A comprehensive method for the structural design and verification of the INNWIND 10MW tri-spar floater

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Abstract. The present paper presents a comprehensive method for the structural design and verification of a floater, accounting for both ultimate limit state (ULS) and fatigue limit state (FLS). Three software are properly combined, a 3D Finite Element Method (FEM) solver, a coupled time domain hydro-servo-aero-elastic tool and a frequency domain hydrodynamic solver. The detailed analysis is performed in the 3D FEM solver through static load cases for ULS and frequency domain stochastic analysis for FLS, while the other tools provide the environmental excitation. The method is used for the detailed design and verification of the INNWIND 10MW tri-spar concrete floater that is assessed in terms of capacity ratios (for ULS) and damage ratios (for FLS), estimated at the most critical connecting positions.

1. Introduction

As wind energy moves to deep offshore due to the high wind potential and the abundantly available space in open seas, offshore floating wind turbines (OFWT) have been brought to the front line of interest within the wind energy community. Numerical tools and design procedures developed for onshore wind turbines (WTs) are being extended, in order to assure on one hand safety and security and on the other, cost-effective solutions for this new demanding concept. To this end, the aero-elastic tools have been properly modified in order to include the wave and the current excitation, the additional degrees of freedom for the 6 rigid body modes of the floater motion and the mooring line modeling in case of OFWTs. In practice, the design of the WT components (tower, drive train and blades), considering both the ultimate limit state (ULS) and the fatigue limit state (FLS), is performed on the basis of IEC design load cases (DLCs). Time domain calculations are performed considering the WT operating in its normal and idling states and also under the occurrence of faults. For composite components (i.e. the blades), the obtained internal loads are then introduced in 2D cross sectional analysis tools, in order to perform FLS and ULS checks at stress level.

Focusing exclusively on the structural design and verification of the floater, its complex geometry (i.e. in case of a semi-submersible floater) calls for a full 3D FEM model rather than a 2D cross sectional tool, in order to accurately provide stress distribution at the connecting points, without considering stress concentration factors that are necessary in case beam theory is applied. Because time domain coupled hydro-servo-aero-elastic simulations including the full 3D FEM model of the floater require substantial computational effort, the practice followed for the design of the blades is suited to the floater as well. Analysis of the coupled structure (WT, floater and mooring lines) using standard engineering tools provide the external loading that will be introduced at a later stage in the 3D FEM solver for the detailed design. The computational effort can be further reduced by performing
(considering the isolated floater) static calculations for ULS and frequency domain stochastic analysis for FLS, instead of performing time domain simulations under the WT and the wave loading applied at the tower base and the wet surface of the floater respectively. In the ULS case the maximum value of the external loading is imposed, while in the FLS case the lifetime power spectral density (PSD) of the corresponding loading, as estimated through the processing of the coupled analysis results.

The present paper presents a comprehensive method for the structural design and verification of a floater, by properly combining 3 software; a 3D FEM solver (SAP2000 [1]), a coupled time domain hydro-servo-aero-elastic tool (hGAST [2]) and a frequency domain hydrodynamic solver (freFloW [3]). Results concerning the detailed design and verification of the INNWIND 10MW tri-spar concrete floater [4], [5] are presented, providing the capacity ratios (for ULS) and the damage ratios (for FLS), estimated at the most critical positions.

2. Numerical tools

2.1. 3D Finite Element Method Solver
Static and dynamic analyses of the floater are performed using Sap2000 [1]. It is a general purpose, civil engineering software, ideal for the analysis and design of any type of structural system. A wide variety of elements (beams, shells and solid elements) is offered. Dynamic methods include response spectrum, power-spectral-density steady-state (for fatigue behaviour with optional damping and complex-impedance properties) and time domain analyses.

Design is fully integrated with the analysis process, providing results before automatically sizing steel members and designing reinforced-concrete sections. Automatic steel, aluminium and concrete design code checks ensure that the analysed structures meet criteria of a variety of international standards, including the American and European ones.

2.2. Coupled hydro-servo-aero-elastic time domain solver
Nonlinear time domain simulations of the coupled floating wind turbine are performed using NTUA’s in-house servo-aero-elastic solver hGAST [2]. hGAST is a multibody, FEM solver, in which the full wind turbine is considered as a multi-component dynamic system having as components the blades, the drive train and the tower; all approximated as beam elements. In the present work the floater is assumed rigid, although modeling through beam elements is also possible, while a dynamic mooring line module with corotational truss elements (finite elements only subjected to extensional loading) provides the necessary stiffness to the floater.

Rotor aerodynamics is simulated using a Blade Element Momentum (BEM) model. An elaborated BEM model is employed that accounts for dynamic inflow, yaw misalignment, and unsteady aerodynamics and dynamic stall effect through the ONERA model [6]. Hydrodynamic loading is based on linear, potential hydrodynamic theory and on the viscous drag term from Morison’s equation. By defining the external environmental wind and wave conditions, the code provides, among others, time histories of internal loads and deflections at the nodes of the components.

2.3. Frequency domain hydrodynamic solver
The hydrodynamic problem considering the floater interacting with the incoming waves is solved in the frequency domain using the in-house hybrid integral equation method freFloW [3]. freFloW solves the Laplace equation in 3D using the Boundary Element Method by adopting the indirect formulation with constant source distributions and by satisfying Garrett’s analytic solution at the matching boundary. The solution procedure provides the exciting loads, the added mass & damping coefficients, the response amplitude operators (RAOs) of the floater and the total (considering both diffraction and radiation loads) 1st order hydrodynamic loading and hydrodynamic pressure on the floater surface.

For the RAOs estimation the following input is needed; the floater external geometry, the mass and the stiffness matrices due to moorings, gravity and hydrostatics as well as, additional mass, stiffness
and damping matrices induced by the WT. The latter are obtained by projecting the WT loads to the floater’s 6 degrees of freedom and by that include the linearized contribution from aerodynamics, inertial and gravitational loading on the WT parts. For their estimation, a simplified reduced order model of hGAST is used, based on modal analysis and steady linearized blade element equations [7].

3. Method for design and verification
The comprehensive method for the floater detailed structural design and verification accounts for both ultimate and fatigue loading conditions, as defined in IEC standards [8], [9]. As already mentioned in the introductory section, static analysis is performed for ULS and frequency domain stochastic analysis for FLS, allowing significant reduction of the computational cost.

The analysis is carried out in SAP2000 3D FEM that permits the accurate detailed design and verification of parts of the structure at the stress level, as for example the joint connections of a semi-submersible floater. The floater is modeled using shell thick elements, while its 6 rigid body modes of motion are included in the model and a generalized stiffness matrix is introduced at the reference point of the floater, taking into account the linearized contribution from the mooring lines, hydrostatics and gravity. The external loading applied on the 3D FEM model of the floater includes the gravity force, the hydrostatic pressure, the hydrodynamic pressure due to waves and sea current and the wind turbine loading communicated through the tower base. The latter includes the contribution from aerodynamics, inertial and gravitational loads transferred to the tower base. The estimation of the gravity force and the hydrostatic pressure applied on the 3D FEM model is trivial, while the estimation of the WT loading and the hydrodynamic pressure is performed through coupled hydro-servo-aero-elastic simulations and solution of the hydrodynamic problem in the frequency domain respectively.

The processing of the loading time histories (3 forces and 3 moments) at the tower base provides; a) the maximum tower base loading (input to the static analysis for ULS) and b) the PSD of the tower base loading (input to the frequency domain stochastic analysis for FLS). For estimating the hydrodynamic loading the maximum pressure over the wet surface of the floater is estimated for ULS and the pressure PSD for FLS. The frequency domain hydrodynamic solver provides the transfer function of the total pressure (from diffraction and radiation problems) \( p(x, \omega)/A \) at any point \( x \) on the floater’s wet surface for unit wave amplitude \( A \), as a function of the wave frequency \( \omega \). By choosing a wave spectrum \( S(\omega) \), i.e. the JONSWAP one, with parameters the peak wave period \( T_p \) and the significant wave height \( H_s \), the pressure PSD and the significant and maximum pressure at a point \( x \) are defined as:

\[
p_{\text{FID}}(x, \omega) = \left[ p(x, \omega)/A \right] S(\omega; T_p, H_s) \quad (1)
\]

\[
p_s(x) = 2 \sqrt{\int_0^\infty p_{\text{FID}}(x, \omega) \, d\omega} \quad (2)
\]

\[
p_{\text{max}}(x) = 1.86 \cdot p_s(x) \quad (3)
\]

Equation (3) guarantees that the maximum pressure over the floater’s surface will be simultaneously applied and in turn the maximum strength for the wet part will be estimated. This will check the wet surface of the floater for local failure due to compression loads. On the other hand, simultaneous application of the maximum pressure over the floater’s wet surface might cancel out the developed moments on parts outside the water. For designing the joint connections i.e. between the tripod and the concrete cylinders, the developed moments should be recorded at these critical points of the structure during the coupled time domain analysis. Then the instantaneous pressure field that produces the maximum moment should be applied to the 3D FEM model.

Regarding the design verification a) for ULS; the 3D FEM solver provides the capacity ratios on the FEM nodes (ratio of maximum calculated stress including safety factors divided by the yield stress of material), while b) for FLS; processing of the provided stress PSDs is required at specific critical points that are selected based on the maximum root mean square (RMS) stress. Based on the provided PSDs, time series of stresses (3 realizations of 1hour) are generated that in turn processed using
rainflow counting in order to provide the stress histograms. The fatigue lifetime is finally calculated based on the S-N curves, under the assumption of linear cumulative damage (Palmgren-Miner rule) proving the damage ratios according to [10].

The DLCs considered for the estimation of ULS and FLS are iteratively repeated until all constrains are fulfilled (capacity and damage ratios should be less than 1). In this process no modification to the main dimensions of the floater (i.e. the length and the outer diameter of each component) is performed and therefore the external forcing provided by the hydro-servo-aero-elastic tool and the frequency domain solver remain unaffected and can be reused. During the loops adjustment mainly of the wall thickness is performed, while the ballast mass is properly modified in order to maintain the overall mass of the floating platform.

4. Detailed design and verification of the INNWIND 10MW tri-spar concrete floater

The present methodology was applied for the detailed design and verification of a tri-spar floating support structure for the DTU 10MW reference WT [11], under Work Package 4 of the INNWIND.EU project. The preliminary design of the floater was performed by the University of Stuttgart [4] and further elaborated in the project. Detailed description of the concrete tri-spar floater, the mooring lines, the wind turbine, the controller and the site conditions can be found in [5].

The floater is formed by 3 concrete vertical cylinders that provide the buoyancy, placed at a distance of 26m from the floater’s center line. Their outer diameter, draft and elevation above sea water lever (SWL) are 15m, 54.5m and 10.5m respectively. At the bottom, heave plates of 22.5m diameter and 0.5m height are considered. A steel tripod is mounted on top of the three cylinders, while the position of the tower base is 25m above SWL. In Table 1, the initial values of the main platform properties from the preliminary design (input) and the final ones derived from the detailed design and verification are presented.

4.1. Time domain simulations

Time domain hydro-servo-aero-elastic simulations have been performed using hGAST in order to estimate the ultimate and the fatigue loads of the coupled floating structure. In Table 2 the list of the performed DLCs is given. Each case corresponds to 1hour simulation. For the fatigue case (DLC1.2) one turbulent field per wind speed has been used, while for the ultimate cases (DLC1.3, 1.6, 6.1, 6.2) three. For the parked DLC6.1 two wave directions have been considered (0°, 30°), while for DLC6.2 three yaw angles (-30°, 0°, 30°) respectively. In the latter DLC the wind and the wave are co-directional. The site conditions considered for the design of the tri-spar floater are for a medium site with 50-year significant wave height 10.9m, peak period varying from 9 -16s, water depth 180m, wind turbine class C and 50-year hub velocity 44m/s.

The maximum tower fore-aft moment is depicted in DLC1.6 at 13m/s and chosen as the most severe load. The maximum fore-aft moment and the other concurrent loads after applying the corresponding safety factors are given in Table 3. By processing the time histories of DLC1.2 and selecting a Weibull distribution with parameters $C$=11m/s and $k$=2, the overall lifetime PSD of the tower base loading is estimated. In Figure 1 the PSD of the tower base fore-aft moment is presented in logarithmic scale. The highest peak at low frequency ~0.043Hz corresponds to the floater pitch mode, the peak at ~0.15Hz to the wave excitation frequency and the peak close to 0.48Hz to 3P frequency.

4.2. Frequency domain analysis

The hydrodynamic properties of the tri-spar floater are estimated using freFLOW solver. Due to the symmetry of the floater’s geometry in the xz plane the computational domain is reduced to half, in order to reduce the computational cost. In total 18573 panels have been used. In Figure 2 the diffraction hydrodynamic loads (fixed floater) are compared against the total hydrodynamic loads (coupled floating floater including the linearized contribution of the WT) for wave angle 0°. The total surge force and pitch moment are almost half of the diffraction ones, while the heave force remains...
almost unchanged. So application of the total pressure field in the 3D FEM model is justified in view of coming up with a cost-effective solution for the tri-spar concrete floater.

### Table 1. 10MW tri-spar concrete floater main properties.

| Part                  | Description                              | Preliminary design | Detailed design |
|-----------------------|------------------------------------------|--------------------|-----------------|
| **Platform**          | Elevation of tower base above SWL (m)   | 25.00              | 25.00           |
|                       | Center of mass below SWL (m)             | 36.02              | **35.97**       |
|                       | Platform total mass (tn)                 | 28268              | 28268           |
|                       | Ballast Mass (tn)                        | 17264              | **15653**       |
|                       | Inertia about center of mass Ixx (tn m\(^2\)) | 18674000      | **17451000**   |
|                       | Inertia about center of mass Iyy (tn m\(^2\)) | 18674000      | **17451000**   |
|                       | Inertia about center of mass Izz (tn m\(^2\)) | 20235000      | **20145000**   |
| **Concrete columns**  | Length (m)                               | 65.0               | 65.0            |
|                       | Distance to the centre (m)               | 26.0               | 26.0            |
|                       | Diameter (m)                             | 15.0               | 15.0            |
|                       | Elevation above SWL (m)                  | 10.5               | 10.5            |
|                       | Mass (tn)                                | 3279.5             | 3279.5          |
| **Heave Plates**      | Thickness (m)                            | 0.5                | 0.5             |
|                       | Diameter (m)                             | 22.5               | 22.5            |
|                       | Mass (tn)                                | 678.7              | **1639.3**      |
| **Tripod**            | Total Height (m)                         | 15.00              | 15.00           |
|                       | Height Outer Cylinder (m)               | 11.00              | 11.00           |
|                       | Diameter Outer Cylinder (m)             | 5.64               | 5.64            |
|                       | Bar Cross-Section height (m)             | 5.64               | 5.64            |
|                       | Bar Cross-Section width (m)              | 5.64               | **4.62**        |
|                       | Wall thickness (cm)                      | 5.64               | 5.64            |
|                       | Mass (tn)                                | 971.3              | **948.36**      |

### Table 2. DLCs definition for time domain simulations

| DLC | Wind | Wave | Seeds | Bins [m/s] | Yaw | Wave | SF |
|-----|------|------|-------|------------|-----|------|----|
| 1.2 | NTM  | NSS  | 1     | 5, 7, 9, 11, 13, 15, 17, 21, 23, 25 | 0   | 0    |    |
| 1.3 | ETM  | NSS  | 3     | 11, 25    | 0   | 0    | 1.35 |
| 1.6 | NTM  | ESS  | 3     | 11, 13, 17, 21, 25 | 0   | 0    | 1.35 |
| 6.1 | EWM  | SSS  | 3     | 41.8      | 0   | 0, 30 | 1.35 |
| 6.2 | EWM  | SSS  | 3     | 41.8      | 0, +/-30 =Yaw | 1.10 |

### Table 3. Maximum tower base loading applied on the tri-spar floater (DLC1.6 at 13m/s, Hs=10.9m, Tp=14.8s. SF=1.3).  

| Fx [kN] | Fy [kN] | Fz [kN] | Mx [kNm] | My [kNm] | Mz [kNm] |
|---------|---------|---------|----------|----------|----------|
| 7472    | 168     | -9736   | -5186    | 621000   | 3679     |
Figure 1. Lifetime PSD of the fore-aft bending moment at the tower base (Weibull with $C=11\text{m/s}$ and $k=2$).

Figure 2. Diffraction (fixed floater) and total hydrodynamic loads (coupled floating floater including the linearized contribution of the WT) of the tri-spar floater for wave angle $0^\circ$.

Table 4. Material properties and mechanical properties.

| Material | Unit Mass $\text{tn/m}^3$ | Young’s Modulus $E$ [kN/m$^2$] | Shear Modulus $G$ [kN/m$^2$] | Poisson’s Ratio [-] |
|----------|-----------------|-----------------|-----------------|-----------------|
| C50/60   | 2.750939358     | 37000000.0      | 15416666.67     | 0.2             |
| Rebar    | 7.752647280     | 199947978.8     | 80769230.77     | 0.3             |
| S450     | 7.752647280     | 210000000.0     | 80769230.77     | 0.3             |

4.3. Floater detailed design

The initial floater characteristics defined in [4] have been initially (for the first loop of the detailed design) introduced in the 3D FEM model, assuming a wall thickness of 0.0564m and 0.4m for the steel members of the jacket and the concrete cylinders respectively. The floater detailed design and verification has been performed for the following parts of the structure; a) the steel tripod, b) the connection between the steel tripod and the concrete cylinders, c) the concrete reinforcement, and d) the heave plates. Also small modifications of the initial design have been performed, concerning; a) a slight reduction of the width of the tripod horizontal legs (from 5.64m to 4.62m), in order to ease the horizontal-vertical leg connection and b) the replacement of the steel heave plates with concrete ones.
The whole structure (steel tripod, concrete cylinders and heave plates) has been modelled using thick-shell elements, while the inclined tubes and the horizontal ties (that form the concrete-steel connection) through linear beam elements. In total 64412 finite elements have been used, 48498 of them for the tripod. The 3D FEM model in SAP2000 is shown in Figure 3. The following stiffness matrix has been introduced at the floater reference point taking into account the linearized contribution from mooring lines, hydrostatics and gravity:

\[
K_{tot} = \begin{bmatrix}
83.28 & 83.28 & 0 & 0.1794 & -2841.52 & 2.3808 \\
83.28 & 0 & 0 & 2834.33 & -7.717 & -156.51 \\
0 & 0 & 5381.41 & 0.7101 & 143.8 & 7.4522 \\
0.1794 & 2834.33 & 0.7107 & 2927178 & 28.939 & 19703 \\
-2841.52 & -7.717 & 143.88 & 28.939 & 2926708 & -832.1 \\
2.3808 & -156.51 & 7.4522 & 19703 & -832.1 & 269575
\end{bmatrix} \text{ (kN, m, rad)}
\]

The materials used for the floater and their corresponding mechanical properties are presented in Table 4. For the concrete parts concrete C50/60 has been selected and rebar B500C for the reinforcement, while for the tripod steel S450 has been used.

### 4.3.1. Detailed design of the steel tripod

The steel tripod is formed by a central cylinder of 7.7m diameter (equal to tower’s diameter), 3 horizontal brackets of 4.62m width and 5.64m height, which are connected to the central cylinder, and 3 vertical legs of 5.64m diameter and 11m height (see Figure 3, Figure 4). Initially a uniform wall thickness of 0.0564m was assumed, but the detailed design showed that at various connection points local reinforcement is needed, as detailed next.

The wind turbine tower is mounted on the central cylinder together with the three horizontal brackets. The central cylinder distributes the stresses to the horizontal brackets and offers rigidity to the overall structure. Local failures were predicted at the upper and rear parts of the central point (Figure 4a, b show the stress concentration) and thus a reinforced steel zone has been introduced, composed of steel plates of varying thickness (0.0564-0.175m). In order to increase the lateral stability of the horizontal members, three equally spaced steel diaphragms of 0.0564m thickness have been placed inside them. Also in order to avoid local failure at the gamma connection between the vertical legs and the horizontal brackets, steel triangular plates of the same thickness are introduced as shown in Figure 4c. Finally the wall thickness of the vertical legs is linearly increased from top (0.0564m) to bottom (0.175m) in order to avoid local failures at the connection point between the tripod and the floater.
concrete cylinders (Figure 4d shows the stress concentration), while four steel plates of 0.0564m thickness, which act as diaphragms have been placed every 2.75m inside the legs in order to increase their strength against buckling.

Figure 4. Critical points of tri-spar floater considered for ULS and FLS verification. Von Mises stress contours [MPa] from ULS case II. Connection points between; a) tower-floater, b) central cylinder-horizontal members, c) vertical legs-horizontal brackets and d) concrete cylinders-steel leg.

Figure 5. Connection between the steel legs of the tripod and the concrete cylinders.
4.3.2. Detailed design of the steel-concrete connection
The connection between the steel legs of the tripod and the concrete columns (Figure 5) is formed by:
a) 12 inclined steel radial rods through which the loading is transferred to 12 positions along the concrete wall, b) 12 horizontal steel ties pinned to the concrete shell that prevent the buckling failure of the concrete wall and c) a steel ring on which the ties are connected. The rods are considered pinned at both ends in order to reduce lateral buckling, while their inclination angle is 60° in order to minimize the horizontal force transferred to the concrete wall. The outer diameter and thickness of both the rods and the ties is 0.5m and 0.02m respectively.

Alternative design of the connection between the steel tripod and the three concrete cylinders could be realized i) through prestressed concrete slabs constructed on top of each cylinder, on which the steel tripod is fixed or ii) through steel plates welded on each steel leg and anchored on the concrete cylinders using steel studs, welded or bolted on the plate. However such solutions were not chosen. On one hand the construction of the prestressed concrete is difficult especially in offshore applications, while the tendons might suffer from corrosion leading to loss of tensile strength and failure of the slab. On the other hand the welding and transportation of the steel plates is difficult because of their large diameter of 15m.

4.3.3. Reinforcement of the Concrete cylindrical columns
The steel reinforcement of the concrete cylindrical columns is defined according to EuroCode2 [12]. \(\Phi 25/180\) is selected for the vertical reinforcement on the side walls and \(\Phi 20/250\) for the horizontal reinforcement at both faces, while \(\Phi 10/200\) confinement hoops are also used.

4.3.4. Heave Plates
The originally considered steel heave plates have been replaced by concrete ones, in order to avoid the steel to concrete connection between the heave plates and the floater base. The geometry of the heave plates is unchanged (0.5m height and 22.5m diameter), while the total mass per plate is 546tn. A double layer \(\Phi 36/65\) is selected for the radial reinforcement and a double layer \(\Phi 36/75\) for the peripheral reinforcement at the top and bottom faces.

4.4. ULS and FLS design verification
In Table 5 the capacity ratios at the most critical positions (also shown in Figure 4) are presented as computed by SAP2000 for the DLC1.6 at 13m/s with \(H_s=10.9m\) and \(T_p=14.8s\). Two cases have been considered regarding the applied pressure along the wet surface of the floater. Case I corresponds to the maximum pressure given by equation (3) and case II to the maximum developed bending moment at the concrete – steel connection at 10.5m above SWL. Case II drives the design of the tripod resulting to maximum capacity ratio 0.78 at the connection point between the tripod leg and the inclined rod and 0.68 at the central connection point between the 3 horizontal steel members. The highest estimated capacity ratio for the steel ties is \(~0.1\) that is rather low. However, its dimensioning is chosen so as to increase the axial stiffness of the structure and in turn reduce the amount of the horizontal force on the concrete wall.

In Table 6 the damage ratios at the most critical positions of Figure 4 are presented based on the method presented in section 3. In the table the S-N considered parameters are also given, assuming weld connections for the steel members, S-N curves in air and number of cycles greater than \(10^7\) [10]. The highest damage ratio is depicted at the connection point between the horizontal members and the central cylinder of the tripod and is equal to 0.93. High damage ratio (0.86) is obtained at the gamma connection between the vertical and the horizontal legs of the tripod as well. Both are local maxima and probably can be reduced by performing another design iteration loop aiming at fatigue reduction.

5. Conclusions
A comprehensive method for the structural design and verification of a floater has been presented, accounting for ULS and FLS. The analysis is performed at the stress level in a 3D FEM solver
considering static analysis for ULS and frequency domain stochastic analysis for FLS, while a coupled
time domain hydro-servo-aero-elastic tool and a frequency domain hydrodynamic solver provide the
external loading. Based on this method, the detailed design and verification of a tri-spar concrete
floating support structure for the DTU 10MW RWT has been performed. For the current detailed
design presented in this work, the FLS seems to drive the design of the floater.

| Table 5. ULS verification: capacity ratios at critical positions (DLC1.6 at 13m/s,
Hs=10.9m, Tp=14.8s) |
|---------------------|----------|----------|
| Critical Position   | Capacity ratios |           |
|                     | I        | II       |
| 1. Central Cylinder -Horizontal Leg Connection | 0.64 | 0.68 |
| 2. Horizontal Leg-Vertical Leg Connection | 0.26 | 0.28 |
| 3. Vertical Leg –Inclined Rods Connection | 0.64 | 0.78 |
| 4. Inclined Rods | 0.46 | 0.54 |
| 5. Ties | 0.08 | 0.09 |

| Table 6. FLS verification: 20 years damage ratios at critical positions |
|---------------------|----------|----------|
| Connection                  | S-N curve parameters | Damage Ratio |
| Type | log(a) | M | |
| 1. Central Cylinder – Horizontal Leg | B2 | 16.856 | 5 | 0.31 |
| 2. Horizontal Leg at inclination point | C | 16.320 | 5 | 0.93 |
| 3. Horizontal Leg –Vertical Leg | B2 | 16.856 | 5 | 0.86 |

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