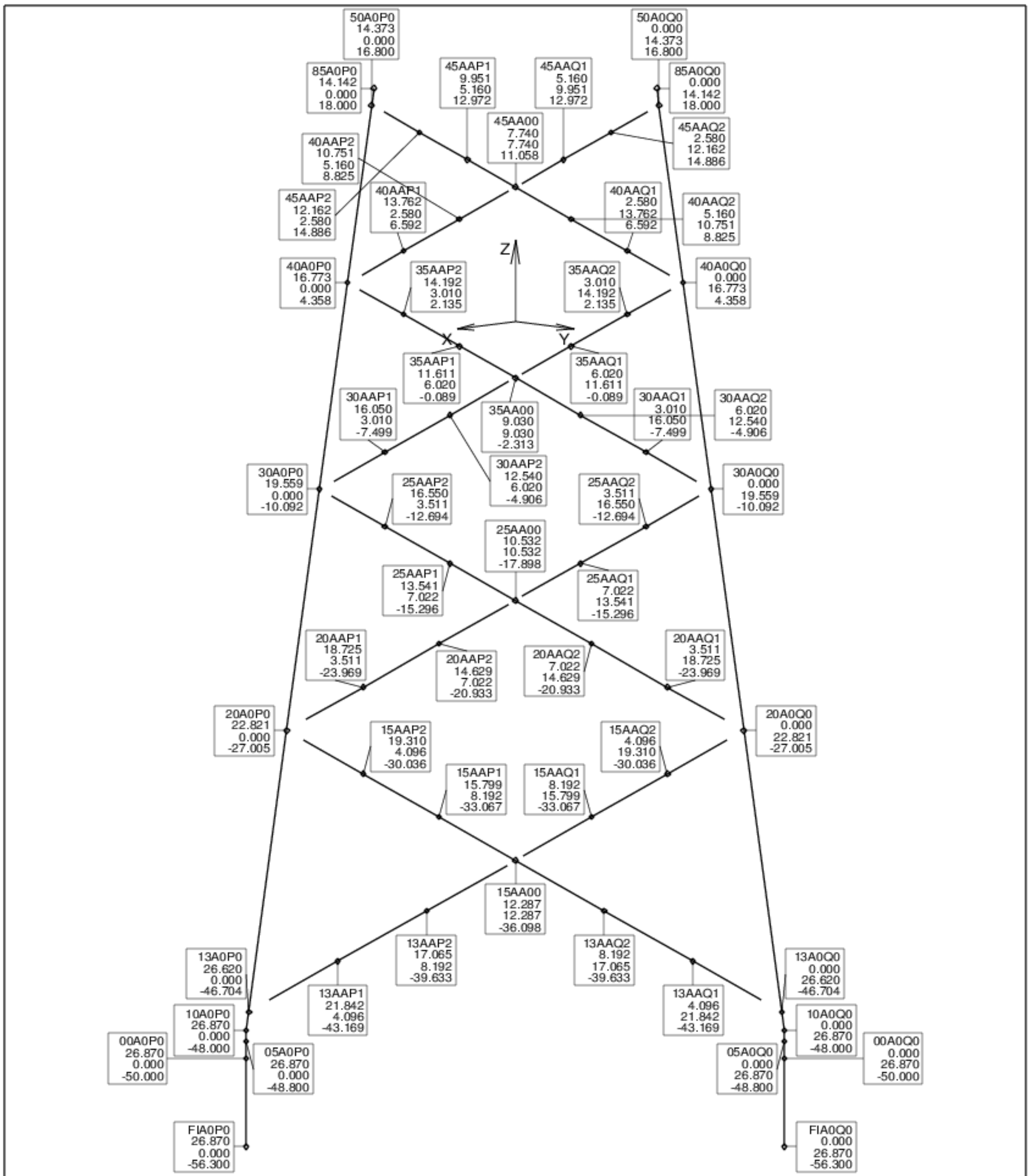
COMPANY: INNWIND.EU
PROJECT: JACKET MODEL 20 MW
SUBJECT : GEOMETRY PLOTS

Stadtdeich 7
GER 20097 Hamburg

Tel: +49 40 302020 0
Fax: +49 40 302020 199

Web: www.ramboll.com/wind
Email: info@ramboll.com

Nodes



Node coordinates (m)

Jacket Side A - View from outside jacket - InWi20

Ramboll Wind

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Limits:	X(0.000, 26.870)	Y(0.000, 26.870)	Z(-56.300, 18.000)
Plot 1:	Date: 2017-09-25	Time: 09:49:17	Job NMJUVR (STPLOT 5.1)

COMPANY: INNWIND.EU
PROJECT: JACKET MODEL 20 MW
SUBJECT : GEOMETRY PLOTS

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GER 20097 Hamburg

Tel: +49 40 302020 0
Fax: +49 40 302020 199

Web: www.ramboll.com/wind
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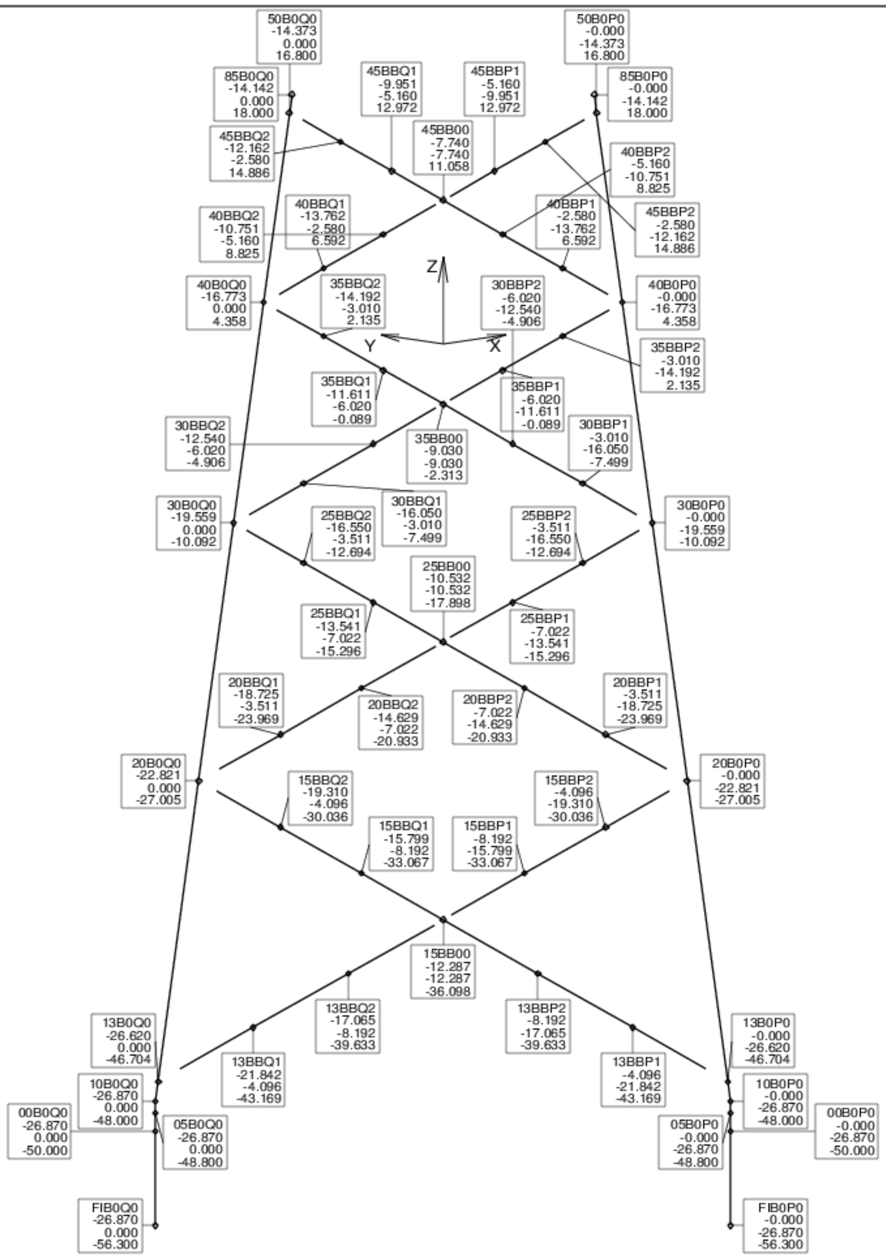

Node coordinates (m)

Jacket Side B - View from outside jacket - InWi20

Ramboll Wind

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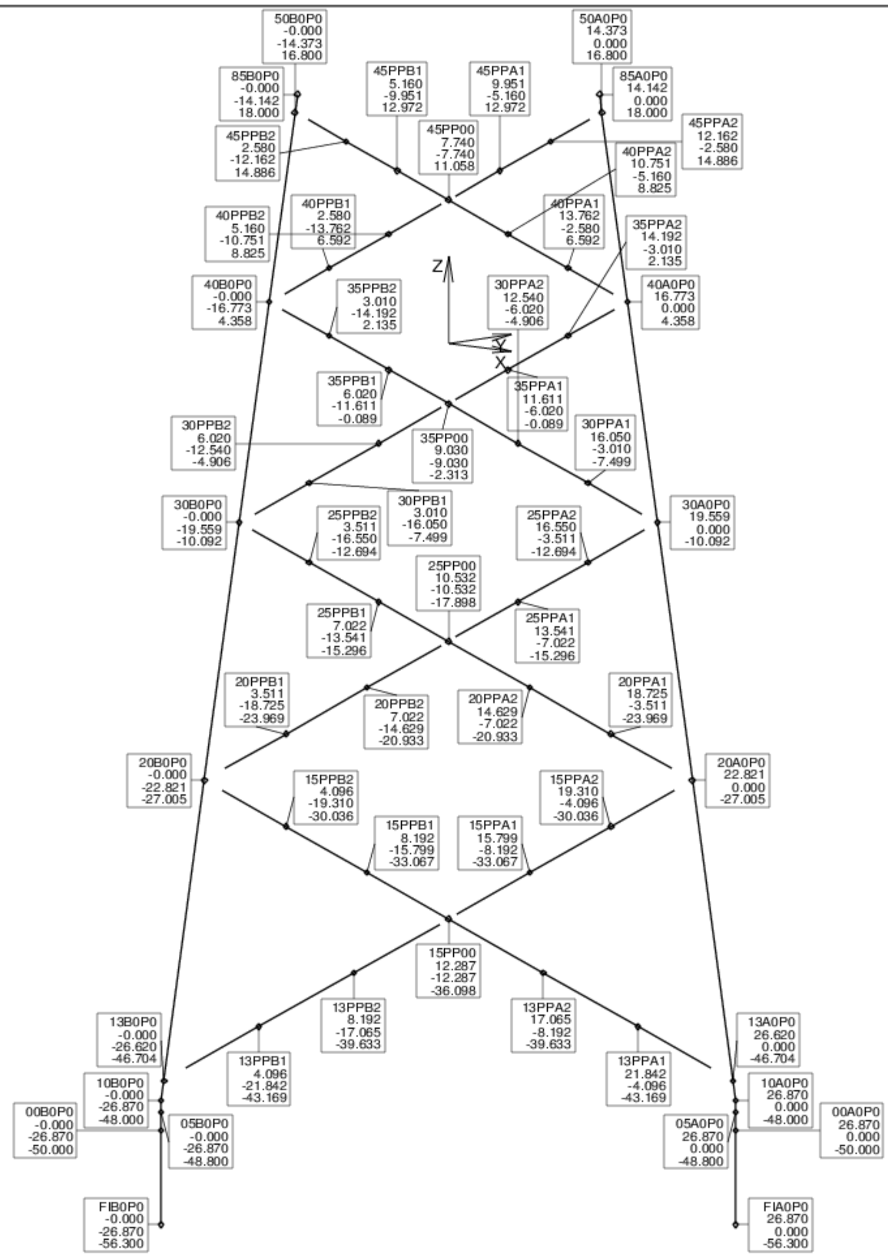
COMPANY: INNWIND.EU
PROJECT: JACKET MODEL 20 MW
SUBJECT : GEOMETRY PLOTS

Stadtdeich 7
GER 20097 Hamburg
Tel: +49 40 302020 0
Fax: +49 40 302020 199
Web: www.ramboll.com/wind
Email: info@ramboll.com



Node coordinates (m)

Jacket Side P - View from outside jacket - InWi20

Ramboll Wind

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Limits:	X(-0.000, 26.870)	Y(-26.870, 0.000)	Z(-56.300, 18.000)		
Plot 3:	Date: 2017-09-25	Time: 09:49:17	Job NMJUVR (STPLOT 5.1)		

COMPANY: INNWIND.EU
PROJECT: JACKET MODEL 20 MW
SUBJECT : GEOMETRY PLOTS

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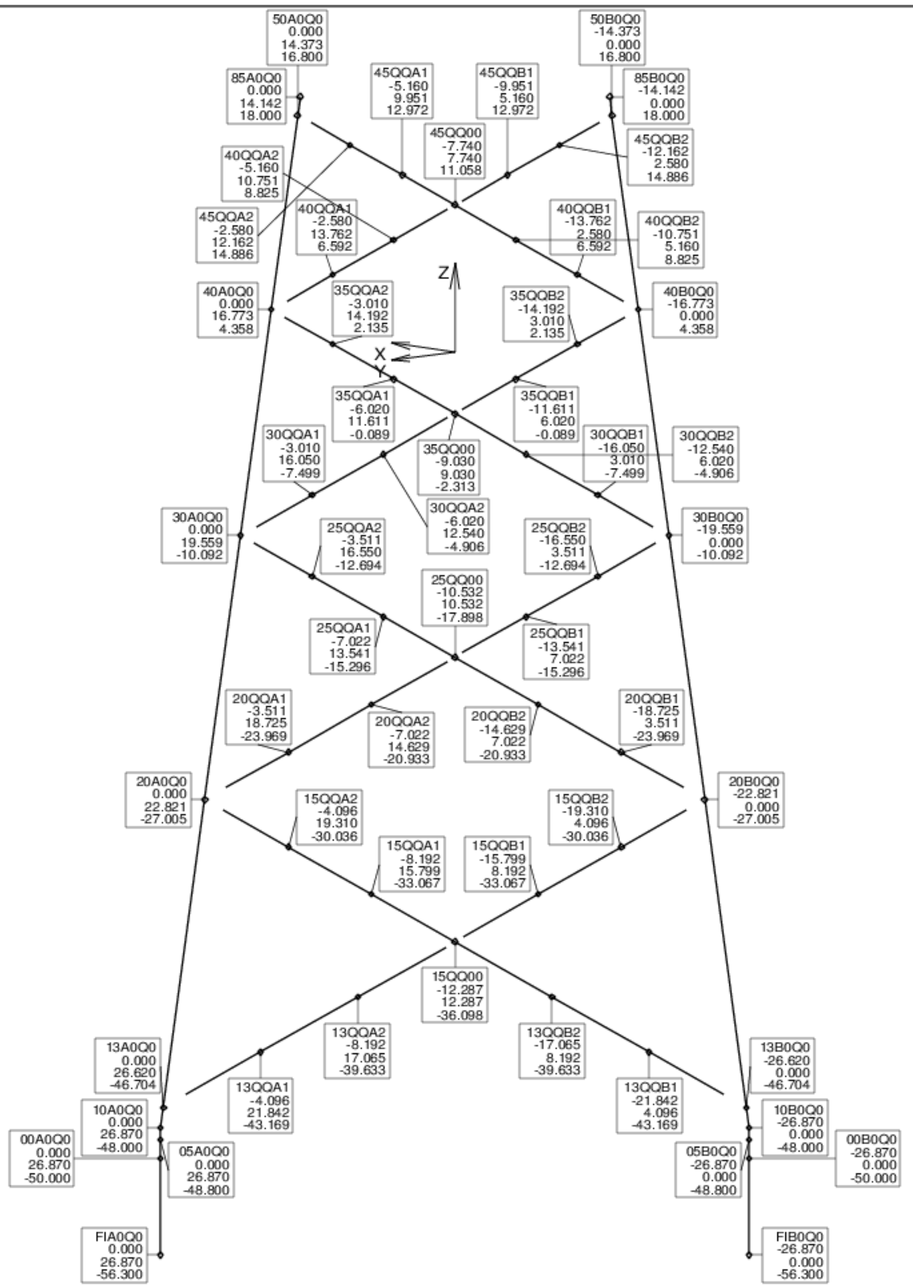
(INNWIND.EU, Deliverable D4.36)



Node coordinates (m)

Jacket Side Q - View from outside jacket - InWi20

Ramboll Wind

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Plot 4:	Date: 2017-09-25	Time: 09:49:17	Job NMJUVR (STPLOT 5.1)

COMPANY: INNWIND.EU
PROJECT: JACKET MODEL 20 MW
SUBJECT : GEOMETRY PLOTS

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APPENDIX B – SOIL PROFILE

DESIGN PARAMETERS FOR SOIL IN01 WITH PILE A0P0P

depth (m)	Soil type	γ' (kN/m³)	I_p (%)	q_u (MPa)	φ (deg)	c_u (kPa)	E (MPa)	ε_{50} (%)	t_c (kPa)	t_t (kPa)	q (MPa)
2.0	SAND	9.00			35.0		5.7		0.0	0.0	0.0
3.3	SAND	9.00			35.0		5.7		0.0	0.0	0.0
9.0	SAND	9.00			35.0		13.7		15.1	15.1	1.1
10.0	SAND	9.00			35.0		16.6		32.8	32.8	2.4
15.0	SAND	9.00			35.0		19.7		47.0	47.0	3.4
20.0	SAND	10.00			35.0		24.4		71.1	71.1	5.1
22.5	SAND	10.50			35.0		27.5		89.8	89.8	6.5
29.0	SAND	11.00			35.0		30.8		105.5	105.5	8.1
32.0	SAND	11.00			35.0		34.0		115.0	115.0	9.9
38.0	SAND	11.00			35.0		36.8		115.0	115.0	11.2
40.0	SAND	11.00			35.0		39.1		115.0	115.0	12.0
90.0	SAND	11.00			35.0		50.3		115.0	115.0	12.0

scale changed

γ' : Submerged unit weight
I_p : Plasticity index
q_u : Unconfined compression strength
φ : Characteristic angle of internal friction
c_u : Characteristic undrained shear strength
E : Modulus of elasticity
ε_{50} : Strain which occurs at one-half of the maximum stress in laboratory undrained compression test
t_c : Unit skin friction, compression
t_t : Unit skin friction, tension
q : Unit tip resistance, compression

Design code: DNV-J101
Partial coefficient on angle of internal friction: 1.00
Partial coefficient on undrained shear strength: 1.00
Partial coefficient on axial bearing capacity: 1.00

Scour: Local scour: 3.3 m
Global scour: 0.0 m
Scour angle: 18.0 deg

Pile tip depth: 38.00 m
Pile tip diameter: 2540.0 mm
Pile tip thickness: 31.8 mm

Ramboll Wind

Subject: InnWind Soil Profile — Program: ROSA 5.00
Prepared: TVB — Checked: BJOS — Approved: TIMF — Rev. 0 Date: 2013-09-25

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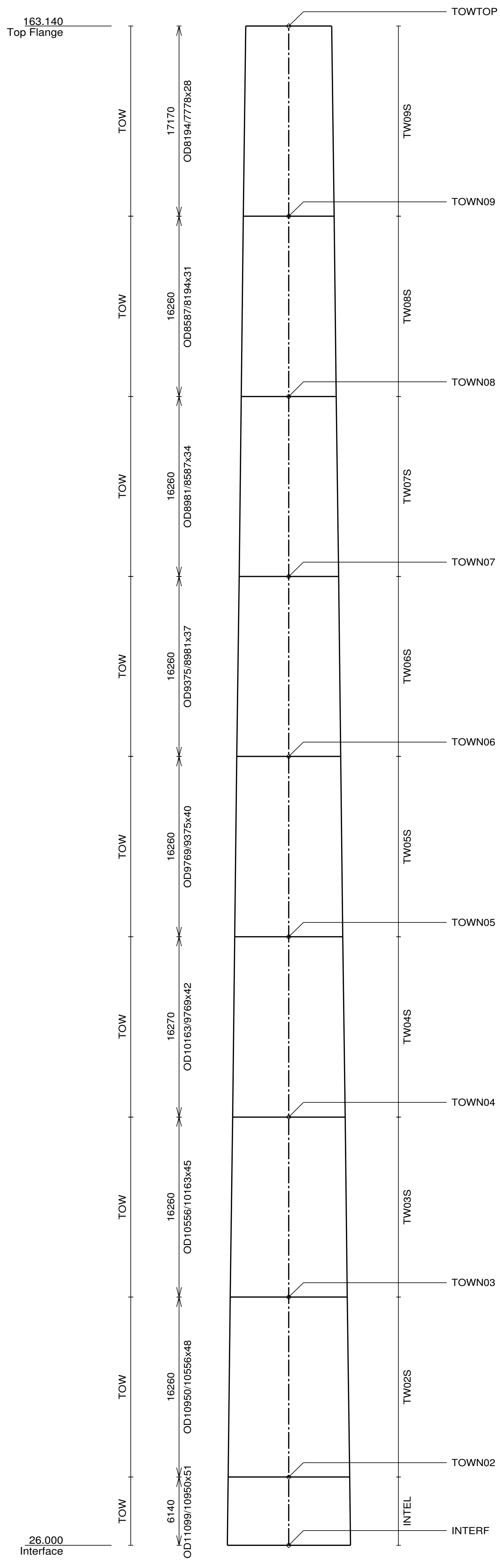
(INNWIND.EU, Deliverable D4.36)

APPENDIX C – TOWER GEOMETRY

The modified geometry of the onshore tower [5] with bottom elevation 26mMSL is shown in this appendix.



NOTES

1. All dimensions in millimeters.
2. All levels in meters.
3. Nominal weight of steel on this drawing:
   TOW 1355.66 tonnes
4. Assumed material density of 8500kg/m3

Rev.	Date	Drw.	Chkd.	Appr.	Description
0	2016.05.XX	KISC	DAKA	xxxx	FOR INFORMATION

Client INNWIND.EU

Planner Ramboll Wind

JACKET MODEL 20 MW

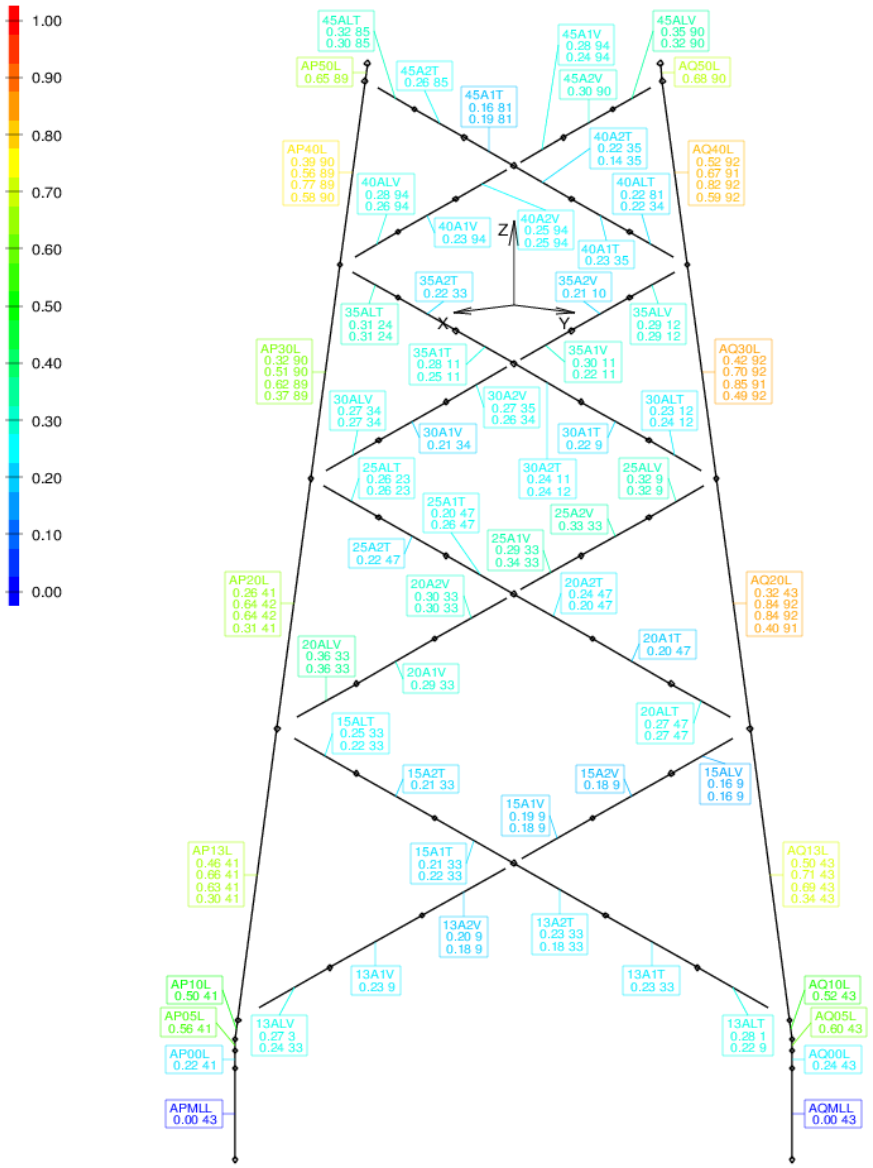
Title TOWER - COMPUTER MODEL - InWi20

Scale	Size	Drawing No.	Rev.
1:435	A3	For information	0

NFA_COBC_FATI_si02.PS

APPENDIX D – PRELIMINARY DESIGN RESULTS

ULS results - beams



Utilization Ratio Summary, Limit = 1.00

Jacket Side A - View from outside jacket

Ramboll Wind

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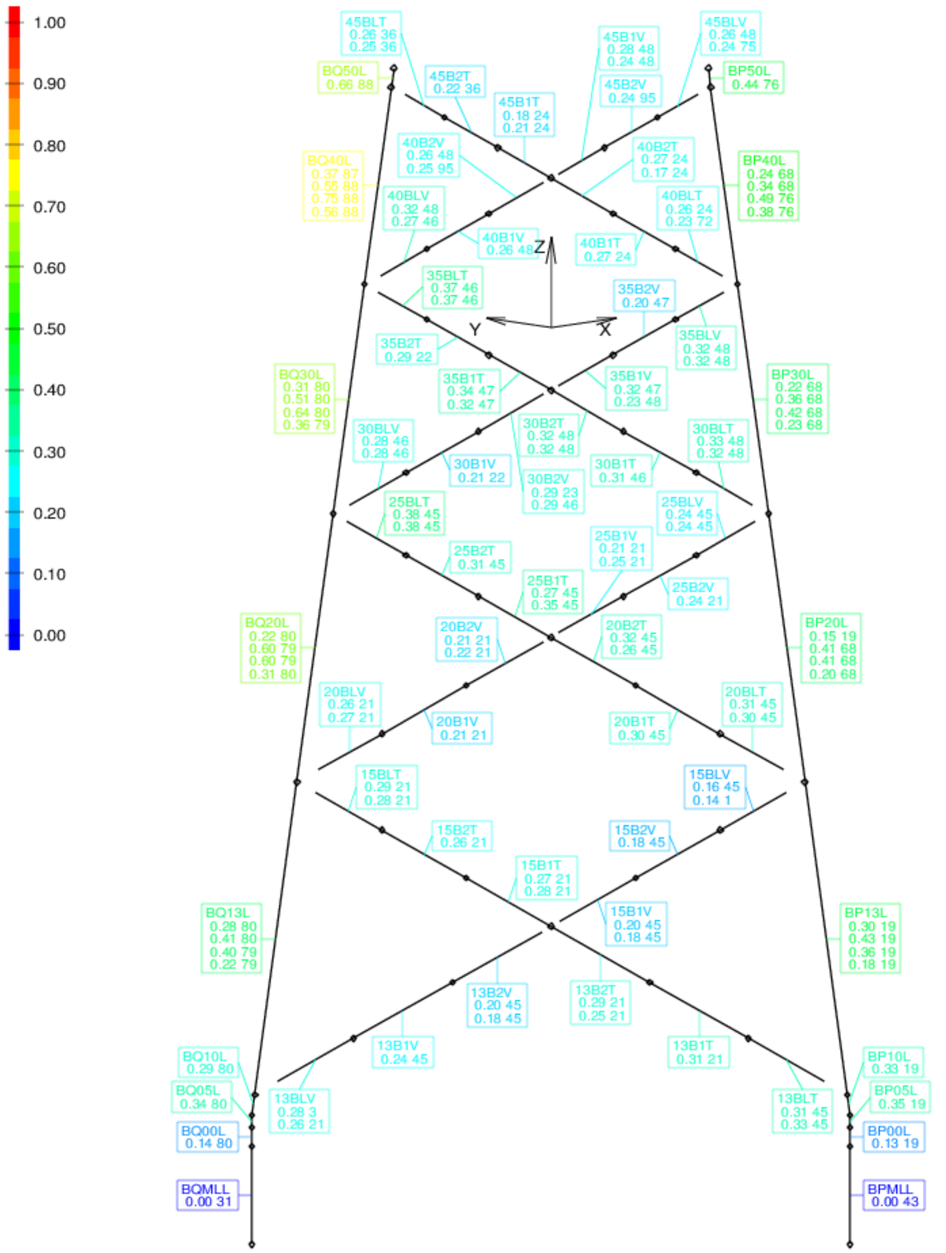
Limits:	X(0.000, 26.870)	Y(0.000, 26.870)	Z(-56.300, 18.000)
Plot 1:	Date: 2017-04-03	Time: 07:40:02	Job EPHQYB (STPLOT 5.1)

COMPANY: INNWIND.EU

PROJECT: JACKET MODEL 20 MW

SUBJECT : STRESS UTILISATION PLOTS - STRESS AND STABILITY CHECK



Utilization Ratio Summary, Limit = 1.00

Jacket Side B - View from outside jacket

Ramboll Wind

Direction:	X: 0.701	Y: 0.701	Z:-0.135
Limits:	X(-26.870, -0.000)	Y(-26.870, 0.000)	Z(-56.300, 18.000)
Plot 2:	Date: 2017-04-03	Time: 07:40:02	Job EPHQYB (STPLOT 5.1)

COMPANY: INNWIND.EU

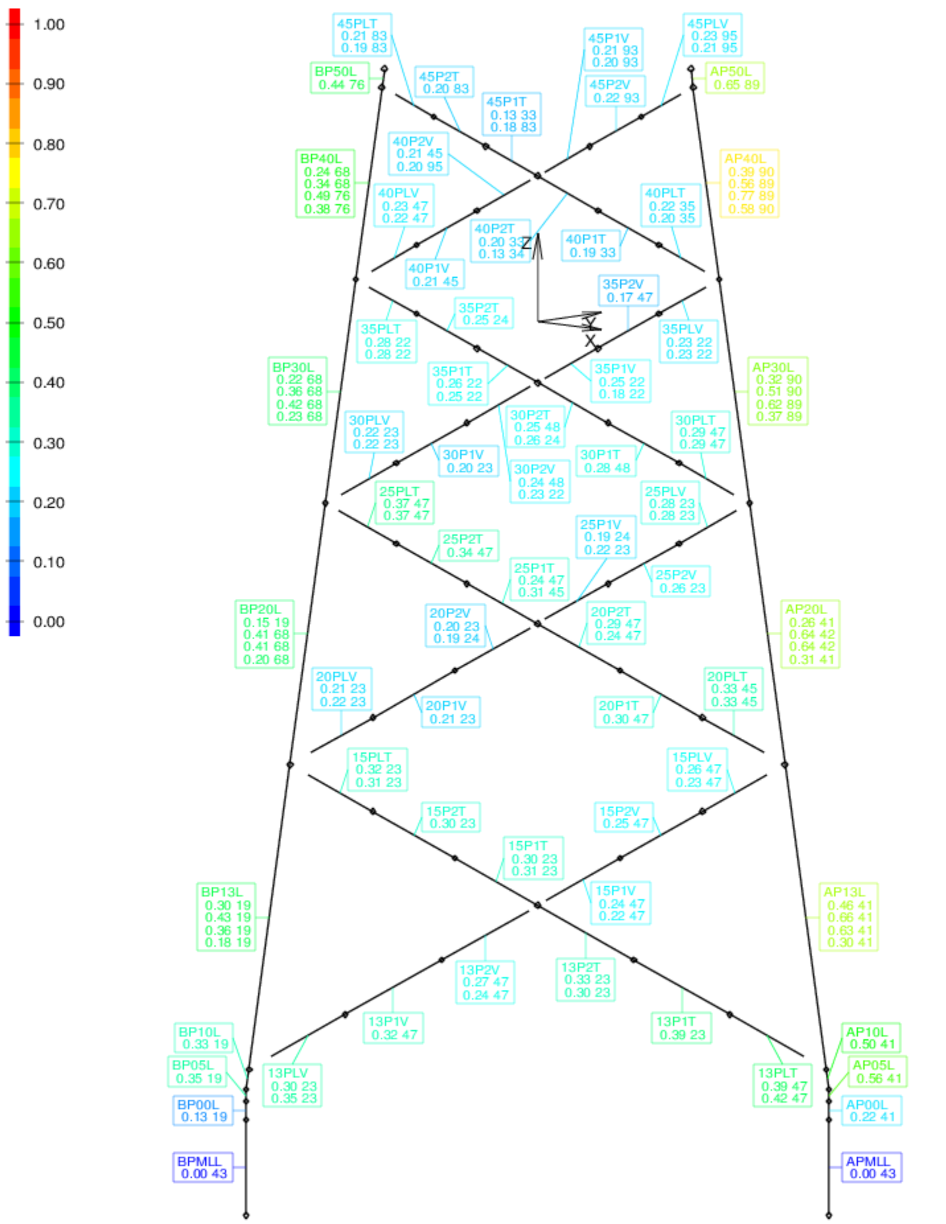
PROJECT: JACKET MODEL 20 MW

SUBJECT : STRESS UTILISATION PLOTS - STRESS AND STABILITY CHECK

Stadtdeich 7
GER 20097 Hamburg

Tel: +49 40 302020 0
Fax: +49 40 302020 199

Web: www.ramboll.com/wind
Email: info@ramboll.com



Utilization Ratio Summary, Limit = 1.00

Jacket Side P - View from outside jacket

Ramboll Wind

Direction:	X:-0.701	Y: 0.701	Z:-0.135
Limits:	X(-0.000, 26.870)	Y(-26.870, 0.000)	Z(-56.300, 18.000)
Plot 3:	Date: 2017-04-03	Time: 07:40:02	Job EPHQYB (STPLOT 5.1)

COMPANY: INNWIND.EU

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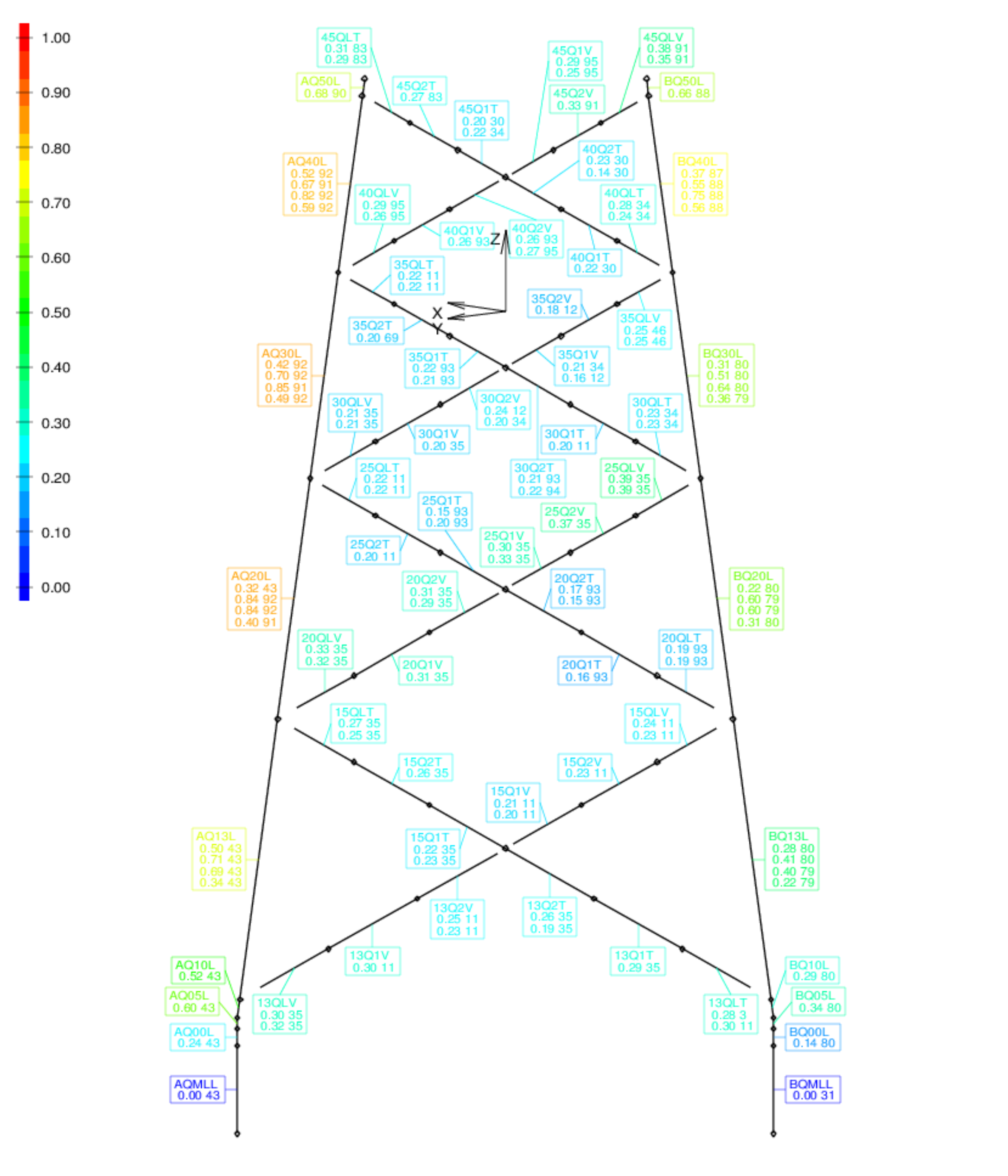

Utilization Ratio Summary, Limit = 1.00

Jacket Side Q - View from outside jacket

Ramboll Wind

Direction:	X: 0.701	Y:-0.701	Z:-0.135
Limits:	X(-26.870, 0.000)	Y(0.000, 26.870)	Z(-56.300, 18.000)
Plot 4:	Date: 2017-04-03	Time: 07:40:02	Job EPHQYB (STPLOT 5.1)

COMPANY: INNWIND.EU
PROJECT: JACKET MODEL 20 MW
SUBJECT : STRESS UTILISATION PLOTS - STRESS AND STABILITY CHECK

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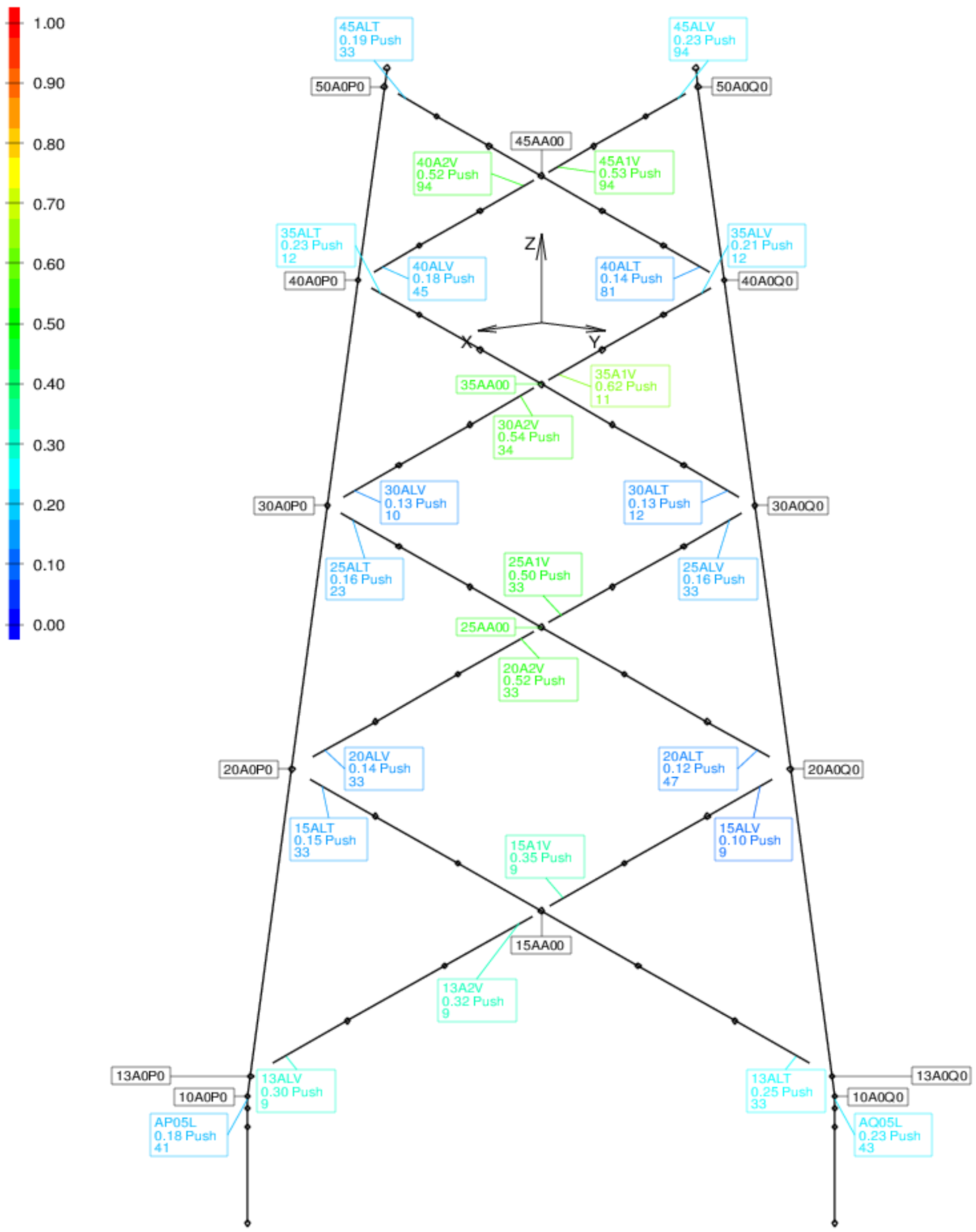
ULS results - nodes

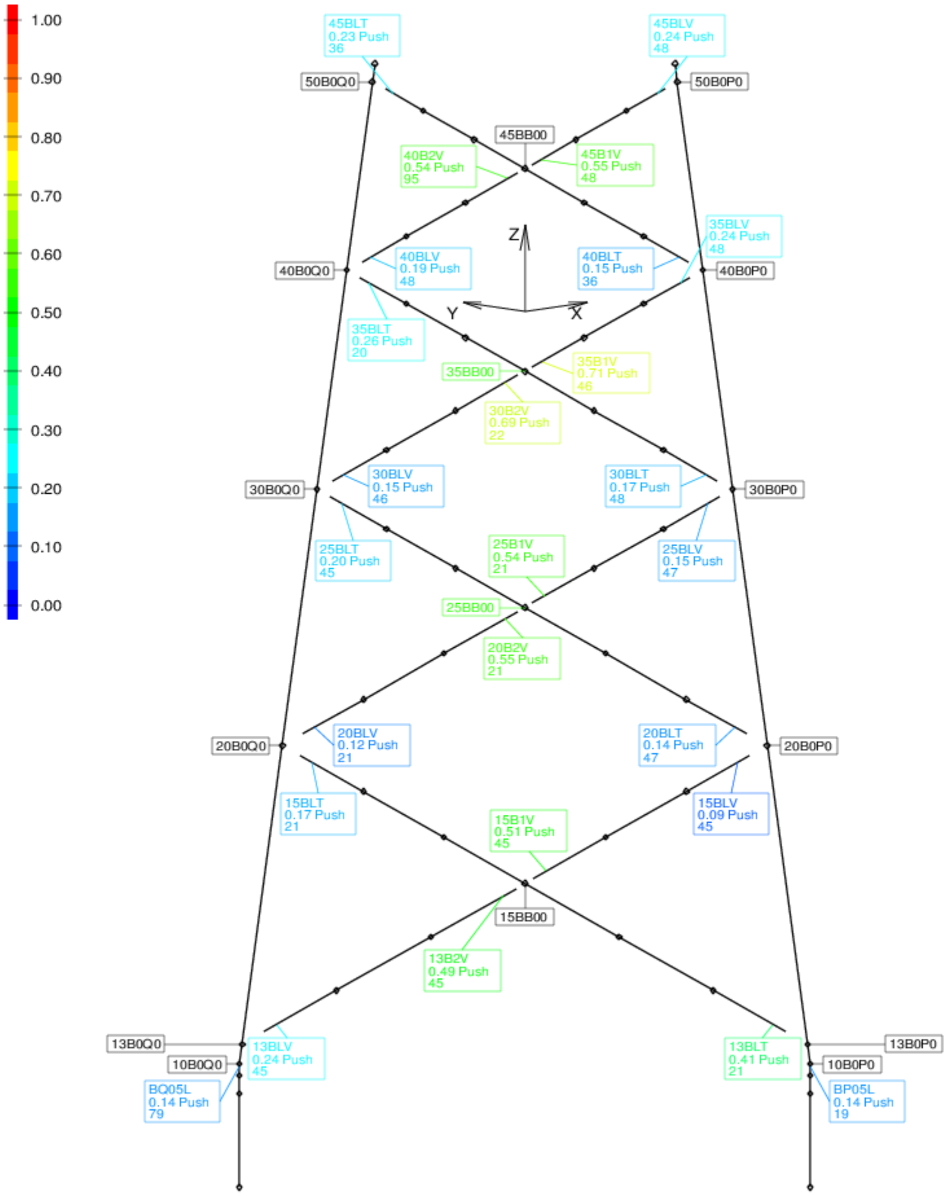


Tubular Joint Stress Check, Maximum Utilization Ratios, Limit = 1.00
Jacket Side A - View from outside jacket

Ramboll Wind

Direction:	X:-0.701	Y:-0.701	Z:-0.135
Limits:	X(0.000, 26.870)	Y(0.000, 26.870)	Z(-56.300, 18.000)
Plot 1:	Date: 2017-04-03	Time: 07:40:03	Job EPHQYF (STPLOT 5.1)

COMPANY: INNWIND.EU
PROJECT: JACKET MODEL 20 MW
SUBJECT : PUNCHING SHEAR UTILISATION PLOTS - PUNCHING SHEAR CHECK



Tubular Joint Stress Check, Maximum Utilization Ratios, Limit = 1.00
Jacket Side B - View from outside jacket

Ramboll Wind

Direction:	X: 0.701	Y: 0.701	Z:-0.135
Limits:	X(-26.870, -0.000)	Y(-26.870, 0.000)	Z(-56.300, 18.000)
Plot 2:	Date: 2017-04-03	Time: 07:40:03	Job EPHQYF (STPLOT 5.1)

COMPANY: INNWIND.EU
PROJECT: JACKET MODEL 20 MW
SUBJECT : PUNCHING SHEAR UTILISATION PLOTS - PUNCHING SHEAR CHECK

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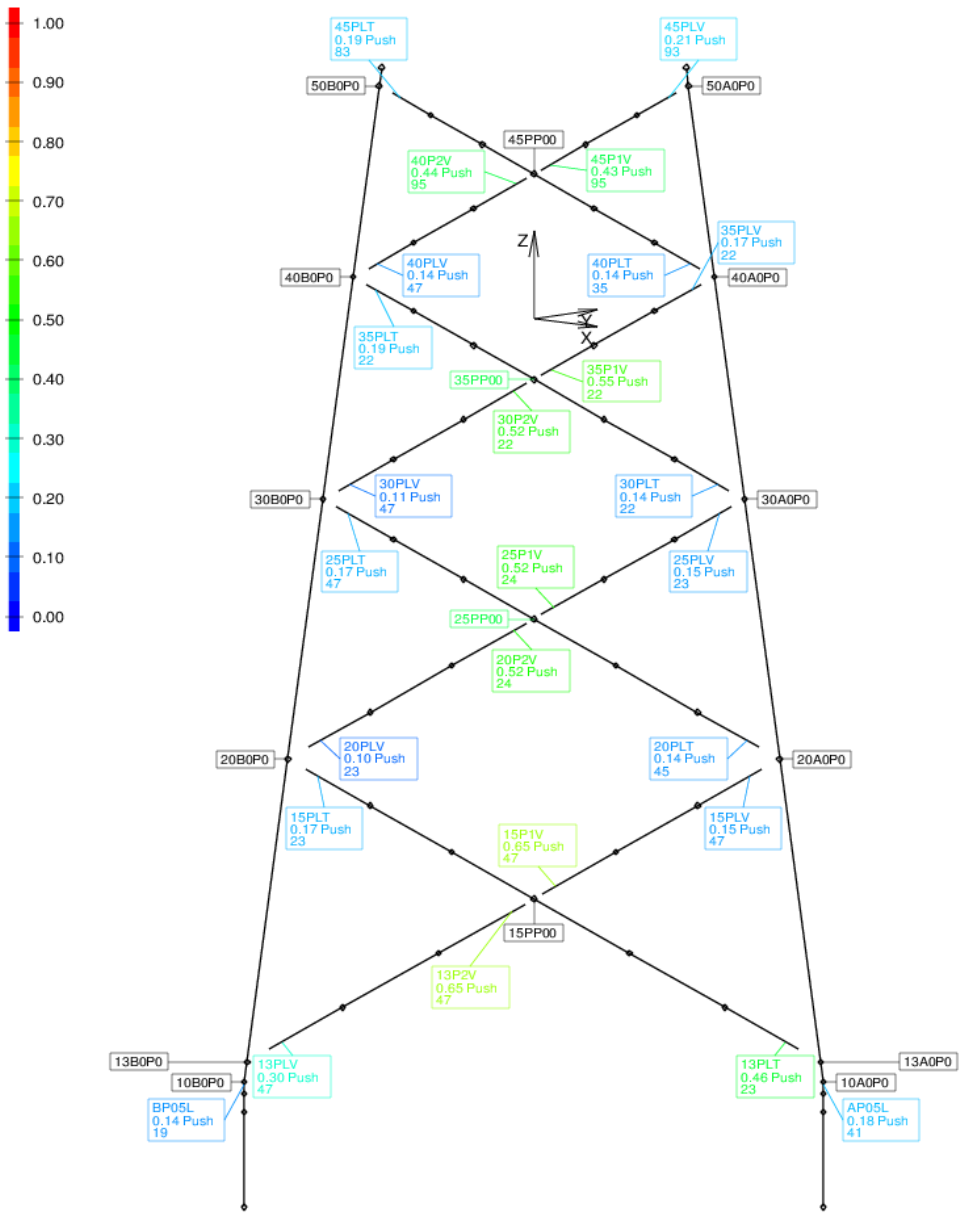

Tubular Joint Stress Check, Maximum Utilization Ratios, Limit = 1.00

Jacket Side P - View from outside jacket

Ramboll Wind

Direction:	X:-0.701	Y: 0.701	Z:-0.135
Limits:	X(-0.000, 26.870)	Y(-26.870, 0.000)	Z(-56.300, 18.000)
Plot 3:	Date: 2017-04-03	Time: 07:40:03	Job EPHQYF (STPLOT 5.1)

COMPANY: INNWIND.EU

PROJECT: JACKET MODEL 20 MW

SUBJECT : PUNCHING SHEAR UTILISATION PLOTS - PUNCHING SHEAR CHECK

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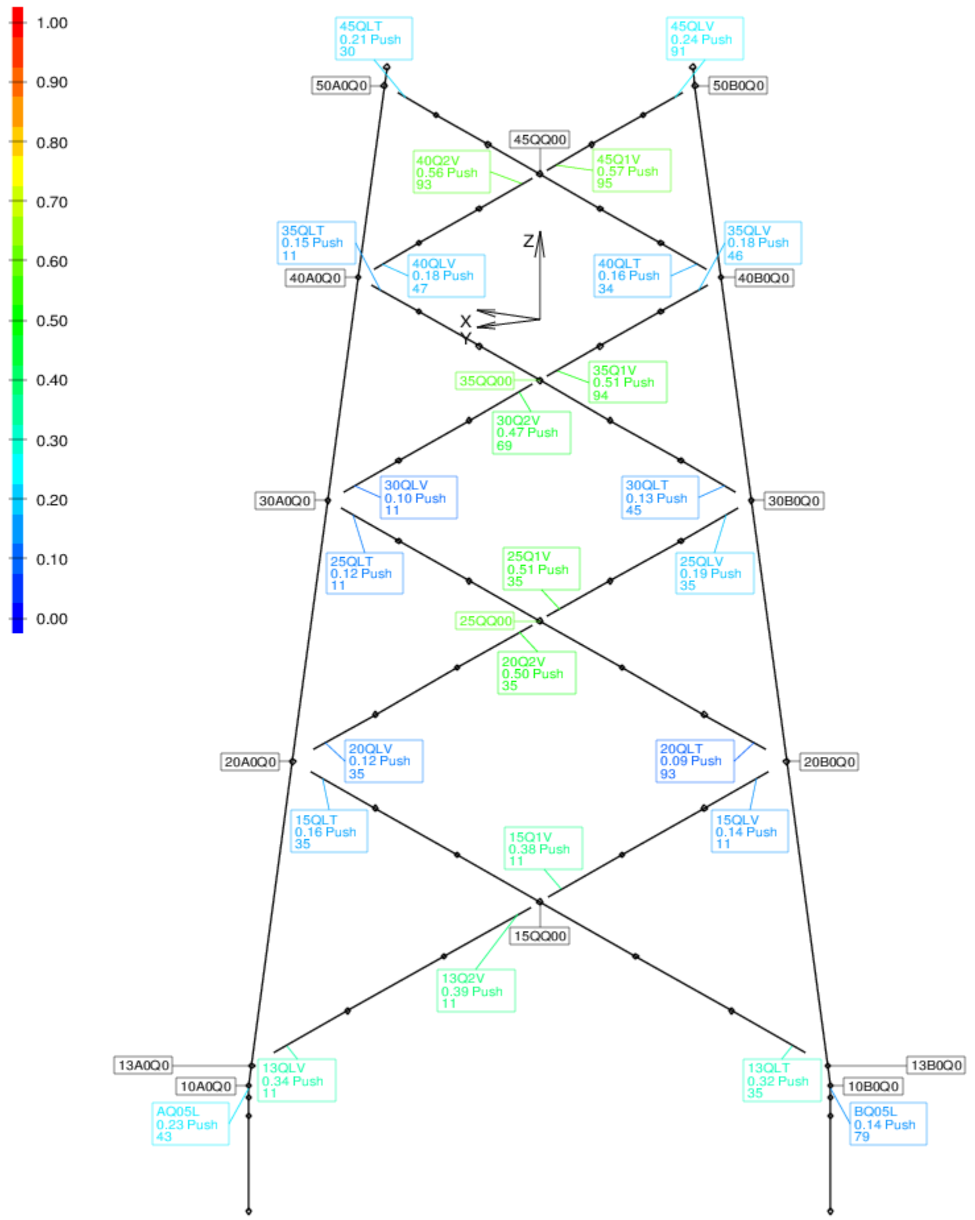

Tubular Joint Stress Check, Maximum Utilization Ratios, Limit = 1.00

Jacket Side Q - View from outside jacket

Ramboll Wind

Direction:	X: 0.701	Y:-0.701	Z:-0.135
Limits:	X(-26.870, 0.000)	Y(0.000, 26.870)	Z(-56.300, 18.000)
Plot 4:	Date: 2017-04-03	Time: 07:40:03	Job EPHQYF (STPLOT 5.1)

COMPANY: INNWIND.EU
PROJECT: JACKET MODEL 20 MW
SUBJECT : PUNCHING SHEAR UTILISATION PLOTS - PUNCHING SHEAR CHECK

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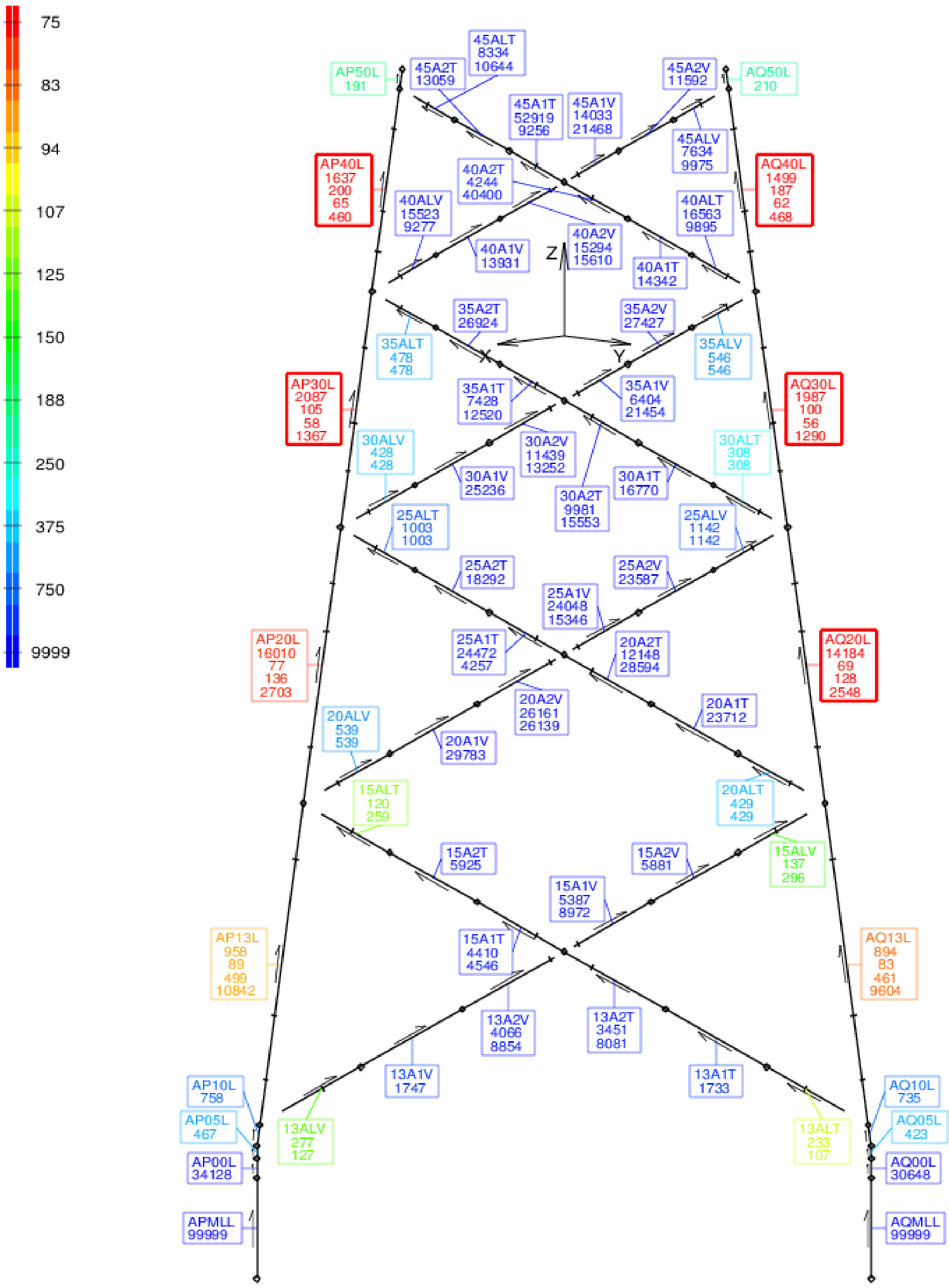
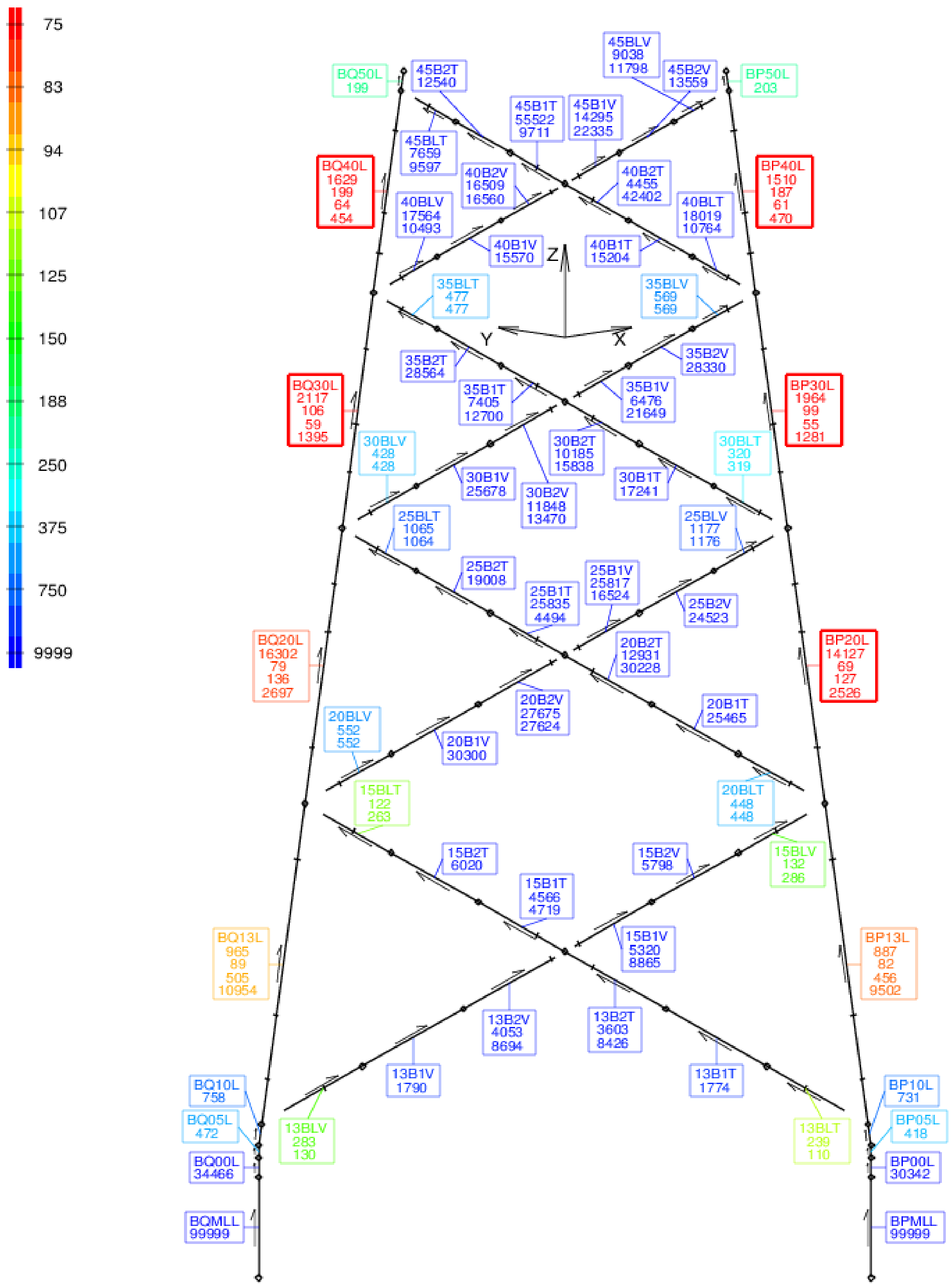
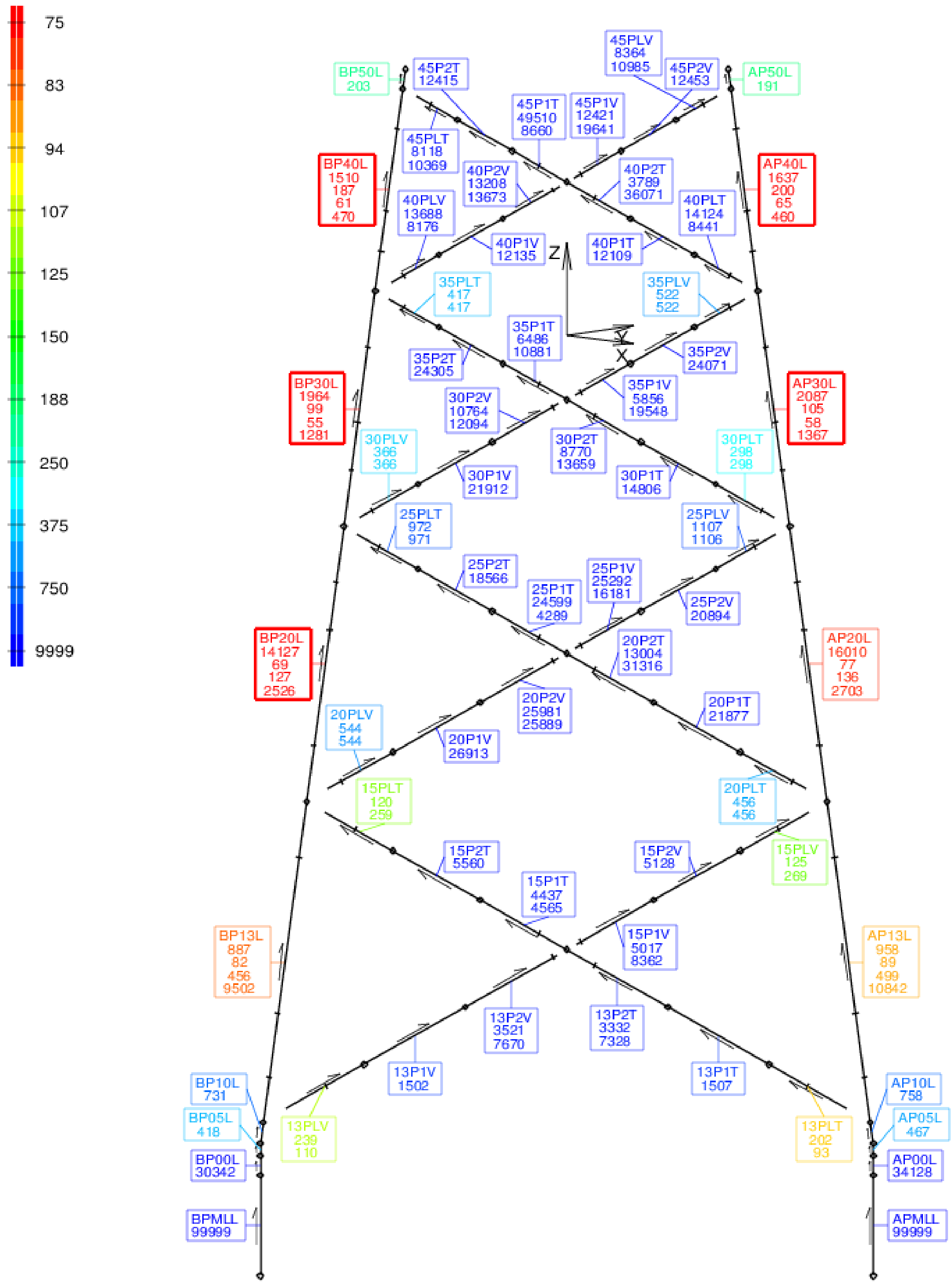
FLS results - Jacket Legs – SN Curve D



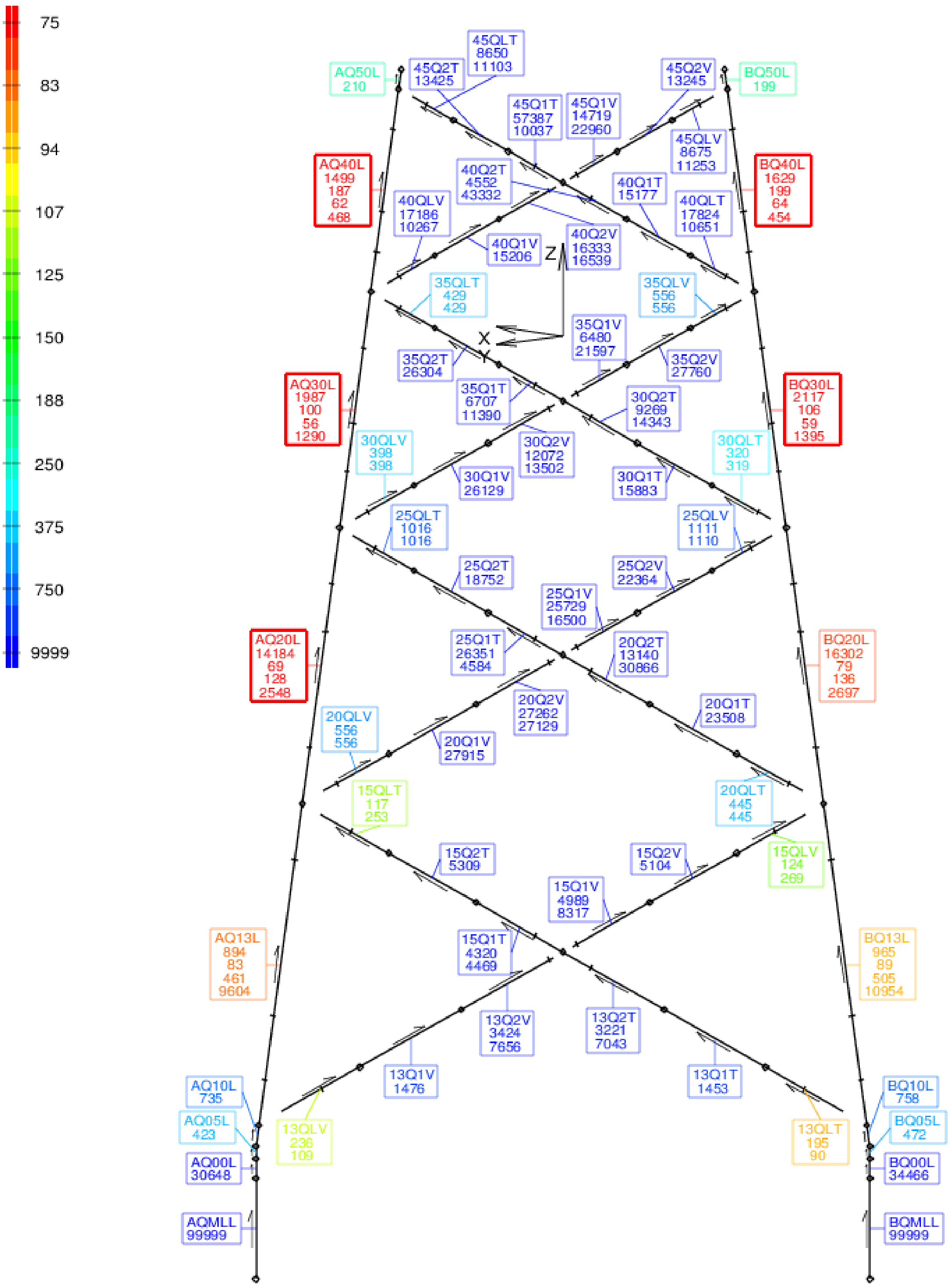
Fatigue Lives, Jacket Side A, SN Curve D to be considered for legs



Fatigue Lives, Jacket Side B, SN Curve D to be considered for legs

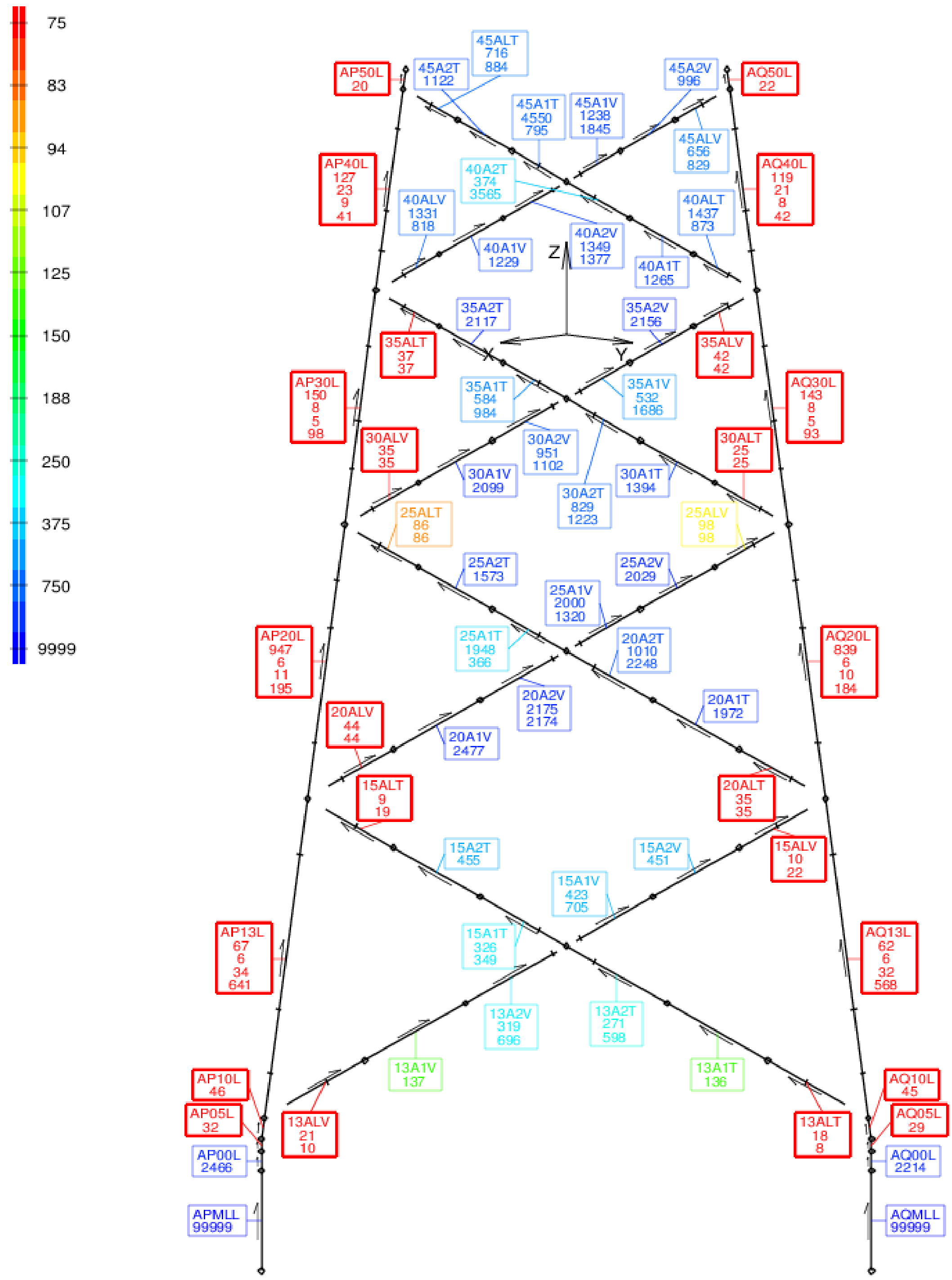


Fatigue Lives, Jacket Side P, SN Curve D to be considered for legs

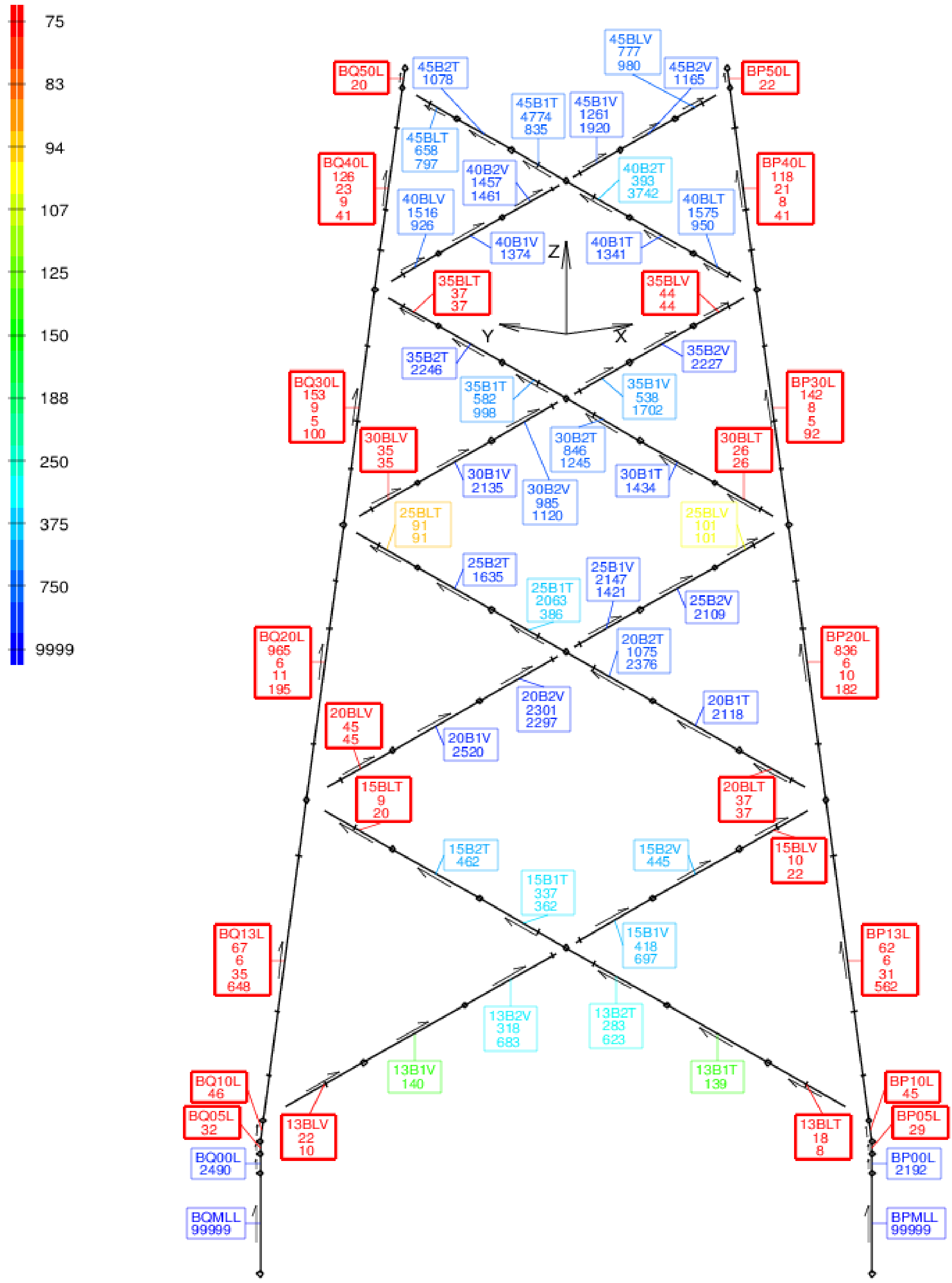


Fatigue Lives, Jacket Side Q, SN Curve D to be considered for legs

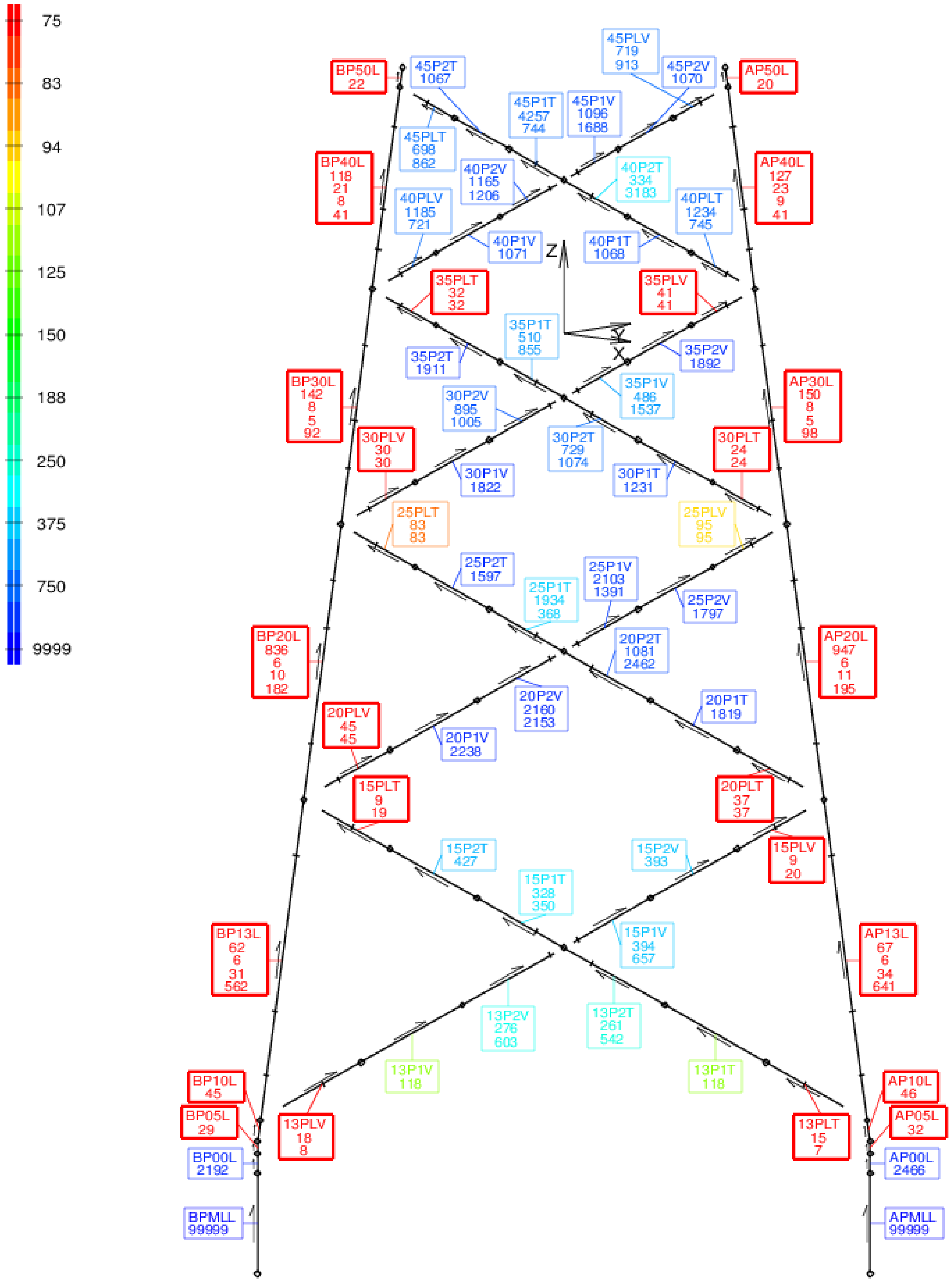
FLS results -Jacket Braces – SN Curve F3



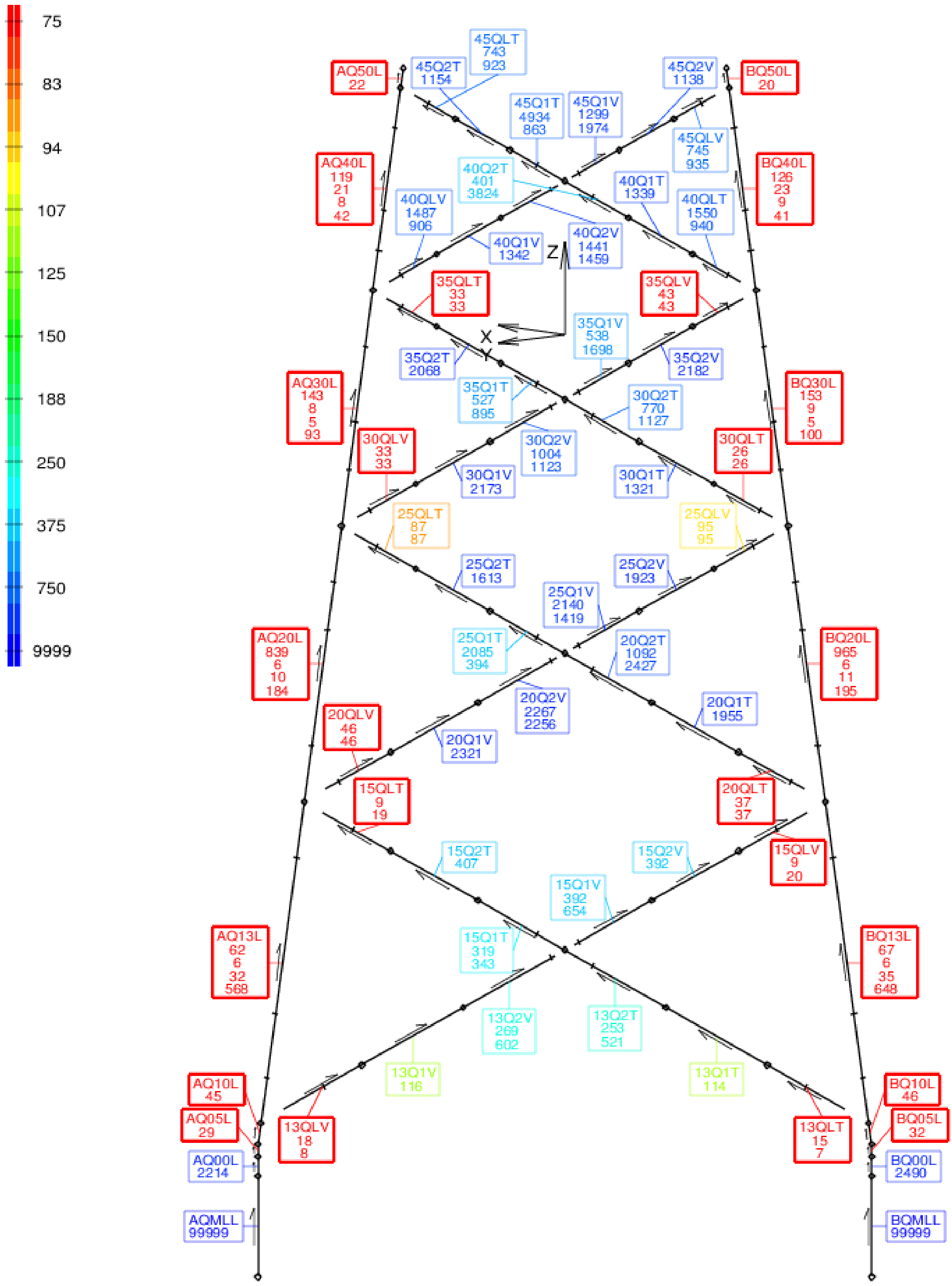
Fatigue Lives, Jacket Side A, SN Curve F3 to be considered for braces



Fatigue Lives, Jacket Side B, SN Curve F3 to be considered for braces

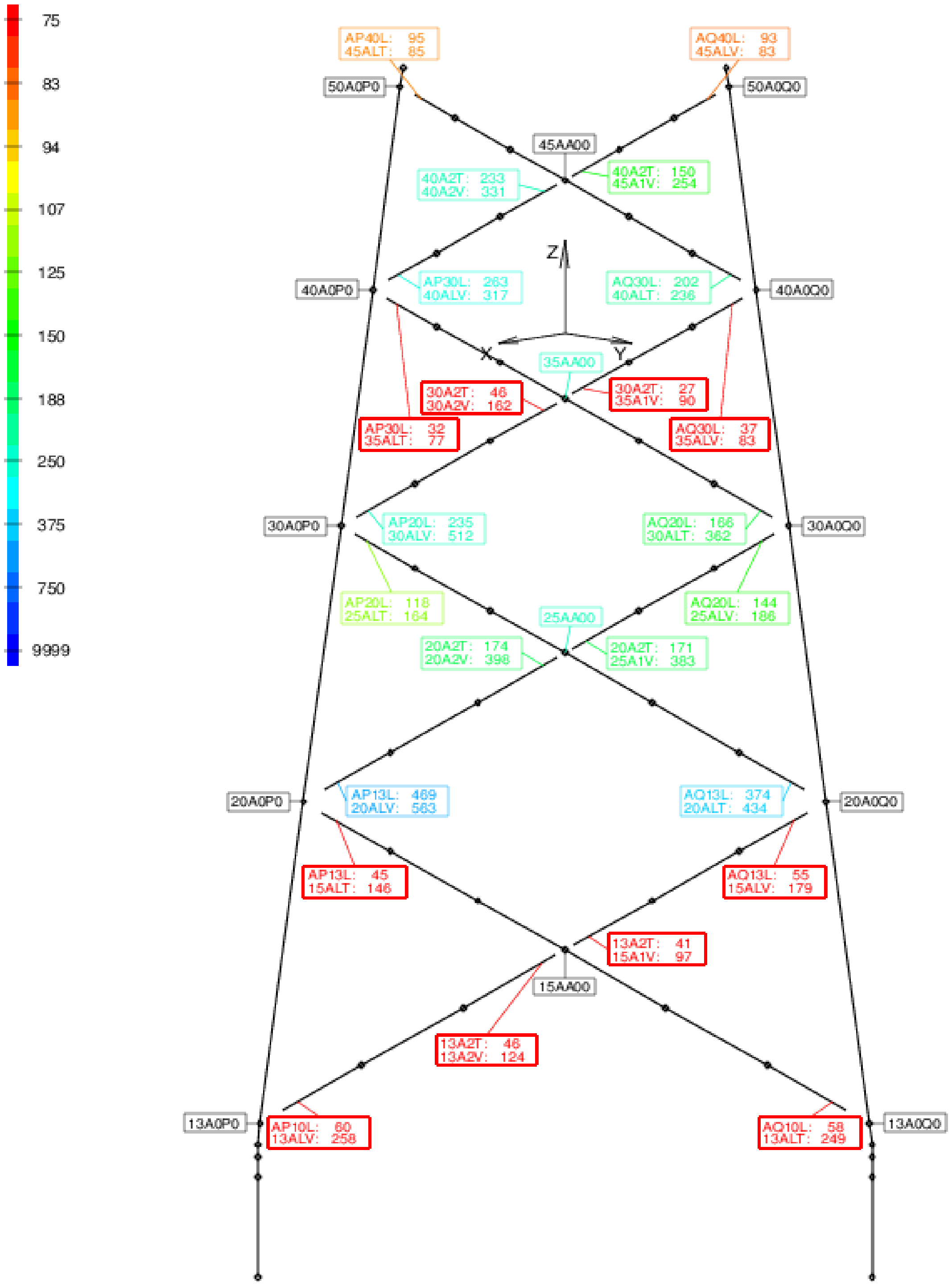


Fatigue Lives, Jacket Side P, SN Curve F3 to be considered for braces

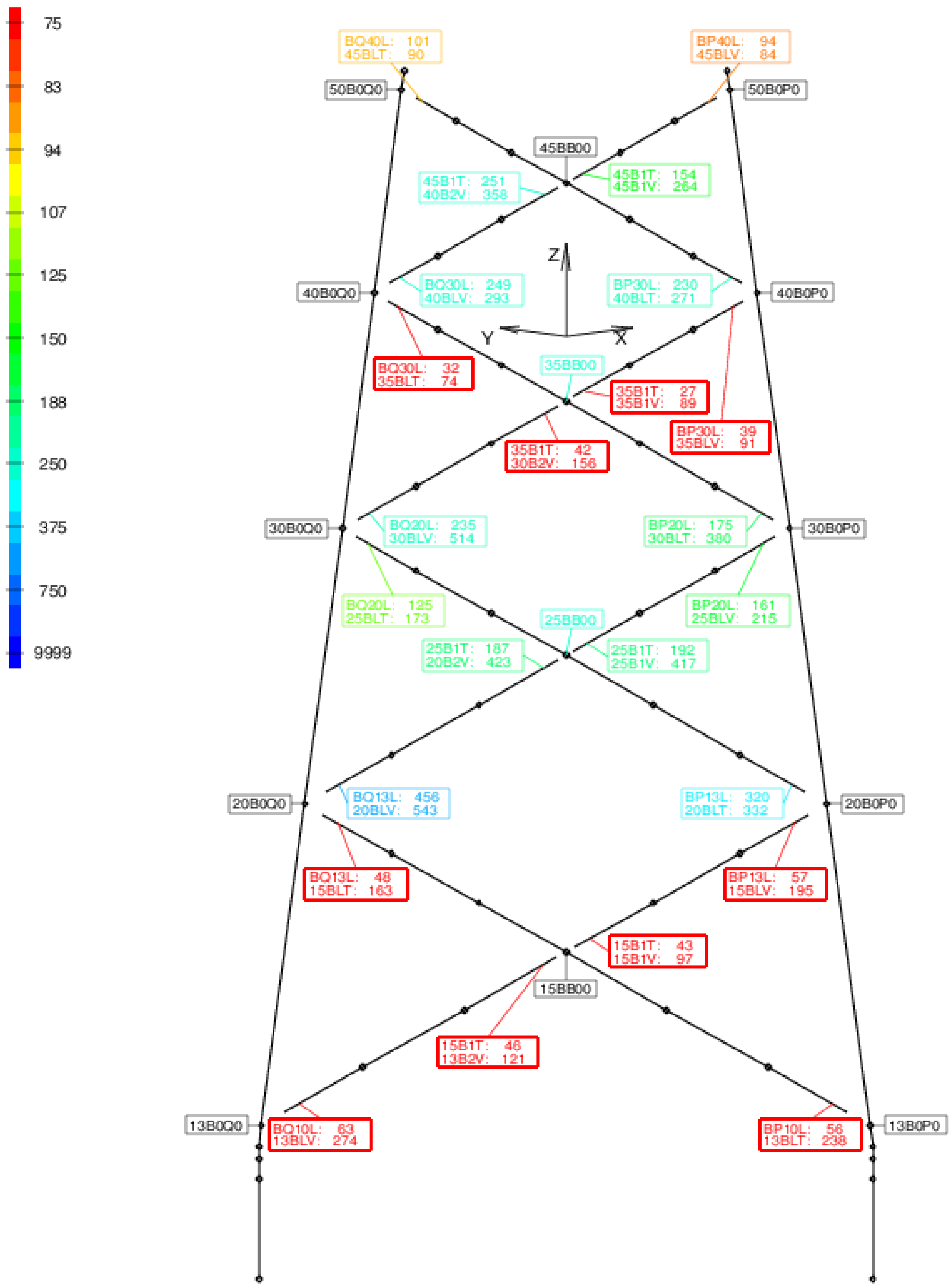


Fatigue Lives, Jacket Side Q, SN Curve F3 to be considered for braces

FLS results -Jacket Joints – SN Curve T

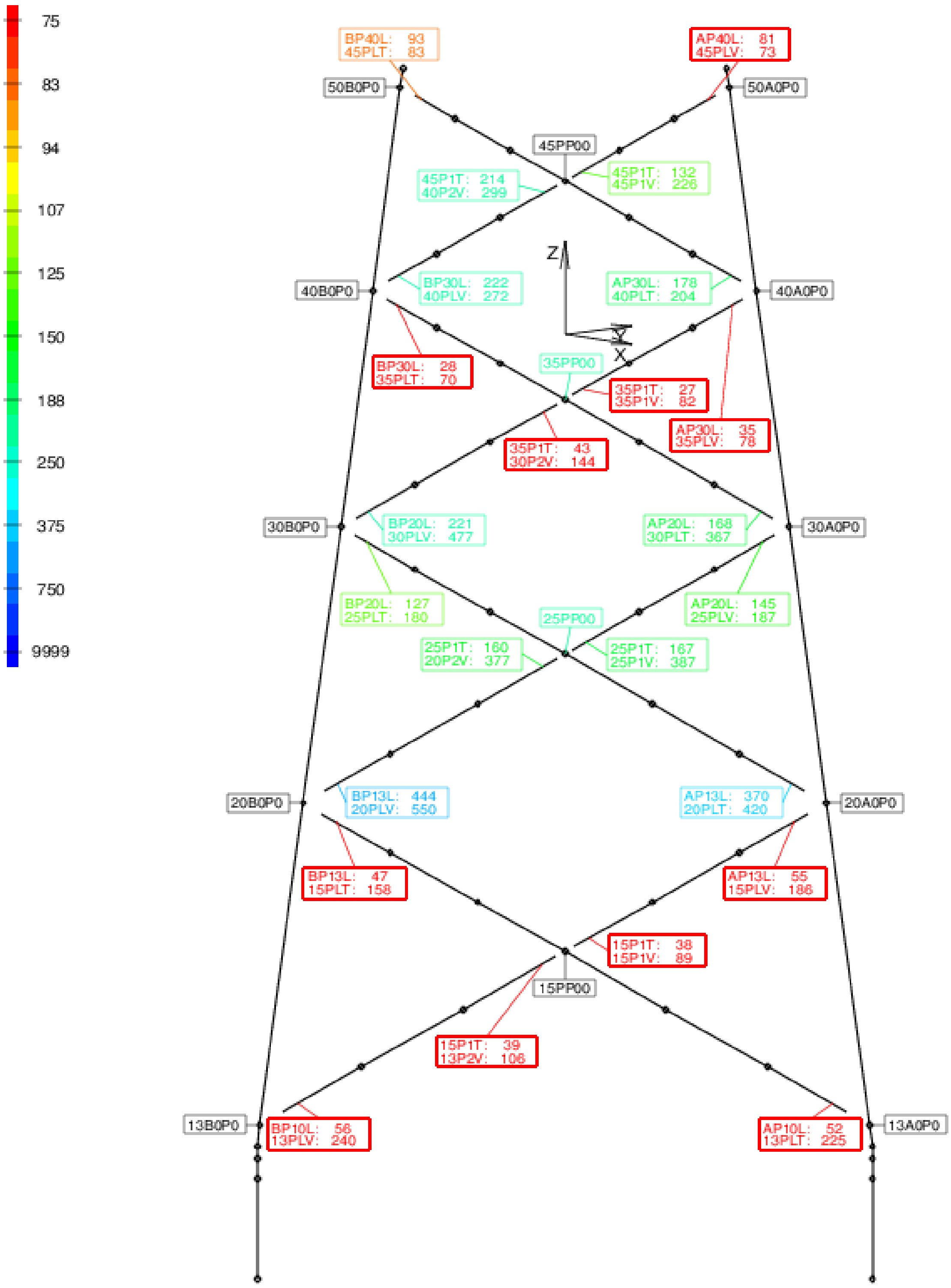


Fatigue Lives, Limit = 75
Jacket Side A - View from outside jacket

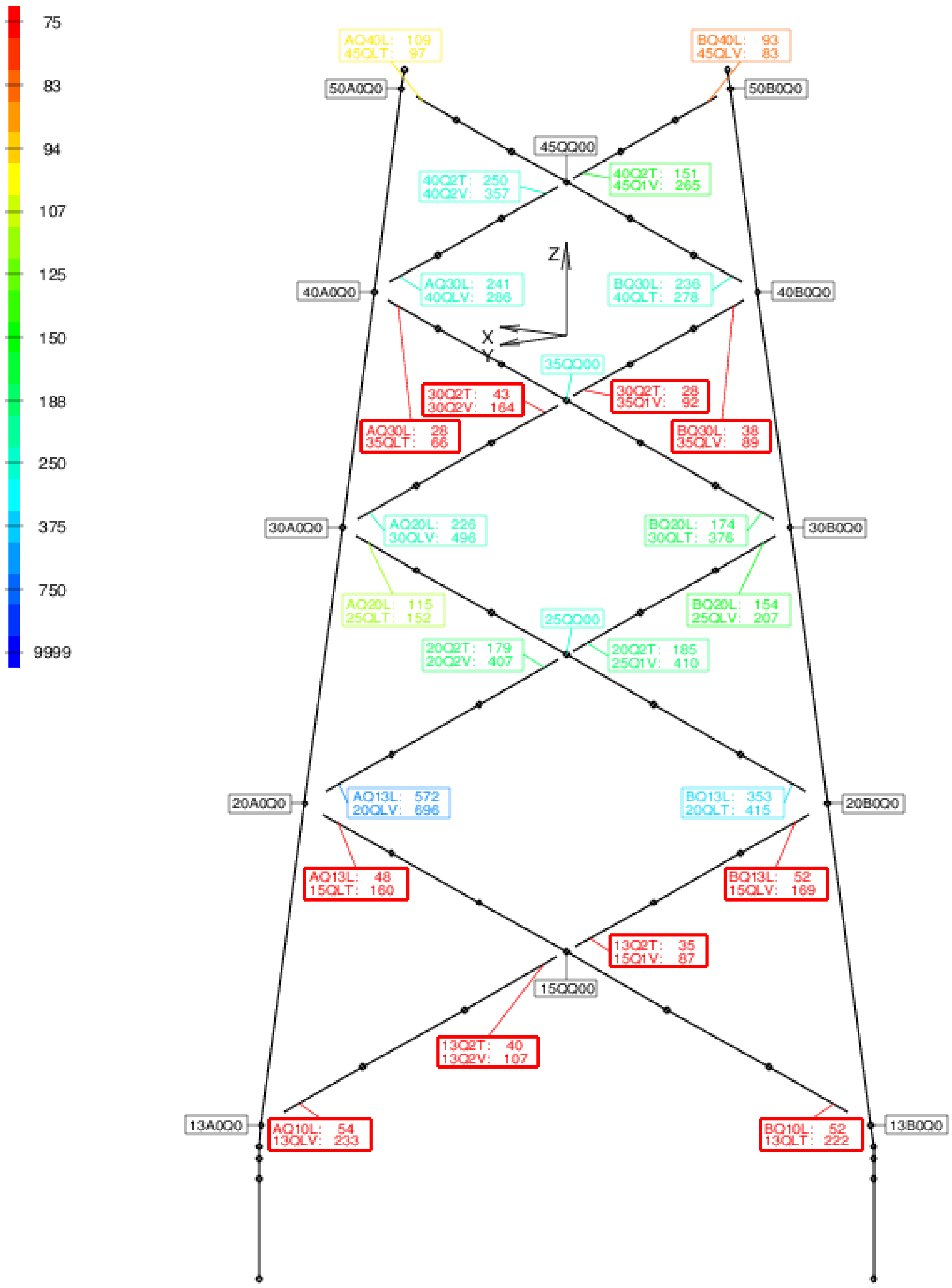


Fatigue Lives, Limit = 75

Jacket Side B - View from outside jacket



Fatigue Lives, Limit = 75
Jacket Side P - View from outside jacket



Fatigue Lives, Limit = 75

Jacket Side Q - View from outside jacket