Structural Design of a Prestressed-Concrete Spar-type floater for 10 MW wind turbines

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Abstract. Prestressed concrete is gaining growing interests as an alternative to steels for the construction material for floating platforms of wind turbines due to its low material costs. Investigation of the characteristics of the concrete structure under the fluctuating loadings are necessary to ensure the structural integrity, where a reference model of both the floating concrete platform and accompanying structural design are highly useful. In this study, a reference model for prestressed-concrete spar-type floater for 10 MW wind turbines is designed together with its concrete structure. Frequency-domain analysis showed that an optimum floater diameter that gives minimum pitch angle could be found. Although the maximum pitch angles were similar for floaters with diameters of 15 m to 17 m, structural design showed that the case with the wall thickness of 0.35 m will subject to prestressed-concrete steel yielding, concrete bending cracks, and shell buckling due to water pressure. Finally the reference model is designed for a floater diameter of 16 m and wall thickness of 0.45 m.

1. Introduction

Prestressed concrete (PC) is gaining growing interest as an alternative to steels for the construction material for floating platforms of wind turbines due to its low material costs and expected high durability reported in previous experiences with concrete barges [1]. Unlike concrete barges, however, floating platforms of wind turbines experience constant high cyclic loadings from the mounted wind turbine and wave excitations, which can lead to various fatal phenomena such as crack propagation. While several existing design standards [2], [3] cover offshore concrete structures, the characteristics of concrete structures under such loadings of floating platforms of wind turbines have not yet fully studied, and intensive research need to be conducted from both structural and material aspects in order to ensure the structural integrity. For the investigation, a reference model of a floating concrete platform with accompanying structural design can be highly useful.

Several designs have been provided for concrete floating platforms for wind turbines. Viselli et al. (2015) [4] and Yu et al.(2018) [5] proposed semi-submersible floaters, Campos et al. (2014) [6] designed a spar-type floater, and Walia et al. (2017) [7] studied TLP type floaters. In these studies, aspects of concrete floaters such as the floater motion, the cost leverage, and construction methods were studied. However, limited information has been provided for the detailed design of the concrete structure.

The aim of this study is to provide a reference model for prestressed-concrete spar-type floater together with its structural design. First, the design of a spar-type floating platform for the DTU 10MW
reference wind turbine is conducted. The effects of the concrete wall thickness, the ballast density, and concrete compression strength to the motion characteristics of spar-type floaters are investigated using frequency domain analysis. Then, using the results from the coupled aero-hydro-servo-elastic analysis for the optimized floater model, the cross-sections of the prestressed concrete floater is designed.

2. Design basis
The design basis for the reference model is presented in this chapter. For the metocean data, those used for the design in Fukushima FORWARD project [9] was used, which are summarized in Table 1. It is noted that although the water depth for the project is 120 m, it was changed to 200 m in this study to allow more flexibility for the design of the reference model. For the wind turbine, the DTU 10MW reference model was used. The basic properties of the wind turbine and the steel tower is summarized in Table 2. The mass, geometry and the controller of the wind turbine were the same as the definitions in reference [5] and [8]. The steel tower was newly designed for present floater, to avoid 3P and 9P of the blade rotation, and to withstand the ultimate strength obtained in Chapter 3.

| Table 1. Summary of the metocean condition |
|-------------------------------------------|
| Water Depth                              | 200 m | Reference turbulence | 0.12 % |
| 10 min average wind speed, 50 years return period | 48.3 m/s (at 60 m height) | Significant wave height, 50 years return period | 11.71 m |
| Wind shear at extreme condition          | 0.1   | Significant wave period, 50 years return period | 13.0 s |

| Table 2. Basic properties of the mounted wind turbine and the steel tower |
|---------------------------------------------------------------------------|
| Hub height | 119 m | Hub mass | 105,520 kg |
| Nacelle mass | 446,036 kg | Hub COG | (-7.073, 0, 119.0) |
| Nacelle COG | (2.687, 0, 118.08) | Tower mass | 628,442 kg |
| Blade mass (3 Blades) | 132148 kg | Tower COG | (0, 0, 63.56) |
| Fairlead height | -22.0 m |

| Table 3. Design basis for the prestressed-concrete floater |
|-----------------------------------------------------------|
| Type of concrete structure | Prestressed Concrete | Installation method | Vorspann System Losinger (VSL) |
| Concrete class | High strength concrete | PC steel (axial direction) | Strand of 7 SWPR7BL, $\phi = 12.7$ mm |
| Concrete strength design | 60 N/mm² | PC steel 0.2 % offset yielding strength | 1580.4 N/mm² |
| Concrete density material | 2500 kg/m³ | Covering depth for PC steel | > 90 mm |
| Concrete module | 33500 N/mm² | Covering depth for reinforcement | > 75mm |
| Concrete ratio Poisson's | 0.2 | Distance between sheath | > 355 mm |
| Main reinforcement grade | D19@300, SD345 | shear reinforcement grade | Lower than D13@300 |
| Distance between reinforcing steel | > 100mm |

The design basis for the PC structure is shown in Table 3. Depending on the amount of the tension introduced in the tendon, prestressed-concrete is categorized into several types; fully prestressed structure that do not allow tensile stress in concrete, partially prestressed structure that allow tensile stress in concrete, and post-tensioned structure that applies the tension on the tendons after the concrete is hardened.
stress in concrete but do not allow cracks. Fully prestressed concrete is most reliable for water-tightness and crack control, while partially prestressed structure allows more flexibility to the design. In present study, the reference model of the concrete structure was designed with partially prestressed structure that allows tensile stress but not cracks. In this study the design strength for concrete was set as 60 N/mm², which is within the range of design strength of the standard precast concrete. Concrete material density and Young’s modulus were determined using the design strength based on the standard of Architectural Institute of Japan (AIJ) [10]. Other design basis such as the covering depth of PC steels and reinforcements, the distance between reinforcing steels and sheath were determined based on the standards of AIJ [10], DNV GL [2], and Eurocode [11], where the most severe regulation of the three was applied.

3. Parametric design of the floater

3.1. Floater geometry

Previous designs for steel spar-type floaters such as Hywind OC3 [12] included a tapered part near the water surface. However, in order to avoid the complexity of the stress flow around the tapered part when the prestress is introduced, a simple cylinder was chosen as the basic geometry of the floater. The outline of the basic geometry of the floater is shown in Figure 1. For a simple cylinder, configurations such as the ballast height, center of gravity, and gyrational radius can be determined from the information of floater diameter, wall thickness, and ballast density using static equilibrium. Figure 2 shows the changes of the ballast height, center of gravity of the system, and gyrational radius with respect to the floater diameter and wall thickness C_t, when the ballast density is fixed to 3800 kg/m³.

![Figure 1. Basic geometry of the floater](image)

![Figure 2. The (a) Ballast height, (b) center of gravity for the whole system, and (c) gyrational radius with respect to floater diameter](image)

3.2. Parametric design using frequency-domain analysis

Frequency-domain analysis was performed to study the dependence of the characteristics of the floater motion on the design parameters: floater diameter, wall thickness, and ballast density. In the design of
a floating platform, there can be several design driving parameters such as pitch motion, nacelle acceleration, and mass of the floater. In this study, maximum pitch incline angle was used to characterize the floater motion. For simplicity, pitch motion was approximated as the sum of the incline angles due to wave excitation and those due to wind turbine thrust force. For pitch motion due to wave excitation forces, one degree of freedom in pitch direction is considered. For the purpose of linearization required in frequency-domain modelling, hydrodynamic force \( f_{\text{hydro}} \) was estimated ignoring the drag force as shown in Eq. 1.

\[
f_{\text{hydro}} = C_M \rho \frac{\pi D^2}{4} \ddot{u}
\]

where \( \rho \) is water density, \( D \) is floater diameter, \( \dddot{u} \) is the acceleration of the water particle, and \( C_M \) is the added mass coefficient. The hydrodynamic rotational moment on the whole floater in pitch direction \( F_{\text{pitch}} \) can be estimated by integrating Eq. 1 over the floater height as Eq. 2.

\[
F_{\text{pitch}} = \int_{z_d}^0 f_{\text{hydro}}(z - z_0)dz
\]

where \( z \) is the height, \( z_d \) is the floater draft, and \( z_0 \) the height of the centre of gravity. The amplitude of the pitch motion under regular wave with angular frequency of \( \omega \) can then be estimated as Eq. 3.

\[
x(\omega) = \frac{X_{st}(\omega)}{\sqrt{\left(1 - \frac{\omega^2}{\omega_n^2}\right)^2 + \left(2\zeta\frac{\omega}{\omega_n}\right)^2}}
\]

where \( \omega_n \) is the pitch natural frequency, \( \zeta \) is the damping ratio, and \( X_{st} \) is the static displacement expressed as Eq. 4, where \( I_{yy} \) is the moment of inertia in pitch direction.

\[
X_{st}(\omega) = \frac{F_{\text{pitch}}(\omega)}{I_{yy}}
\]

Pitch motion in frequency domain was obtained by multiplying Eq.3 by the spectrum of input irregular wave. The maximum pitch angle was obtained by applying the inverse FFT to the response function, where 12 phases of random seeds were generated, and the maximum pitch angles in the 1 hour time-series were averaged to obtain the final maximum pitch angle. In present study, irregular wave of 11.71 m significant wave height and 13.4 s wave period with JONSWAP spectrum was used as input, which gave the maximum result for the cases defined for DLC6.1 as shown in Table 5 below. The pitch incline due to wind turbine thrust was calculated using the equilibrium of the rotational moments about fairlead height of the thrust force and the restoring force. For the thrust force, the maximum thrust force of the DTU 10MW wind turbine installed onshore, 1824.4 kN [8] was used.

One case of the calculated pitch angle is shown in Figure 3 to show the characteristics of the dependence of the pitch angles on the floater diameter. Here, pitch angles originate from the wave excitation and the thrust force, and the sum of the two on the floater diameter were estimated with the wall thickness and the ballast density fixed as 0.45 m and 3800 kg/m³ respectively. It is seen from the figure that the maximum pitch angle due to wave excitation increased with the increase of the floater diameter, while the pitch incline due to thrust force became smaller for larger diameters. These characteristics gave an optimum floater diameter with a minimum summed pitch angle.

![Figure 3](image)

**Figure 3.** Contribution of the wave excitation force and the thrust force to the total pitch angle

The analysis described above was performed for various combinations of design parameters to study their effect on the floater motion. Figure 4 shows the dependency of the maximum pitch angle on the...
wall thickness and the floater diameter for draft 100 m and 120 m when the ballast density was fixed as 3800 kg/m³. It is seen from the figure that for both drafts, with the increase of the wall thickness, the optimum floater diameter increased and the pitch angle at the optimum diameter decreased. For floaters with 100 m draft, the minimum maximum pitch angle was about 13 deg, while for floaters with 120 m draft it was about 10 deg. The draft of the floater was fixed to 120 m for the cases in the following discussions.

Figure 4. Effect of concrete wall thickness on the maximum pitch angle for floater with (a) 100 m draft depth and (b) 120 m draft depth

Figure 5 shows the dependence of the maximum pitch angle on the ballast density and concrete density when the wall thickness is 0.45 m. It is seen from Figure 6 (a) that both the optimum floater diameter and the pitch angle at the optimum diameter became smaller with larger ballast density. Figure 6 (b) shows that the effect of concrete density on the pitch angle was limited. Although the concrete density can vary with the design strength, material composition, and construction process, the effect of the uncertainty on the floater motion was found to be limited. From the results shown in Figure 5 and 6, three combinations of the floater diameter, wall thickness, and ballast density that gave smaller maximum pitch angles were selected as shown in Table 4 for further analysis.

Table 4. Properties of the floater models designed based on frequency-domain analysis

| Model   | Draft | Floater Diameter | Wall Thickness | Ballast density | Ballast Height | Concrete mass | Ballast mass |
|---------|-------|------------------|----------------|----------------|----------------|---------------|--------------|
| Model1  | 120 m | 15 m             | 0.35 m         | 3800 kg/m³     | 23.18 m        | 5,839,165 kg  | 14,140,485 kg|
| Model2  | 120 m | 16 m             | 0.45 m         | 3800 kg/m³     | 22.06 m        | 7,968,709 kg  | 15,005,575 kg|
| Model3  | 120 m | 17 m             | 0.55 m         | 3800 kg/m³     | 21.03 m        | 10,303,236 kg | 15,858,885 kg|

3.3. Design load estimation using time-domain analysis

Time-domain analysis is performed for the three models shown in Table 4 to obtain more accurate floater motion and the sectional forces. An in-house code for the aero-hydro-servo-elastic analysis, NK-UTWind [13] is used, where the equation of motion shown in Eq. 5 is calculated.

\[
\begin{align*}
&M_{6N,6N} \ddot{\mathbf{x}}_{6N,1} + C_{6N,6N} \dot{\mathbf{x}}_{6N,1} + K_{6N,6N} \mathbf{x}_{6N,1} = \{F_{hydro} + F_{lines} + \mathcal{P}_{buoyancy} + \mathcal{P}_{aero}\}_{6N,1}
\end{align*}
\]
where \{M\} is the mass matrix, \{K\} is the stiffness matrix, and \{C\} is the Rayleigh damping matrix. \(F_{\text{lines}}\) is the forces from the mooring lines, \(F_{\text{buoyancy}}\) is the restoring force, and \(F_{\text{aero}}\) is the force from wind turbine. \(F_{\text{hydro}}\) is the hydrodynamic forces that can be modeled with both Morison’s equation and potential flow theory. The Morison’s equation shown in Eq. 6 is used in this study.

\[
F_{\text{hydro}} = \rho \frac{\pi D}{4} \dot{v} + C_m \rho \frac{\pi D^2}{4} (\dot{v} - \dot{x}) + C_D \frac{1}{2} \rho D (\dot{v} - \dot{x}) |\dot{v} - \dot{x}|
\]

where \(C_D\) is the drag coefficient. The mooring force \(F_{\text{lines}}\) is calculated with the quasi-static catenary equation. The aerodynamic and the inertia forces from the wind turbine is calculated with FAST and passed to NK-UTWind as \(F_{\text{aero}}\) at the tower-top node. From NK-UTWind, the displacement and velocity of the tower-top node is passed to FAST at each time-step. Added mass coefficients and drag coefficients were all set at 1.0, and 0.1 % of the Rayleigh damping was introduced for the purpose of numerical stabilization. Additional stiffness of 98000 kN/rad was introduced in the yaw direction to stabilize the yaw motion. The controller of DTU 10MW model provided in [8] was directly used without any optimization. As the controller was designed for onshore environment, this can give severer loadings for the floater.

Two of the design load cases defined in the design standard [14] for ultimate condition were analyzed in this study: DLC 1.6 for the operational condition in severe sea state, and DLC 6.1 for the parked condition in extreme sea state. Values used for each load case is summarized in Table 5. The mooring line was designed as shown in Table 6 and was used for the three floater models.

### Table 5. Summary of cases for time-domain coupled analysis

| DLC Name | Mean wind speed (m/s) | Turbulence intensity (%) | Significant wave height (m) | Significant wave period (s) |
|----------|-----------------------|--------------------------|-----------------------------|-----------------------------|
| DLC1.6   | 11.40                 | 14.90                    | 11.71                       | 11.1                        |
|          | 15.00                 | 13.48                    | 11.71                       | 14.3                        |
| DLC6.1   | 51.72                 | 10.30                    | 11.71                       | 12.7                        |
|          |                       |                          |                             | 14.3                        |

### Table 6. Principal particulars of the mooring system

| Horizontal distance of anchor to Platform Centerline | 800 m | Mooring line weight in water | 224.789 kg/m |
|-----------------------------------------------------|-------|-----------------------------|---------------|
| Initial line length                                  | 850 m | Mooring line axial stiffness | 1,110,000,000 N |

The pitch motions and the maximum stress for the three models obtained from the time-domain analysis are summarized in Table 7. It is seen from the table that the maximum pitch angles were about 10 deg for all three models, which was consistent with the results of the frequency domain analysis. Figure 6 shows the distribution of the maximum sectional forces for the three models. From these results, the axial and shear stress at the section edge were obtained as shown in Figure 7, using which the structural design of the prestressed-concrete was conducted.

### Table 7. Summary of the results of the time-domain analysis

|                | Max. Pitch at CoG (deg) | Max. acc (G) | Max. wall tension (N/mm²) | Min. wall tension (N/mm²) | Max. wall shear (N/mm²) | Min. wall shear (N/mm²) |
|----------------|-------------------------|--------------|--------------------------|---------------------------|-------------------------|--------------------------|
| Model1         | 10.37                   | 0.43         | 20.3                     | -39.7                     | 0.94                    | -1.2                     |
| Model2         | 10.25                   | 0.44         | 14.3                     | -26.3                     | 0.75                    | -0.89                    |
| Model3         | 10.25                   | 0.43         | 10.7                     | -18.7                     | 0.66                    | -0.71                    |
Figure 6. Distribution of the maximum values of (a) shear force, (b) bending moment along the floater for DLC1.6

Figure 7. Distribution of the maximum values of (a) axial stress and (b) shear stress along the floater for DLC1.6

4. Structural design of the prestressed-concrete floater

Structural design of the prestressed-concrete floater was conducted using the results of the time-domain analysis. For the assessment of the ultimate strength, the yielding of the PC steel, the occurrence of the bending cracks, shear cracks, and shell buckling of the concrete were considered. It is noted that fatigue strength is not considered in this initial design stage.

4.1. Structural assessment methods

As mentioned above, concrete structure was designed with partially prestressed concrete that allows flexural tensile stresses but not cracks in the concrete. The required amount of prestressed tension to prevent cracks was calculated with Eq. 7.

$$P_e = \left(\frac{M_d}{Z_t} - \sigma_{tb} - \frac{N_d}{A_c}\right) / \left\{\frac{1}{A_c} + e/Z_t\right\}$$  \hspace{1cm} (7)

where $P_e$ is the required amount of pretension, $M_d$ and $N_d$ are the maximum bending moment and the axial force obtained from time-domain analysis, $Z_t$ is the section modulus, $A_c$ is area of concrete, $e$ is the horizontal distance from PC steel to the center of gravity of the section, and $\sigma_{tb}$ is the concrete flexural tensile strength. $\sigma_{tb}$ is which is estimated using the concrete design strength $f'_c$ as shown in Eq. 8 based on reference [15].

$$\sigma_{tb} = 0.56\sqrt{f'_c}$$ \hspace{1cm} (8)

The occurrence of the bending crack was assessed using the ratio of the axial stress to the allowable compressive stress. Based on reference [15], the assessment was assessed for short-term stress $\sigma_S$ and the long-term stress $\sigma_L$, which are defined as shown in Eqs 9 and 10 respectively.

$$\sigma_S = \frac{(P_{max} + N_d)}{A_c} + \frac{M_d}{Z_t}$$ \hspace{1cm} (9)

$$\sigma_L = \frac{(P_{max} + N_d)}{A_c}$$ \hspace{1cm} (10)
where \( P_{\text{max}} \) is the maximum prestressed tension. The allowable compressive stress of concrete was calculated as Eq. 11 \[11\], where \( f_c(L) \) and \( f_c(S) \) is the long-term and short-term allowable compressive stress respectively.

\[
f_c(L) = 0.45F_c \quad f_c(S) = 0.6F_c
\]

(11)

The occurrence of the shear crack was assessed using the ratio of the stress \( \sigma_1 \) represented in Eq. 12 to the concrete tensile strength.

\[
\sigma_1 = (\sigma_x + \sigma_y)/2 + 1/2 \cdot \sqrt{((\sigma_x - \sigma_y)^2 + 4 \cdot \tau^2)}
\]

(12)

where \( \sigma_x \) is the axial stress, \( \sigma_y \) is the circumferential stress, and \( \tau \) is the shear stress. \( \sigma_y \) is estimated from the hydrostatic pressure. The tensile strength of concrete \( f_t \) is calculated with Eq. 13 \[15\].

\[
f_t = 0.23\pi^{2/3}\]

(13)

Shell buckling of the concrete structure was assessed for water pressure, shear stress, and axial stress separately based on reference \[16\]. First, shell buckling due to water pressure was assessed. The maximum allowable pressure to shell buckling was obtained with Eq. 14.

\[
P_{k0} = ((n^2 - 1)/12) \cdot (E/(1 - \nu^2)) \cdot (C_t/r_o)^3
\]

(14)

where \( P_{k0} \) is the maximum allowable pressure, \( n \) is the number of buckling lobe, \( E \) is the Young’s modulus, \( \nu \) is the Poisson's ratio, \( C_t \) is the wall thickness, and \( r_o \) is the outer radius of the section. Shell buckling due to shear stress was assessed using the maximum torsional shear stress \( \tau_{k2} \) calculated with Eq. 15, where \( r_i \) is the inner radius and \( T_k \) is the maximum torsional moment estimated with Eq. 16, where \( P_w \) is the water pressure.

\[
\tau_{k2} = 2T_kr_o/(\pi(t_o^4 - r_i^4))
\]

(15)

\[
T_k = 0.85T_{k0}(1 - P_w/P_{k0})^{3/4} \cdot T_{k0} = (1/3) \cdot \sqrt{(2/3) \cdot (n^2 - 1) \cdot (E \cdot \pi)/(1 - \nu^2)^{3/4}} \cdot \sqrt{2}\pi r_o^5
\]

(16)

Shell buckling due to axial stress was assessed using the maximum axial stress under torsional shear, \( \sigma'_{k} \), calculated with Eq. 17.

\[
\sigma'_{k} = \sigma_x(1 - (\tau_{k2}/\tau_x)^{n})
\]

(17)

where \( \tau_x \) is the shear stress, \( \sigma_x \) is the axial compression buckling stress obtained with Eq.18.

\[
\sigma_x = E/\sqrt{3(1 - \nu^2)} \cdot (t/r_o) \cdot 0.4
\]

(18)

4.2. Ultimate strength assessment results

Results of the structural assessments for the ultimate strength is discussed in this section. As DLC1.6 was generally the governing case instead of DLC 6.1, only the results for DLC1.6 are shown due to the limitation of the space.

4.2.1. PC steel yielding. The ratio of the prestress tension \( P_e \) to the steel yielding strength \( P_y \) is shown in Figure 8. It is seen that for Model 1 with wall thickness of 35 cm, the PC steel yielded at the heights around -20 m. For Model 2 and Model 3 with thicker wall thickness, tension in PC steel was under the yielding strength for all heights.

4.2.2. Concrete bending cracks. Figure 9 shows the ratios of the axial stress of concrete to the allowable long-term and short-term compressive stress. It is seen from the figure that for Model 1 the short-term axial stress exceeded the allowable stress for heights above around -80 m mainly due to the bending moment. Long-term axial stress exceeded the allowable stress at the bottom part which could be attributed to the increase in the axial stress due to ballast materials. For Model 2 and Model 3, both the short-term and long-term axial stress were under the allowable stress for all heights.

4.2.3. Concrete shear cracks. The ratio of \( \sigma_1 \) in Eq. 12 to the concrete tensile strength is shown in Figure 10. It is seen that \( \sigma_1 \) in Eq.12 were below the concrete tensile strength for all heights in all three models.
4.2.4. Shell buckling. The results for the shell buckling assessments are shown in Figure 11. Figure 11 (a) shows that Model 1 subjected to shell buckling due to water pressure in the lower part of the floater while other two models withstood the pressure. Figure 11 (b) and (c) show the assessment results for shell buckling due to axial stress and shear stress respectively. Figure shows that due to the water pressure, Model 1 also subjected to shell buckling, while Model 2 and 3 were intact for these failure modes.
Figure 11. Assessment of ultimate shell-buckling strength

Figure 12. Sectional view of the concrete structure for spar-type floater for 10 MW wind turbines

Table 8. Concrete mix proportion for the reference model

| Water-cement ratio | Maximum size of coarse aggregate | Air volume | Fine aggregate ratio | Coarse aggregate | Admixture |
|--------------------|----------------------------------|------------|----------------------|-----------------|-----------|
| 35 %               | 5 mm                             | 2.5 %      | 44.5 %               | Crushed stone, 955 kg/m³ | Air entraining and high-range water reducing agent, 5 kg/m³ |
| Water              | 175 kg/m³                        |            |                      | Crushed sand, 765 kg/m³ |           |
| Cement             | ordinary Portland cement (JIS R 5210), 500 kg/m³ |            |                      |                 |           |

4.3. Reference model for the prestressed-concrete floater

From the results of previous chapters, Model 2 was chosen for the reference model and the structural design for the prestressed concrete was conducted based on its geometry.
is shown in Figure 12. PC steel was installed with 96 sheaths of 88 mm diameter. In each sheath, 22 PC steel strands were installed and the gaps were filled with grout of the same design strength with the concrete. Width fixing reinforcements are usually required in the concrete casting process and usually do not contribute to the overall structural strength. However, considering that the reinforcements can contribute to some nonlinear behavior of the structure, they were introduced in the reference model. The mix proportion of the concrete was also designed as shown in Table 7. The design was based on “C-BOAT500™”, the concrete barge that has been in use for more than 40 years.

5. Conclusion
In this study, a reference model for prestressed-concrete spar-type floater for 10 MW wind turbines was designed together with its concrete structure. Frequency-domain analysis showed that an optimum floater diameter that gives minimum pitch angle could be found. Although the maximum pitch angles were the similar for floaters with diameters of 15 m to 17 m, the case with the wall thickness of 0.35 m will subject to PC steel yielding, concrete bending cracks, and shell buckling due to water pressure. Finally the reference model was designed for the floater diameter of 16 m and wall thickness of 0.45 m.

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