**Joint Earthquake and Wave Action on the Monopile Wind Turbine Foundation:
An Experimental Study**

by

Xiang Yuan Zheng*
Division of Ocean Science and Technology
Shenzhen Graduate School, Tsinghua University
Tsinghua Campus, University Town, Shenzhen, China, 518055

Hongbin Li & Weidong Rong
School of Naval Architecture
Dalian University of Technology
Ling Gong Road 2, Dalian, China, 116024

Wei Li
POWERCHINA Huadong Engineering Corporation,
Offshore Wind Power R&D Center of POWERCHINA Huadong
Hangzhou, China, 310014

*Corresponding Author:
Email: zheng.xiangyuan@sz.tsinghua.edu.cn
Tel: +86 (0755) 2603-6292

Key Words: Joint Action, Earthquake, Morison force,
Dynamic amplification factor, Monopile foundation

July 2015

(Revised)

ABSTRACT

Offshore wind turbines in regions of active seismicity are under a potential threat caused by the joint earthquake and ocean wave action. Taking NREL-5MW monopile wind turbine as the prototype, this study is devoted to probe the joint action of strong earthquakes and moderate sea conditions. A series of shake table model tests of scale 30:1 have been carried out by the contemporary world unique facility (Earthquake, Wave and Current Joint Simulation System) to investigate the structural response in dry flume, low and high calm water levels, with and without regular or random waves. Particular interest has been in the nacelle acceleration subjected to strong ground motions as well as to random waves. The experimental results of nacelle peak accelerations and corresponding dynamic amplification factors show that the joint earthquake and wave action is important for the proper evaluation of structural response. Ignoring the effect of wave action in seismic analysis will lead to underestimation of structural response, especially when the monopile foundation is dynamically sensitive. The coupling of earthquake and wave actions is sourced not only from the relative velocity of vibration but also from the initial condition induced by waves. It has been also derived that the peak acceleration excited by a moderate sea condition is comparable to that by a moderate earthquake.

1 INTRODUCTION

For the design of offshore wind turbine structures, the two most important environmental loads are usually aerodynamic wind load and ocean wave load. However, earthquake load will dominate the design in areas of active seismicity, like the Eastern coast of China and the Western coast of USA, for instance. In these areas, the earthquake epicenter can be located in the continental shelf extending from the costal land to offshore undersea, thereby threatening the safety of offshore wind turbines.

Figure 1 shows the design seismic intensities of three rapidly booming wind farm regions in China, i.e., Bohai Bay (denoted "1" in red font in the Figure), East China Sea (near Shanghai, "2"), and western bank of Taiwan Straits ("3"). It can be seen from Table 1 that the level of intensities in these regions ranges from VI to VIII, corresponding to **peak ground accelerations (PGA)** of 0.09-0.38g in the Seismic Ground Motion Parameters Zonation Map [1]. It is worthwhile to note that the circular monopile is one of the main foundations for wind turbines being employed in these regions.

Table 1 PGA versus Seismic Intensity^[1]

PGA	0.04-0.09g	0.09-0.19g	0.19-0.38g	0.38-0.75g	>0.75g
Intensity	VI	VII	VIII	IX	X

In design codes and standards, e.g., DNV-OS-J101 [2], the earthquake load is often considered using the popular response spectral analysis of displacement, velocity or acceleration. Occasionally, time simulation analysis of structural response to seismic excitation also finds its application in the offshore wind turbine design. However, the effective but expensive shake table experimental studies of the structural performance of offshore wind turbines were rarely carried out. So far there have been very limited research works [3, 4] addressing the seismic behavior of wind turbines using shake table tests. Nevertheless, these works in reality did not have physical sea waves simulated in laboratory. The study by Li et al. [3] utilized a wind-wave equivalent ground motion as the excitation input to the shake table which as a matter of fact could not reflect the real

wave-wind-earthquake joint action on the wind turbine structure. Till the end of 2012 no laboratory was able to simulate earthquake and ocean waves in a single facility.

On the other hand of design consideration, load combination of seismic load with wave or/and wind load has not been adopted by the industrial society [2] yet, as the coincidence of a strong earthquake event and a severe wave/wind events is extremely rare. Yet damage due to the joint action of quake and wave/wind has neither been reported around the world. However the collected data for the most recent five decades in Asia showed that several strong earthquakes have occurred in winter or spring season in which the occurrence probability of moderate or even severe sea states was high. For example, on December 26 of 2006, doublet quakes of magnitude $M=7$ attacked Pingtung Taiwan just within a couple of minutes [5] and in this event the earthquake wave propagated across the Taiwan Straits and hit the western bank of the straits ("3" in Fig. 1). As another example, on November 8 of 2011 a quake of $M=6.9$ occurred in East China Sea and a number of cities in region "2" felt this quake [6]. Thus, it should be recognized that a significant earthquake can occur with a moderate sea condition, simultaneously. Also, the known 2011 off the Pacific coast of Tohoku Earthquake [7, 8, 9] has reminded us of the possible strike on offshore structures not only by the main shock induced tsunami in the form of giant waves but by aftershocks. Hence, it would be interesting to investigate the joint earthquake-wave action on the structural response of offshore wind turbines, particularly in a laboratory environment.

This research objective would not be fulfilled without the Earthquake, Wave and Current Joint Simulation System (EWCJSS in short hereafter) that was accomplished by Dalian University of Technology in December 2012. This paper is devoted to presenting the results and finding from the experiments conducted there in April 2014. The NREL-5MW monopile wind turbine [10] was used as the prototype for a series of 30:1 scale model tests.

2. EWCJSS AND EXPERIMENT SETUP

EWCJSS is still unique in the world by the end of 2014 as it possesses the capability of simulating waves, current and earthquake in the same wave flume. Fig. 2 shows a picture of this flume taken from the monitoring room in the second floor of the flume building.

The flume dimensions are 21.6 x 5 x 1m, whereas the maximum water depth for model tests can reach 0.8m. The elliptical shake table is located in the center of the flume. The major and transverse diameters are 4m and 3m. The maximum dead load that the shake table can carry is 10,000kg. The table shakes in three degree of freedoms: horizontal in long axis, vertical, and pitch in the same plane. The maximum horizontal displacement and acceleration are respectively 75mm and 1.0g. The frequency range of all motions varies from 0 to 50Hz. For the wave generation system that is installed at the end of the flume, the maximum wave height it can make is 0.33m, with wave period ranging from 0.5s to 4s. It should be noticed that the shake table and the wave maker work independently. In other words, ideally there exists negligible oscillation of water incurred by the motion of shake table.

In our study, only the horizontal ground motion is simulated by the shake table as our focus is placed on the joint earthquake-wave action in the collinear direction. The vertical ground acceleration is more associated with buckling and so is neglected. Compared with horizontal and vertical motions, the pitch motion of ground is of less significance for the dynamics of a monopile foundation.

For a tall structure like offshore wind turbine that has a heavy end mass on the top, it is necessary to adopt the Froude (Gravity) similitude law in the model tests to account for the gravitational effect. Meanwhile, the model will have to satisfy the prerequisite elastic and geometric similarities. In this case, in terms of scale $\lambda=30:1$ (Full/Model), the original overall weight of nacelle and rotor will be scaled down from 350ton to 389kg and the weight of tower is scaled down from 487ton to 541kg, including the self weight of tower as well as the blast weight. Note that the tower is in a tapering shape [10]. In our study we added a transition part of 0.833m to the model, or 25m to the bottom of prototype, to represent the submerged wet portion of tower in water. This transition portion is of uniform diameter (Fig. 3). Therefore, the 1st tower fore-aft free vibration frequency will decrease, changing from 0.324Hz [10] to 0.248Hz by ANSYS finite element analysis [11]. Since the study assumes the bottom of the tower to be a fixed end and does not consider the actual piling length beneath the seabed and soil behavior, this calculated frequency for a cantilever beam could be slightly higher than the frequency of actual wind turbine structures installed in the offshore site.

Figure 3 shows the geometry of the scaled tower including thickness. Due to the difficulty of fabricating thin steel plate in rolling and welding processes, a uniform thickness of 1mm was used, while in the prototype [10] the thickness varies from 24.7mm in the top to 35.1mm in the bottom.

Figure 4 illustrates the tower having been erected onto the shake table, but temporarily without ballast. Figure 5 is a picture taken during nacelle installation. Because the hydrodynamic force is heavily dependent on the tower diameter, it can be observed from the picture that we purposely placed the lead ballast to the body of tower at locations above the maximum water level. Therefore, the hydrodynamic force only acts on the submerged portion that is shorter than 0.833m. However, it is worth mentioning that ballast weight accounts for more than 95% of tower's total weight (see Fig. 4).

Table 2 gives the comparison of the characteristics of the prototype and the underlying model, particularly on the 1st fore-aft vibration frequency. This frequency of model, 1.707Hz, is obtained from the decay test to be discussed later on in Section 6.1. It is very close to the desired frequency $0.324(\lambda)^{1/2}=1.774\text{Hz}$ after scaling down from the prototype [10], in that the length of tower is extended, thickness of 1mm is used, the actual nacelle and rotor weight in model is 372kg and, the actual ballast weight is 489kg, and that the integration of ballast lead blocks with tower may somehow increase the section modulus and stiffness.

Table 2 Comparison of geometry and 1st fore-aft vibration frequency
of NREL-5MW prototype and model ($\lambda=30:1$)

	Length (m)	Thickness (mm)	1st fore-aft frequency f_1 (Hz)	Scaled Frequency $\lambda^{1/2}f_1$ (Hz)
Prototype	87.60	24.7 - 35.1	0.324	1.774
Model	3.753	1	1.707	-

The total weight of model is 877kg, significantly lower than the approximate buckling load of 6000kg. Therefore, albeit the thickness of tower model is small, global buckling will not happen even under a vertical PGA of 0.7g which corresponds to intensity IX in Table 1.

3. EXCITATIONS AND LOAD CASES

In order to dig out the wave effects on the structural response excited simultaneously by the earthquake ground motion, shake table tests for four main scenarios: no water (dry flume), calm water, low water level (LW) of 0.5m, high water level (HW) of 0.7m have been carried out.

The low water level scenario is used to model the maximum water depth of 15m for wind farms in Bohai Bay (region “1” in Fig. 1), while the high water level models the maximum water depth of 21m in the western bank of Taiwan Strait (region “3” in Fig. 1). Moderate wave conditions for LW and HW are specified in Table 3. Both regular waves and random waves with short and long periods are taken into account. Conditions for HW are slightly more severe. JONSWAP spectral formation [2] is employed in the random wave generation by the flap of wave maker.

Table 3 Wave conditions for low and high water levels (LW & HW)

	Regular wave 1 H=0.1m T=0.75s	Regular wave 2 H=0.18m T=1s	Regular wave 3 H=0.32m T=2.1s	Random wave 1 Hs=0.12m Tp=1s $\gamma=3.3$	Random wave 2 Hs=0.18m Tp=2.1s $\gamma=3.3$
LW	X	X		X	
HW		X	X	X	X

So far, the subsea strong earthquake monitoring in offshore areas has not been carried out by most countries. The records of subsea strong earthquakes are not only rare but also confidential. Thence, for ground motion input to shake table, we chose the WE component of Chichi wave (Taiwan Chichi earthquake, M=7.3, 1999), the 230° component of El Centro wave (California Imperial Valley earthquake, M=6.4, 1979) and the NS component of Tangshan wave (Tangshan earthquake, M=7.6, 1976). These three typical records, as depicted in Fig. 6a, have been popularly used by the industrial and academia societies, although their characteristics could differ from the actual subsea strong ground motions. Note that the epicenter of Tangshan earthquake was geographically close to the Bohai Bay such that NS

Tianjin wave is very suitable for the seismic analysis of wind turbine structures there, while the Chichi seismic record is very meaningful for wind turbines operating in the offshore areas of southeast China. El Centro wave is of special interest as it is very appropriate for structures of long periods ranging from 1.5s to 5.5s. Note that the natural period ($T_n=3.1s$) of NREL-5MW prototype falls into this range. Choosing these three wave records has, to some extent, considered the field effect arising from soil amplifications.

Table 4 gives the peak ground acceleration (PGA) and the half cycle period of PGA (defined as the period between PGA and its neighboring significant crest or trough), the spectral peak frequency (Fig. 6b) and the average zero-crossing period of these three seismic records. It can be seen that among them the El Centro record contains the strongest magnitude of 428Gal (cm/s^2), belonging to the intensity level of IX in Table 1. Also, its peak frequency of 0.32Hz is pretty close to the resonant frequency of the NREL-5MW prototype wind turbine (see Table 2).

Table 4 Characteristics of chosen ground motion records

Seismic waves	PGA (Gal)	Intensity level	Half cycle period for PGA (s)	Peak freq. (Hz)	Average crossing period (s)
Chichi	75.2	VI	0.29	1.15	0.16
El Centro	428	IX	0.35	0.32	0.25
Tianjin	145.8	VII	0.58	1.12	0.22

Also, as observed from the spectra of these three seismic ground motions in Fig. 6b, El Centro wave contains the richest components nearby the 1st fore-aft frequency of prototype NREL-5MW wind turbine structure, while Tianjin wave is rich at high-frequency components. Thereby, the dynamic behavior of NREL-5MW monopile wind turbine subjected to the three excitations should be different. The structural resonance induced by El Centro wave will be most remarkable.

In addition to the abovementioned seismic simulations, decay tests and white noise tests are carried out, as illustrated in Table 5.

Table 5 Excitations for the four scenarios

	Decay test	White Noise	Ocean waves	Chichi wave	El Centro wave	Tianjin wave
#1 No water	X	X		X	X	X
#2 Calm water	X	X		X	X	X
#3 LW: 0.5m	X	X	X	X		X
#4 HW: 0.7m	X	X	X	X	X	X

In total, as many as 52 load cases combining Table 3 and Table 5 have been simulated in EWCJSS model tests. They include various levels of loading of ground motions. For instance, for the scenario of no water, the Tianjin wave was loaded at three levels: 34.3%, 68.587%, 100%, in order to capture the structural response under earthquake intensities of VI, VII and VIII. Restricted by the length of this paper, only results for significant load cases will be given.

4. DATA MEASUREMENT

Accelerations and displacements of tower vibration at various height levels were measured. Also, measured data of strains at critical locations was recorded.

Figure 7 shows the placement of 7 accelerometers, in which accelerometer in channel #48 measured the acceleration of nacelle fore-aft vibration in the direction of shake table motion and waves, #49 measured the nacelle acceleration in the transverse side-to-side direction, #35 measured the acceleration at the welding connection between tapering tower and the submerged transition part, #37 is used to calibrate the ground motion input of the three seismic waves. In the experiments, the three seismic waves in Fig. 6a were more or less modified and then fed to the servo of shake table. Accelerometers #35 and #37 are waterproof. All acceleration data were collected at resolution of 256Hz.

Displacements were measured using the non-contact optical technology of Imetrum Video Gauge, of resolution up to 37.5Hz. All displacements in major and transverse directions were recorded at four heights A, B, C, D, as shown in Fig. 7.

Also, strain gauges were placed around the sections of tower at A, C, D, E to measure the time history of strain variations. For sections A, C, D, four strain gauges in the circumferential direction were used, while in section E where was the welding and stiffening tip of tower bottom, 12 strain gauges were used. Figure 8 gives the **planar** arrangement of strain gauges. Waterproof strain gauges were fixed to the surface of tower by special gluing. The strain measurement, at resolution of 125Hz, was based on the Fiber-Bragg-Grating sensing technology and the optical transmission of signal from sensors to receivers.

Meanwhile, we employed two cameras of 60fps for the video recording of all 52 model tests.

5. GOVERNING EQUATION FOR VIBRATION

Excited by earthquake only, the motion of wind turbine structure relative to the rigid ground can be written in the following familiar governing equation for a multi-degree-of-freedom system (MDOF) [12]:

$$[M]\{\ddot{x}\} + [C]\{\dot{x}\} + [K]\{x\} = -[M]\{\ddot{x}_g\} \quad (1)$$

where $\{x\}$ is displacement vector along the tower, $\{\ddot{x}_g\}$ is ground acceleration vector for all degree of freedoms. In the elastic stage, matrices $[M]$, $[K]$, $[C]$ are considered time invariant. With the joint action of waves, the external forcing functions need to take the relative motion between ground motion and fluids into account. Penzien [13] gave the corresponding modified governing equation for a fixed offshore structure.

$$[M]\{\ddot{x}\} + [C]\{\dot{x}\} + [K]\{x\} = \rho(C_m - 1)[V]\{\ddot{x}_w - \ddot{x}_g\} + \rho[V]\{\ddot{x}_w\} + \rho C_d[A]\{\dot{x}_w - \dot{x}_g\} \left| \dot{x}_w - \dot{x}_g \right| \quad (2)$$

where the three terms on the right-hand-side are respectively earthquake force, Morison inertial force, nonlinear Morison drag force; ρ is water density, $\{\ddot{x}_w\}$ and $\{\dot{x}_w\}$ are water particle acceleration and velocity vectors; C_m and C_d are inertia and drag coefficients; matrices $[V]$ and $[A]$ were regarded as the effective drag area and volume [13]. However, we

understand that when we adopt the discrete MDOF system like Eq. (1) and Eq. (2), for a monopile wind turbine, $[A]$ is associated with the area per unit length for the submerged transition part, i.e. $[A] = 0.5D$, with D being the diameter, and $[V] = \pi/4D^2$ as the volume per unit length of the bottom transition part. One thing implicit but not considered by [13] is that the water surface varies during the short earthquake action, so that kinematics and the resultant Morison forces above the calm water level need to be estimated using an approximate stretching method such as Wheeler [14] or constant stretching. Taking the water surface as a calm water level usually underestimates the total wave load because the underlying water depth is shallow and waves tend to be nonlinear. Also, the free surface introduces a higher-order wave load on the structure which may boost the structural dynamic response at **multiple frequencies like** 2ω [15]. Nonetheless, this free surface issue can be numerically resolved with ease by the discrete time integration method like Newmark- β algorithm [16] while calculating the total wave force at each time step.

Equation (2) actually was meant for rigid structures and it ignored the relative velocity between structural motion and fluid kinematics. Now, denote the total displacement of tower as $\{x_t\} = \{x_g\} + \{x\}$, then in the present research the governing equation should be rewritten as:

$$[M]\{\ddot{x}_t\} + [C]\{\dot{x}_t\} + [K]\{x_t\} = [C]\{\dot{x}_g\} + [K]\{x_g\} + 0.25\pi D^2 \rho \{\ddot{x}_w\} + 0.25\pi D^2 \rho (C_m - 1) \{\ddot{x}_w - \ddot{x}_t\} + 0.5\rho C_d D \{\dot{x}_w - \dot{x}_t\} |\dot{x}_w - \dot{x}_t| \quad (3)$$

after some manipulations, we have

$$[M]\{\ddot{x}\} + [C]\{\dot{x}\} + [K]\{x\} = -[M]\{\ddot{x}_g\} + 0.25\pi D^2 \rho \{\ddot{x}_w\} + 0.25\pi D^2 \rho (C_m - 1) \{\ddot{x}_w - \ddot{x}_g - \ddot{x}\} + 0.5\rho C_d D \{\dot{x}_w - \dot{x}_g - \dot{x}\} \left| \dot{x}_w - \dot{x}_g - \dot{x} \right| \quad (4)$$

Some wind farms are in estuary and current speed U_c needs to be included in the forcing function, then Eq. (4) is further rewritten as follows.

$$[M]\{\ddot{x}\} + [C]\{\dot{x}\} + [K]\{x\} = -[M]\{\ddot{x}_g\} + 0.25\pi D^2 \rho \{\ddot{x}_w\} + 0.25\pi D^2 \rho (C_m - 1) \{\ddot{x}_w - \ddot{x}_g - \ddot{x}\} + 0.5\rho C_d D \{\dot{x}_w - \dot{x}_g - \dot{x} + U_c\} \left| \dot{x}_w - \dot{x}_g - \dot{x} + U_c \right| \quad (5)$$

If the submerged foundation is in non-uniform shape, the constant diameter D in preceding equations should be replaced by vector $\{D\}$ to reflect the actual configuration of foundation and the corresponding Morison forces at various heights.

Compare Eq. (4) with Eq. (2), we can find that Eq. (2) ignores not only velocity $\dot{x}$ in the drag term but also $\ddot{x}$ in the inertia term. In both inertia and drag terms of Eqs. (3-5), the ground acceleration needs to be integrated once to obtain the base velocity $\dot{x}_g$. It is worthwhile to point out that because Morison drag term in Eq. (4) is a nonlinear forcing function involving three velocities, the response under the joint action of earthquake and waves does not equal the summation of response separately under each action. Earthquake action and ocean wave action are coupled. If structure's relative velocity $\dot{x}$ is much smaller than the ground velocity and fluid kinematics, which is the vibration case of a rigid jacket platform in shallow water, as an example, Eq. (2) hence holds its validity.

The relative motion in Eqs. (2-4) will result in a slight increase in the hydrodynamic damping. Therefore, the overall damping for the four scenarios we are considering (no water, calm water, LW & HW waves) could somehow differ from each other, especially when the structure is flexible and water is deep. In general, the overall damping consists of the following terms.

$$\xi = \xi_{steel} + \xi_{soil} + \xi_{aerodynamic} + \xi_{radiation} + \xi_{viscous,hydro} + \xi_{apparatus} \quad (6)$$

For the vertical cantilever tower in this study that models the non-production state of rotor, apparently the overall damping consists of steel and hydrodynamic damping only.

The above discussion is for non-breaking waves. We also have to emphasize that the fluid field and associated fluid kinematics are assumed to be unaffected by the moving boundary condition (in-situ seafloor or shake table in this study) and neither by the moving body of slender monopiles. In other words, we tackle the situation where the monopile foundation is subjected to far-field strong seafloor motion and its joint action with independent ocean waves. Otherwise, the problem will become more complicated. This reiterates the independence of earthquake excitation and water wave propagation in the flume. In this sense, Eq. (4) is not appropriate for the vibration of a large-volume offshore structure because of diffraction and radiation issues.

6. EXPERIMENTAL RESULTS AND DISCUSSION

6.1 *Decay tests*

Decay tests are very useful to identify the structure's free vibration characteristics under various circumstances. We now compare the hammering decay tests for four circumstances in the model tests:

- a) Initial stage, no water (dry flume);
- b) Calm water, low water level;
- c) Calm water, high water level;

d) Eventual damaged stage, no water. Before this stage, the structure has just experienced 100% El Centro seismic loading and a local compressive yielding at the welding connection was observed due to that severe excitation. Then this decay test was implemented afterwards.

Among all 52 model tests, these four tests are labeled #1, #10, #25 and #52 in the sequential order.

For illustration purpose, only nacelle acceleration decay curves of a) and d) are shown in Fig. 9, while the 1st-mode fore-aft natural frequencies obtained from Fourier spectrum and the damping ratios derived from exponential decay method for all 4 cases are shown in Table 6.

As expected, the free vibration frequencies for the first three cases a), b), c) are almost identical because the structure is still in the elastic undamaged state and because the water depth is much smaller than the height of tower, hydrodynamic damping in calm water can hardly affect the overall response of structure in the absence of waves. Low and high water levels almost make no difference. After the tower structure suffered damage due to El Centro excitation, as in case d), its stiffness was reduced and therefore the natural frequency decreased from 1.707Hz to 1.508Hz.

Table 6 Fore-aft natural frequencies f_n and damping ratios for Cases a) - d)

	a)	b)	c)	d)
Natural frequency (Hz)	1.707	1.695	1.695	1.508
Damping ratio (%)	1.32	1.30	1.35	1.36

Also, Table 6 shows that all damping ratios in Table 1 are in the range from 1.3% to 1.4%. However, Devriendt et al. [17] suggested that the overall damping without aerodynamic contribution in Eq. (5) very likely falls into [0.8%, 1.2%]. Their finding was based on an extensive analysis of the site collected data for wind turbines in Belgian North Sea. In our model tests, since the tower's surface was covered with a number of lead blocks for ballast purpose, the frictions between tower and these metal blocks may have contributed to the increase of damping.

For other types of offshore structures in deepwater, like a guyed tower discussed in [18] and a tall jacket platform in [19], it has been found from the numerical calculations that the hydrodynamic damping are significant. As a result, in calm and mild sea waves, the seismic response of structures could be reduced, luckily in a favorable manner. However, this is very different from the situation of monopile foundations in shallow water.

6.2 Response to earthquake alone

The acceleration response of nacelle is important because it is not only crucial to the safety of contained power generating system, but also associated with shear force acting on the tower. According to the Elasticity-Froude similitude law, the accelerations of model test and prototype are identical. Before the shake table starts to move in accordance with a given seismic input, an artificial zero ground acceleration of length more than 10 times natural period is simulated in advance, to largely damp off the initial transience. Figure 10(a, b, c) present model's measured nacelle acceleration in dry flume, respectively subjected to Chichi, El Centro and Tianjin earthquake waves, with different levels of loading factor.

Seen here and aforementioned, the resonance by El Centro is most pronounced. The response exhibits beautiful ring-shaped decay, while such tendency is not evident in the response driven by Tianjin wave. Table 7 gives PGA, PSA (Peak Structural Acceleration) and

dynamic amplification factor (DAF=PSA/PGA) of these three responses. Different signs of PGA are respectively for the peak and the trough in time history. DAF for El Centro driven response is largest, reaching 1.15. DAF for Tianjin case is only 0.39, while DAF for Chichi case is 0.80, between the two cases. The remarkable difference in DAF can be well explained by the spectral characteristics shown in Fig. 6b. In addition, we understand that all DAFs are large, not only due to resonance but also the small damping as shown in Table 6.

**Table 7 Peak ground acceleration (PGA), Peak structural acceleration (PSA)
and Dynamic amplification factor (DAF)**

Load cases	PGA (m/s ²)	PSA (m/s ²)	DAF	DAF
	+ / -	+ / -	+ / -	
Chichi	2.45 / -2.25	1.76 / -1.81	0.72 / 0.80	0.74
El Centro	1.86 / -1.17	1.28 / -1.35	0.69 / 1.15	0.72
Tianjin	2.82 / -2.74	0.86 / -1.08	0.30 / 0.39	0.38
RW1	0 / 0	0.050 / -0.065 $\sigma = 0.009$	- -	- -
RW2	0 / 0	0.069 / -0.642 $\sigma = 0.006$	- -	- -
Chichi + RW2	2.76 / -2.33 Averaged	1.69 / -1.90 Averaged	0.61 / 0.82 Averaged	Aver.=0.69 Max=0.73
El Centro + RW2	1.47 / -1.25 Averaged	1.20 / -1.15 Averaged	0.82 / 0.92 Averaged	Aver.=0.82 Max=0.98
Tianjin + RW2	2.74 / -2.60 Averaged	1.04 / -1.04 Averaged	0.38 / 0.40 Averaged	Aver.=0.39 Max=0.40

* DAF is defined as the amplification by PSA/PGA with same signs, while |DAF| in this table adopts the traditional definition in seismic response spectrum, i.e., $|PSA|_{\max}/|PGA|_{\max}$, regardless of signs.

An interesting observation during almost all 52 model tests is that the vibration track of nacelle did not strictly follow the fore-aft line (the main axis of shake table) but appeared in somehow elliptical or so-called whirling tracks with simultaneous side-to-side vibration. Figure 10(d) gives the nacelle side-to-side acceleration under the excitation of El Centro seismic wave. The 1st side-to-side vibration frequency is 1.688Hz that can be easily identified from plotting its spectrum. It is very close to the fore-aft natural frequency f_n in Table 6. Also, it can be seen from Fig. (10b) and Fig. (10d) that the side-to-side acceleration is slightly smaller than the motion along the main fore-aft direction. Such transverse motion cannot be avoided, because the imperfection during fabrication has caused unevenness, although subtle, in every section of the tower and therefore in the whole structure. Asymmetries in not only stiffness distribution but also mass distribution, or perhaps as well as the eccentricity of nacelle's weight centre, all contribute to the coupling between fore-aft and side-to-side motions. Such coupling causes the stress at critical locations to increase and is potentially harmful to the safety of structures. As an actual wind turbine tower carries ladders and other apparatus, and the misalignment of tower and nacelle may happen during offshore installation, it is believed that the actual structural vibration track in an earthquake event is whirling too.

6.3 Response to waves alone (High water level)

Figure 11(a, b) show measured nacelle fore-aft acceleration records of model, excited by Random Wave 1 and 2 in high water level (HW). Considering that wave conditions are moderate and earthquake is the dominant excitation in our study, compared with seismic responses in the preceding section, the acceleration driven by sea waves are much smaller. However, the magnitudes are not negligible. For these two circumstances, the maximum accelerations reach 0.065 and 0.069 m/s^2 .

Particularly, if the occurrence of a moderate earthquake (like Chichi ground motion we use here: 75.2Gal, Intensity VI) coincides with a moderate wave condition, which is an event of higher occurrence probability, it can be derived from the DAF in Table 7 that the earthquake induced maximum nacelle acceleration is $0.752 \cdot |\text{DAF}| = 0.557 \text{ m/s}^2$, where $|\text{DAF}| = 0.74$. Hence, the ratio of sea wave induced acceleration to that by earthquake is

0.069:0.557=1:8. This is the situation that might occur in wind farms located in offshore Taiwan Strait. For another example, if an earthquake similar to the Tangshan event is simulated but of 50% magnitude (Intensity VI), it can be derived from Table 2 & 6 that the maximum nacelle acceleration now reaches $1.458/2*|DAF|=0.278 \text{ m/s}^2$ with $|DAF|=0.38$ in Table 7. Thus, the ratio of sea wave induced acceleration to that by earthquake is 1:4. This would be the situation where wind farms in Bohai Bay will have to face during their lifetime. Both examples imply that the hydrodynamic wave load is not always negligible when considered together with the earthquake load.

Nonetheless, it still needs to be recognized in the viewpoint of structural dynamics that because the earthquake excitation usually contains rich components whose frequencies are pretty close to the structural natural frequency while the dominant frequency of ocean waves is much lower, in many scenarios the wave load is secondary to the earthquake load.

From Table 7, in addition, it can be noticed that the ratio of model's PSA to its standard deviation σ is larger in RW2 than in RW1. This is because RW2's wave period is closer to T_n and its wave height is larger. Morison drag force in RW2 therefore accounts for a higher weight in the total wave load, which causes the structural response to gain stronger non-Gaussianities.

6.4 Response to earthquake and waves

It should be noted that the structural response to earthquake excitation is transient, while the response to water waves possesses stationary and ergodic characteristics. As the actual ground motion seldom lasts longer than 40 seconds, when scaled down by $(\lambda)^{1/2}=5.48$ in our model tests, the shake table input motion and the forced vibration only lasts for a few seconds. For EWCJSS model tests, we do not know whether the occurrence of PGA (positive and negative maxima) will coincide with the largest wave crest or trough acting on the column bottom. Assuming coincidence will lead to conservative results. On the other hand, for the purpose of investigating joint action, we designed many episodes, each episode corresponding to a short seismic action riding on the long water wave action. During the process of wave making that lasts for hundreds of seconds, shake table motion repeats, but

with adequate time ($>10T_n$) allocated to each episode in order to let the transient earthquake response die out.

Figure 12(a, b, c) show measured nacelle acceleration under the joint action by Chichi earthquake and Random Wave 2, El Centro wave and RW2, Tianjin wave and RW2, respectively. As many as 9, 12, 14 joint action episodes occur in these three model tests to purposely simulate the interaction of loads. The respective averaged PGA, PSA, DAF and $|DAF|$ are shown in Table 7. One has to realize that at the end of each joint action episode, though there is no shake table excitation, the structure continues to vibrate because of water wave action. Therefore, the coupling of these two actions not only comes from the relative velocity in Eq. (4) but also should be attributed to the random initial conditions caused by wave action, which is the phenomenon in the actual world.

The peak acceleration data and DAF ratios in Table 7 show that under the joint action of a strong earthquake and a moderate wave condition, the general trend is that DAF and $|DAF|$ are larger than those under earthquake action alone. This is particularly evident for structural response under the excitation of EL Centro wave and Random Wave 2. It can be seen that the traditionally defined $|DAF|$ for 12 joint action episodes is 0.82 on average, which is 13% higher than earthquake alone, $|DAF|=0.72$. For Chichi earthquake and RW2, though averaged and maximum $|DAF|$ are both smaller than that by earthquake alone, the averaged DAF is actually higher. This can be also supported by the amplitude spectra depicted in Fig. 13. At 1st- and 2nd-mode frequencies, the amplitudes by Chichi and RW2 are higher than Chichi input alone. Thus, ignoring the effect of wave action in seismic analysis as long been adopted in class rules will underestimate the structural response.

Also, from the amplitude spectra in Fig. 13, we know that 2nd-mode frequency is 15.21Hz. Namely the corresponding frequency when scaled back to the full scale is $15.21/(\lambda)^{1/2}=2.777\text{Hz}$ which is very close to the value of 2.936Hz of NREL-5MW prototype [10]. This again manifests the high accuracy of model design and setup in this study.

7. CONCLUDING REMARKS

This research work is focused on the joint action of a strong earthquake and a moderate sea condition on a typical monopile wind turbine foundation, for addressing the potential

challenge to wind farms in seismically active regions. Bohai Bay, one of the three underlying regions treated in this study, in history has experienced $M \geq 7$ earthquakes for over four times [20]. The other two offshore regions also have had frequent quake events reported since the 1950's. For the wind farms currently being constructed or planned in these regions, the consideration of structural safety for popularly used monopile foundations has driven the authors to find out the mechanism of structural vibration under environmental loads as well as their sophisticated interaction.

Extensive model tests of scale ratio 30:1 have been carried out in the world unique EWCJSS facility to investigate the structural response of NREL-5MW prototype's nacelle. These include decay tests, white noise tests, and tests by various earthquake excitations in dry flume, low and high water levels, with and without regular and random waves. Due to the length of this paper, not all measurements were presented.

Peak acceleration of nacelle exceeding 180Gal has been observed (Table 7) during a number of tests as the ground motion input by shake table is strong. Also, the coupling between fore-aft and side-to-side nacelle motions has been reported. Plots of measured acceleration time histories and their amplitude spectra, as well as analysis of dynamic amplification DAF and $|DAF|$, have shown that the joint earthquake and wave action is important for properly evaluating the structural response. Ignoring the effect of wave action in seismic analysis will underestimate structural response. Using the experimental data, it has been derived that the acceleration response by a moderate sea condition is comparable to that by a moderate earthquake, e.g, 1:4. Thus, assuming the prevailing dominance of earthquake action over water wave action can be risky. Further, it has been pointed out that the initial condition of earthquake driven structural vibration is actually subjected to ocean waves.

The underlying water depths treated in this study are shallow (less than 21m), pertaining to most wind farms worldwide. Hydrodynamic damping's contribution to the overall damping is small. For a more flexible foundation (for instance $T_n = 4 - 5s$) in deeper water depth (25-35m), the super-harmonic 2ω and 3ω wave load may boost the magnitude of structural response. As revealed by Penzien et al. [21], this in turn will intensify the coupling with earthquake action as shown in Eq. (4), therefore affecting the hydrodynamic force and damping, as well as the resultant total response. Further numerical analysis using finite

element calculation must be undertaken to quantify the relative weights of seismic and hydrodynamic loading, with soil-structure interaction included.

ACKNOWLEDGEMENT

The research funds from National Science Foundation of China (Grant No. 51379035 and No. 41206075) and from State Key Laboratory of Coastal and Offshore Engineering of Dalian University of Technology (Grant No. SL2012-3) to support this study are gratefully acknowledged. The authors are also grateful to Professor JW Dai from Institute of Engineering Mechanics, China Seismological Bureau for offering the seismic records used in this study.

- [1] Committee for Standardization Management China. Seismic ground motion parameters zonation map of China (recommended). GB18306. 2012.
- [2] Det Norske Veritas. DNV-OS-J101. Design of offshore wind turbine structures. 2014.
- [3] Prowell I, Veletzos M, Elgamal A and Restrepo J. Shake table test of a 65 kW wind turbine and computational simulation. 14th World Conf. on Earthquake Eng., Beijing, China. 2008.
- [4] Li J, Zhang Z and Chen J. Experimental study on vibration control of offshore wind turbines using a ball vibration absorber. *Energy and Power Eng.* 2012. (4):153-157.
- [5] Liu CY, Liu JY, Chen WS, Li JZ, Xia YQ, and Cui XY. An integrated study of anomalies observed before four major earthquakes: 2004 Sumatra M9.3, 2006 Pingtung M7.0, 2007 Chuetsu Oki M6.8, and 2008 Wenchuan M8.0. *J of Asian Earth Sciences.* 2011. 41(4-5):401–409.
- [6] USGS, Significant Earthquakes of the World 2011. Earthquake hazard program. 2011. http://earthquake.usgs.gov/earthquakes/eqarchives/significant/sig_2011.php.
- [7] Koshimura S, Hayashi S and Gokon H. The impact of the 2011 Tohoku earthquake tsunami disaster and implications to the reconstruction. *Soils and Foundations.* 2014. 54(4):560–572.
- [8] Ichihara H, Hamano Y, Baba K and Kasaya T. Tsunami source of the 2011 Tohoku earthquake detected by an ocean-bottom magnetometer. *Earth and Planetary Science Letters.* 2013. (382):117–124.
- [9] Lin A, Ikuta R and Rao G. Tsunami run-up associated with co-seismic thrust slip produced by the 2011 M_w 9.0 off Pacific Coast of Tohoku earthquake. *Japan Earth and Planetary Science Letters*, 2012. (337–338):121-132.
- [10] Jonkman J, Butterfield S, Musial W and Scott G. Definition of a 5-MW reference wind turbine for offshore system development. Technical report. NREL NREL/TP-500–38060. 2009.
- [11] ANSYS, Inc. ANSYS standard user's manual. 2006.
- [12] Clough RW and Penzien J. *Dynamics of Structures.* McGraw-Hill. 1975.
- [13] Penzien J. Structural dynamics of fixed offshore structures. *Proc. 1st Int. Conf. on the Behavior of Offshore Structures (BOSS'76).* 1976. (1):581–592.

- [14] Wheeler JDE. Method for calculating forces produced by irregular waves. *J. Petroleum Tech.*. 1970. (249):359-367.
- [15] Liaw CY and Zheng XY. Inundation effect of wave forces on jack-up platforms. *Int. J. of Offshore & Polar Eng.* 2001. 11(2): 87-92.
- [16] Newmark NM. A method of computation for structural dynamics. *ASCE J. Eng. Mech.* 1959. 85(EM3):67-94.
- [17] Devriendt C, Jordaens PJ, Sitter GD and Guillaume P. Damping estimation of an offshore wind turbine on a monopile foundation. *IET Renew. Power Gener.* 2013. 7(4):401–412.
- [18] Brynjolfsson S and Leonard JW. Effects of currents on the stochastic response to earthquakes of multiple degree of freedom models of guyed offshore towers. *Eng. Structures.* 1988. 10(3):194–203.
- [19] Venkataramana K and Kawano K. Nonlinear dynamics of offshore structures under sea wave and earthquake forces. *Sadhana, Springer.* 1995. 20(2-4):501-512.
- [20] Wang J. Historical earthquakes and a tsunami in Bohai Sea. *Acta Seismologica Sinica.* 2007. 29(5):549-557.
- [21] Penzien J, Kaul MK and Berge B. Stochastic response of offshore towers to random sea waves and strong motion earthquakes. *Computers & Structures.* 1972. 2(5-6):733–756.

Figure Legends

- Fig. 1 Seismic intensity in China offshore regions
- Fig. 2 View of EWCJSS from monitoring room
- Fig. 3 The geometry of tower in model test (Unit: mm)
- Fig. 4 Tower erected without lead ballasts on surface
- Fig. 5 Installation of nacelle onto the ballasted model
- Fig. 6a Original strong seismic motion of WE Chichi, 230⁰ El Centro, NS Tianjin earthquake waves
- Fig. 6b **Unaveraged** amplitude spectra of original WE Chichi, 230⁰ El Centro, NS Tianjin earthquake waves
- Fig. 7 Placement of accelerometers
- Fig. 8 Placement of strain gauges (Left: section A, C, D; Right: section E)
- Fig. 9 Free vibration decay of nacelle acceleration in circumstance a) and d)
a) Initial: dry flume; d) After damage by El Centro wave: dry flume
- Fig. 10a Nacelle fore-aft Acceleration - excited by modified Chichi wave (300% loaded)
- Fig. 10b Nacelle fore-aft Acceleration - excited by modified El Centro wave (46.7% loaded)
- Fig. 10c Nacelle fore-aft Acceleration - excited by modified Tianjin wave (200% loaded)
- Fig. 10d Nacelle side-to-side Acceleration - excited by modified El Centro wave (46.7% loaded)
- Fig. 11a Nacelle fore-aft Acceleration - excited by Random wave 1 (HW)
- Fig. 11b Nacelle fore-aft Acceleration - excited by Random wave 2 (HW)
- Fig. 12a Nacelle fore-aft Acceleration - excited by modified Chichi seismic wave (300%) and Random wave 2
- Fig. 12b Nacelle fore-aft Acceleration - excited by modified El Centro seismic wave (46.7%) and Random wave 2
- Fig. 12c Nacelle fore-aft Acceleration - excited by modified Tianjin seismic wave (200%) and Random wave 2
- Fig. 13 **Averaged** amplitude spectra of nacelle fore-aft acceleration: Chichi (300%) vs. Chichi (300%) + RW2; **Number of Fast Fourier Transform points: 16384.**

Fig. 1 Seismic intensity in China offshore regions

Fig. 2 View of EWCJSS from monitoring room

Fig. 3 The geometry of tower in model test (Unit: mm)

Fig. 4 Tower erected without lead ballasts on surface

Fig. 5 Installation of nacelle onto the ballasted model

Fig. 6a Original strong seismic motion of WE Chichi,
230⁰ El Centro, NS Tianjin earthquake waves

Fig. 6b **Unaveraged** amplitude spectra of original WE Chichi,
230° El Centro, NS Tianjin earthquake waves

Fig. 7 Placement of accelerometers

Fig. 8 Placement of strain gauges
(Left: section A, C, D; Right: section E)

Fig. 9 Free vibration decay of nacelle acceleration in a) and d)
a) Initial: dry flume; d) After damage by El Centro wave: dry flume

Fig. 10a Nacelle fore-aft Acceleration

- excited by modified Chichi wave (300% loaded)

Fig. 10b Nacelle fore-aft Acceleration
- excited by modified El Centro wave (46.7% loaded)

Fig. 10c Nacelle fore-aft Acceleration
- excited by modified Tianjin wave (200% loaded)

Fig. 10d Nacelle side-to-side Acceleration
- excited by modified El Centro wave (46.7% loaded)

Fig. 11a Nacelle fore-aft Acceleration - excited by Random wave 1 (HW)

Fig. 11b Nacelle fore-aft Acceleration - excited by Random wave 2 (HW)

Fig. 12a Nacelle fore-aft Acceleration - excited by
modified Chichi seismic wave (300%) and Random wave 2

Fig. 12b Nacelle fore-aft Acceleration - excited by
modified El Centro seismic wave (46.7%) and Random wave 2

Fig. 12c Nacelle fore-aft Acceleration - excited by
modified Tianjin seismic wave (200%) and Random wave 2

Fig. 13 Averaged amplitude spectra of nacelle fore-aft acceleration: Chichi (300%) vs. Chichi (300%) + RW2; Number of Fast Fourier Transform points: 16384.