Feasibility of a very large floating structure as an offshore wind foundation: effects of hinge numbers on wave loads and induced responses

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Abstract

Floating offshore wind is a rapidly-growing technology attracting global interest. To date, most of the demonstrated concepts for offshore floating wind are based on a simple "one turbine – one platform" system, which may not be the most efficient approach for manufacturing, transportation and onsite installation. Very large floating structures (VLFSs), which allow for operation of multiple-turbines, may be an effective alternative to traditional floating foundations. However, the large bending moment caused by waves has been a major concern for a VLFS foundation. Adding hinges into the structure may help alleviate the bending moment. Based on the discrete-module-beam-bending based hydroelasticity method, the

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effects of hinge numbers on the bending moment are investigated in detail and presented in this paper. Overall, the bending moment is reduced while the vertical displacement is increased by adding hinges, which indicates a compromise in choosing hinge numbers. In addition, a feasibility study for applying the multi-hinged VLFS as a floating wind platform is provided. It demonstrates the existence of wind turbines may further reduce the wave induced bending moment but enlarges the total bending moment by introducing the still water bending moment. The effect of wind turbines on the vertical displacement of the multi-hinged VLFS is insignificant.

Keywords

Floating offshore wind platform; Very Large Floating Structure (VLFS); Hydroelasticity; Hinge

Notation

The following symbols are used in this paper:

λ	*Wavelength*
ξ	*The (complex) displacement vector of each rigid submodule*
$\Psi(\omega)$	*Added mass*
ω	*Wave frequency*
Ξ_J	*Constraint matrix*
A	*Incoming wave amplitude*
$B(\omega)$	*Radiation damping*
C	*Hydrostatic restoring coefficient matrix*
F_A	*Added mass force*
F_E	*Wave excitation force*
F_J	*Force vector for the hinge connection*
F_{Hs}	*Hydrostatic force*

F_{Rd}	*Radiation damping force*
F_{St}	*Structural deformation induced force*
F_z	*Shear force at the hinge*
K_e	*Beam element stiffness matrix*
K_{St}	*Stiffness Matrix of the entire structure*
L	*Length*
M	*Mass matrix*
N	*Submodules number*
Δz	*Vertical displacement*

1. Introduction

The wind resources in deep water region and further offshore area have attracted huge interest along with the continuing development in offshore wind energy. Deep waters tend to have a greater wind resource which could lead to the operation of large turbines more efficiently. Larger wind turbines offshore require different support structures including floating foundations. Currently, a number of different concepts are under development, the major of which are spar type, semi-submersible type, tension-leg-platform type and barge type floating foundation. To date, most of the demonstrated concepts for offshore floating wind development are still based on a simple “one turbine - one platform” system, where a single floating foundation only supports one turbine. However, this system may not be the most efficient approach to manufacturing the floating platform, and is likely to also increase the cost of transportations as well as onsite installation. Therefore, a question has been raised: “Could two or more turbines sit on one platform?” By doing so, the usage of one platform serving multi-turbines may significantly reduce the cost of manufacture, transportation and installation for a wind farm. Floating Power Plant AS (2013) developed a pilot multi-turbines platform (i.e. P37) with 3 wind turbines installed (as shown in **Fig. 1**) which has been tested over a period of several years and has produced joint power to the grid in a real offshore environment (Floating Power Plant AS, 2013). As demonstrated in the P37 platform, a large deck space created between each turbine on the platform leads to potential development for multi-purposes use. Additionally, the large deck area offers more space for operation and maintenance activities and thus indicates a potential reduction on the operating expenditure of offshore wind development.

Following this trend, the very large floating structure (VLFS) can be potentially considered as a promising alternative for floating offshore wind foundations considering their potential to maximize the power generating capacity to drastically reduce some dangerous and costly offshore operations and to bear a low maintenance cost. Despite that the VLFS (especially for the mat-type VLFS) bears some similar features as the barge-type floating foundation for offshore wind such as large surface areas and thus large wave loads, the former has some unique advantages over other types of floating foundations (mainly because of its much larger surface area than the barge type and other types of floating foundations). First, the space between two or more turbines on the upper surface of the VLFS may be considered for multi-purpose use including wave energy utilization and aquaculture. Second, offshore wind farms deployed far offshore may induce difficulties associated with the connection of wind turbine to the grid. The VLFS offer enough space for energy storage facilities, helping solve the grid connection problem. Third, due to the large available space of the VLFS, the operation and maintenance of the wind turbines is much simplified as done on land. This kind of VLFS could offer a large deck space for utilising wind turbines and other energy conversion units. In addition, the installation of energy conversion units, as well as the operation and maintenance activities, could be substantially simplified because the large deck area offers workspace similar to an onshore project. However, the floating offshore wind foundation based on a VLFS presents new challenges. A very large floating structure is a unique concept of ocean structure primarily because of its unprecedented length, displacement and associated hydroelastic response, analysis and design (Suzuki*, et al.*, 2006). It also has unprecedented challenges associated with a long design life compared with other oceanic structures. To date, the concept of deploying offshore wind farm on a VLFS has not been demonstrated. However, some applications of VLFSs have indicated the possibility and feasibility in playing a role in offshore wind. In 1995, the Japanese Mega-Float programme was established to create a very large floating structure for an airport development. Two experimental demonstration cases were built through the programme. In the experimental Phase 1, a 300 m × 60 m × 2 m (length × width × depth) structure was built. Following Phase 1, Phase 2 was started in 1998. A 1000 m × 120 m × 3 m (length × width × depth) airport was built to test the feasibility of floating airport with landing and take-off of small airplanes. It is noted that this floating airport completed in 1998 is the only Mega-Float that has ever been built. However, it confirms the feasibility of VLFS and the Technological Research Association of Mega-Float concluded in 2001 that a 4000 m length floating airport is feasible.

As an application of Mega-Float technology, offshore floating wind farms were subsequently investigated (Inoue, 2005, Suzuki, 2005, Yago, 2003). A sailing wind farm was proposed by Manabe*, et al.* (2008) as shown in **Fig. 2**.

The VLFS could potentially integrate the construction, installation and maintenance activities on the floating platform itself, it offers a future solution on the development of deepwater offshore wind farm. However, due to the large waterplane area and shallow draft of a VLFS, its behaviour under wave action is dominated by elastic deformations. This fluid-structure interaction is known as hydroelasticity.

When head waves passing a barge type VLFS, a strong hydrodynamic bending moment is observed. This bending moment could cause structural failure once it reaches the limiting strength of the material. For a VLFS, one possible solution to alleviate the maximum bending moment on the structure is to introduce interconnected hinges onto the structure. Thus, a single-module VLFS is changed to a hinged multi-module VLFS. At the hinge joint, no internal bending moment is translated. This could potentially significantly reduce the maximum bending moment acting on the whole VLFS. To date, a VLFS connected by a single hinge is well documented through numerical model development. However, for a multi-hinge VLFS, there is still lack of understanding of the hydroelastic response of the structure. One of the contributions of the present numerical model is the capability of predicting the hydroelastic response of a VLFS with the multi-hinge connection.

For a hinged VLFS, the hydrodynamic response of the structure was investigated by Newman (1994). However, the elastic deformation of the structure was neglected. Later on, Kim*, et al.* (1999) studied a five-module VLFS in the linear frequency domain by taking into account of the hydroelastic response. Fu*, et al.* (2007) demonstrated a numerical method to predict the hydroelastic response of a flexible, floating, interconnected structure using three-dimensional hydroelasticity theory (Wu, 1984) by taking into account the interconnected hinges. Based on the multi-rigid-body dynamics and beam bending method developed by Lu*, et al.* (2019), Sun*, et al.* (2018) discussed the coupled effects of wave dynamics and structural deformation on a hinged two-modules structure. The vertical displacement, force and bending moment of the hinged VLFS are presented in detail. Zhang*, et al.* (2018) developed a time-domain discrete-module-beam-bending based hydroelasticity method for estimating the transient response of VLFSs under unsteady external loads. The interconnected hinge effect on the overall hydroelastic response was also discussed in their study. To date, a VLFS connected by a single hinge is well documented through numerical model development. However, for a multi-hinged VLFS, there is still lack of understanding of the hydroelastic response of the structure. Wu*, et al.* (1993) analysed the hydroelastic response of a 5-module VLFS with Flexible Module and Flexible Connector. Based on linear hydroelasticity, Riggs*, et al.* (2000) investigated an interconnected VLFS under Rigid Module and Flexible Connector as well as Flexible Module and Flexible Connector models. In their work, five modules connected with hinges at the deck were numerically simulated to obtain the wave-induced response. Stansby*, et al.* (2015) developed devices comprising rigid floats connected by flexible structural elements. However, the elasticity of the floats is not considered.

Based on the above literature, adding the hinge connector into the VLFS alters the hydroelastic responses. However, to the authors' best knowledge, little information has been found on the hydroelastic effects of hinge numbers regarding the design optimisation of VLFS for the purpose of floating offshore wind foundation. The present study is aimed to discuss the effects of the interconnected hinges numbers on the hydroelastic response of the VLFS, including vertical displacement, force and bending moment. A VLFS with different numbers of hinges (0, 1, 3 and 7 hinges) is numerically simulated under different regular wave frequencies. Finally, by assuming the wind turbines as static external loads, a preliminary feasibility study is performed for a multi-hingled VLFS with two wind turbines installed.

2. Methodology

2.1. Description of the VLFS structures

A schematic of the mat-type VLFS is shown in **Fig. 3**. Several types of VLFS are considered including a continuous VLFS, a VLFS with a single hinge, a VLFS with three hinges and a VLFS with seven hinges, to investigate the effects of hinge numbers on dynamic response of the structure. It is noted that the rotational stiffness of the hinge is zero and more details are presented in Section 2.2. The main dimensions and physical properties of the benchmark VLFS are given in **Table 1**. It is noted that all VLFS structures shown in **Fig. 3** are derived from the benchmark VLFS given in **Table 1**, which means that the hinged VLFS is obtained by dividing the continuous VLFS at the locations of hinges.

In the present study, only a preliminary investigation is made on the feasibility of a VLFS concept as a floating foundation for offshore wind turbine, whose focus is put on multi-hinge effects (to reduce the bending moment of the large structure). At this stage, there is no appropriate design of mooring system for the VLFS and the mooring system is ignored. The second order wave loads mainly affect the low frequency horizontal motion of the VLFS with mooring system. Therefore, for the present study where the mooring is ignored, only the first order wave frequency loads and motion is considered.

2.2. Discrete-module-beam-bending based hydroelasticity method

The discrete-module-beam-bending based hydroelasticity method (Lu, *et al.*, 2019, Sun, *et al.*, 2018, Zhang and Lu, 2018, Zhang, *et al.*, 2018) is adopted to solve the hydroelastic response of a VLFS (which may be continuous, single-hinged or multi-hinged) under regular waves. The hydroelasticity method is derived in the framework of linear potential flow theory, assuming that (i) the fluid is inviscid and incompressible; (ii) the fluid motion is irrotational; and (iii) the motion of structure is small. Here, a single-hinged VLFS is taken as an example to briefly introduce the procedure of this method, as illustrated in **Fig. 4**.

First, each module of the single-hinged VLFS is uniformly divided into several rigid submodules, for example, N_1 submodules for module 1 of length L_1 and N_2 for module 2 of length L_2. Thus, the length of a submodule for module 1 and module 2 is L_1/N_1 and L_2/N_2, respectively. The (complex) displacement vector of each rigid submodule is denoted as $\xi^{(m)} = \left[\xi_1^{(m)}\ \xi_2^{(m)}\ \xi_3^{(m)}\ \xi_4^{(m)}\ \xi_5^{(m)}\ \xi_6^{(m)}\right]^{\mathrm{T}} (m = 1, 2, \dots, N_1 + N_2)$ with $\xi_j^{(m)}\ (j = 1, 2, 3)$ being the translational displacement along x, y and z axis, respectively and $\xi_j^{(m)}\ (j = 4, 5, 6)$ the rotational displacement around the x, y and z axis, respectively. The total displacement for the whole structure is $\boldsymbol{\xi} = \left[\left(\boldsymbol{\xi}^{(1)}\right)^{\mathrm{T}}\ \left(\boldsymbol{\xi}^{(2)}\right)^{\mathrm{T}} \dots \left(\boldsymbol{\xi}^{(N_1+N_2)}\right)^{\mathrm{T}}\right]^{\mathrm{T}}$ with the dimension of $6(N_1 + N_2) \times 1$. For a complex displacement vector $\xi^{(m)}$, its absolute value represents the magnitude of the displacement while its phase indicates the phase difference between incident waves and the displacement of the structure.

Multi-rigid-body hydrodynamics theory is used to obtain the wave excitation force $\boldsymbol{F_E}$, the added mass $\boldsymbol{\Psi}(\omega)$ (or the added mass force $\boldsymbol{F_A} = \omega^2\boldsymbol{\Psi}(\omega)\xi$) and the radiation damping $\boldsymbol{B}(\omega)$ (or the radiation damping force $\boldsymbol{F_{Rd}} = \mathrm{i}\omega\boldsymbol{B}(\omega)\xi$). It is noted that i is the imaginary unit which satisfies $\mathrm{i}^2 = -1$. The hydrostatic force is $\boldsymbol{F_{Hs}} = -\boldsymbol{C}\xi$ with C being the hydrostatic restoring coefficient matrix (and it can be obtained through hydrostatic analysis). The inertia force is $\boldsymbol{F_{In}} = \omega^2\boldsymbol{M}\boldsymbol{\xi}$ with $\boldsymbol{M}$ being the mass matrix. The dimensions are all $6(N_1 + N_2) \times 6(N_1 + N_2)$ for $\boldsymbol{\Psi}(\omega)$, $\boldsymbol{B}(\omega)$, C and $\boldsymbol{M}$ while $6(\boldsymbol{N_1} + \boldsymbol{N_2}) \times 1$ for $\boldsymbol{F_E}$, $\boldsymbol{F_A}$, $\boldsymbol{F_{Rd}}$, $\boldsymbol{F_{Hs}}$

and $\boldsymbol{F_{In}}$. Details of the above-mentioned matrices and their derivation can be referred to Zhang, *et al.* (2018).

By assuming that all external forces and the physical properties such as mass and moment of inertia of each submodule are concentrated on its centre of gravity, each rigid submodule is simplified as a generalized lumped mass. A beam element which follows the geometrical and physical properties of the original structure is used to connect two adjacent lumped masses to consider the effect of structural deformation. The structural deformation induced force on all lumped masses is denoted as $\boldsymbol{F_{St}} = -\boldsymbol{K_{St}}\boldsymbol{\xi}$, where $\boldsymbol{K_{St}}$ (whose dimension is $6(N_1+N_2)\times 6(N_1+N_2)$) is the stiffness matrix of the entire structure and it is given by overlaying each beam element stiffness matrix $\boldsymbol{K_e}$ (whose dimension is 12×12) according to the standard process of finite element method. The expressions for the above-mentioned matrices are given in **Appendix A**.

Special attention is paid to the connection between two modules of the VLFS. The connection only allows the relative rotation around the axis passing through the hinge centre and parallel to y axis. The displacements for the N_1^{th} and $(N_1+1)^{\text{th}}$ lumped masses are constrained by the hinge connection as follows,

$$\Xi_{\mathrm{J}}\xi=\left[\begin{array}{cccccccccccccc} & 1&0&0&0 & 0 & 0 & -1&0&0&0 & 0 & 0 & \\ & 0&1&0&0 & 0 & \left(\frac{L_1}{2N_1}+\frac{L_{\mathrm{G}}}{2}\right) & 0&-1&0&0 & 0 & \left(\frac{L_2}{2N_2}+\frac{L_{\mathrm{G}}}{2}\right) & \\ \mathbf{0}_{5\times(6N_1-6)} & 0&0&1&0 & -\left(\frac{L_1}{2N_1}+\frac{L_{\mathrm{G}}}{2}\right) & 0 & 0&0&-1&0 & -\left(\frac{L_2}{2N_2}+\frac{L_{\mathrm{G}}}{2}\right) & 0 & \mathbf{0}_{5\times(6N_2-6)} \\ & 0&0&0&1 & 0 & 0 & 0&0&0&-1 & 0 & 0 & \\ & 0&0&0&0 & 0 & 1 & 0&0&0&0 & 0 & -1 & \end{array}\right]\xi=\mathbf{0} \tag{1}$$

where Ξ_{J} is the constraint matrix with the dimension of $5\times 6(N_1+N_2)$ due to the existence of the joint (hinge connection).

The force (and moment) vector for the hinge connection is denoted as $F_J = \left[F_{J1}, F_{J2}, F_{J3}, F_{J4}, F_{J6}\right]^T$. F_{J1}, F_{J2} and F_{J3} represent the force in the direction of x, y and z axis, respectively while F_{J4} and F_{J6} are the moment around x and z axis, respectively. Due to the free rotation around the y axis, the component $\mathrm{F}_{\mathrm{J}5}$ (the moment around y axis) is zero and it is not included in the force vector. For the hydroelastic analysis using the discrete-module-beam-bending based approach, the equations of motion are established on all lumped masses (or the centres of gravity of all submodules). Therefore, the force vector for the hinge is transformed into the equivalent forces on the two adjacent lumped masses (i.e. the N_1^{th} and $(N_1+1)^{\mathrm{th}}$ lumped masses) through the formula $\Xi_J^T F_J$.

By considering the equilibrium of the forces exerted on all lumped masses and the displacement continuity conditions due to the existence of the hinge (see Eq. 1), the equations of motion for the interconnected two-module VLFS can be obtained,

$$\begin{bmatrix} -\omega^2\left(\mathbf{M}+\mathbf{A}(\omega)\right)-\mathrm{i}\omega\mathbf{B}(\omega)+\mathbf{C}+\mathbf{K}_{\mathrm{St}} & \Xi_{\mathrm{J}}^{\mathrm{T}} \\ \Xi_{\mathrm{J}} & \mathbf{0} \end{bmatrix} \begin{bmatrix} \xi \\ \mathbf{F}_{\mathrm{J}} \end{bmatrix} = \begin{bmatrix} \mathbf{F}_{\mathrm{E}} \\ \mathbf{0} \end{bmatrix} \tag{2}$$

It is noted that by removing the items related to Ξ, $\Xi_{\mathrm{J}}^{\mathrm{T}}$ and $\mathbf{F}_{\mathrm{J}}$, Eq. (2) represents the equation of motion for a continuous VLFS structure. By modifying the matrix components of Ξ, $\Xi_{\mathrm{J}}^{\mathrm{T}}$ and $\mathbf{F}_{\mathrm{J}}$ to cover the forces and displacement constraints due to the existence of hinges at other locations (for example, multi-hinged VLFS), Eq. (3) can be used to describe the equation of motion for a multi-hinged VLFS.

The procedure for calculating the displacement and force at any position of the VLFS after solving Eq. (2) is illustrated in **Appendix B.**

2.3. Verification of the numerical method.

The discrete-module-beam-bending based hydroelasticity method is verified by the three-dimensional hydroelaticity method based on modal expansion approach (Fu*, et al.*, 2007). The 300 m long Mega-Float prototype structure (Yago and Endo, 1996) has been chosen as the reference VLFS for the verification study. Details of the principal dimensions of the reference structure are illustrated in **Table 2**. The structure is a one-hinge interconnected structure. It is noted that the present numerical model is verified under waves with no unsteady external dynamic loads.

For a one-hinge interconnected VLFS, the present numerical results on the vertical displacement Δz (normalized by the incident wave amplitude A) along with the VLFS are compared with numerical predictions performed by Fu*, et al.* (2007).

Two different incident wavelengths (150 m and 300 m) are calculated for the verification. As shown in **Fig. 5**, the present numerical results show a good agreement compared with the predictions provided by Fu*, et al.* (2007). Therefore, the present numerical model can be used with some confidence in future VLFS hydroelasticity simulations.

3. Results and Discussion

The hydroelatstic responses of the benchmark VLFS (whose properties are given in **Table 1**) under different regular waves are investigated using the present numerical model. The current study covers wavelength from 75 m to 600 m and wave directions of 0, 45 and 90 degrees. As shown in **Table 3**, 11 cases are discussed to reveal the effect of hinge numbers on the moment and displacement distribution along the VLFS, such as the bending moment, shear vertical force, vertical and rotational displacements as well as the force and displacement at the hinge connections. It is noted that the wave amplitude is chosen as 1 m, which means that the results presented in this paper should be regarded as the quantities per unit of wave amplitude.

3.1. Bending moment of the VLFS.

Fig. 6, presents the bending moment (the moment along Y axis) along the VLFS with a different number of hinges under regular waves of wavelength from 75 m to 600 m and a wave direction of 0 degree. When the wavelength (λ) is close to the length of the VLFS (for example, λ = 600 m), the continuous body without hinge connectors has the largest bending moment at the middle point of the VLFS. By adding on the hinge, the bending moment drastically drops, especially at the middle part of the structure (where the bending moment is zero due to the existence of the hinge connection). However, for a shorter wavelength, adding one hinge into the structures does not reduce the maximum bending moment along the structure. Instead, the existence of one-hinge slightly increases the maximum bending moment along the structure (occurring at a given position of the upstream module along wave incidence direction) when the wavelength is less than the overall length of the structure. When $\lambda = 199\ m$, the maximum bending moment of one hinge VLFS is very close to the continuous VLFS. However, the maximum bending moment for a one hinge VLFS at the second part (from 300 to 600 m of VLFS) is still larger than the value for a continuous VLFS. By introducing more hinges (i.e. 3 and 7 hinges), the maximum bending moment along the VLFS starts to drastically decrease for all the wavelength considered here. As seen in **Fig. 6**, the bending moment along a 7 hinges VLFS is extremely small compared to the continuous and one hinge body. Overall, adding one hinge into the VLFS can not effectively reduce the maximum bending moment along the VLFS whereas a further increase of hinge numbers could significantly reduce the bending moment along the VLFS.

The effect of hinge numbers on the bending moment of the VLFS may be further altered by incident wave directions. As indicated in **Table 3**, two other wave directions, i.e. 45 and 90 degrees, are considered. A VLFS with 7 hinges is investigated due to the relatively small bending moment. The results are shown in **Fig. 7**. It can be seen that the wave direction of 0 degree corresponds to the largest bending moment of the VLFS, followed by the oblique waves of 45 degree, and the beam-sea waves (i.e. wave direction of 90 degree) leads to the smallest bending moment. Therefore, from the point of view of structural integrity of the VLFS, the most critical condition is wave direction of 0 degree (i.e. wave propagating along the X axis).

3.2. Displacement of the VLFS.

Fig. 8 shows the vertical displacement Δz (normalized by the incident wave amplitude A) along the VLFS with a different number of hinges under regular waves of wavelength from 75 m to 600 m and a wave direction of 0 degree. Overall, the introduction of the hinge connection leads to an increase of vertical displacement of the VLFS (compared with a continuous VLFS without hinge connection) but the effects of hinge number vary according to the wavelength. For wavelength comparable to the length of the VLFS (for example, λ = 600 m), the maximum vetical displacement (ignoring the location near free ends) is quite similar for hinged VLFSs of different hinge numbers (i.e. 1, 3 and 7 hinges), which is almost twice of that for a continuous VLFS. Along the longitudinal direction of the VLFS, the range of variation of the vertical displacement becomes smaller for a larger hinge number. For wavelength shorter than the overall length of the VLFS, the maximum vertical displacement (also ignoring that near free ends) is quite varied for different number of hinges, showing an increasing trend with the increase of hinges. For example, for wavelength equal to 199 m and 150 m, the fluctuation of the vertical displacement for a seven-hinged VLFS is significant while this trend is relatively insignificant for other hinge numbers. For a quite short wave (i.e. λ = 75m), except at the fore section, the vertical displacement is small for a VLFS of different configurations (i.e. with different hinge numbers). This may be due to the fact that the energy of short waves is rapidly dissipated along the longitudinal direction of the VLFS.

The effect of hinge numbers on the displacement of the VLFS is further altered by incident wave directions. It is noted that under oblique (45 degree) and beam-sea (90 degree) waves, the roll motion (or the rotational angle around X axis) is also deserved to be investigated.

As shown in **Fig. 9**, overall, the increase of wave incidence angle from 0 to 90 degree leads to a decrease in the maximum vertical displacement of the VLFS. For beam-sea (90 degree) waves, the vertical displacement remains nearly unchanged along the longitudinal direction of the VLFS.

Fig. 10 shows the rotation angle of the seven-hinge VLFS around the x and y axis for different wave frequencies and wave incidence angles. As indicated by **Fig. 10** (a), for wave direction of 45 degree, the rotation angle around x axis is less than 0.25 degree per unit of wave amplitude and the maximum value occurs for wave frequency of 0.46 rad/s (i.e. wavelength = 297 m). For beam-sea waves (wave direction of 90 degree), with the wave frequency varying from 0.32 to 0.64 rad/s, the rotation angle around x axis increases from 0.6 to 2.18 degree per unit of wave amplitude. **Fig. 10** (b) shows that for wave frequencies considered, the rotation around y axis is less than 3.5 degree per unit wave amplitude. The rotation angle for wave direction of 0 degree is larger than that for 45 degree. For small wave frequencies (or large wavelength), the amplitude of rotation angle varies little with respect to the longitudinal position of the VLFS. However, for relatively large wave frequency (i.e. ω = 0.64 rad/s), there is an observable difference of the rotation angle along the longitudinal direction of the VLFS.

3.3. Forces and displacements at the hinge connection

One of the significant advantages of the present numerical approach is that the shear force at the hinge point could be captured. For an interconnected VLFS, the hinge connection is a key component which is of great significance for the structural integrity. In this subsection, the vertical shear force F_z, torsional moment M_x and the vertical displacement Δz of the hinges is investigated. **Fig. 11** shows the vertical shear force and torsional moment at the hinge connection for VLFSs with a different number of hinges and under different wave frequencies and wave incidence directions.

As shown in **Fig. 11** (a), for a single hinged VLFS, the wave frequency (or wavelength) has a significant effect on the vertical shear force of the hinge. More specifically, the maximum shear force at the hinge is observed at ω = 0.4 rad/s ($\lambda = 400\ m$), with a value of 1.539×10^7 N. For relatively short waves (for example, ω ≥ 0.55 rad/s, or $\lambda \leq 200\ m$), the shear force becomes much smaller, i.e. around 1/5 of the maximum value.

By introducing two more hinges into the structure, the shear force F_z observed at each hinge point is shown in **Fig. 11** (b). A maximum value of 1×10^7 N is founded, which is smaller than that for a single hinged VLFS. **Fig. 11** (b) also indicates that for three-hinged

VLFS, the maximum shear force of the hinge is related to its location with respect to the fore part of the VLFS, i.e. being larger for a closer distance to the fore part (which is the upstream direction of the incident wave). The wave frequency (or wavelength) corresponding to the maximum vertical shear force of the hinge is larger (or shorter) for the hinge being closer to the fore part of the VLFS. For example, Hinge 1 experienced a large shear force at a higher wave frequency (i.e. a shorter wavelength) compared with other 2 hinges. This may be due to the fact that the wave energy transferred onto the structure decreases by increasing the distance from the fore part of the VLFS.

When the VLFS has 7 hinges in the structure, the shear force trend on the hinges is different from that for both 1 or 3 hinges cases, as shown in **Fig. 11** (c). At low wave frequencies (i.e. when the wavelength is relatively long), the shear force on each hinge is quite similar. The shear force on the hinge increases by increasing the wave frequency. However, when the wave frequency is shifted to a higher value, for the first three hinges (hinge 1, 2 and 3), two peaks are observed along with the increasing of wave frequency. The first peak is found around $\omega = 0.6\ \mathrm{rad/s}$, and the second peak is observed around $\omega = 0.7\ \mathrm{rad/s}$. Between the two peaks, a trough can be seen in the figure around $\omega = 0.64\ \mathrm{rad/s}$. For the rest four hinges (hinge 4, 5, 6 and 7) similar distributions as one or three hinged VLFS cases are observed.

For oblique waves (i.e. wave direction of 45 degree), the maximum vertical shear forces for all 7 hinges occur at relatively large wave frequencies, i.e. $\omega > 0.8\ \mathrm{rad/s}$ (or wavelength $<$ 96 m). Unlike the head wave (wave direction of 0 degree) case where the maximum vertical shear force occurs for hinge 1 (which is the closest to the fore part of the VLFS), for oblique waves, Hinge 4 (the hinge at the middle of the VLFS) bears the maximum vertical shear force whose value is quite similar to that for the head wave condition (around 4.2×10^6 N per unit wave amplitude).

Fig. 11 (e) - (f) presents the torsional moment, i.e. M_x, at the hinge connection. For oblique waves, hinge 4 is subjected to the largest torsional moment of $2.4 \times 10^8\ \mathrm{N \cdot m}$ per unit wave amplitude at wave frequency of $0.38\ \mathrm{rad/s}$. For the other three pairs of hinges, i.e. hinge 1 and 7, hinge 2 and 6, hinge 3 and 5, each pair shows a similar trend of the torsional moment with respect to the wave frequency. The maximum torsional moment for the pair- hinge 1 and 7 is the smallest among all three pairs, being less than $1.5 \times 10^8\ \mathrm{N \cdot m}$. For beam sea waves (i.e. wave direction of 90 degree), the torsional moment is smaller than that for oblique waves. In contrast, for three pairs of hinges, i.e. hinge 1 and 7, hinge 2 and 6, hinge 3 and 5, the pair of hinge 1 and 7 bears the largest torsional moment, which shows a monotonically increasing trend with the increase of wave frequency from 0.2 to 1.0 rad/s. The torsional moment for hinge 4 (the hinge at the middle of the VLFS) is insignificant compared with other hinges.

Fig. 12 presents the distribution of the vertical shear force along the VLFS as well as the vertical shear force at the hinge location. Overall, the local extreme of the vertical shear force occurs at the hinge location. For the wave frequencies considered, the shear forces of the hinges in the middle position (i.e. $150\ \mathrm{m} < x < 450\ \mathrm{m}$) of the VLFS varies a little with respect to the longitudinal position of the hinge. An increasing trend of the shear force at the hinge location is observed with the increase of the wave frequency from 0.32 rad/s to 0.64 rad/s.

Apart from the shear force on the hinge, the vertical amplitude at the hinge point is also provided in the present study, as shown in **Fig. 13**. It is noted that the wave frequency of the maximum vertical amplitude is observed at a slightly lower wave frequency compared to the shear force. The displacement at the hinge shows a similar trend as the shear force on the hinge for 1 and 3 hinges case. As seen in **Fig. 13** (b), the hinge experienced large shear force also has the largest displacement among all three hinges. However, for a seven hinges VLFS (see **Fig. 13** (c)), the displacement on each hinge alters to a different pattern compared with the shear force. Only one significant peak can be found in **Fig. 13** (c) at a relatively high wave frequency range, which is closed to the second peak range observed in **Fig. 11** (c). When the hinge is more close to the bow of the structure (incidence wave direction), it will experience a large shear force and displacement compared with the hinges far away from the bow.

3.4. Applying wind turbines on the VLFS

Based on the regular wave simulations, a feasibility study for deploying wind turbines on a seven hinges VLFS is provided. Two 5 MW wind turbines (see **Table 4**) were built on the seven hinges VLFS with a sketch shown in **Fig. 14**. There is a yaw system of the wind turbine which is responsible for the orientation of the wind turbine rotor towards the wind. If the wind comes from the beam sea direction (i.e. 90 degree), the two wind turbines bears the side-by-side layout and thus avoids the wake effect. However, if the wind comes from the head sea direction (i.e. 0 degree), the two wind turbines are in the front-rear arrangement which means the wake effect may be an issue. Considering that the distance of two wind turbines is around 4 times of the rotor diameter, the wake effect may be less significant. As demonstrated by van der Laan*, et al.* (2019), the rotor space can be as small as just over 1 time of the rotor diameter.

As a preliminary study, the two wind turbines are considered as steady external loads on the VLFS.

First, the hydrostatic stability of the VLFS is briefly discussed for the beam sea wind direction. According to Jonkman*, et al.* (2009), the thrust of the 5 MW wind turbine under the wind speed of 11.38 m/s is around $F_T = 827$ kN. The hub height above the deck of the VLFS is $H = 90$ m. The hydrostatic restoring coefficient of the whole VLFS structure for the roll motion (i.e. rotation around the x axis) is $C_{44} = 1.08 \times 10^{11}$ N · m. Then the inclination angle of the whole structure induced by the thrust is $\alpha = 2F_T H / C_{44} = 0.0014\, rad = 0.08°$. This is a rather small inclination angle, which means that the hydrostatic stability of the VLFS is ensured.

For the seven-hinge VLFS without wind turbines, the still water bending moment is zero as the mass is assumed to be uniformly distributed along the VLFS. However, by deploying two wind turbines (which are assumed to be two point masses) on the deck of the VLFS, the still water bending moment becomes non-zero due to the non-uniform distribution of the mass along the VLFS. The procedure for calculating the still water bending moment caused by two wind turbines (i.e. two point masses) can be referred to Section 4.1.2 in Zhang*, et al.* (2018). The result is shown in **Fig. 15**. It can be seen that the maximum still water bending moment is around 7.3×10^7 N · m, which occurs near the location of wind turbine. As the hinge connection is characterized by free rotation around the y axis, the bending moments on the first and eighth submodules caused by the wind turbines are not transferred to other submodules, leading to almost zero bending moment in the middle section of the VLFS.

Fig. 16 presents the distribution of the bending moment along the longitudinal direction of the seven-hinge VLFS with or without two wind turbines on the deck. Overall, by deploying two wind turbines, the maximum bending moment at the hinged modules around the wind turbine is further decreased. For example, for $\omega = 0.32$ rad/s, the bending moment of the VLFS with wind turbines at around $X = 550$ m is about 59% of that without wind turbines. For relatively long waves (i.e. $\omega = 0.32$ rad/s), there is a slight increase of the bending moment on the central hinged modules by deploying wind turbines on the VLFS. For $\omega = 0.46$ rad/s and 0.64 rad/s, the bending moment on the central hinged modules bears insignificant difference for the VLFS with or without wind turbines. For short waves (i.e. $\omega = 0.64$ rad/s), deploying the wind turbines only reduces the bending moment of the hinged modules around the wind turbine in the upstream region of the VLFS whereas a slight increase of bending moment is observed in the downstream region.

It is noteworthy that only wave induced bending moment is presented in **Fig. 15**. If the total bending moment, including both the still water and wave induced bending moment, is considered, it can be found that the deployment of two wind turbines increases the total bending moment of the seven-hinge VLFS with the increasing rate depending on the wave amplitude. For example, for wave amplitude of 1 m, the maximum total bending moments of the VLFS with two wind turbines are $7.4 \times 10^7\ \mathrm{N \cdot m}$ for $\omega = 0.32$ rad/s, $8.1 \times 10^7\ \mathrm{N \cdot m}$ for $\omega = 0.46$ rad/s and $9.5 \times 10^7\ \mathrm{N \cdot m}$ for $\omega = 0.64$ rad/s. For the VLFS without wind turbines, these three values are $1.6 \times 10^6\ \mathrm{N \cdot m}$, $1.5 \times 10^7\ \mathrm{N \cdot m}$ and $4.5 \times 10^7\ \mathrm{N \cdot m}$, respectively. As a result, the ratio of the total bending moment for the VLFS with and without wind turbines is 46 for $\omega = 0.32$ rad/s, 5.4 for $\omega = 0.46$ rad/s, and 2.1 for $\omega = 0.64$ rad/s. If the wave amplitude is 5 m, these three ratios of the total bending moment for the VLFS with and without wind turbines are 10, 1.5 and 1.0, for $\omega = 0.32$ rad/s, 0.46 rad/s and 0.64 rad/s, respectively, respectively. Overall, a larger wave amplitude corresponds to a larger wave induced bending moment, which means that the portion of the still water bending moment in the total becomes smaller.

Fig. 17 shows the distribution of the vertical displacement along the longitudinal direction of the seven-hinge VLFS with or without two wind turbines on the deck. It can be seen that the vertical displacement does not change obviously by adding two wind turbines onto the VLFS, especially for $\omega = 0.32$ rad/s and 0.46 rad/s. For $\omega = 0.64$ rad/s (relatively short waves), there is a slight increase in the vertical displacement (with the maximum percentage of 15%) by deploying two wind turbines. Therefore, the present study may demonstrate that a multi-hinge VLFS design will benefit by adding the wind turbines onto the structure, especially in reducing the bending moment.

4. Conclusions

This paper presents a numerical study focusing on the effect of hinge number on the dynamic response of a VLFS. The discrete-module-beam-bending based hydroelasticity method has been applied to analyse the hinge effects. Numerical simulations provide substantial details on the bending and torsional moment, vertical and rotational displacement along the VLFS, as well as the shear force on hinges, which further leads to a feasibility study for deploying wind turbines on the VLFS.

Good agreement has been demonstrated between the present numerical method and previous numerical results. The present numerical simulations reveal that adding one hinge into the VLFS can not reduce the maximum bending moment along the VLFS. However, introducing more hinges could significantly reduce the bending moment along the VLFS. In addition, when the distance between each hinge is equal to half of the incoming wavelength, the vertical displacement along the VLFS is significantly increased.

For a seven-hinged VLFS, a unique two-peak phenomenon for the vertical shear force of the first three hinges is observed along with the increasing of wave frequency. Additionally, the wave frequency of the maximum vertical amplitude is observed at a slightly lower wave frequency compared to the shear force.

A feasibility study for deploying wind turbines on a seven-hinged VLFS is provided. It demonstrates that, the wave induced bending moment of a multi-hinge VLFS is further reduced by adding the wind turbines onto the structure whereas the total bending moment is enlarged due to the introduction of still water bending moment. The effect of the deployment of wind turbines on the vertical displacement of the VLFS is insignificant.

Finally, it is noteworthy that the proper modelling of the aerodynamic loads and the coupled analysis of wind- and wave-induced loads and responses deserve further investigations. Moreover, if a targeted sea state is available, it is worth performing the stress analysis for the cross sections of the VLFS for the purpose of structural integrity evaluation.

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Data availability

Some or all data, models, or code that support the findings of this study are available from the corresponding author upon reasonable request.

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Tables

Table 1 Benchmark VLFS dimensions and geometrical properties

Physical properties of the VLFS	**Prototype**
Length (m)	600
Width (m)	60
Depth (m)	2
Design draft (m)	0.5
Vertical bending stiffness (N·m^2)	4.77×10^{11}
Design operated water depth (m)	2000
Mass (kg)	1.845×10^{7}

Table 2 Principal dimensions of the 300 m long Mega-Float prototype structure.

Physical properties of the VLFS	**Prototype**
Length (m)	300
Width (m)	60
Depth (m)	2
Design draft (m)	0.5
Vertical bending stiffness (N·m^2)	4.77×10^{11}
Design operated water depth (m)	58.5
Mass (kg)	9.225×10^{6}

Table 3 Regular wave cases for the numerical simulation

Regular wave case	**Wave amplitude (m)**	**Wave frequency (rad/s)**	**Wavelength (m)**	**Wave period (s)**	**Wave direction (degree)**
Case 1	1	0.32	600	19.6	0
Case 2	1	0.40	400	15.9	0
Case 3	1	0.46	297	13.8	0
Case 4	1	0.56	199	11.3	0
Case 5	1	0.64	150	9.8	0
Case 6	1	0.9	75	6.95	0
Case 7	1	0.32	600	19.6	45
Case 8	1	0.46	297	13.8	45
Case 9	1	0.64	150	9.8	45
Case 10	1	0.32	600	19.6	90
Case 11	1	0.46	297	13.8	90
Case 12	1	0.64	150	9.8	90

Table 4 5MW wind turbine properties (Robertson, *et al.*, 2016)

Rating	5 MW
Rotor Orientation, Configuration	Upwind, 3 Blades
Rotor Diameter	126 m
Hub Height above the deck of VLFS	90 m
Rotor Mass (just blade mass)	6.70×10^{4} kg
Nacelle and hub Mass	4.779×10^{5}kg
Tower Mass	1.778×10^{5}kg