The 2020 World Congress on
Advances in Civil, Environmental, & Materials Research (ACEM20)
25-28, August, 2020, GECE, Seoul, Korea

Effect of wind field simulation approach on the response of a compliant offshore tower

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ABSTRACT

The economic production of petroleum in deep water requires innovative structures that often test the limits of fixed platform design technology. The growing demand for deepwater offshore platforms has thus initiated the development of economical but equally reliable alternatives in the last decade or so. As a result, there are quite a few new design concepts of offshore platforms available presently for deepwater applications. One of the promising concepts is compliant offshore towers, which are developed to counter the high stiffness requirements of shallow water platforms. Articulated towers are among the compliant offshore structures that freely oscillates with wind and waves, as they are designed to have low natural frequency than that of ocean waves. This paper deals with the dynamic behavior of a double-hinged articulated tower under two different wind field simulation approaches (single-point and multi-point). The analysis includes the nonlinearities due to fluctuating buoyancy, variable added mass, and instantaneous tower orientation. Hamilton's principle is used to derives the non-linear equation of motion. The equations of motion are solved in the time domain by using the Wilson-θ method. The root mean square (RMS) and maximum values along with salient power spectral density function (PSDF) of deck displacement and bending moment are presented under high and moderate sea states. The results establish that the multiple-point analysis, which includes lack of correlation over the entire structure, results in response estimates lower than that of the single-point formulation.

Keywords: Multi-point wind field; dynamic analysis; compliant tower; wind-induced response, offshore.

1. INTRODUCTION

As the development of offshore oil and natural gas extends into deeper waters, an increase in the economic attractiveness of offshore platforms becomes necessary. As the depth of water increases, the size of conventional fixed leg platforms is approaching

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the economic limit. As a result, several new structural systems have been developed to improve the water depth capability of offshore structures. Some of the promising concepts are tension leg platforms, guyed, and articulated tower platforms that take advantage of the effect of compliance, i.e., yield to the environmental forces (Li and Kareem, 1990).

The wind-induced response of an articulated tower is mainly due to the drag force in the direction parallel to the wind flow. This drag force is the function of space and time due to spatial variation of mean and fluctuating components of the wind speed. Typically, structural components on the platform deck obstruct the approaching longitudinal fluctuating velocity and transform it into a fluctuating force. The fluctuations are sensed by the platform, which results in oscillations in the along wind direction. Response in the along wind direction contains resonant contributions induced by turbulence. Compliant platforms are having low natural frequencies and admit the fluctuating wind energy in the resonant mode. Relevant studies related to wind-induced vibrations on compliant platforms such as TLPs, Semi-submersible, Spar, and Guyed towers can be found in (Kareem, 1985; Kareem *et al.*, 1987; Bisht and Jain, 1998; Ormberg *et al.*, 2003; Zeng *et al.*, 2006; Islam *et al.*, 2012; Oyejobi *et al.*, 2016; Ibrahim and Jameel, 2018; Antoniou *et al.*, 2019). The vibration control strategies of offshore platforms can be found in the review paper by Zhang *et al.* (2017). Chandrasekaran *et al.* (2013) developed a mathematical model for the analysis of triceratops, a new generation offshore platforms, under wind loads at a water depth of 600 m. Experimental investigations on TLP were carried out by (Chandrasekaran and Nassery, 2017) for controlling its response using passive dampers and showed effective control of surge response. While most of the researchers use single-point wind field simulations for the analysis of compliant towers, only a few works (Kareem, 1985; Kareem *et al.*, 1987) were reported on both single-point and multi-point wind field simulations, which has important practical applications.

Few studies dealing with the dynamic analysis of single hinge articulated towers have been reported in (Datta and Jain, 1990; Bar-Avi and Benaroya, 1996; Ghorai *et al.*, 2015; Kushal and Solomon, 2019). The design of a 3-legged articulated tower for wind turbines under offshore conditions was given by Philip *et al.* (2015). It showed that the suggested tower conforms well to the Indian Ocean conditions. In a recent study, (Nagavinothini and Chandrasekaran, 2019) analyses offshore triceratops in ultra-deep waters under the wind, wave, and current forces and showed that response increases with the increase in wave height and wind speed, but lesser than a surge in all sea states. However, the literature lacks double-hinged articulated towers, particularly in the wind environment. In a review paper, Zaheer and Islam (2008) noted that the design of offshore structures significantly influenced by various environmental loads, particularly in the context of compliant towers, which are sensitive to wind and wave loads. In another review paper, existing approaches used to estimate hydrodynamic and aerodynamic loads on floating offshore wind turbines was presented by Lamei and Hayatdavoodi (2020). Ghorai *et al.*, (2015) compared the response of a single and double hinged articulated tower under random waves. They showed that for the double-hinged tower, both hinge rotations are well below the permissible limits of 10˚ from the vertical position for satisfactory platform operations. Comparative response of multi-legged articulated towers under different wave theories was analyzed by (Aslam *et al.*, 2013a, 2013b).

Other studies on double-hinged towers, which verified the efficiency of the articulation system under the action of various environmental loads, are (Zaheer and Islam, 2008, 2010, 2017). Dynamic response of sea-crossing railway bridge under correlated wind and waves was studied by Fang *et al.*, (2019). The developed model involves a multi-point fluctuating wind field simulation. The study found to be imperative for the assessment of structural and vehicle safety of sea-crossing railway bridge. In another study, the extreme response of a sea-crossing bridge tower under correlated wind and waves was investigated by Fang *et al.*, (2019) to predict the structural response. The fluctuating wind load was modeled as a multi-point correlation. In one of the authors' recent studies, Zaheer and Islam (2020), the response of a bi-articulated offshore tower to ocean currents are compared under high and moderate sea states. However, in all these studies, wind formulation was employed either as single-point or multiple-point. The previous discussion reveals that comparative investigation on single-point and multiple-point wind field simulations for double-hinged towers is yet to be explored, making the present study novel. In the present study, a comprehensive mathematical model, Double Hinged Articulated Loading Platform (DHALP), is developed for investigating the aerodynamic response of the tower. The basis for the development of DHALP is that the behaviour of the articulated tower is comparable with other offshore compliant platforms, such as TLPs, Buoyant Leg Structures (BLS), and multi-legged articulated towers (Chandrasekaran and Madhuri, 2015).

2. METHODOLOGY

2.1 The Physical Model

A schematic of the double-hinged articulated tower is shown in Fig.1. It consists of a ballast chamber attached to the lower shaft of length L_1 and a universal joint at the bottom end O_1, which is fixed to the rigid foundation block at the seabed. The upper part of the structure consists of a buoyancy chamber attached to the shaft of length L_2 through another articulated joint O_2. The deck and other attachments are provided on the top of the buoyancy chamber. The length of the top tower is such that the buoyancy chamber remains below the water surface, and therefore hydrodynamic forces on the system are minimized.

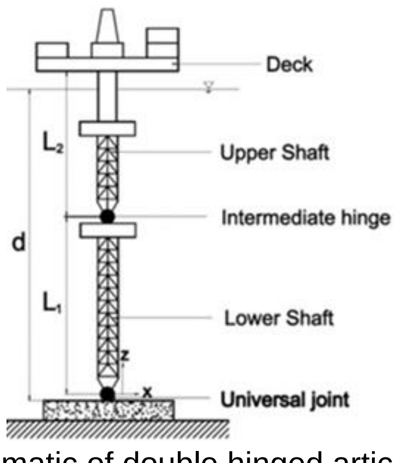

Fig. 1 Schematic of double hinged articulated tower

The articulation system consists of two devices (see Fig. 2 and Fig. 3). The rubber articulated joint, primarily supports the horizontal and vertical reactions. A wide torsional frame, as depicted in Fig. 3 was provided to resist the torsional moment (Zaheer and Islam, 2010).

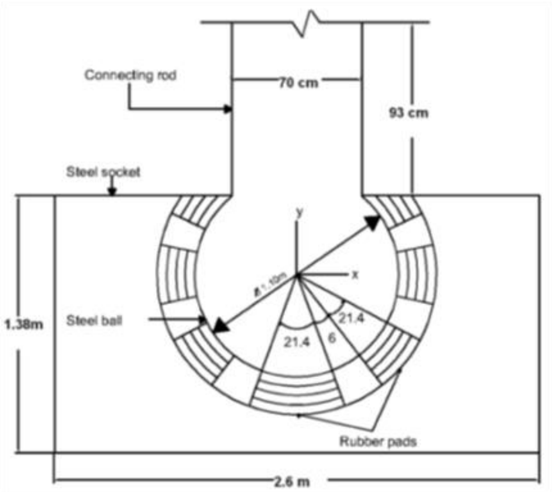

Fig. 2 Articulated joint

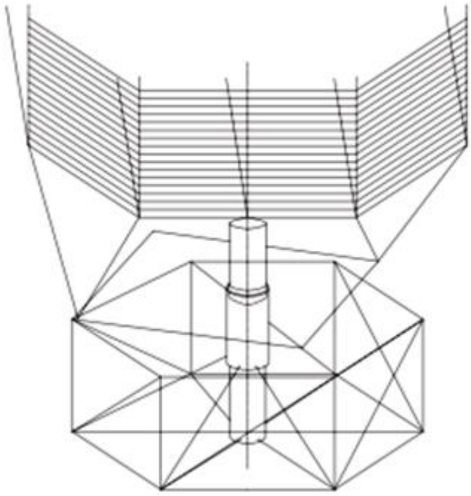

Fig. 3 Articulation system: Torsional

2.2. Mathematical Modeling

2.2.1 Governing Equations The equations of motion are derived using Lagrange's equation. This approach provides several advantages over the Newtonian method, like eliminating free body diagrams with interaction forces between the members (Craig, 1983). The ALP model consists of a two-degree-of-freedom system: rotations θ_1 and θ_2, about the vertical axis (planer motion). The equations of motion are derived for large displacements under the following assumptions.

2.2.2 Assumptions Following assumptions have been made for deriving the EOMs.

- The tower has a uniform mass per unit length.
- The tower stiffness is infinite.
- The deck mass is considered to be concentrated at the end of the tower.
- The structure is statically stable due to buoyancy forces.
- Drag and inertia coefficients in Morison's equation are assumed to be constant.
- The waves are linear, having random wave heights.

The first and second non-linear EOM of the articulated tower takes the form as:

$$(I_{1t} + m_{2t}L_1^2 + m_d L_1^2)\ddot{\theta}_1 - \left[m_{2t}c_2L_1\dot{\theta}_2\, sin(\theta_2 - \theta_1)\right]\dot{\theta}_2 + m_{2t}L_1c_2\, cos(\theta_2 - \theta_1)\,\ddot{\theta}_2$$
$$+\left[\{(F_1b_1 - W_1c_1) + (F_2 - W_2 - W_d)L_1\}\frac{sin\,\theta_1}{\theta_1}\right]\theta_1 = F_{\theta 1}(t) \quad (1)$$

$$\left(I_{2t} + I_d + m_d L_p^2\right)\ddot{\theta}_2 + \{m_{2t}c_2L_1\, cos(\theta_2 - \theta_1)\}\ddot{\theta}_1 + \left\{m_{2t}c_2L_1\dot{\theta}_1\dot{\theta}_2\, sin(\theta_2 - \theta_1)\right\}\dot{\theta}_1$$
$$+\left[\left(F_2b_2 - W_2c_2 - W_dL_p\right)\frac{sin\,\theta_2}{\theta_2}\right]\theta_2 = F_{\theta 2}(t) \quad (2)$$

In the above equation of motion, 1 stands for the lower tower while 2 stands for the upper tower. The step by step procedure of formulating EOM from the first principle has been given in Islam *et al.* (2009).

2.2.3 Forcing function The combined action of aerodynamic and hydrodynamic forces which constitutes the forcing function is given by:

$$F_\theta = F_a\{u(z), u', \dot{x}\} + F_d(\dot{u}, v_c, \dot{x}) + F_i(\ddot{x}) \quad (3)$$

in which $F_a\{u(z, u', \dot{x})\} =$ aerodynamic force, $F_d(\dot{u}, v_c, \dot{x}) =$ drag force, and $F_i(\ddot{x}) =$ inertia force. These environmental forces are elaborated in the following sub-sections.

2.3. Representation of the wind velocity field

The portion of the articulated tower that is above the water level is subjected to aerodynamic force in the windward direction, primarily resulting from drag. This force comprises of mean and fluctuating components. The dynamic wind force is produced by the latter component of the wind velocity. In order to formulate the total fluctuating wind load acting on the articulated tower, wind velocity fluctuations are described as a single- or multi-point wind field.

2.3.1 Mean wind simulation Either logarithmic law or power-law simulates the mean or static wind. In this study, the logarithmic law of wind speed variation with height is used and is given by:

$$\mathrm{u}(\mathrm{z}) = \mathrm{u}(\mathrm{z}_{\mathrm{ref}}) \times \ln\frac{z}{z_0} / \ln\frac{10}{z_0} \quad (4)$$

where $u(z_{ref})$ is the velocity of wind at 10 m height; z the vertical coordinate above MSL at 33.0 m and z_0 the length of roughness (0.005 m) for a rough sea surface.

2.3.2 Fluctuating wind velocity Past researchers have used several spectral models for wind fluctuations. Hare, Ochi and Shin spectrum is applied as it possesses high energy at low frequencies, and it can suitably represent many offshore conditions (Myrhaug, 2007).

$$\begin{aligned}
\frac{nS_u(z,n)}{U_*^2} &= 583 f_* && 0 < f_* \le 0.003 \\
\frac{nS_u(z,n)}{U_*^2} &= \frac{420 f_*^{0.70}}{\left(1 + f_*^{0.35}\right)^{11.5}} && 0.003 \le f_* \le 0.1 \\
\frac{nS_u(z,n)}{U_*^2} &= \frac{838 f_*}{\left(1 + f_*^{0.35}\right)^{11.5}} && 0.1 \le f_*
\end{aligned} \quad (5)$$

where $f_* = nz/u(z)$ is the dimensionless frequency, and U_* is the friction velocity.

2.4. Wind loading

2.4.1. Single-point wind field The wind forces can be treated as a single-point if the size of gusts λ is large enough compared to the typical dimension D of the structure ($\lambda / D >> 1$), which means that the wind velocity field is assumed to be fully correlated. This assumption is quite valid for structures with small spatial size (Kareem and Dalton, 1982). The wind force on the platform using single-point formulation is given by:

$$F(t) = \frac{\rho_a C_D A_p U^2(t)}{2} \tag{6}$$

in which $U(t) = u(z) + u'$; A_p is the tower projected area above MSL. The mean wind load is given by:

$$\overline{F} = \frac{\rho_a C_D A_p u^2(z)}{2} \tag{7}$$

and the fluctuating wind force is given by:

$$F(t) = \rho_a C_D A_p u(z) u'(t) \tag{8}$$

2.4.2. Multiple-point wind field For large offshore articulated towers, full correlation assumption is impractical and may give conservative results under aerodynamic loads. Therefore, in such cases, multi-point-statistics is employed to incorporate the effects of partial correlation (Kareem and Dalton, 1982). The multi-point simulation means that the fluctuating wind velocity varies along the projected area. A Spatio-temporal function defines the fluctuating wind field as:

$$U(y,z,t) = u(z) + u'(y,z,t) \tag{9}$$

in which $u(z)$ is the mean wind, and $u'(y,z,t)$ the two-dimensional Spatio-temporal fluctuating wind.

The fluctuating with wind force in surge direction on the platform area A_p is given by:

$$F_a(y,z,t) = \rho_a C_d \int_{A_p} u(z) \int u'(y,z,t)\, dydz \tag{10}$$

For evaluating Eq. (10) the wind velocity is to be simulated at n locations on the tower deck. Then, Eq. (10) is discretized as:

$$F_a(y,z,t) = \rho_a \sum_{i=1}^{n} C_{d_i} A_i u(z) u_i'(t) \tag{11}$$

in which A_i and C_{d_i} are the segmental area and drag coefficient, i represents the i^{th} segment and $u_i'(t)$ the simulated fluctuating velocity at the i^{th} segment.

2.4.3. Dynamic force due to wind loading After modeling the wind field either by single- or multiple-point, the velocity field is transformed into the aerodynamic loading. The wind force per unit projected area of the tower deck is given by:

$$F_a(y,z,t) = 0.5\rho_a C_p(y,z)[u(z) + u'(y,z,t) - \dot{x}(t)]^2 \quad (12)$$

here $F_a(y,z,t)$ is the force per unit area, varies in space and time coordinates, ρ_a the air density; $C_p(y,z)$ the pressure coefficient at a height z and horizontal coordinate y; $\dot{x}$ the structural velocity in the surge direction; $u(z)$ the mean wind velocity, and $u'(y,z,t)$ the fluctuating wind velocity.

2.5 Wave loading

Modified Morison's equation has been employed to estimate the hydrodynamic load. The force on the member of j^{th} tower at i^{th} location due to fluid-structure interaction is given by:

$$F_h(t) = 0.5\rho_w C_D D_{ji}\left(\dot{u}_{fi} - r_{ij}\theta_j + v_c\right)\left|\dot{u}_{fi} - r_{ij}\theta_j + v_c\right| + \frac{\pi}{4}C_M \rho_w \pi D_{ji}^2 \ddot{u}_{fi} \pm \frac{\pi}{4}\rho_w D_{ji}^2 (C_M - 1) r_{ij}\ddot{\theta}_j \quad (13)$$

where C_D and C_M are drag and inertia coefficients; v_c is the velocity of current; D_{ji} is the diameter of the j^{th} tower for i^{th} element; r_{ij} is the distance of the i^{th} element from the hinge of the j^{th} tower. $\dot{u}_{fi}$ and $\ddot{u}_{fi}$ are the water particle velocity and acceleration normal to the displaced j^{th} tower at i^{th} location; θ_j is the tilt angle of the j^{th} tower and ρ_w is the mass density of seawater. The last term in Eq. (13) is due to the added mass. The positive sign is used when sea surface elevation η is below the mean sea level and vice-versa.

The sea surface is assumed to be a Gaussian-ergodic process. In contrast, the sea surface elevation is assumed to be a superposition of infinite small harmonic waves having a randomly distributed phase. The PSDF of sea surface elevation is a graphical representation of the energy content of various harmonics present in it. Here, the DNV version of the Pierson-Moskowitz spectrum is used.

$$S_\eta = \frac{H_s^2 T_z}{8\pi^2}(T_z f)^{-5} \exp\left[-\frac{1}{\pi}(T_z f)^{-4}\right] \quad (14)$$

where f is the frequency (cycles/sec); H_s is the significant wave height (m), T_z is the wave period (sec), and S_η is the P-M sea surface elevation spectrum.

3. GENERATION OF TIME HISTORY RESPONSE

The time histories of articulated tower displacement are generated by the Wilson-θ method. Time histories of responses must be of sufficient length so that r.m.s responses attain the steady-state values. In this study, the simulated length of time histories is taken as three hours. The transition phase of the oscillation, which is about 10 times the time period of the structure, has been ignored. For all the responses, the structure is assumed to be initially at rest. The time histories are obtained at a time interval of 0.7 sec. The PSDF of the responses is obtained by direct Fourier transform of the response time histories using a fast Fourier Transformation technique (FFT).

4. NUMERICAL STUDY

The dynamic response of a double-hinged articulated tower under single- and multiple-point wind field simulation has been carried out in a water depth of 420 m. The two segments idealized tower with a lumped mass at the top is used in the present investigation (see Fig. 1). Each segment is divided into 50 elements. The length of the bottom and top tower are taken as 260 m and 210 m, respectively. The structural mass of each shaft of the tower is taken as 200 KN/m. The deck and ballast mass are taken as 25000 KN and 448.4 KN/m, respectively. The position of buoyancy chamber from the mid hinge is taken as 135 m. The following data represents a tower for which the effective diameters of the shaft for drag, the buoyancy, and the added mass, as well as inertia, are 17 m, 7.5 m, and 4.5 m, respectively. Likewise, the same effective diameters for the buoyancy chamber are 20 m, 19.5 m, and 7.5 m. For the double-hinged tower, the natural frequencies were found as ω_1 =0.14 rad/sec (first mode) and $\omega_2 =$ 0.42 rad/sec (second mode) using the developed source code DHALP (Double Hinged Articulated Loading Platform). The code was developed using FORTRAN Power Station.

The characteristics of sea states are given in Table 1, whereas the wind characteristics are presented in Table 2. Detail of nonlinearities considered in the study is given in Table 3. The details of various components on the deck are shown in Fig. 4. For plotting PSDFs, the wind fluctuations are represented by Ochi and Shin wind spectrum. The wind spectra corresponding to wind velocities of 25 m/sec and 15 m/sec are shown in Fig. 5. The sea states corresponding to these wind velocities are labeled as high sea state (18.0 m, 13.6 sec) and moderate sea state (6.5 m, 8.15 sec).

Table 1 Characteristics of sea states

Wind velocity (m s^{-1})	wave height H_s (m)	Wave period T_z (sec)	Wave frequency (rad/sec)	Sea state designation
25	18.0	13.6	0.29 rad/sec	High sea state
15	6.50	8.15	0.55 rad/sec	Moderate sea state

Table 2 Wind characteristics

Parameter	Value
Wind drag coefficient	1.81
Mean wind velocity	15 and 25 m/sec
Air density	1.27 kg/ m^3
Reference elevation, z_{ref}	33.0 m
Equivalent area of tower superstructure	1557 m^2

A uniform current velocity of 1.0 m/sec is adopted in the analysis. The drag and inertia coefficients are assumed as 0.6 and 2.0. In order to satisfy the ergodicity, the duration of simulated time histories is taken as 3 hours, with a sampling interval of 0.7 sec. The results for deck displacement, upper hinge roration, central hinge shear, and bending moment are summarized in Tables 4 and 5, respectively, for high and moderate sea states. Salient PSDFs are shown in Figs. 6- 9.

Table 3 Types of nonlinearities

Nonlinearities	Details of nonlinearities
Geometric nonlinearities	Large difference in the diameter of the buoyancy chamber and the tower's shaft
Force nonlinearities	Wave force introduces nonlinearity due to • Fluctuating buoyancy • Variable added mass • Instantaneous tower orientation

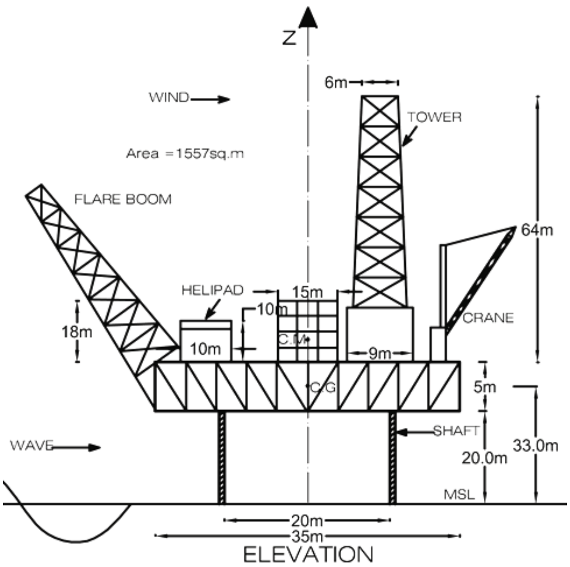

Fig. 4 Full scale dimensions of tower superstructure

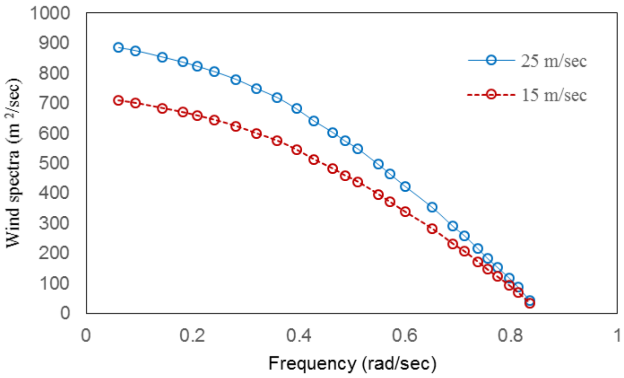

Fig. 5 Ochi and Shin spectra plot for different wind velocities

5. DISCUSSION OF RESULTS

5.1. Response under high sea state (H_s=18.0 m, T_z=13.6 sec)

Table 4 presents the RMS and maximum response of the tower for single-point and multiple-point wind field simulation for deck displacement, hinge rotation, hinge shear, and bending moment, respectively. When the multiple-point simulation is used, the RMS and maximum deck displacement response are reduced by 3.40% and 24.44%, respectively. Further, statistical results show that multiple-point analysis results in response estimates, which are generally lower than the single-point formulation, where the full correlation is tacitly assumed over the entirety of the structure. PSDF of the deck displacement response under high sea state is shown in Fig. 6. Two significant peaks characterize the response spectra. The first peak occurs near the vicinity of the first natural frequency (0.14 rad/sec) of the tower, while the second peak occurs at the towers second frequency (0.42 rad/sec). It is also seen that the application of multiple-point wind field simulation brings down the peak of the PSDF at salient frequencies. The smaller peaks in between these higher peaks are the characteristic of a non-linear articulated tower system. Because of stress reversals, hinge shear in the articulated joints causes fatigue. Therefore, its systematic evaluation is critical for the safety and survival of the tower. The statistical quantities for central hinge shear in Table 4 are reduced by 3.60%, and 12.93%, respectively, which signifies the importance of multiple-point wind field simulation. The PSDF of bending moment under high sea state is presented in Fig. 7. Two appreciable peaks appear in the response PSDF. The first most prominent peak occurs at a low frequency (0.06 rad/sec), showing the influence of wind on the bending moment response. The second peak occurs at the tower's first frequency. Similar to deck displacement response, multi-point analysis decreases the spectral energy to a large extent at respective frequencies.

Table 4 Comparison of responses under high sea state

Responses ↓ Statistics →	RMS			Maximum		
	Single-point	Multiple-point	Decrease in response (%)	Single-point	Multiple-point	Decrease in response (%)
Deck displacement (m)	11.2	10.8	3.4	16.2	12.2	24.4
Upper hinge rotation (rad.)	3.8×10^{-2}	3.7×10^{-2}	3.1	5.9×10^{-2}	4.9×10^{-2}	17.7
Central hinge shear (N)	2.5×10^{7}	2.4×10^{7}	3.6	4.9×10^{7}	4.3×10^{7}	12.9
Bending moment (Nm)	4.4×10^{10}	4.2×10^{10}	5.2	-1×10^{11}	-1×10^{11}	5.55

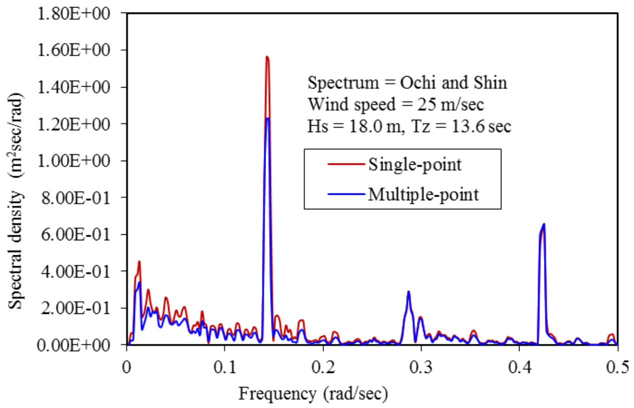

Fig. 6 PSD of surge response under high sea state

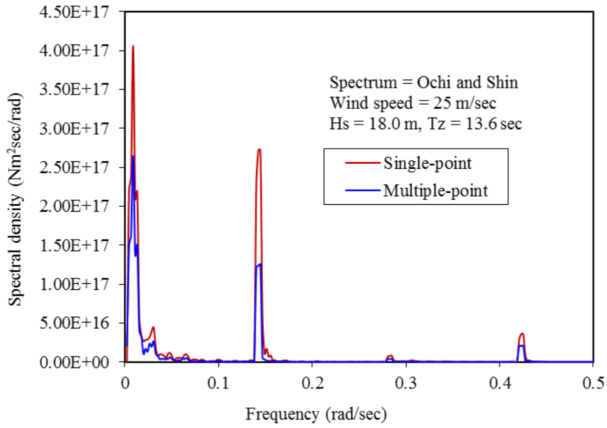

Fig. 7 PSD of bending moment response under high sea state

5.2 Response under moderate sea state (H_s=6.50 m, T_z=8.15 sec)

The PSDF of deck displacement under moderate sea state, as shown in Fig.8, is depicted by two prominent peaks. The first not so significant peak appears at a low-frequency, which corresponds to the peak frequency of the wind spectrum. The second spectral peak occurs at the first natural frequency of the tower (0.14 rad/sec). By comparing the deck displacement response under both wind simulations, it is seen that multi-point analysis significantly influences the deck displacement response of double articulated towers.

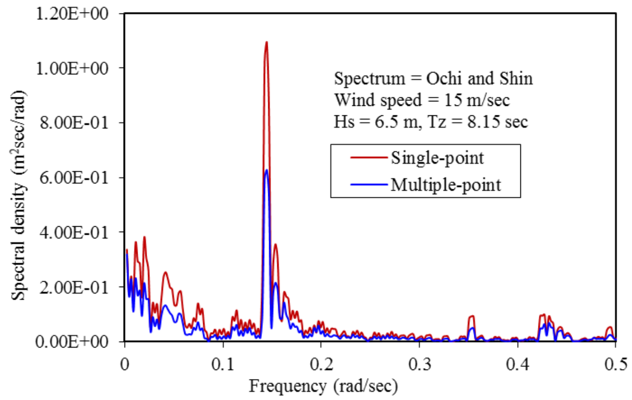

Fig. 8 PSD of surge response under moderate sea state

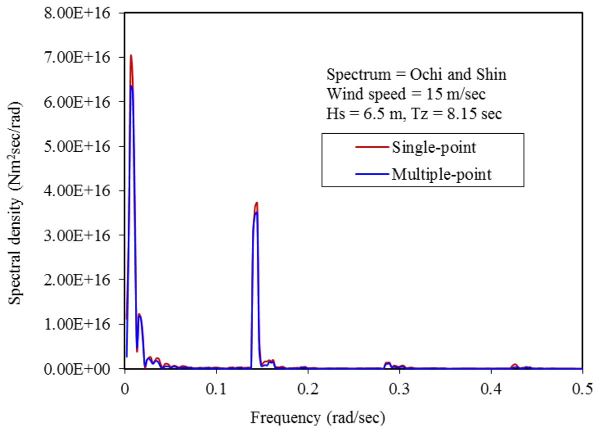

Fig. 9 PSD of bending moment response under moderate sea state

Table 5 Comparison of responses under moderate sea state

Responses ↓ Statistics →	RMS			Maximum		
	Single-point	Multiple-point	Decrease in response (%)	Single-point	Multiple-point	Decrease in response (%)
Deck displacement (m)	4.1	4.0	1.5	6.4	5.5	13.0
Upper hinge rotation (rad.)	1.4×10^{-2}	1.4×10^{-2}	1.4	2.6×10^{-2}	2.4×10^{-2}	5.8
Central hinge shear (N)	9.5×10^{6}	9.2×10^{6}	3.2	2.3×10^{7}	2.3×10^{7}	2.1
Bending moment (Nm)	1.5×10^{10}	1.4×10^{10}	5.8	-3.6×10^{10}	-3.4×10^{10}	6.1

Table 5 shows that multiple-point simulation lowers the RMS and maximum responses by 1.47% and 13.02%, respectively. Hinge shear response quantities in Table 5 under multi-point simulation are reduced by 3.24% and 2.15%, respectively. PSDF of bending moment response for moderate sea state is characterized by two peaks, as shown in Fig. 9. The second peak occurs at the tower's first frequency, which is not as significant as the first one. A decrease of about 6% is observed in RMS and maximum bending moment response while considering the effect of multiple-point wind simulation (Table 5).

6. CONCLUSIONS

From the dynamic analyses of the double-hinged articulated tower, it can be concluded that tower response is sensitive to the dynamic effects of wind. The multiple-point analysis results in response quantities, which are generally lower than the single-point formulation. The RMS and maximum response in the deck displacement are reduced by 3.40% and 24.44%, respectively when the multiple-point formulation is used under high sea state. The same response is reduced by 1.5% and 13.0% under moderate sea state. Therefore, it is argued that a conservative estimate of responses under single-point simulation can be effectively mitigated by using the multi-point formulation of wind loads, which results in the economical, safe, and reliable design of the tower. The energy content of PSDFs under multiple-point wind field is altered as compared to that with single-point simulation. It implies the significance of wind field simulation on the dynamics of a double-hinged tower. The distribution of energy against frequencies provides valuable data used in the design of a non-linear articulated tower system.

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NOMENCLATURE

L_1	length of the lower shaft
L_2	length of the top shaft

r_{2j}	position vector of an element in the top tower measured from the mid hinge
θ_1	hinge rotation of the lower shaft
θ_2	hinge rotationof the top shaft
F_θ	Forcing function due to environmental loads at any instant of time
I_{1t}	moment of inertia of the lower shaft
I_{2t}	moment of inertia of the top shaft
$\overline{m}_{2t}$	mass of the top tower
m_a	added mass of the structure
m_2	mass of the top shaft
m_{ac}	time-invariant added mass upto MSL
m_{af}	fluctuating added mass
m_d	mass of the deck
I_d	moment of inertia of the deck
L_p	height of c.g of the deck above mid hinge
P_{cm}	height of c.g above the deck
F_1	buoyancy force in the lower shaft
F_2	buoyancy force in the top shaft
W_1	weights of the lower shaft
W_2	weights of the upper shaft
W_d	weight of the deck
b_2	center of buoyancy in the upper shaft from mid hinge
c_2	center of mass in the lower shaft from mid hinge
F_a	aerodynamic force
F_d	fluid drag force
F_i	fluid inertia force
$\mathrm{C_p}$	wind pressure coefficient
$\dot{\mathrm{x}}$	structural velocity in the horizontal direction
u	mean wind velocity
u'	fluctuating wind velocity
A_a	projected area of the tower normal to the wind flow
$\mathrm{u}(\mathrm{z}_{\mathrm{ref}})$	reference velocity at a height of 10 m above MSL
$S(f_i)$	spectral density of one sided sea surface elevation spectrum at the frequency f_i