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Response of a Double Pendulum Compliant Offshore Tower to Collinear Wave and Current Forces

Mohd Moonis Zaheer[1], Nazrul Islam[2], Moazzam Aslam[3]

Associate Professor, Civil Engineering Section, University Polytechnic, AMU Aligarh, 202002, India[1]

Professor, Department of Civil Engineering, Islamic University, P.B.170, Madina, (KSA)[2]

Research Scholar, Department of Civil Engineering, Jamia Millia Islamia, New Delhi, 110025, India[3]

Abstract: Compliant offshore platforms are designed to allow controlled movement of the platform as an alternative to near rigid resistance to environmental forces by fixed platform designs. The dynamic response of a double articulated offshore loading structure to collinear wave and a steady current is studied for large and moderate sea states. The governing equations of motion are derived by the Lagrange's method where the wave and current forces are computed by a modified Morison's equation which takes account for the relative velocity of the water particle with respect to the oscillating tower. The analysis is carried out in time domain assuming rigid body motion, and the solution is generated by Wilson-**θ** method integration scheme. Numerical studies are conducted to compare the responses under large and moderate sea states with and without current forces. Results show that the presence of current in the wave field increases the responses in all the cases taken for the study.

Keywords: Articulated loading platform; Offshore structures; Compliant towers; Random wind; Dynamic response.

I. Introduction

As discoveries of hydrocarbons are made in deeper waters which are beyond the economic application of fixed platform technologies, alternative types of production platforms are required. Compliant platforms which move with environmental forces to a limited extent, rather than resisting their effects, present one solution to this problem.

An articulated loading platform (ALP) is one of the compliant structures which are designed for deep water offshore applications. Typically these structures are supported near the sea bed through a universal joint. The tower shaft, extending through the water surface consists of a ballast chamber, a steel lattice structure, buoyancy chamber, and a chimney supporting a platform above the water surface.

An articulated loading platform is also recognized by other names; articulated tower, buoyant tower, and articulated loading column. This type of structure is being used as an efficient means of mooring and oil loading terminal for tankers in open waters, flare towers as well as production riser and control tower in remote offshore environment.

II. Literature Review

Relevant information about dynamic analysis of ALP is available in various references. Jain and Kirk (1981) give an analysis of the dynamic response of double hinged articulated tower to non-collinear waves and current. The hydrodynamic loading neglecting tangential component has been calculated by modified Morison's formula. The study showed that the deck possess 3D complex whirling motion under waves with non-collinear currents. McNamara and lane (1984) discusses the finite element analysis of multi-hinged articulated towers. The study shows that the rigid body approximation yield results, close to finite element analysis. Hence, simpler rigid body formulation was justified. Hanna et al. (1988) gives a new concept of Tension Restrained Articulated Platform (TRAP). Analytical studies conducted on a three-segment TRAP under the action of wind, wave and currents. Study concluded that multi articulation concept is an attractive option for deepwater applications. Helvacioglu and Incecik (1988) gives an analysis and model test of single and double hinged towers subjected to wave and wind forces. The predicted responses of both the towers were comparable with the test results. Sellers and Niedzwecki (1992) furnishes valuable inputs on a general

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mathematical model of multi hinged articulated tower. The model was shown to be useful in changing the natural period of a structure prior to detailed member sizing and weight or buoyancy adjustments. Will et al. (1999) describes the design and installation of Baldpate compliant piled tower in the Gulf of Mexico. Islam and Ahmad (2003) include analytical results of double pendulum articulated tower under long crested and directional random sea. It was concluded that the presence of mid hinge causes large variation in shear forces at the hinges.

Review of the aforementioned references indicates the need for further study and development of analytical techniques especially for bi-articulated towers. The overall goal of the present study is to investigate the wave induced response of the platform in a realistic offshore environment.

III. ASSUMPTIONS AND STRUCTURAL IDEALIZATION

1. The platform is idealized as an inverted double pendulum with total mass and buoyancy of each pendulum is assumed to act respectively, at the centre of gravity and centre of buoyancy.
2. Damping matrix is assumed proportional to the mass and stiffness matrix.
3. Drag and inertia coefficients are assumed to be independent of the orientation of the tower.
4. For computation of hydrodynamic forces on the tower, it is divided into small elements and the load intensity between these small elements is assumed to remain constant. These elements have uniform properties over the segments of uniform diameters.
5. Diffraction effects are assumed to be insignificant as the member dimensions being small compared to incident wavelengths.

IV. DESCRIPTION OF THE PROBLEM

A schematic of the double hinged articulated tower is shown in Fig.1 (a). 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 at O_2. The upper part of the buoyancy chamber has a surface piercing cylinder of the same diameter as that of upper shaft. The deck and other attachments are provided on the top of buoyancy chamber. The net buoyancy force of the tower system is altered by changing the ballast size in the lower part of the tower. 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.

V. EQUATIONS OF MOTION

With the above idealization and assumptions, the equation of motion for the resulting two degree of freedom system is given as:

$$[M]\{\ddot{x}\}+[C]\{\dot{x}\}+[K]\{x\}=\{F(t)\} \tag{1}$$

The structural mass $[M]$ is assumed to be lumped at the deck and at intermediate hinge. Hence, it is diagonal in nature and constant. The added mass due to the water surrounding the structural members and arising from the modified Morison equation has been considered upto MSL. The presence of off-diagonal term in the mass matrix indicates contribution in added mass due to the hydrodynamic loading. The distribution of the inertia forces due to added mass is same as that due to the structural mass. The mass matrix and the stiffness matrix of double hinged ALP have been derived from the first principle presented by Islam et.al (2009). The forcing function $F(t)$ is the time dependent random hydrodynamic loading due to long crested random sea. Current velocity may be added to the water particle velocity due to waves.

VI. HYDRODYNAMIC LOADING

Wave forces

The time histories of the water particle velocity and acceleration have been evaluated using Pierson-Moskowitz spectrum (Pierson-Moskowitz, 1964). Here, a random wave is represented by the one sided DNV version of the Pierson-Moskowitz spectrum.

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

where f is the frequency in cycles per second, T_z is the zero up crossing period, S_η is the single sided P-M sea surface elevation spectrum and H_s is the significant wave height.

The wave superposition technique suggested by Goda (1970) is adopted for the simulation of sea surface elevation. The component wave frequencies are non-correlating. The random selection of component frequency is repeated for each run of the sea surface elevation spectrum. Based on the performed studies, the asymptotic approach to the Gaussian distribution is found time-consuming for the number of component waves above 50 and hence the simulation is carried out with 50 component waves. The length of the simulated wave record is controlled so that about 4000 data points are generated in one run. Here, record length of 32400 s is chosen which gives more than 4096 (2^{12}) data points.

Water particle kinematics is calculated using Airy's linear wave theory along with the stretching modifications suggested by Chakrabarti (1971).

The wave load intensity F_h on the tower is evaluated using the modified Morison equation, which is given as:

$$F_h(t) = \left[\frac{1}{2}\rho_w D_d C_D (\dot{u} - \dot{x} + V_c)|\dot{u} - \dot{x} + V_c|\right] + \frac{\pi}{4}\rho_w C_M D_i^2 \ddot{u} \pm \frac{\pi}{4} D_i^2 (C_m - 1)\rho_w \ddot{x} \tag{3}$$

where V_c is the velocity of current; D_d and D_i are effective drag and inertia diameters; ρ_w is the mass density of water; $\dot{u}, \ddot{u}$ are the normal water particle velocity and acceleration; $\dot{x}$ and $\ddot{x}$ are the structure velocity and acceleration; and C_D and C_m are drag and inertia coefficients. The last term in the above equation is the added mass term. The positive sign is used when η is below MSL, and the negative sign is used when η is above MSL. In the present study, current is assumed to be constant throughout the water depth.

VII. STATISTICS OF ALP DYNAMIC RESPONSE

The ALP design requires the consideration of serviceability and survivability in the extreme environmental conditions. Failure due to large displacements in extreme conditions and the provision of adequate fatigue life of the articulated joint necessitate a realistic prediction of deck displacement, hinge rotations and hinge shear stresses. The spectral analysis gives information on the r.m.s statistics of the random phenomena. The statistical response quantities (maximum, minimum, mean and standard deviation) get modified when major nonlinearities are taken into consideration. This can be achieved by simulated time histories of responses generated for a set of waves with and without current forces.

Although the dynamic response of ALPs subjected to random sea waves has been obtained by many investigators, the same for double hinged ALP's has not been widely investigated. Therefore, there is a need for further investigation of response statistics and spectral description of response time histories.

VIII. NUMERICAL STUDY

A double hinged ALP in 420 m deep water has been chosen for the numerical study. The idealized tower consists of a two segment vertical line cantilever, having a lumped mass at the top (Fig.1 (b)). Each cantilever is discretized into 50 elements. Characteristics of the platform and the environment used in the present study are reported in Table 1. The angle of attack of long crested sea is assumed as zero degree. The natural frequencies of the system for the two modes of vibration are 0.14 rad/sec and 0.42 rad/sec, respectively.

Dynamic response of double hinged ALP under long crested sea is obtained with and without current forces. Random waves are represented by the DNV version of one-sided Pierson-Moskowitz energy spectrum. Two random sea states are considered, namely, large sea state (H_s =18.0 m, T_z =13.6 sec) and moderate sea state (H_s = 6.5 m, T_z = 8.15 sec). These sea states adequately cover the condition of significant dynamic excitation. Each sea state is simulated for duration of one hour. Note that simulated length excludes the initial transient non-stationary phase of the response due to initial conditions. Response PSDFs and important statistical parameters such as maxima, minima, mean and standard deviation are obtained for various responses.

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IX. RESULTS AND DISCUSSION

1. Response under large sea state (H_s =18.0 m, T_z =13.6 sec)

In this section, the platform response to random sea waves with and without current forces has been studied. A linear profile of current is assumed. ALP response due to random sea is found to be significantly influenced in the presence of current. The steady drift is enhanced due to the presence of current velocity introducing pseudo static effect as shown in various time histories.

1.1 Surge Response

All operational activities such as exploration and drilling on the ALP deck are strongly influenced by the surge response at this level. The maxima and minima for the surge response without current forces range from +2.16 m to -2.17 m, while the mean value is -0.02 m as presented in Table 2. When current forces are included in the response calculation, the maxima and minima becomes +2.42 m and -1.79 m, respectively (Table 3). However, the mean value becomes positive (+0.102 m). This is due to drifting of the platform to right hand side under current forces as shown in Fig. 2. The response PSD is shown in Fig. 3. It shows a peak of mild intensity at structures fundamental frequency under no current condition. In case of surge with current, the corresponding peak energy reduces. It is mainly due to the fact that current in the direction of wave propagation tends to lower the wave height. A cluster of peaks occur in the vicinity of 0.3 rad/sec which is nothing but the frequency of dominant wave loading.

1.2 Tilting Motion Response

Tilting motion response directly influences the platform serviceability and survival. The time traces for upper tilting motion are shown in Fig. 4. Table 2 presents the maximum and minimum response for upper tilting motion as $+1.56 \times 10^{-2}$, rad and -1.59×10^{-2} rad, respectively. The mean and standard deviation of upper tilting motion values are -2.01×10^{-5} rad and $+5.70 \times 10^{-3}$ rad, respectively. These values at the central hinge have their own significance with respect to operational constraints. Again, when current is considered along with waves, it influences the response in terms of static offset as presented in Table 3. The PSD of upper tilting motion (Fig. 5) shows a solitary peak at the wave frequency.

1.3 Hinge Shear Response

The dynamic response at the hinge joints where the shaft is allowed to tilt relative to the base is of great importance, especially in relation to the fatigue damage. Central hinge shear time trace show the random behaviour under large sea state (Fig. 6). The time histories show the introduction of smaller fluctuations (high frequency content) in between the large scale motion. The presence of high frequency oscillations in the response is responsible to shorten the expected fatigue life of the platform. The PSD of central hinge shear is shown in Fig. 7. There are several peaks shown but the maxima occur approximately at 0.30 rad/sec which is close to the wave excitation frequency. Another peak, at three times the wave frequency is also seen in the PSD.

The statistical Table 2 presents the maximum and minimum values as $+2.10 \times 10^7$ N and -2.00×10^7 N, respectively in the central hinge. The mean value is obtained as $- 2.45 \times 10^4$ N. A somewhat lesser values are observed for base hinge shear highlighting the significance for the design of central hinge. Table 3 shows the statistics of hinge shear when current is included in the analysis. The mean values of hinge shear under current condition are positive (2.68×10^5 N and 2.31×10^5 N) as opposed to negative values under no current (-2.45×10^4 N and -1.41×10^4 N).

1.4 Bending Moment Response

Fig. 8 shows a segment of bending moment response time history at node 24. The PSDF of bending moment in Fig. 9 is characterized by a single peak at the frequency of wave loading. Table 2 presents the maximum and minimum values of bending moment response as $+1.80 \times 10^{10}$ Nm and $– 2.65 \times 10^{10}$ Nm, respectively, while, the mean value is $–8.54 \times 10^7$ Nm. Maximum bending moment in the presence of current (Table 3) increases to $+3.36 \times 10^{10}$ Nm, while, minimum value is $–2.11 \times 10^{10}$ Nm. Mean value in the presence of current increases to 1.39×10^8 Nm because of the static force of current.

2. Response under moderate sea state (H_s =6.5 m, T_z =8.15 sec)

ALP dynamics gets significantly modified under moderate sea state. The response behaviour is entirely different when compared to large sea state. Hence, it is important to analyse the platform for a wide range of probable sea states. The following section deals with the platform behaviour under moderate sea state

2.1 Surge Response

The surge time history in Fig. 10 shows a periodic response. In the presence of current, the platform is displaced to the positive side from the mean position. The platform then oscillates in a random fashion. The current and wave response are observed to be lower than that due to wave alone (Table 5). It may be due to the fact that static current pushes the structure to static lateral offset, and oscillation is therefore, controlled.

The PSD in Fig. 11 show two distinct peaks. These peaks correspond to natural frequencies of the platform. Most prominent peak occurs at 0.14 rad/sec which is platform's fundamental frequency. Other peak occurs in the vicinity of 0.42 rad/sec which is due to excitation of second mode of vibration. As can be seen from the PSD, influence of second mode of vibration is rather insignificant in the overall platform motion. This pattern is very much different than that of large sea state as discussed in the previous section. It showed that the response is influenced significantly by the intensity of sea state. It is also seen that when inline current is considered, peak of the energy reduces.

2.2 Tilting Motion Response

In case of moderate sea state, tilting motion response time history shows beating phenomenon (Fig. 12). In other words, high and low amplitude (responses) following each other in a sinusoidal wave form. The PSD of time history (Fig. 13) show two frequencies near 0.15 rad/sec. The twin peaks occur is due to the oscillation pattern of high and low amplitude motion following each other. This twin peak behaviour is not observed in case of large sea state. Such behaviour show that the excitation frequencies is linked with the sea states and participating frequencies.

2.3 Hinge Shear Response

In case of (6.5 m, 8.15 sec) wave, the power spectral density function (Fig. 14) shows that excitation frequencies occur at structural frequencies only. Several smaller peaks in the PSD represent the nonlinearity in the system. This pattern is different than the case under large sea state, where the response is governed by wave forcing.

The random time history (Fig. 15) shows the fluctuations about the mean. The mean central hinge shear in the presence of current is $+8.95 \times 10^4$ N while the standard deviation is 4.43×10^5. On comparing the response with base hinge shear it is observed that design of central hinge is more crucial because of high shear concentration. The same values of mean and standard deviation under no current condition are observed as -2.82×10^3 N and 1.53×10^6 N, respectively. This shows that presence of current has influenced the response in a significant way.

2.4 Bending Moment Response

Bending moment response PSD for small sea state (Fig. 17) is distinctly different than that for large sea state (Fig. 9). Fig. 17 show peaks at the structural frequencies while, Fig. 9 showed a distinct peak only at the wave frequency. The maximum bending moment along with current are observed to be higher as compared to the case with no current. The time history shown in Fig. 16 verifies this claim.

X. CONCLUSIONS

1. The energy content of PSDs in the presence of current is significantly reduced as compared to that with no current case. It is mainly due to the current induced damping in the ALP system.
2. The statistical characteristics under large and moderate sea state show the extreme response values which are imperative for serviceability and survivability of the tower. Mean values for various parameters are higher when current is considered along with wave.
3. Upper tilting motion response is found to be higher than the lower tilting motion. However, the maximum tilt in each case is less than 4° for successful platform operations.
4. The responses under moderate sea state show the participation of second mode apart from exciting an appreciable response at the structure's first frequency. In case of large sea state, the mean square response is contributed by the amplification at the peak frequency of the wave loading.
5. Hinge shear time histories give a quantitative estimate of the fatigue stresses in the articulated joints. The presence of high-frequency (smaller) oscillations in between large scale motion in the response time traces can shorten the expected fatigue life of the articulation points.

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TABLE 1: Principal characteristic of platform and the environment

Parameter	Value
Water depth, d	420 m
Height of bottom tower, L_1	260 m
Height of top tower, L_2	210 m
Structural mass of top tower	2.0×10^4 kg/m
Structural mass of bottom tower	2.0×10^4 kg/m
Mass of ballast	44.84×10^3 kg/m
Deck mass, m_d	2.5×10^6 kg
Position of buoyancy chamber from mid hinge	135 m
Structural frequency (first and second mode)	0.14 rad/sec, 0.42 rad/sec
Mass density of sea water, ρ_w	1025 kg/m^3
Coefficient of drag and inertia, (C_D , C_I)	0.6, 2.0
Current velocity, v_c	1.0 m/s
Tower's Shaft	
Effective drag diameter	17.0 m
Effective diameter for buoyancy	7.50 m
Effective diameter for inertia	4.5 m
Effective diameter for added mass	4.5 m
Buoyancy chamber	
Effective drag diameter	20 m
Effective diameter for buoyancy	19.5 m
Effective diameter for inertia	7.5 m
Effective diameter for added mass	7.5 m

TABLE 2: Statistical response of ALP under large sea state (Hs =18.0 m, Tz =13.6 sec)

Response	Mean	S.D	Maximum	Minimum
Surge	-0.02	0.716	2.16	-2.17
Lower tilting motion	-5.95×10^{-5}	2.19×10^{-3}	5.43×10^{-3}	-5.74×10^{-3}
Upper tilting motion	-2.01×10^{-5}	5.70×10^{-3}	1.56×10^{-2}	-1.59×10^{-2}
Base hinge shear	-1.41×10^{4}	6.58×10^{6}	2.00×10^{7}	-1.90×10^{7}
Central hinge shear	-2.45×10^{4}	6.80×10^{6}	2.10×10^{7}	-2.00×10^{7}
Bending moment	-8.54×10^{7}	1.19×10^{10}	1.80×10^{10}	-2.65×10^{10}

TABLE 3: Statistical response of ALP under large sea state (Hs =18.0 m, Tz =13.6 sec) with current (1.0 m/sec)

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Response	Mean	S.D	Maximum	Minimum
Surge	0.102	0.719	2.42	-1.79
Lower tilting motion	2.40×10^{-4}	2.16×10^{-3}	6.04×10^{-3}	-4.23×10^{-3}
Upper tilting motion	1.62×10^{-4}	5.71×10^{-3}	1.72×10^{-2}	-1.51×10^{-2}
Base hinge shear	2.31×10^{5}	6.23×10^{6}	2.46×10^{7}	-2.27×10^{7}
Central hinge shear	2.68×10^{5}	6.08×10^{6}	2.51×10^{7}	-2.19×10^{7}
Bending moment	1.39×10^{8}	1.21×10^{10}	3.36×10^{10}	-2.11×10^{10}

TABLE 4: Statistical response of ALP under moderate sea state (Hs =6.5 m, Tz =8.15 sec)

Response	Mean	S.D	Maximum	Minimum
Surge	-2.98×10^{-4}	0.099	0.211	-0.217
Lower tilting motion	4.48×10^{-7}	5.32×10^{-4}	1.10×10^{-3}	-1.10×10^{-3}
Upper tilting motion	-1.96×10^{-6}	1.13×10^{-3}	2.35×10^{-3}	-2.35×10^{-3}
Base hinge shear	-2.12×10^{3}	1.41×10^{6}	2.90×10^{6}	-2.90×10^{6}
Central hinge shear	-2.82×10^{3}	1.53×10^{6}	3.20×10^{6}	-3.20×10^{6}
Bending moment	1.85×10^{7}	1.65×10^{9}	2.73×10^{9}	-3.11×10^{9}

TABLE 5: Statistical response of ALP under large sea state (Hs =6.5 m, Tz =8.15 sec) with current (1.0 m/sec)

Response	Mean	S.D	Maximum	Minimum
Surge	0.219	0.048	0.279	-0.081
Lower tilting motion	2.52×10^{-4}	1.65×10^{-4}	1.39×10^{-3}	-7.74×10^{-4}
Upper tilting motion	2.30×10^{-4}	3.52×10^{-4}	2.57×10^{-3}	-2.09×10^{-3}
Base hinge shear	8.76×10^{4}	4.79×10^{5}	3.42×10^{6}	-3.04×10^{6}
Central hinge shear	8.95×10^{4}	4.43×10^{5}	4.75×10^{6}	-3.77×10^{6}
Bending moment	4.08×10^{7}	6.94×10^{8}	3.74×10^{9}	-3.11×10^{9}

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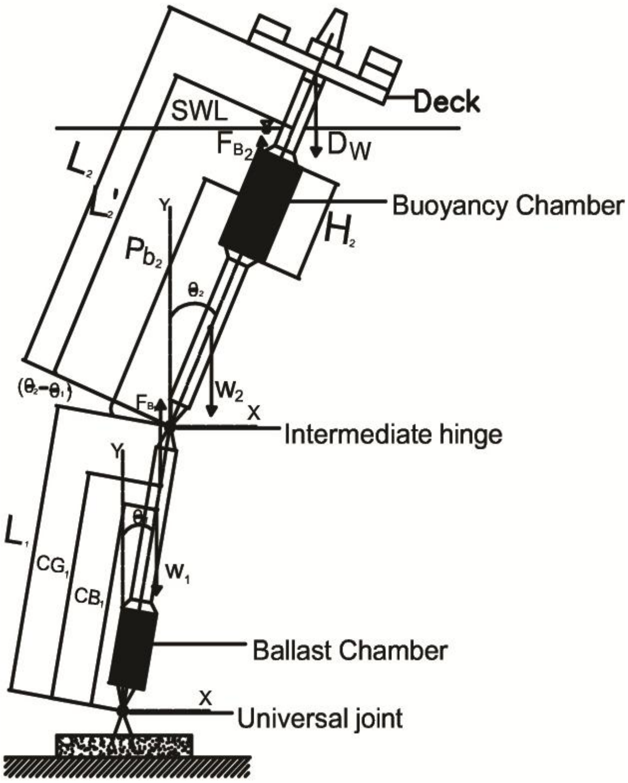

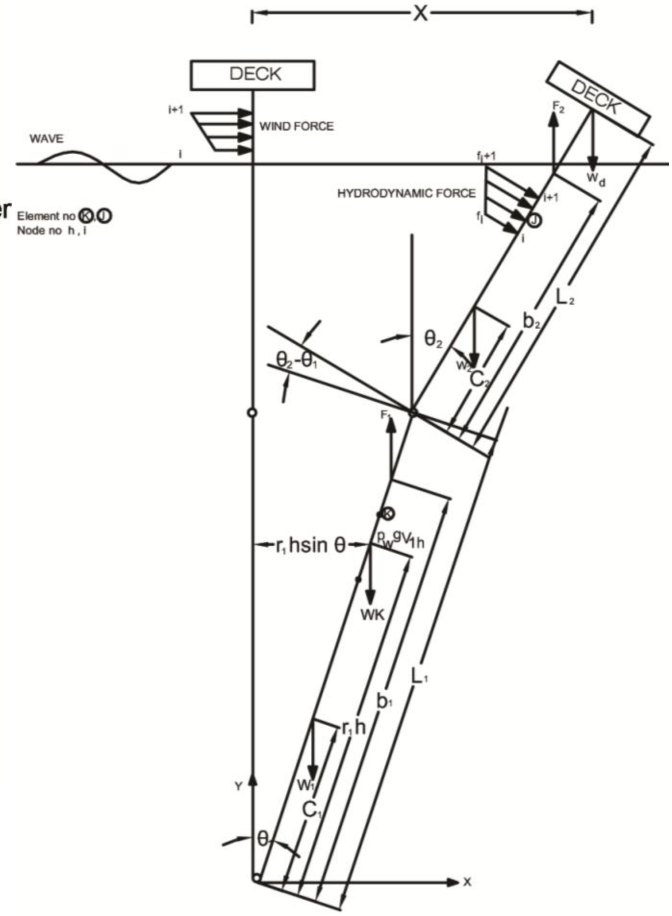

Fig. 1(a) Mathematical Model of double hinged ALP **Fig. 1(b) Schematic of double hinged articulated tower**

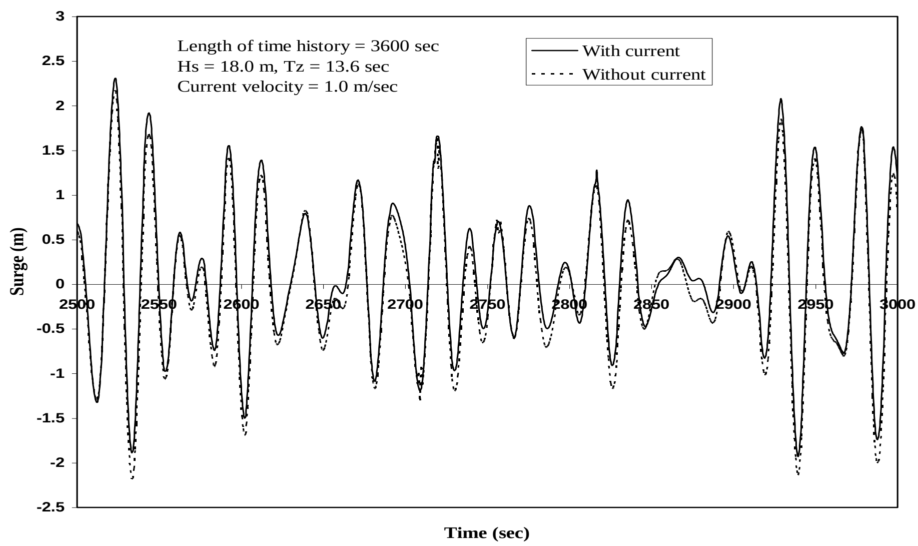

Fig. 2 Surge response time history under large sea state

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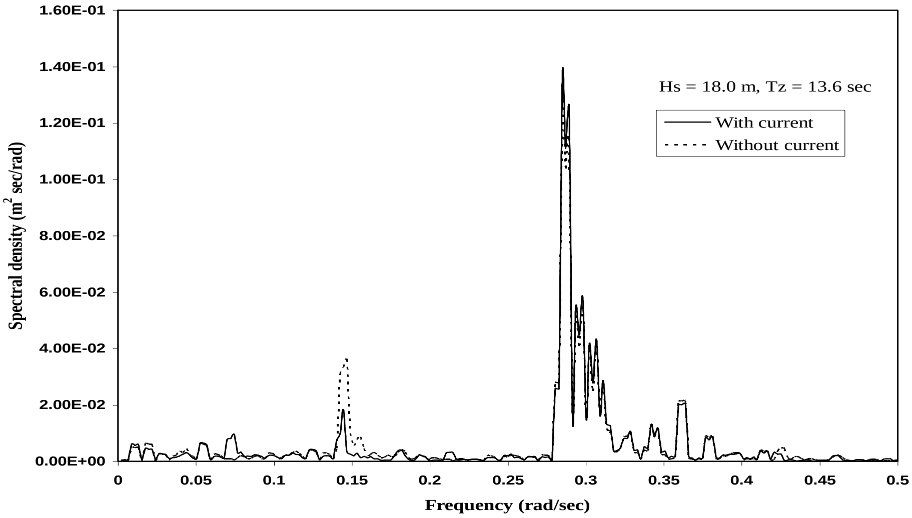

Fig. 3. PSD of surge response under large sea state

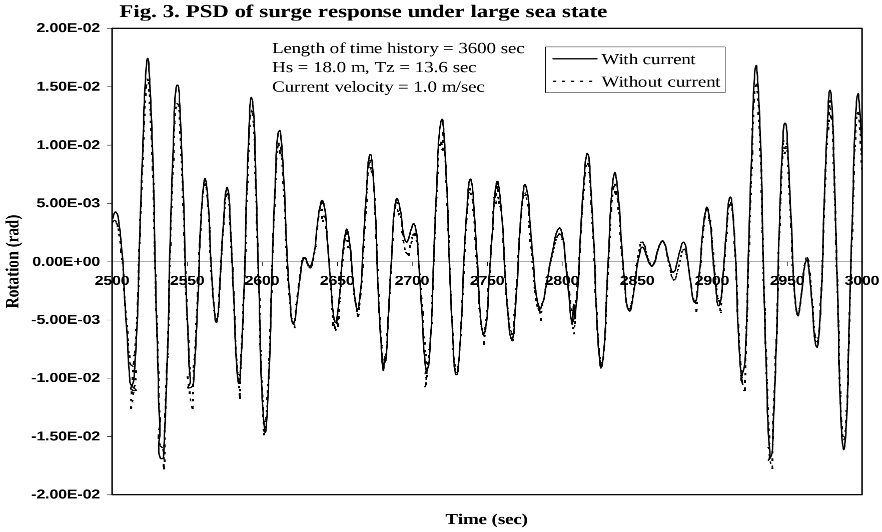

Fig. 4. Upper tilting motion response time history under large sea state

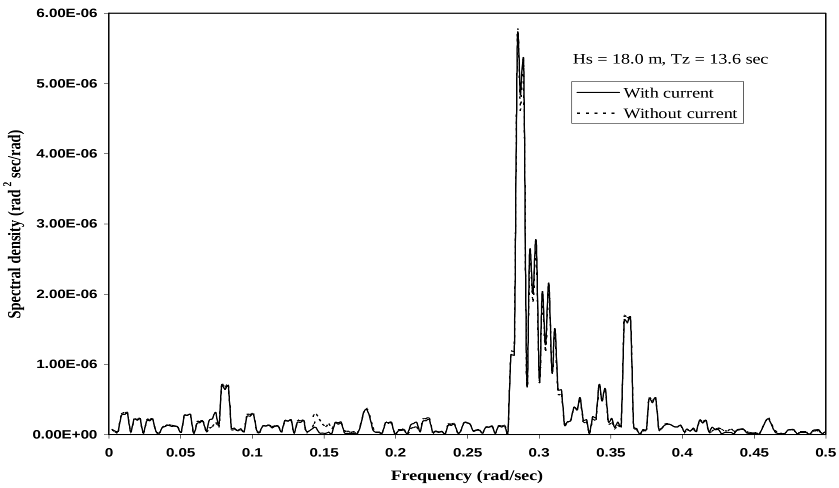

Fig. 5. PSD of upper tilting motion response under large sea state

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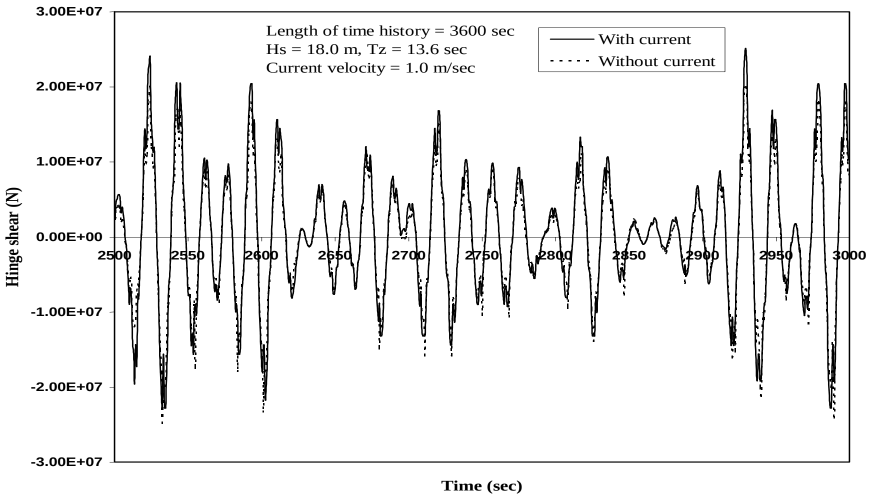

Fig. 6. Central hinge shear response time history under large sea state

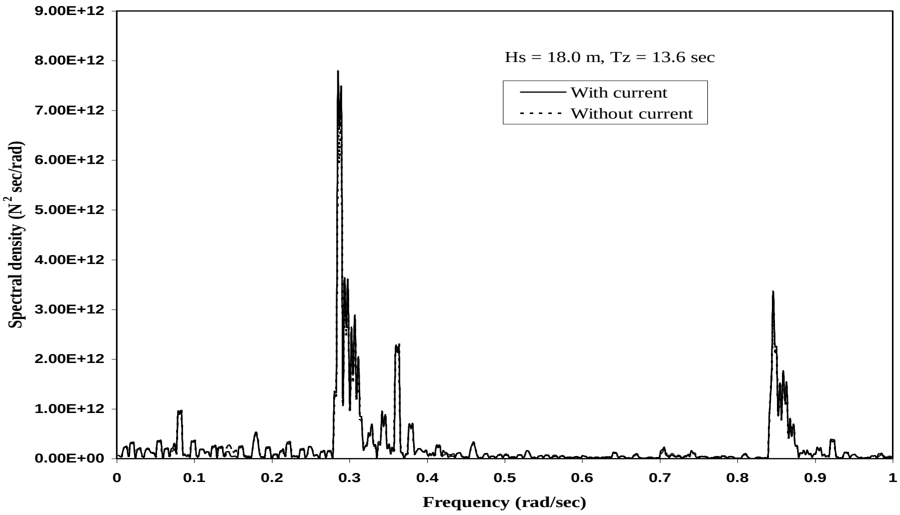

Fig. 7. PSD of central hinge shear response under large sea state

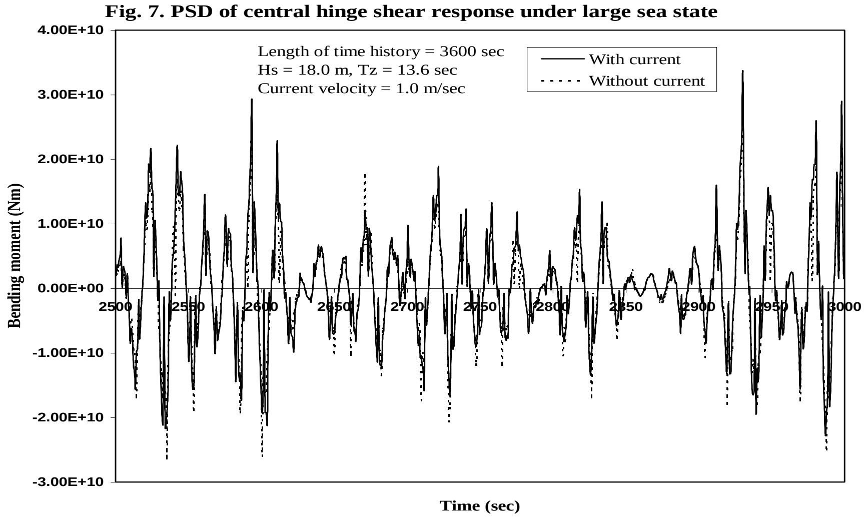

Fig. 8. Bending moment response time history under large sea state

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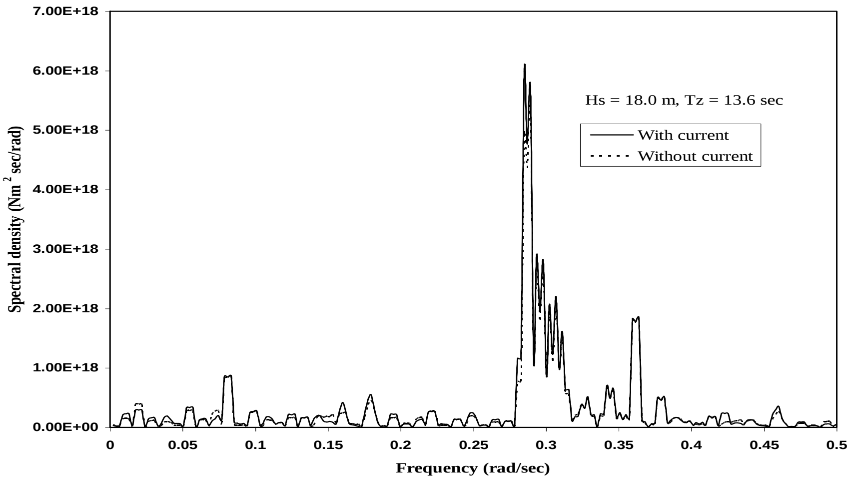

Fig. 9. PSD of bending moment response at node 24 under large sea state

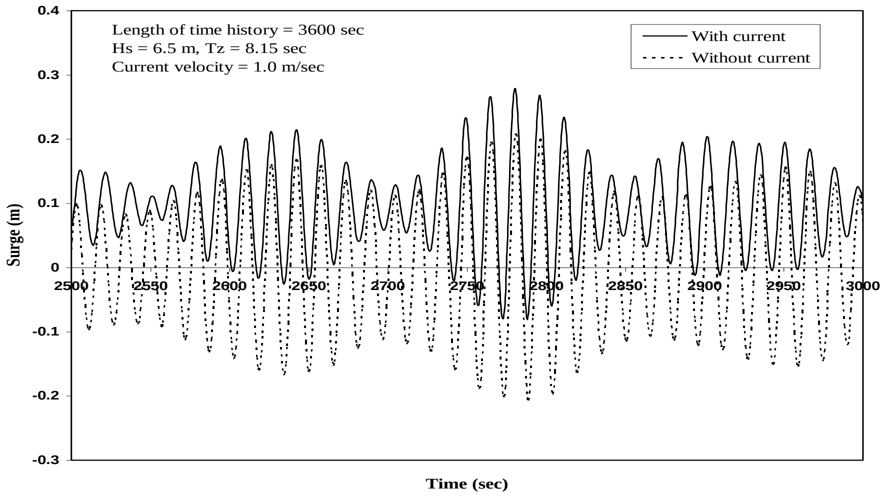

Fig. 10. Surge response time history under moderate sea state

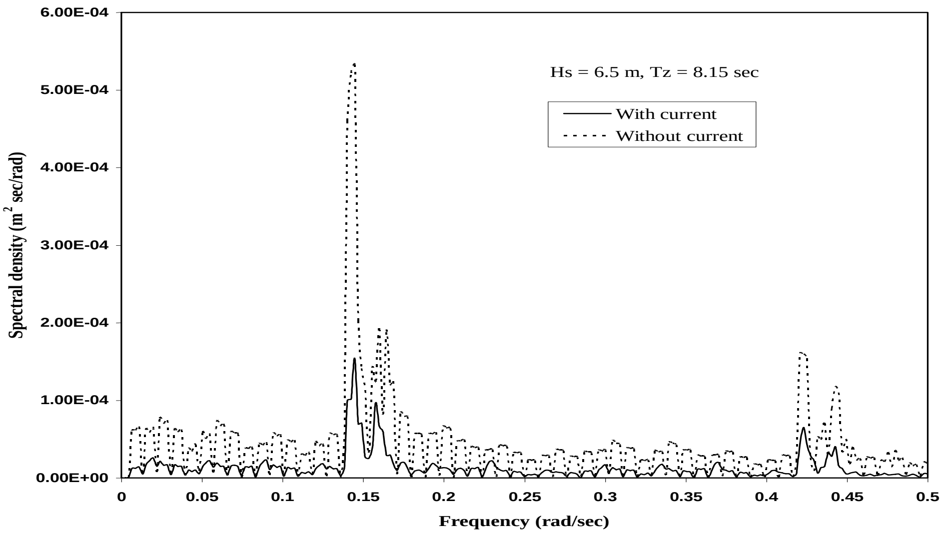

Fig. 11. PSD of surge response under moderate sea state

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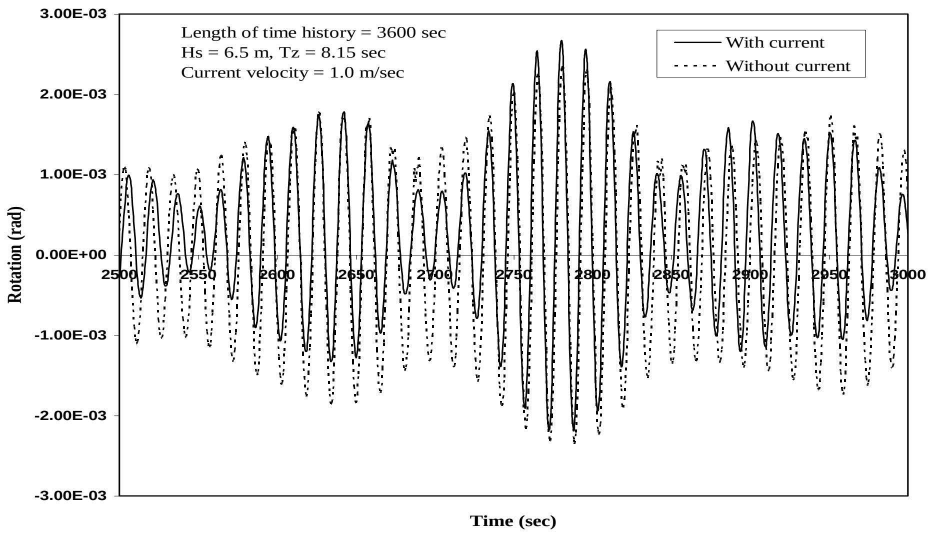

Fig. 12. Upper tilting motion response time history under moderate sea state

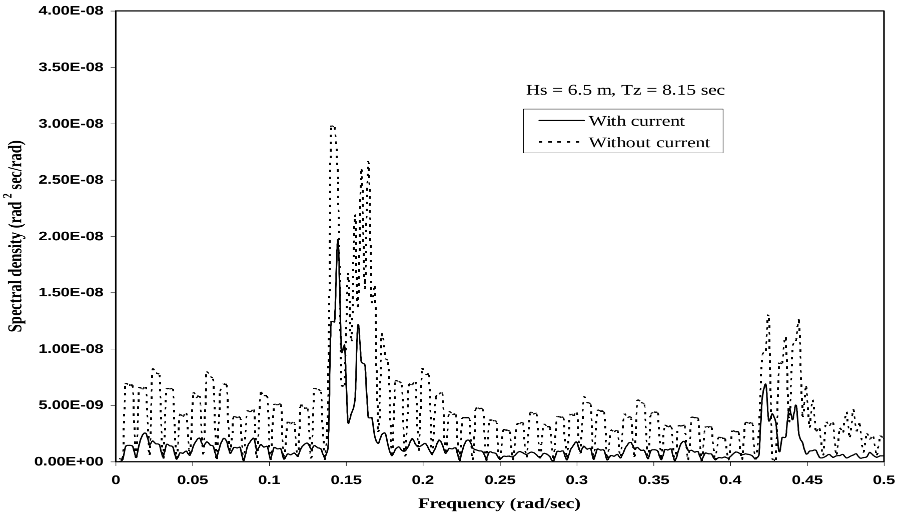

Fig. 13. PSD of upper tilting motion response under moderate sea state

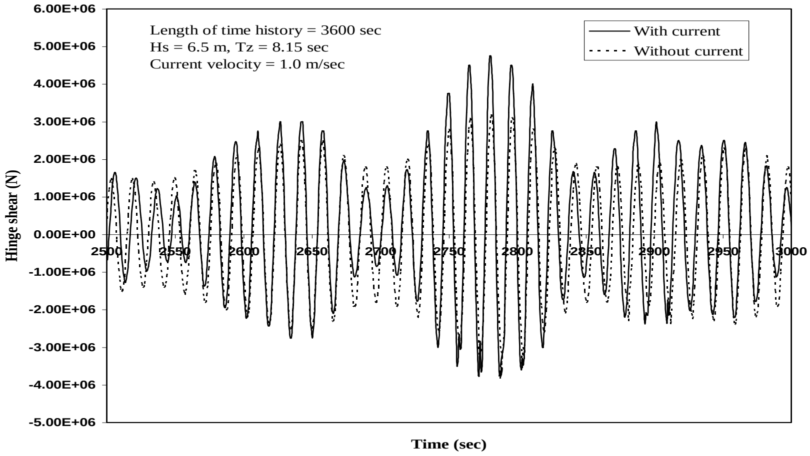

Fig. 14. Central hinge shear response time history under moderate sea state

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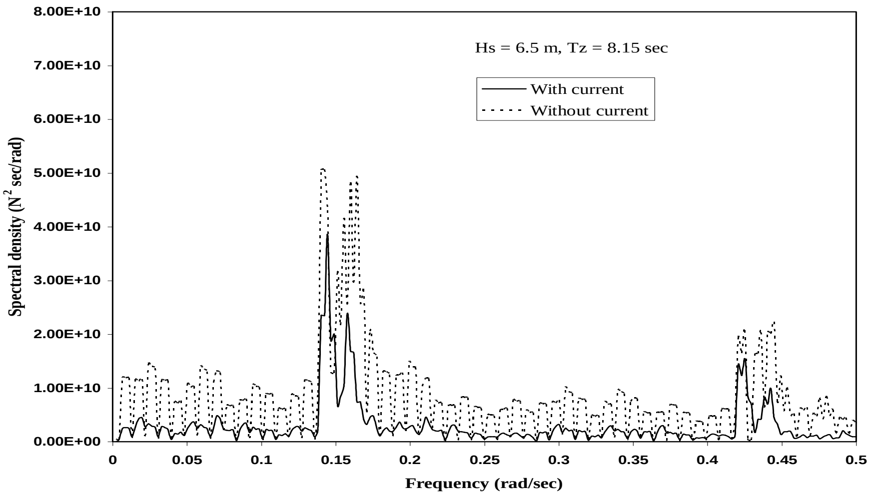

Fig. 15. PSD of central hinge shear response under moderate sea state

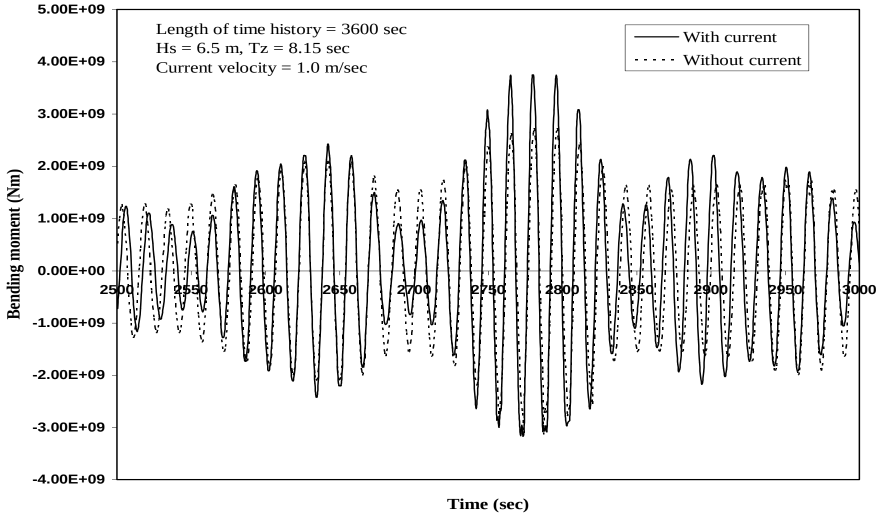

Fig. 16. Bending moment response time history at node 24 under moderate sea state

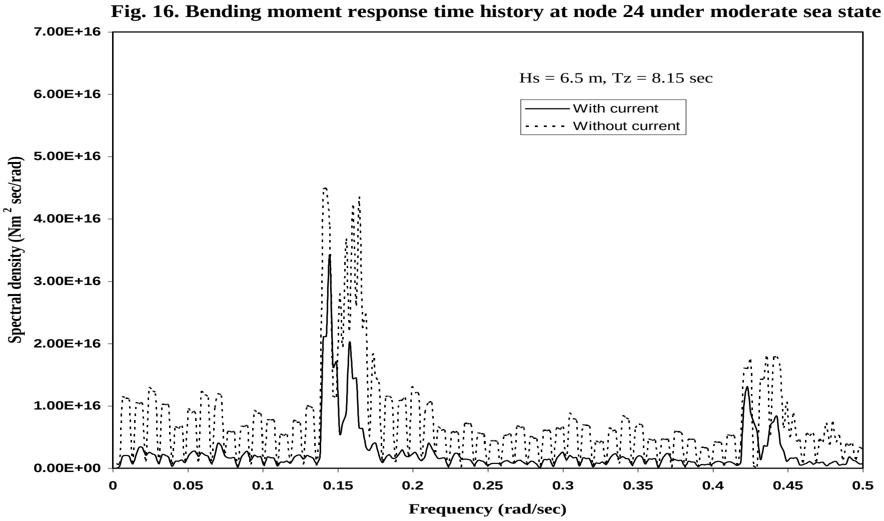

Fig. 17. PSD of bending moment response at node 24 under moderate sea state

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BIOGRAPHY

Moonis Zaheer received his B.Sc (Engg.) degree in Civil Engineering from Aligarh Muslim University, Aligarh, India, in 1994, the Master's degree in Building Engineering from Aligarh Muslim University, Aligarh, India, in 1996 and obtained his Ph.D.degree in Structures from Jamia Millia Islamia, New Delhi, India, in the year 2009. He has guided one Ph.D and published more than 20 papers in International Journals and Conferences. At present he is Assistant Professor in the College of Architecture and Planning, University of Dammam, Kingdom of Saudi Arabia. His teaching and research areas include structural design and structural dynamics.

Nazrul Islam received his B.Sc (Engg.) degree in Civil Engineering from Aligarh Muslim University, Aligarh, India, in 1984, the M.E.degree in Structures from University of Roorkee, Roorkee, India in 1990 and obtained his Ph.D.degree in Structure from Indian Institute of Technology, Delhi, India in the year 1998. He has guided four Ph.D" s and published more than 60 papers in International Journals and Conferences. At present he is a Professor in Civil Engineering Department, Islamic University, Madina Munawwarah, Kingdom of Saudi Arabia. His teaching and research areas include design of offshore structures and structural dynamics.

Moazzam Aslam received his degree in Civil Engineering, B.E (Civil) in the year 1994 and M.Tech (Structures) in the year 2004, from Aligarh Muslim University, Aligarh, India. He has submitted his Ph.D in Structures to Jamia Millia Islamia University, New Delhi, India in Nov. 2013. He has published three papers in International Journals and Conferences. He has served more than twenty years services in the department of telecommunications in construction field and at present he is working as an Executive officer in BSNL Civil Wing. His research areas include design of offshore structures and structural dynamics.