

Mitigation of Coupled Wind-Wave-Earthquake Responses of a 10 MW Fixed-Bottom Offshore Wind Turbine

Yang YANG ^{a,b,*}, Musa BASHIR ^b, Chun LI ^a, Constantine MICHAELIDES ^c, Jin WANG ^b

^a School of Energy and Power Engineering, University of Shanghai for Science and
Technology, Shanghai, 200093, P.R. China

^b Department of Maritime and Mechanical Engineering, Liverpool John Moores University,
Byrom Street, L3 3AF, Liverpool, UK

^c Department of Civil Engineering and Geometrics, Cyprus University of Technology,
Saripolou 2-8, 3036, Limassol, Cyprus

Abstract: In this paper we present a study on the mitigation of dynamic responses of a 10 MW monopile offshore wind turbine under coupled wind-wave-earthquake excitations. We have developed and validated the generic seismic coupled analysis and structural control architecture tool to overcome the limitation of numerical tools when examining the wind-wave-earthquake coupling effects. We investigated the dynamic responses of a 10 MW monopile offshore wind turbine under different loading combinations and found that the earthquake loading increases the tower-top displacement and pile-cap moment by 47.6% and 95.1%, respectively, compared to the wind-wave-only condition. It is found that the earthquake-induced vibration in the fore-aft direction is mitigated by the wind and wave loadings due to the energy dissipated by the aerodynamic and hydrodynamic damping. In addition, the tower responses are dominated by the earthquake excitation. In order to alleviate the tower vibration induced by the earthquake, we implemented the structural control capability within the tool using tuned mass dampers. The tuned mass dampers with appropriately selected design parameters achieve a larger mitigation on the tower-top

24 displacement for the earthquake-only condition compared to the coupled-loading scenario.
25 The reason is that the tuned mass damper is only effective in mitigating tower vibration, and it
26 is not capable of reducing the tower elastic deformation which is the major contribution of the
27 tower displacement for the coupled-loading condition. In addition, we have found that a
28 heavier tuned mass damper requires a lower tuned frequency to achieve a larger mitigation. A
29 configuration for the mitigation control of the 10 MW offshore wind turbine is suggested by
30 using a 5% mass ratio of the tuned mass damper.

31 **Keywords:** Offshore Wind Turbines; Tuned Mass Dampers; Wind-Wave-Earthquake
32 Analysis; Structural Control; Earthquake Excitation;

34 **1 Introduction**

35 The climate action demands lower emissions of greenhouse gases by decreasing energy
36 consumption and transitioning to low-carbon or zero-carbon resources. Development of
37 renewable energy resources offers the most efficient action in reducing carbon emissions for
38 moderating the global warming [1]. According to the study by Liang et al. [2], the average
39 CO₂ abatement cost decreases by 0.7 EUR for every 1% increase of the capacity factor of
40 renewable power resources. Moreover, the renewable energy sector has been at the forefront
41 of realizing the sustainability goals by playing a significant role in providing access to basic
42 and clean electric power to people, especially those living in developing countries and remote
43 areas with huge difficulties in accessing electricity grid facilities. In addition, the renewable
44 energy sector has continued to serve as a vehicle for social mobility in providing 10.3 million
45 jobs worldwide as estimated by the International Renewable Energy Agency (IRENA) [3].

46 Further development of renewable energy technologies will produce significant economic and
47 environmental benefits in moving humanity towards achieving sustainability goals.

48 Offshore wind offers a promising pathway to accelerating transitions to sustainability
49 goals due to its availability and high capacity factor. As indicated in the outlook report of the
50 International Energy Agency (IEA) [4], the offshore wind energy market has expanded by
51 nearly 30% per year between 2010 and 2018, and the global offshore wind capacity is
52 expected to increase by over 20 GW per year in the coming decade. It is noted that there are
53 more than 40% of Offshore Wind Turbines (OWTs) expecting to be installed in the coastal
54 areas of China, Mediterranean and the United States, which are earthquake-prone. The
55 seismic hazards necessitate the examination of the coupling effects between wind, wave and
56 earthquake loadings in the design of OWTs operating in these areas.

57 Early-stage seismic studies employed the response spectrum method [5-6] to estimate the
58 load demand of a wind turbine under an earthquake event. The linear modal properties
59 including the mode shapes and mass distribution were used as recommended in the seismic
60 design codes of conventional buildings [7]. However, the difference between a wind turbine
61 and conventional buildings is that the aerodynamic load acting on the rotor is as significant as
62 earthquake excitations. In order to consider the wind effect, these studies [8-11] simplified the
63 aerodynamic loads as time-varying rotor thrusts that were calculated externally in an
64 uncoupled manner, meaning that the pitch velocity of the rotor induced by the tower vibration
65 under an earthquake event was neglected. The aerodynamic load, however, is sensitive to the
66 relative speed between inflow wind and rotor, especially for large-scale OWTs. The coupled
67 effect of wind and earthquake loadings must be examined in the seismic analysis of wind

68 turbines.

69 In order to address the research need, a seismic module was added into the commercial
70 software tool, Bladed [12]. A recent study by Santangelo *et al.* [13] investigated the influence
71 of the coupling effect between wind and earthquake for a 5 MW wind turbine using Bladed.
72 Similarly, Asareh [14-15] implemented the seismic analysis capability into FAST by
73 developing an additional module that used the big-mass method to calculate earthquake
74 excitations [16-17]. A fictive platform with big-mass rigidly connecting the wind turbine base
75 was placed beneath the ground. The stiffness and damping of the platform, depending on the
76 mass, were used to determine the earthquake loads. Asareh *et al.* [18] investigated the
77 dynamic behaviours of a 5 MW wind turbine influenced by earthquake intensity and wind
78 speed using the FAST-Seismic. However, it is noted that the definition of the fictive mass
79 depends on the experience of users. Furthermore, this method is incapable of considering the
80 soil-structure interaction (SSI) effect that would be more significant under an earthquake
81 event. Yang *et al.* [19] further improved the method of earthquake load calculation used in
82 FAST-Seismic by using the Wolf model. The influence of aerodynamic damping on the
83 seismic behaviour of a 5 MW wind turbine was investigated for different earthquake loading
84 scenarios.

85 The studies reviewed above investigated the seismic behaviour of land-based wind
86 turbines. As numerous offshore wind farms are located in earthquake-prone sites such as
87 south-eastern coastal areas of China, coastal areas of south-eastern Europe and the west
88 coastal areas of the US, it is vital to perform seismic analysis of OWTs. Kim *et al.* [20]
89 conducted a fragility analysis of a 5 MW monopile OWT subjected to earthquake loadings

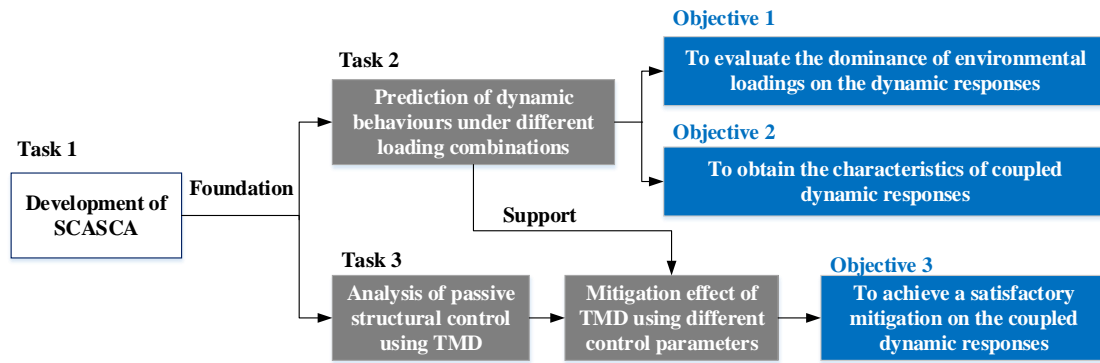
90 under the parked state. The SSI was modelled using p - y curves. Mo *et al.* [21] developed a
91 weak-coupled model of a 5 MW OWT in OpenSees. The fragility of the support structure was
92 investigated under different operating conditions. Alati *et al.* [22] compared the dynamic
93 responses of two types of fixed-bottom OWTs subjected to wind and earthquake loadings
94 using Bladed. The SSI effect was examined using the linear coupled-springs model. Yang *et al.*
95 [23-24] investigated the linear and nonlinear SSI effects on the seismic behaviour of a 5 MW
96 OWT using a newly developed numerical tool based on FAST. The dominance of wind, wave
97 and earthquake loadings was discussed for the 5 MW wind turbine.

98 However, in all the above mentioned literatures the focus has been on wind turbines
99 whose capacity is up to 5 MW. Due to the demand for reducing installation and maintenance
100 costs of OWTs as part of requirements for reducing Levelised Cost of Electricity (LCoE), the
101 development of 10 MW-class wind turbines is attracting significant attention. Consequently, it
102 is imperative to investigate the coupling effects of wind, wave and earthquake loadings for 10
103 MW OWTs located within earthquake-prone areas including some particular coastal areas of
104 Europe, China and the US. Furthermore, mitigation studies are required to reduce the risk of
105 potential damage caused by an earthquake.

106 In order to address the identified research gap, this study aims to investigate the dynamic
107 behaviour of a 10 MW OWT subjected to coupled wind wave and earthquake loadings. In
108 addition, a study on mitigation of the coupled dynamic responses is examined to reduce the
109 risk of potential damage during an earthquake event. In order to conduct the research and
110 achieve its aims, a generic Seismic Coupled Analysis and Structural Control Architecture
111 (SCASCA) is developed to conduct fully coupled simulations of OWTs subjected to wind,

112 wave and earthquake loadings. The seismic analysis capability is implemented into an open
113 source numerical tool, FAST (version 7.02) [25], by modifying its source code with regards to
114 the structural modelling. In addition, the SCASCA tool is further improved to be capable of
115 performing structural control analysis based on the Tuned Mass Damper (TMD) [26] for
116 mitigating the coupled responses. The SCASCA offers a generic capability of performing
117 seismic analysis of different wind turbines compared to FAST-Seismic developed by Asareh
118 *et al.* [14], since the approach of earthquake load calculation employed in SCASCA is
119 independent of the researcher's experience. The superiority of SCASCA compared to Bladed
120 is that SCASCA is capable of examining the vertical excitation of an earthquake. In addition,
121 the frequency contents of the input ground motion can be adjusted in order to be consistent
122 with the target response spectrum of a specific site.

123 Fig. 1 presents the research tasks and objectives of this paper. With the use of SCASCA,
124 dynamic responses of the 10 MW monopile OWT [27] under different loading combinations
125 are obtained and compared in order to illustrate the dominance of the environmental loadings.
126 The effectiveness of a TMD in alleviating tower vibration caused by the coupled loads is
127 investigated. Rational parameters of a TMD with a specified mass ratio are obtained by
128 conducting parametric and sensitivity analyses of the control parameters. The maximum
129 tower-top displacement is reduced significantly by an appropriate TMD under both coupled
130 and earthquake-only environmental conditions.



131

132

Fig. 1: Research tasks and objectives of this paper

133 2 Development of SCASCA

134 A generic tool named SCASCA is developed in order to investigate and moderate the
 135 seismic behaviour of a 10 MW OWT under coupled wind-wave-earthquake loadings. The
 136 capabilities of seismic analysis and structural control are implemented within the FAST
 137 (version 7.02) numerical tool [25]. The subsequent sections present an overview of the
 138 original FAST as well as of the development of SCASCA.

139 2.1 Overview of FAST

140 National Renewable Energy Laboratory (NREL) developed a fully coupled
 141 aero-hydro-servo-elastic tool, FAST, for the design of horizontal axis wind turbines [25]. The
 142 original version of FASTv7.02 used in this study integrates four major modules: AeroDyn,
 143 ElastDyn, ServoDyn and HydroDyn. Aerodynamic and hydrodynamic loads are computed in
 144 the AeroDyn and HydroDyn modules, respectively. The ServoDyn module deals with the
 145 adjustments of blade pitch angles and generator speed for normal power production through a
 146 dynamic link library. In the ElastDyn module, the wind turbine is modelled as a multi-body
 147 dynamic system consisting of rigid and flexible structural elements. The equation of motion
 148 of the dynamic system is derived using the Kane method [28]. The linear modal approach is

149 used to predict aero-elastic responses of the blades and tower. The capabilities of seismic
150 analysis and structural control can be implemented by modifying the source code of the
151 ElastDyn module.

152 FAST has been extensively used in industrial and academic studies due to its
153 well-validated accuracy and credibility. The open source nature of FAST encourages
154 researchers to implement new capabilities for the design of wind turbines. FAST is an ideal
155 option to be used as the foundation for the development of SCASCA.

156 ***2.2 Implementation of seismic analysis capability***

157 The big-mass method is one of the commonly-used approaches in the calculation of
158 seismic loads of civil engineering structures. It assumes that the structure above the ground
159 behaves as a rigid body under the influence of a fictive big-mass body beneath the ground.
160 The fictive big-mass body follows the input ground motion, resulting in seismic load acting
161 on the structure. This method is efficient in capturing intense variations of structural responses
162 during an earthquake event. The implementation of this method only requires an estimation of
163 the seismic load based on simple equations and without the need to modify the equation of
164 motion of the wind turbine. Asareh *et al.* [14] used this method to develop the Seismic module
165 and integrated it into FAST. However, it is noted that the definition of the fictive mass
166 depends on the experience of the users. Furthermore, this method is incapable of considering
167 the SSI effect that would be more significant under an earthquake event.

168 In order to address the limitations of the big-mass method, this study modifies the
169 equation of motion of the wind turbine in FAST based on a generic theory that has been
170 extensively applied in civil engineering. For a monopile OWT, FAST treats the pile and tower

171 as one integrated support structure. FAST employs the linear modal approach in the structural
 172 modelling of the support structure. The equation of motion for each of the considered i^{th}
 173 modal degrees of freedom (DOFs) of the support structure subjected to wind, wave and
 174 earthquake loadings is derived as follows:

$$175 \quad \omega_i^2 q_i + 2\xi_i \omega_i \dot{q}_i + \ddot{q}_i = (a_{\text{eq}} \gamma_i + F_{\text{aero},i} + F_{\text{hydro},i} + F_{\text{gra},i}) / m_i \quad (1)$$

176 where q_i , \dot{q}_i and \ddot{q}_i are, respectively, the modal displacement, velocity and acceleration
 177 of the i^{th} mode. ω_i and ξ_i are the angular frequency and damping ratio of the i^{th} mode,
 178 respectively. a_{eq} is the input earthquake acceleration. $F_{\text{aero},i}$, $F_{\text{hydro},i}$ and $F_{\text{gra},i}$ are,
 179 respectively, the generalized aerodynamic, hydrodynamic and gravity loads corresponding to
 180 the i^{th} mode. m_i is the modal mass associated with the i^{th} mode. γ_i is the earthquake
 181 participation factor associated with the i^{th} mode that is denoted as:

$$182 \quad \gamma_i = \int_0^H \rho(h) \cdot \phi_i(h) \cdot dh \quad (2)$$

183 where H is the length of the support structure. $\rho(h)$ is the mass density of the support
 184 structure and $\phi_i(h)$ is the normalized modal shape of the i^{th} mode of the support structure.

185 The rotor-nacelle-assembly (RNA) is simply treated as a lumped mass atop the support
 186 structure for the seismic load calculation. The corresponding seismic load of the RNA,
 187 $F_{\text{eq,RNA}}$, is derived as:

$$188 \quad F_{\text{eq,RNA}} = a_{\text{eq}} \cdot m_{\text{RNA}} \quad (3)$$

189 where m_{RNA} is the total mass of RNA.

190 It is apparent that the prediction of seismic load only depends on the modal shapes of the
 191 structure and the input earthquake acceleration. The method implemented in this study is
 192 generic and applicable to an arbitrary wind turbine. Furthermore, the SSI effect is considered

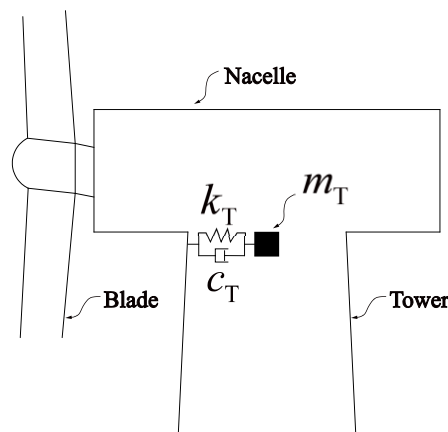
193 properly using the Winkler spring-dashpot model when calculating the modal shapes of the
194 structures.

195 The seismic loads are added into the generalized forces within FAST when modelling the
196 equation of motion of the wind turbine. The source code of FAST is modified accordingly
197 based on the equations presented above in order to implement the fully coupled seismic
198 analysis capability.

199

200 **2.3 Structural control**

201 In order to moderate and mitigate the dynamic responses of an OWT subjected to
202 earthquake loadings, a passive structural control module is developed using the TMD method.
203 The basic concept of the TMD method is to place a mass damper at an appropriate location
204 for dissipation of energy from external excitations. In this study, two independent TMDs are
205 orthogonally placed at the tower-top to mitigate longitudinal and lateral responses of the
206 support structure due to coupled wind-wave-earthquake loadings as presented in Fig. 2.



207

208 Fig. 2: Schematic diagram of TMD location

209 The implementation of TMD requires modifications in the modelling of the equation of

210 motion of the wind turbine in FAST. The force produced due to the motion of the TMD is
 211 added into the generalized forces, *i.e.* the right terms in Eq. (1). The TMD force F_{TMD} in each
 212 direction is derived as follows:

$$213 \quad F_{\text{TMD}} = -k_{\text{T}} \cdot x_{\text{TMD}} - c_{\text{T}} \cdot \dot{x}_{\text{TMD}} \quad (4)$$

214 where x_{TMD} and \dot{x}_{TMD} are the TMD displacement and velocity, respectively. m_{T} , k_{T} and
 215 c_{T} are the mass, stiffness and damping of the TMD, respectively.

216 The motion of the TMD is influenced by the nacelle dynamics associated with
 217 centrifugal force, Euler force and Coriolis force. The TMD acceleration \ddot{x}_{TMD} can be denoted
 218 as follows:

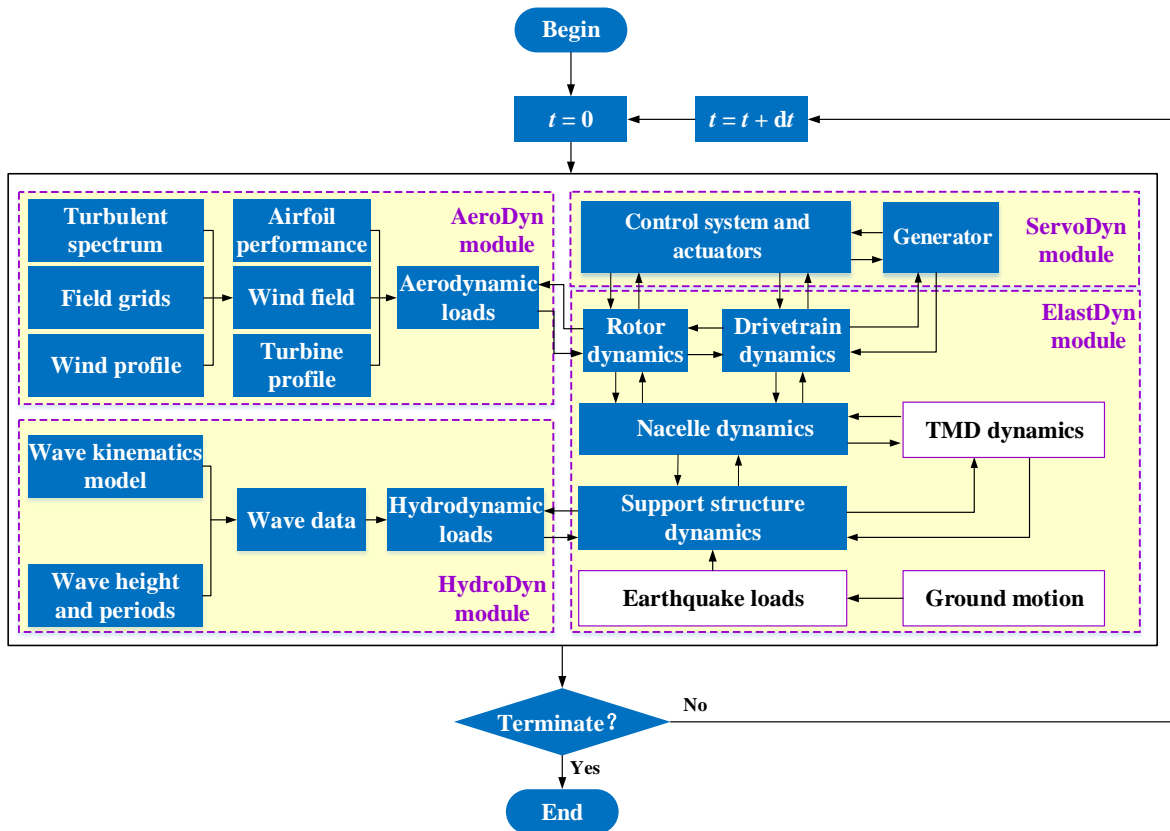
$$219 \quad \ddot{x}_{\text{TMD}} = -\ddot{x}_{\text{N}} - \omega_{\text{N}} \times (\omega_{\text{N}} \times x_{\text{TMD}}) - \alpha_{\text{N}} \times x_{\text{TMD}} - 2\omega_{\text{N}} \times \dot{x}_{\text{TMD}} - F_{\text{TMD}}/m_{\text{T}} \quad (5)$$

220 where \ddot{x}_{N} is the nacelle acceleration. ω_{N} and α_{N} are, respectively, the translational and
 221 rotational angular velocities of the nacelle. $\omega_{\text{N}} \times x_{\text{TMD}}$, $\alpha_{\text{N}} \times x_{\text{TMD}}$ and $2\omega_{\text{N}} \times \dot{x}_{\text{TMD}}$ denote
 222 the contributions of the centrifugal force, Euler force and Coriolis force, respectively.

223

224 ***2.4 Validation of the SCASCA tool***

225 Fig. 3 presents the flowchart of SCASCA for every time step of an analysis. In every
 226 time step, dt , of a simulation in SCASCA, the earthquake loads acting on the support structure
 227 are calculated based on the input ground motion. The TMD kinematics and kinetics are
 228 coupled with the dynamics of the nacelle and support structure when solving the equations of
 229 motion of the offshore wind turbine.



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Fig. 3: Flowchart of SCASCA for every time step of an analysis

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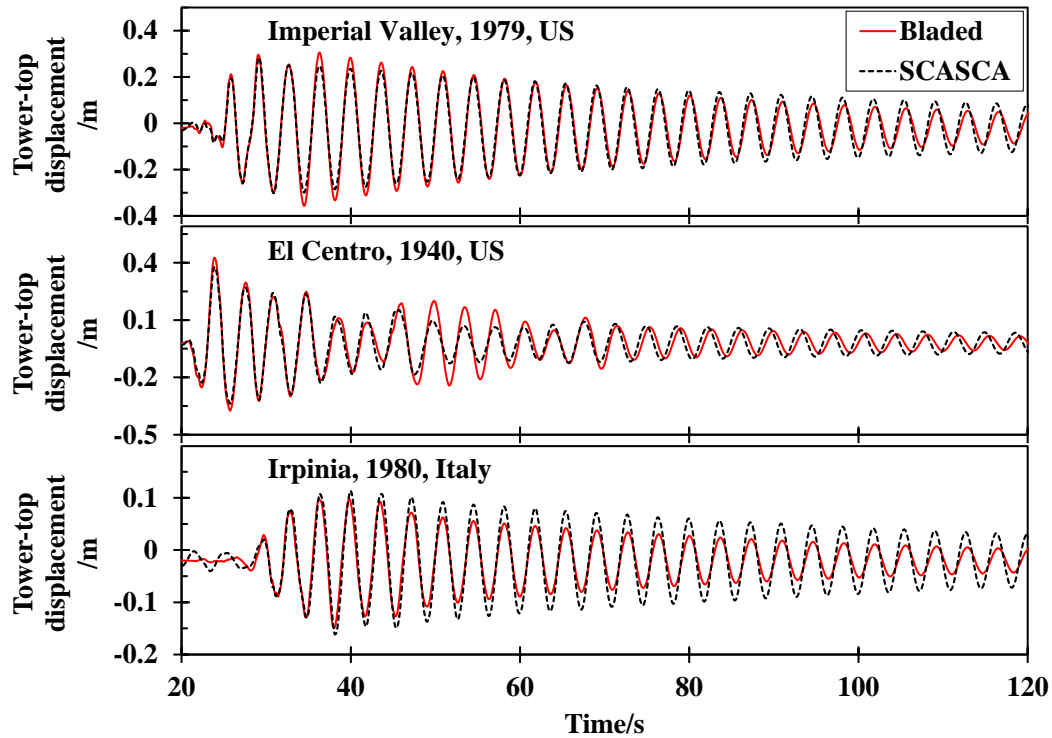
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In order to validate SCASCA, a comparison of the horizontal excitation of an earthquake against Bladed is presented. Fig. 4 presents the tower-top displacements of the NREL 5 MW monopile OWT [23] obtained by Bladed and SCASCA, respectively, for different earthquake loadings. For each of the simulations, the earthquake is assumed to occur at the 20th s. To avoid the influence of the difference between FAST and Bladed in predicting aerodynamic loads, the wind turbine is only subjected to the earthquake loading. As can be seen from Fig. 4, the earthquake-induced responses of the wind turbine calculated by SCASCA agree very well with the results from Bladed for each level of the ground motions. SCASCA efficiently captures the drastic variation of the tower response under an earthquake scenario as confirmed by the agreements between the two numerical analysis tools.



242

243

Fig. 4: Comparison of tower-top responses of the NREL 5 MW wind turbine subjected to

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different ground motions between Bladed and SCASCA

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It is noted that SCASCA addresses the limitation of the commonly-used commercial

246

software, Bladed [9], in handling the vertical earthquake excitation. The accuracy of SCASCA

247

in examining the vertical earthquake excitation is validated by comparing it with

248

NREL-Seismic tool that employed the big-mass approach for earthquake load prediction. Fig.

249

5 presents the tower-base vertical shear-force of the wind turbine under different earthquake

250

loadings. The mass of the fictive platform adopted in NREL-Seismic code is 7.0×10^6 kg, that

251

is the value recommended for the land-based NREL 5 MW wind turbine. As can be seen from

252

Fig. 5, the vertical shear-force at the tower-base predicted by SCASCA follows the same trend

253

with similar magnitudes compared to the results calculated by NREL-Seismic for each of the

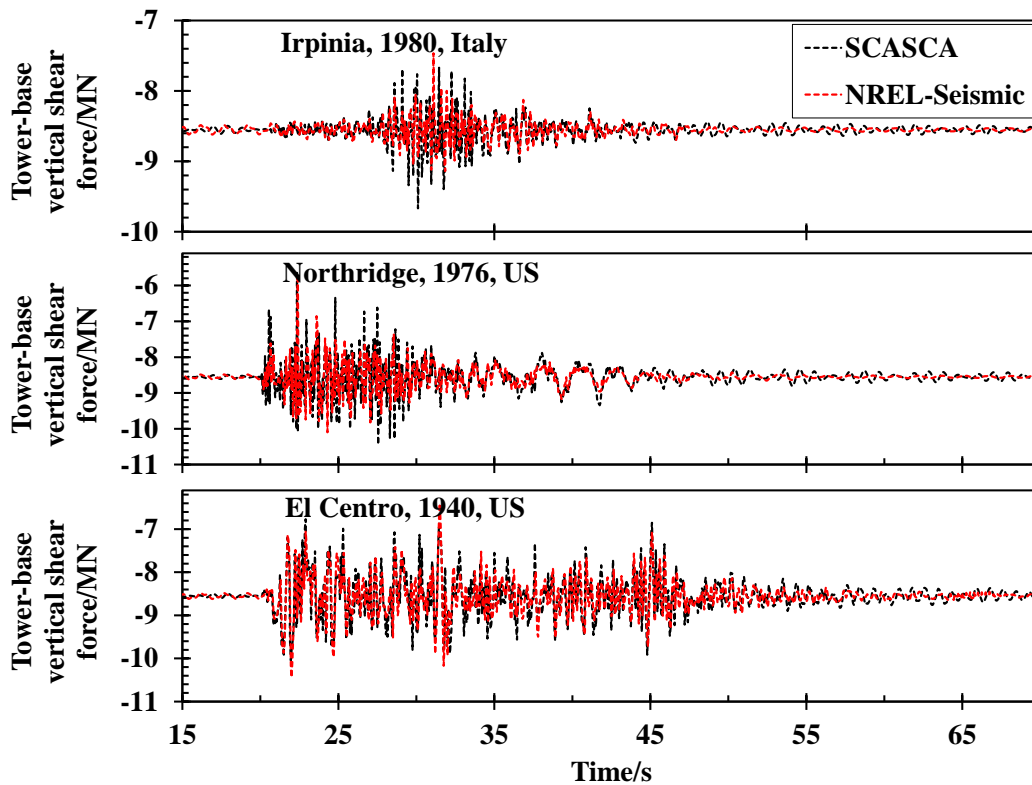
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earthquake events. The result of SCASCA is slightly larger than the result of NREL-Seismic

255

for the Irpinia earthquake record. The minor discrepancy between the results is due to the fact

256 that the fictive platform mass defined in NREL-Seismic program was for the land-based wind
257 turbine, resulting in a relatively smaller prediction of the earthquake load. For the other two
258 earthquake events, the differences between the results of SCASCA and NREL-Seismic are
259 insignificant. The overall agreements between the results are good, indicating that the
260 capability of examining vertical earthquake excitation is well implemented within SCASCA.



261
262 Fig. 5: Comparison of tower-base vertical shear-force of the NREL 5 MW wind turbine
263 subjected to different vertical ground motions between NREL-Seismic and SCASCA
264

265 The comparisons above verify that SCASCA has a high accuracy in performing seismic
266 analysis of OWTs. Since the linear modal approach is used for the structural modelling, the
267 stiffness of the structures is assumed to remain unchanged under an earthquake event,
268 implying that SCASCA tool is incapable of examining nonlinear material characteristics in
269 the determination of a plastic damage.

270 **3 Numerical modelling of the 10 MW offshore wind turbine**

271 ***3.1 Design characteristics of the 10 MW monopile wind turbine***

272 The 2012 Light Rotor project carried out in the collaboration between Technical
273 University of Denmark (DTU) and Vestas was aimed at developing a light-weight blade for
274 10+ MW wind turbines [29]. BECAS and HAWCStab2 were used to conduct the lay-up
275 design and aero-elastic stability analysis of the blades. The DTU reference land-based wind
276 turbine was developed by assembling the blades with other essential structural components
277 including hub, tower and nacelle.

278 Offshore application of the DTU 10 MW wind turbine requires structural strength
279 enhancements on the support system to guarantee safety and integrity of the entire wind
280 turbine system. Velarde [27] developed four monopiles for the DTU 10 MW wind turbine
281 operating in different water depths (20 m ~ 50 m) by considering the nonlinear SSI effects.
282 The dimensions of the baseline land-based tower were enlarged against more severe offshore
283 environmental loadings. Since monopile type OWTs are more suitable for water depths within
284 15 m to 30 m, the monopile designed for the 30 m water depth is adopted in this study. The
285 corresponding up-scaling factors for the tower diameter and thickness are 1.25 and 1.3,
286 respectively. The diameter and thickness at tower top are modified to 6.25 m and 35.0 mm,
287 respectively. The diameter and thickness at tower base are changed to 9.00 m and 66.5 mm,
288 respectively. The schematic diagram and a summary of main specifications of the DTU 10
289 MW monopile OWT are presented in Fig. 6 and Table 1, respectively.

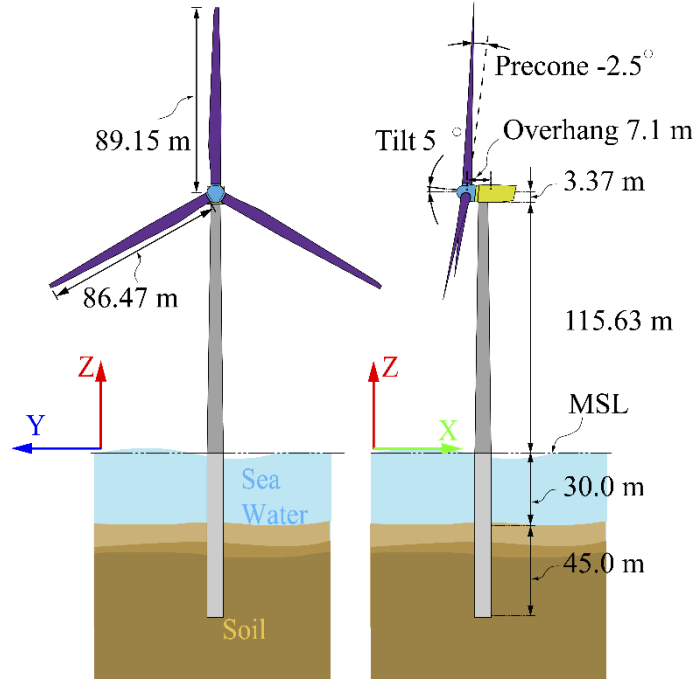


Fig. 6: Schematic diagrams of the DTU 10 MW OWT

Table 1: Main specifications of the DTU 10 MW OWT

Specification (Unit)	Value	Specification (Unit)	Value
Rated power (MW)	10.0	Nacelle mass (kg)	4.46×10^5
Cut-in/cut-out speeds (m/s)	4/25	Tower mass (kg)	1.20×10^6
Rated wind speed (m/s)	11.4	Tower height (m)	115.63
Cut-in/rated rotor speeds (rpm)	6/9.6	Tower top diameter (m)	6.25
Rotor diameter (m)	178.3	Tower base diameter (m)	9.0
Hub diameter (m)	5.6	Tower top thickness (mm)	35.0
Gear box ratio(-)	50	Tower base thickness (mm)	66.5
Shaft tilt angle (°)	5.0	Monopile diameter (m)	9.0
Hub height (m)	119.0	Monopile thickness (mm)	110.0
Rotor mass (kg)	227,962	Monopile length (m)	75
Blade pre-cone angle (°)	-2.5	Monopile mass (kg)	1.96×10^6

3.2 Modelling of soil-structure interaction (SSI) effects

The selected site of the wind turbine has a single soil layer of sand with a saturated soil weight of 20 kN/m^3 and an internal friction angle of 36° . The pile-soil interaction is

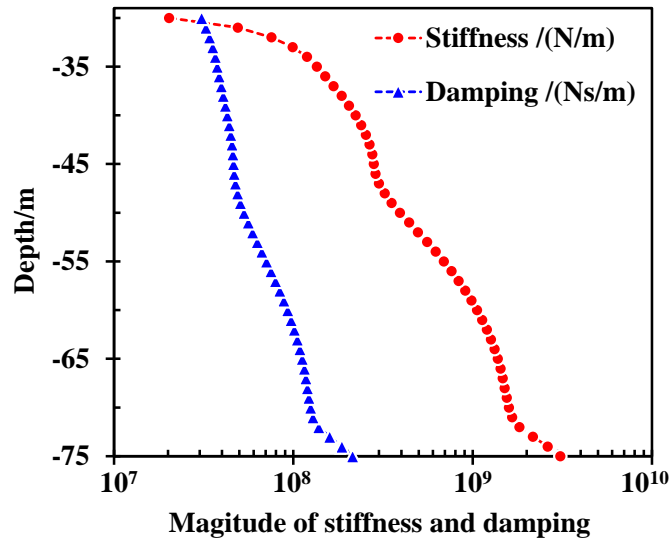
296 represented by the Winkler spring-dashpot model [24Error! Bookmark not defined.] to
 297 consider the soil effect. The stiffness of each spring is derived by force-displacement
 298 relationships (p - y curves). By applying different cyclic loads at the mudline of the monopile,
 299 the p - y curves along the embedded length of the monopile were obtained using the finite
 300 element software Plaxis 3D. Regarding the soil damping due to radiation and hysteretic
 301 effects, the model developed by Gazetas *et al.* [30] is used to determine the soil damping as
 302 follows:

$$303 \quad C_s = 6\sqrt{\rho_s G_s} D_m \left(\frac{\omega_m D_m}{\sqrt{G_s / \rho_s}} \right)^{-1/4} + 2\beta_s \frac{k_s}{\omega_m} \quad (6)$$

304 where C_s is the soil damping; ρ_s and G_s are the density and shear modulus of the soil,
 305 respectively. D_m is the monopile diameter. ω_m is the 1st-order natural angular frequency of
 306 the support structure. k_s is the stiffness derived from the p - y curves and β_s is the hysteresis
 307 damping ratio with a value of 5%.

308 The stiffness and damping distributions along the embedded pile subjected to a lateral
 309 force of 30 MN are presented in Fig. 7. The stiffness close to the seabed level is about two
 310 orders lower than that at the bottom of the monopile. The soil reaction F_s due to relative
 311 displacement d_s and velocity v_s between the soil and monopile under external loadings is
 312 given as:

$$313 \quad F_s = -k_s \cdot d_s - C_s \cdot v_s \quad (7)$$



314

315 Fig. 7: Linear stiffness and damping of the Winkler spring-dashpot model of the 30 m water
 316 depth monopile

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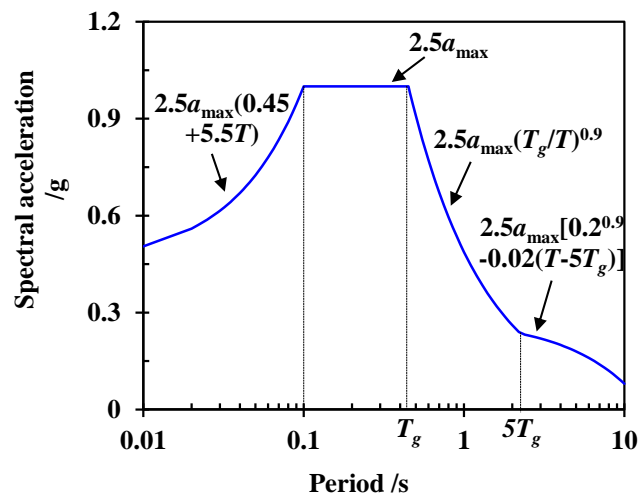
318 4 Seismic behaviour of the 10 MW wind turbine

319 4.1 Scaling of the ground motion

320 The monopile used in this study was designed for a typical medium-stiff soil with an
 321 internal friction angle of 36° . The saturated and effective unit weight of the soil are 20 kN/m^3
 322 and 17 kN/m^3 , respectively. In order to be consistent with the design of the monopile, a
 323 medium-stiff site in the eastern coast of China is chosen for the case study of the 10 MW
 324 monopile OWT.

325 The ground motion recorded in the 1979 Imperial Valley earthquake event is chosen as
 326 the input earthquake acceleration. In order to ensure that the frequency contents of the
 327 selected ground motion is consistent with the geological characteristics of the specific area,
 328 the response spectrum of the ground motion is modified to match a target response spectrum

329 that is defined in accordance with the seismic design code. Fig. 8 presents a seismic response
 330 spectrum with a design acceleration of 0.40 g and a damping ratio of 5 % defined in
 331 accordance with the Chinese code for seismic design of buildings [31]. T_g is a site depended
 332 characteristic parameter that denotes the ratio between the design spectral acceleration (a_{\max})
 333 and the spectral acceleration at 1.0 s. According to the Chinese seismic design code, the value
 334 of T_g is chosen as 0.43 s for a medium-stiff site in the eastern coastal areas of China. For the
 335 ninth-level seismic design intensity, the longitudinal design acceleration is chosen as 0.40 g.
 336 The ratio between the design acceleration magnitudes of the longitudinal and lateral ground
 337 motions is 1:0.85. The target response spectra corresponding to the horizontal ground motions
 338 are obtained accordingly.

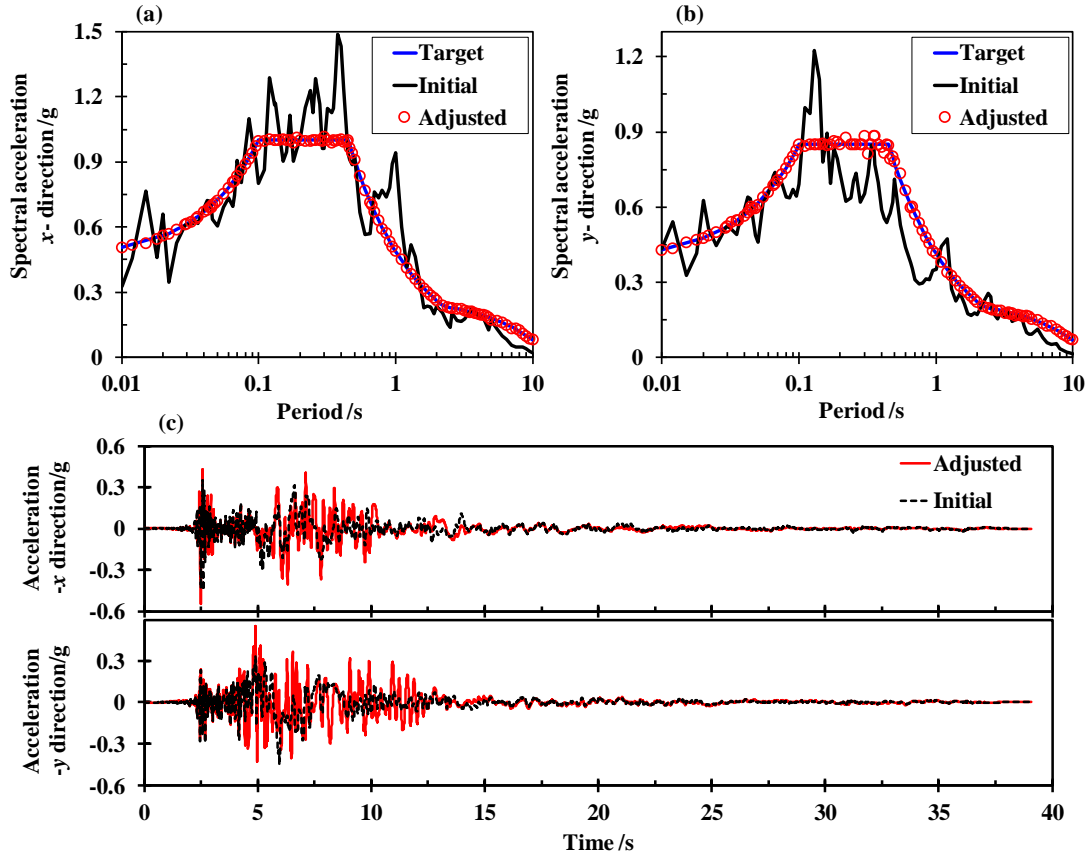


339
 340 Fig. 8: Target response spectrum for a China's eastern coastal site

341
 342 The application of the ground motion scaling is to eliminate the spectral misfits between
 343 the initial and the target response spectra. The scaling of the longitudinal and lateral ground
 344 motions is conducted using the “RspMatch” code developed by Atik *et al.* [32]. In the scaling
 345 process, different wavelet components are iteratively added into the initial earthquake record

346 to adjust its frequency characteristics until the spectral misfit to the target spectrum falls
347 below a given tolerance value. Fig. 9 presents the response spectra and accelerograms of the
348 initial and adjusted ground motions. The initial response spectrum is the spectral acceleration
349 of the 1979 Imperial Valley earthquake recorded by El Centro Array #6 station. The adjusted
350 response spectrum corresponds to the earthquake acceleogram modified by the “RspMatch”
351 code in the scaling process.

352 From Fig. 9-(a) and Fig. 9-(b), it is observed that the response spectrum of the adjusted
353 ground motion in each of the horizontal directions agrees very well with the target spectrum.
354 It means that the adjusted ground motion is capable of representing the earthquakes in the
355 target site. The accelerograms indicate that the peak of ground acceleration (PGA) of the
356 adjusted ground motion in the longitudinal direction is around 0.40 g. This means that the
357 adjusted ground motion has satisfied the requirement of the scaling process.



358
 359 Fig. 9: Response spectra and accelerograms of the initial and adjusted ground motions in the
 360 longitudinal (x -aligned with wind and wave) and lateral (y) directions
 361

362 *4.2 Coupled responses due to wind-wave-earthquake loadings*

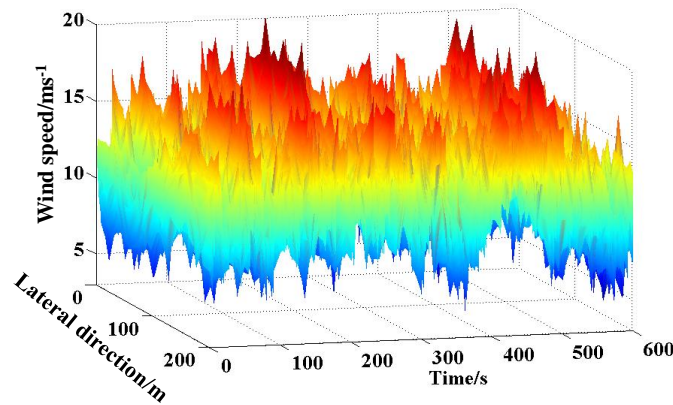
363 In order to evaluate the contribution of the earthquake loading to the coupled responses
 364 of the 10 MW OWT, three different loading scenarios examined in this study are presented in
 365 Table 2. The wave direction is assumed to be aligned with the inflow direction of the wind.

366 Table 2: Loading scenarios for the simulations

Load cases	Wind speed (m/s)	Wave height (m)	Wave period (s)	Earthquake (-)
Earthquake-only	(-)	(-)	(-)	Imperial Valley
Wind-wave-only	11.4	6.0	12.5	-
Coupled-loading	11.4	6.0	12.5	Imperial Valley

367

368 The full-field turbulent wind is generated using TurbSim [33] based on the Kaimal
369 spectrum. Fig. 10 presents the wind speeds at the hub height. The spatial and time-domain
370 variations of the wind speed have confirmed the turbulent features of the generated wind
371 field.

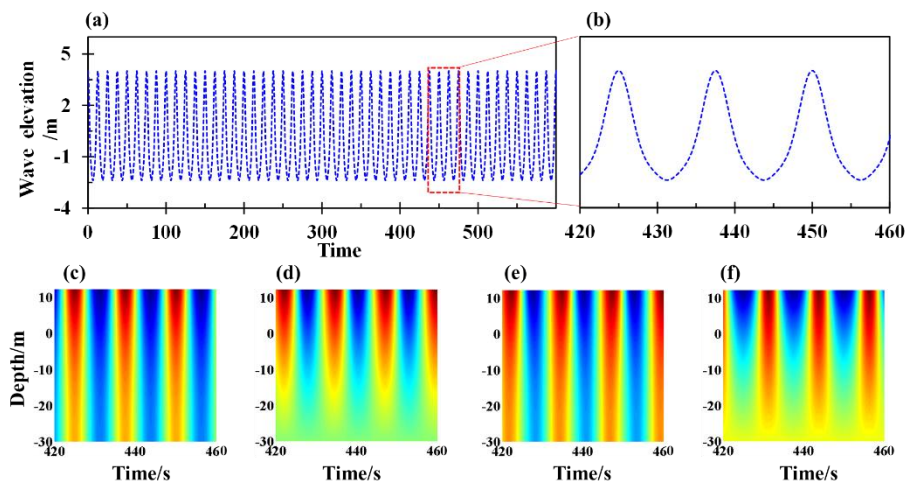


372

Fig. 10: Wind speed at the hub height

373

374 The baseline FAST is only capable of generating linear waves based on Airy wave theory
375 but accepts user-defined waves in a specific format. In order to consider nonlinearity of the
376 waves, the kinematics of the nonlinear waves are reproduced based on the third-order Stokes
377 wave theory [34]. Fig. 11 presents the wave kinematics versus depth.



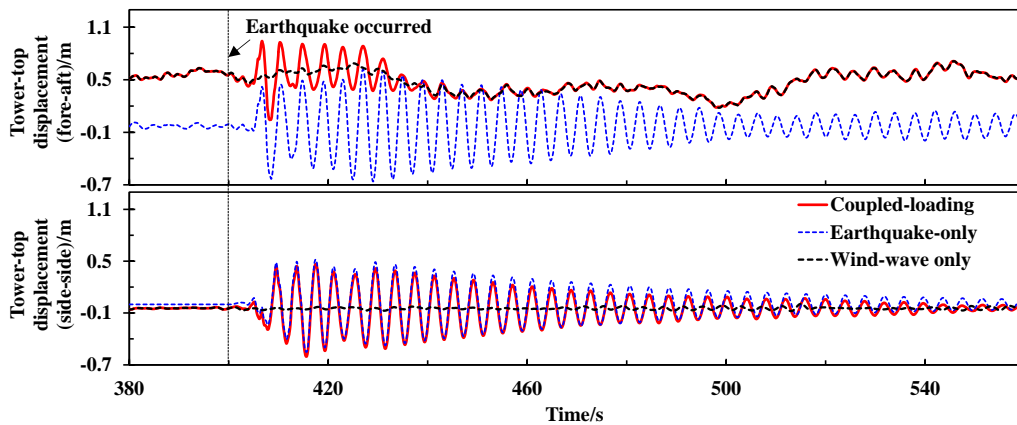
378

379 Fig. 11: Wave elevation and kinematics of the nonlinear wave. (a) wave elevation; (b)
380 enlarged vision of wave elevation; (c) longitudinal velocity; (d) vertical velocity; (e)

381 longitudinal acceleration; (f) vertical acceleration

382

383 Each of the simulations has a duration of 600 s and a time step of 0.002 s. The
384 earthquake excitation is added at the 400th s to avoid the influence of the transient behaviour
385 of the wind turbine. Fig. 12 presents a comparison of the tower-top displacements of the 10
386 MW monopile OWT under the three loading scenarios.



387

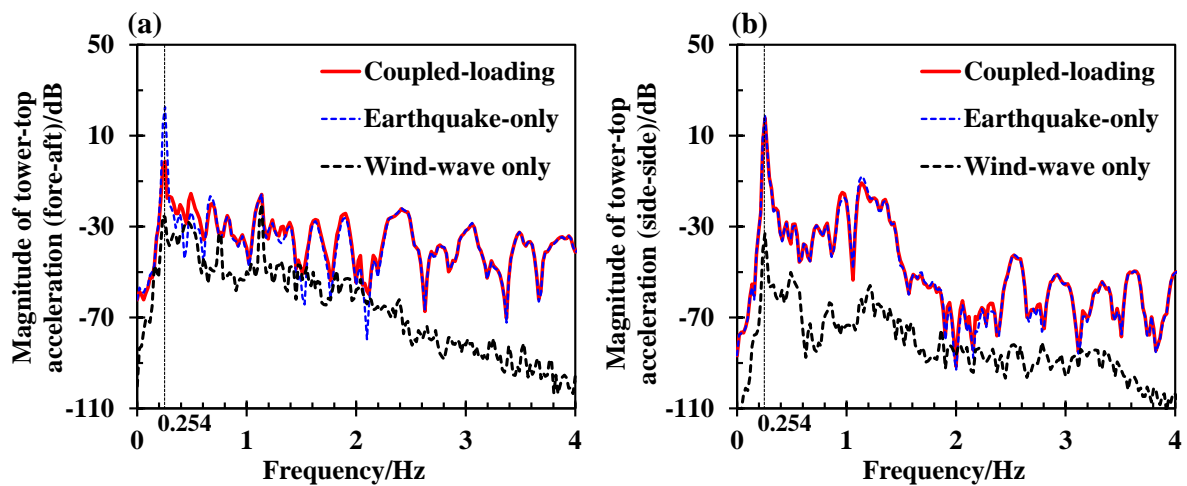
388 Fig. 12: Tower-top displacement time series of the 10 MW OWT subjected to different

389 loading combinations

390 For the earthquake-only scenario, both the tower-top's fore-aft and side-side
391 displacements fluctuate periodically with large amplitude after the earthquake occurred. The
392 variation ranges of the tower-top's fore-aft and side-side displacement are -0.66 m~0.62 m
393 and -0.55 m~0.52 m, respectively. The tower vibrates more severely in the fore-aft direction
394 due to the stronger component of the ground motion. After the high intensity excitation (>440
395 s), the tower-top displacements start to decay. The decay ratio of the fore-aft tower-top
396 displacement under the earthquake-only condition is smaller than that of the coupled-loading
397 condition. Moreover, the fore-aft tower-top displacement resulted from the coupled-loading
398 varies within the range of 0.04 m to 0.94 m. This is comparatively smaller than the variation

399 range corresponding to the earthquake-only condition. The observations indicate that the
 400 tower vibration in the fore-aft direction is mitigated by the wind and wave loadings. The
 401 reason behind this is that the presences of wind and wave provide aerodynamic and
 402 hydrodynamic damping for dissipating the energy from the earthquake excitation. The fore-aft
 403 tower-top displacement fluctuates within the range of 0.48 m to 0.71 m when the wind turbine
 404 operates under the wind-wave-only condition. The fluctuation over the simulation is much
 405 smaller than that of the other two loading scenarios. This implies that the vibration induced by
 406 the wind and wave is much less severe compared to the vibration caused by the earthquake,
 407 although the average displacement contributed by the elastic deformation is higher.

408 The spectral curves of the tower-top accelerations of the 10 MW monopile OWT under
 409 the three loading scenarios are obtained using the Welch spectrum method and are presented
 410 in Fig. 13.

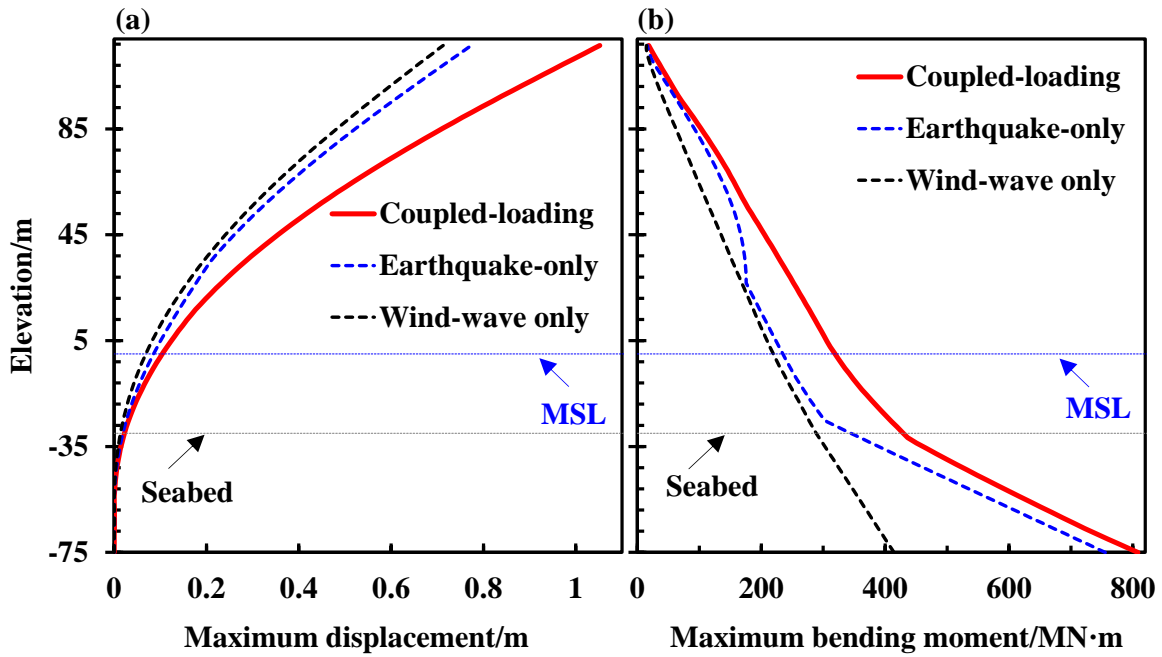


411
 412 Fig. 13: Welch spectral curves of the tower-top accelerations of the 10 MW OWT under
 413 different loading scenarios for: (a) fore-aft direction and (b) side-side direction.

414
 415 The first-order natural mode of the support structure is the main contributor to the

416 tower-top vibration for both the fore-aft and side-side directions as confirmed by the peak
417 magnitude presence at 0.254 Hz. It is noted that the fore-aft magnitude of the coupled-loading
418 condition is much lower than that of the earthquake-only scenario. This further confirms that
419 wind and wave loadings have positive effects in mitigating the earthquake-induced vibration.
420 Due to the absence of wind in the side-side direction, the peak magnitudes at the first-order
421 natural frequency of the support structure agree well between the coupled-loading and
422 earthquake-only conditions. In addition, the spectral magnitudes of the tower-top acceleration
423 from the wind-wave condition are significantly smaller than those from the remaining two
424 loading conditions. This observation confirms that the earthquake is the dominant loading of
425 the tower vibration.

426 Fig. 14 presents the maximum resultant displacement and bending moment along the
427 support structure elevation for the three examined loading conditions. The tower-top
428 displacement resulting from the earthquake-only condition is slightly larger than that of the
429 wind-wave condition. This implies that the magnitude of the tower vibration caused by the
430 earthquake excitation is larger than the elastic deformation due to the wind-wave loading. The
431 tower-top displacement resulting from the coupled-loading exceeds 1.0 m, which is much
432 larger than the values of the other two loading scenarios. Compared with the wind-wave
433 condition, the earthquake enhances the tower-top displacement by 47.6% and the pile-cap
434 bending moment by 95.1%.



435

436 Fig. 14: Maximum responses along the support structure elevation of the three examined

437 loading scenarios: (a) displacement and (b) bending moment

438

439 The maximum tower bending moment due to wind-wave condition increases linearly
 440 with the support structure elevation, which is significantly different from the variation trend
 441 corresponding to an earthquake event. The tower bending moment varies more severely with
 442 the elevation of the embedded portion when the wind turbine is subjected to the earthquake
 443 loading. For an arbitrary elevation, the bending moment of the support structure under the
 444 earthquake-only condition is slightly larger than that of the wind-wave condition. This
 445 indicates that the earthquake loading is the dominant excitation of the OWT. The maximum
 446 bending moments of the support structure at the seabed and pile-cap locations under the
 447 coupled-loading condition are 428 MN·m and 808 MN·m, respectively. For the wind-wave
 448 scenario, the bending moments at the seabed and pile-cap are 288 MN·m and 414 MN·m,
 449 respectively. The earthquake enhances the loads by 48.7% and 95.1%, respectively. It means

450 that the monopile beneath the seabed suffers much stronger loads compared to the portion
 451 above the ground. It is noted that the thickness of the monopile remains unchanged for the
 452 embedded portion in the original design [24]. The results obtained in this study suggest that
 453 the monopile thickness should increase with soil depth for the seismic resistance design of
 454 wind turbines operating in earthquake-prone sites.

455

456 **5 Mitigation control using TMDs**

457 The previous results have indicated that the earthquake loading significantly enhances
 458 tower vibration and bending moment of the OWT. In order to reduce the risk of structural
 459 damage potentially caused by earthquake loadings, TMD is employed to mitigate the tower
 460 vibration and loads on the 10 MW monopile OWT under an earthquake event.

461 ***5.1 Sensitivity of control parameters***

462 For the TMD with a mass, m_T , and a stiffness, k_T , the tuned frequency f_T can be
 463 denoted as:

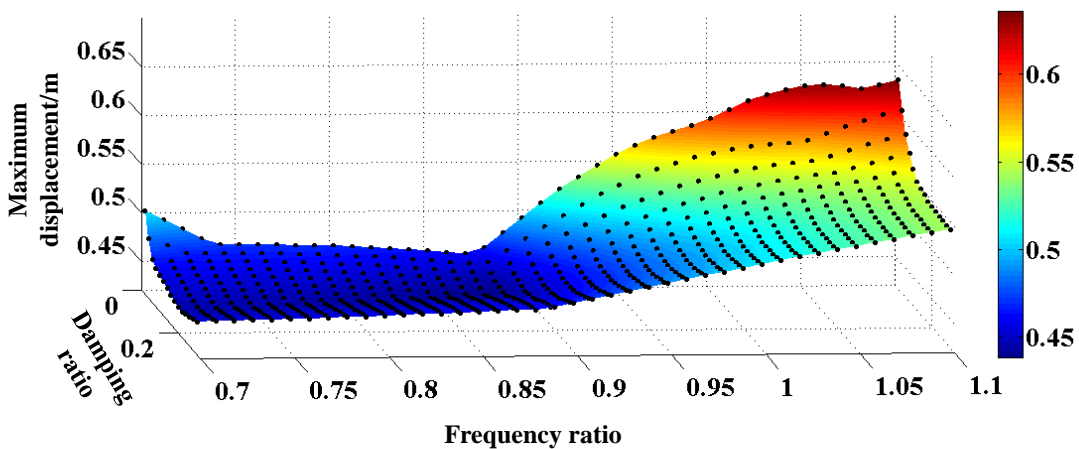
$$464 \quad f_T = \sqrt{k_T / (m_T \cdot 4\pi^2)} \quad (8)$$

465 The mass and first-order natural frequency of the 10 MW OWT are m_{WT} and f_{WT} ,
 466 respectively. The tuned frequency ratio λ and mass ratio μ are defined as follows:

$$467 \quad \begin{cases} \lambda = f_T / f_{WT} \\ \mu = m_T / m_{WT} \end{cases} \quad (9)$$

468 The mitigation effect on the seismic behaviour of the wind turbine is sensitive to the
 469 control parameters including the tuned frequency and damping of a TMD. To obtain the best
 470 mitigation effect, a sensitivity analysis of the control parameters is performed for the

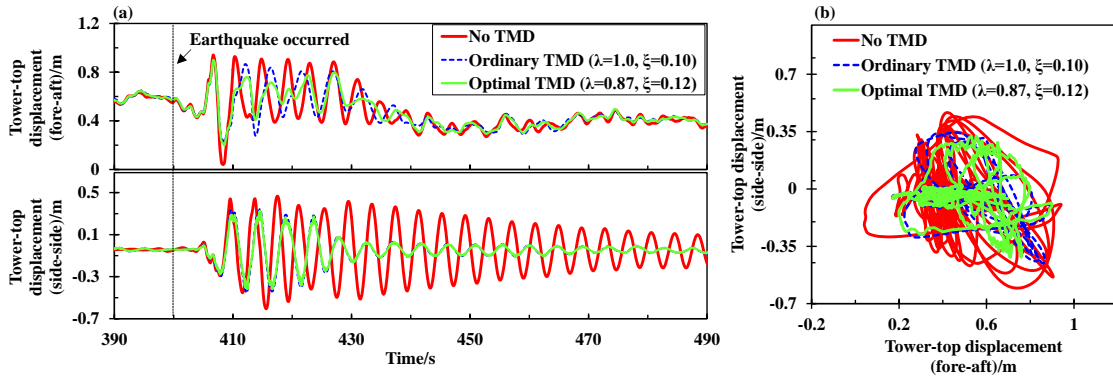
471 earthquake-only condition. Fig. 15 presents the maximum tower-top displacement of the
 472 OWT under the control of a 5% mass ratio TMD with different tuned parameters. The
 473 maximum tower-top displacement of the OWT without the TMD is 0.76 m. All the examined
 474 TMDs are effective in reducing the tower-top displacement as can be observed from Fig. 15.
 475 The mitigation effect is sensitive to the damping ratio for the frequency ratio within the range
 476 from 0.88 to 1.1. A higher damping ratio leads to a relatively larger reduction of tower-top
 477 displacement. The mitigation effect of the TMD is more sensitive on the frequency ratio. The
 478 tower-top displacement decreases significantly with decrease in the frequency ratio. The TMD
 479 with a frequency ratio lower than 0.85 has similar mitigation effects on the tower-top
 480 displacement. The largest mitigation is achieved at 42.5% by using the TMD with a frequency
 481 ratio of 0.87 and a damping ratio of 0.12.



482
 483 Fig. 15: Tower-top displacement versus frequency and damping ratios of TMDs

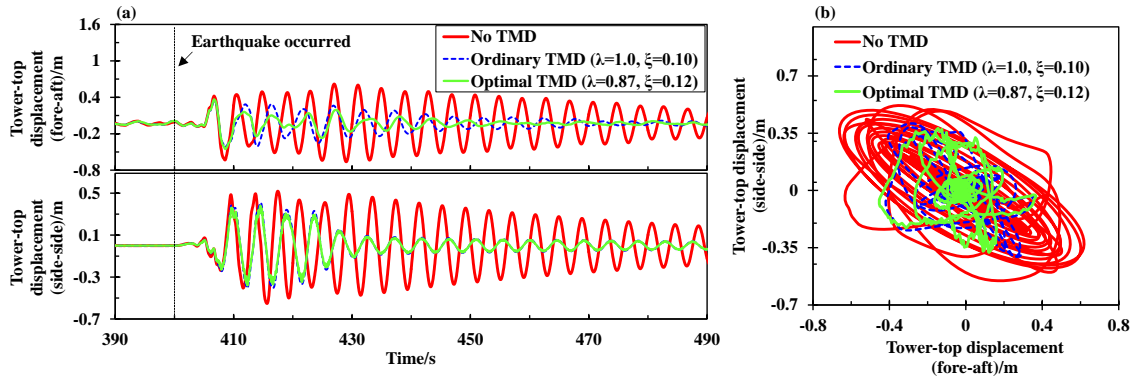
484 Fig. 16 presents the tower-top displacements of the OWT under the control of an optimal
 485 TMD against an ordinary TMD for the coupled-loading condition. Both of the TMDs are
 486 effective in reducing the peak of the fore-aft tower-top displacement. The TMDs accelerate
 487 the decay process to a more stable level after the strong ground motion (>410 s). It is noted
 488 that the tower-top displacement over the simulation of the optimal TMD scenario is smaller

489 compared to the ordinary TMD with a frequency ratio of 1.0 and a damping ratio of 0.1. The
 490 tower-top trajectories imply that the tower vibrates in a smaller range under the control of the
 491 optimal TMD. The observations indicate that the optimal TMD can better alleviate the
 492 earthquake-induced responses compared to the ordinary TMD.



493
 494 Fig. 16: Tower-top displacements of the coupled-loading scenario: (a) time-domain variation
 495 and (b) tower-top trajectory
 496

497 The tower-top displacements under the earthquake-only condition for different TMD
 498 configurations are presented in Fig. 17. The fluctuations of the tower-top displacements in
 499 both fore-aft and side-side directions are significantly mitigated by the TMDs. The standard
 500 deviations of the fore-aft and side-side tower-top displacements are reduced by 70.4% and
 501 56.8 % respectively. Similar to the results of the coupled-loading scenario, the optimal TMD
 502 is more efficient in eliminating the fluctuation of the tower-top displacement caused by the
 503 earthquake, resulting in a narrower range of the tower-top motion trajectory.



504

505 Fig. 17: Tower-top displacements of the earthquake-only scenario: (a) time-domain variation

506

and (b) tower-top trajectory

507

508 **5.2 Effects of mass ratio on response mitigation**

509

Mitigation of the dynamic responses of the wind turbine subjected to an earthquake

510

excitation is also affected by the mass ratio of the TMD. In order to investigate the influence

511

of the mass ratio with respect to the mitigation effect, sensitivity analysis of the frequency and

512

damping ratios is performed on the TMDs with different mass ratios. The optimal frequency

513

and damping ratios corresponding to different mass ratios are presented in Fig. 18. The linear

514

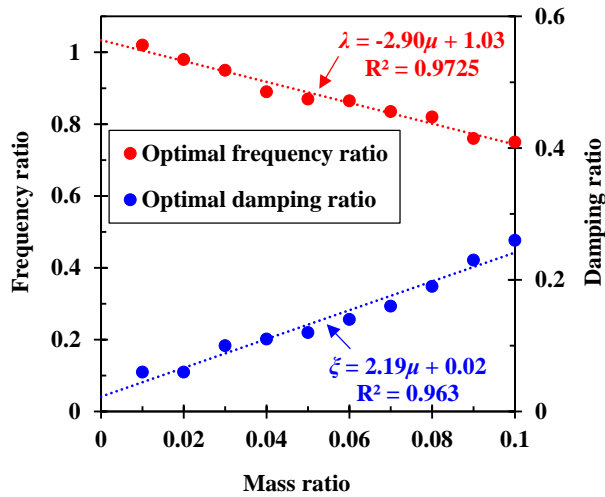
fitted lines of frequency and damping ratios can be used to obtain the optimal control

515

parameters corresponding to an arbitrary mass ratio without performing numerous parametric

516

analysis.



517

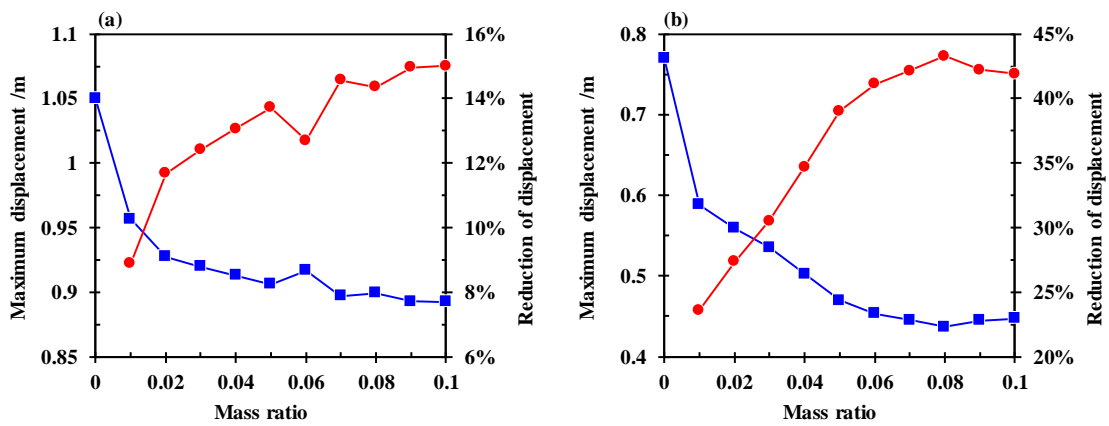
518 Fig. 18: Optimal frequency ratio and damping ratio versus mass ratio

519

520 It is noted that the optimal frequency ratio decreases with increase in the mass ratio,
 521 which is opposed to the variation trend of the optimal damping ratio. This is because the TMD
 522 located at the tower-top affects the natural frequency of the wind turbine system, which
 523 decreases with increase in the TMD mass. The mitigation of structural responses is achieved
 524 only when the tuned frequency of the TMD is close to the natural frequency of the OWT.
 525 Therefore, a smaller tuned frequency is required to obtain the best mitigation effectiveness for
 526 a heavier TMD.

527 Fig. 19 presents the mitigation effect of the TMDs with different mass ratios for both the
 528 coupled-loading and earthquake-only scenarios. For the coupled-loading scenario, the
 529 reduction in the maximum tower-top displacement increases with the mass ratio. The
 530 maximum tower-top displacement can be reduced by over 10% if the mass ratio of the TMD
 531 is larger than 0.01. It is noted that the alleviation effect of the TMD is insignificant when the
 532 mass ratio is larger than 0.06. The same observation can be made for the earthquake-only
 533 condition. In addition, the TMD with a mass ratio of 0.05 is able to reduce the maximum

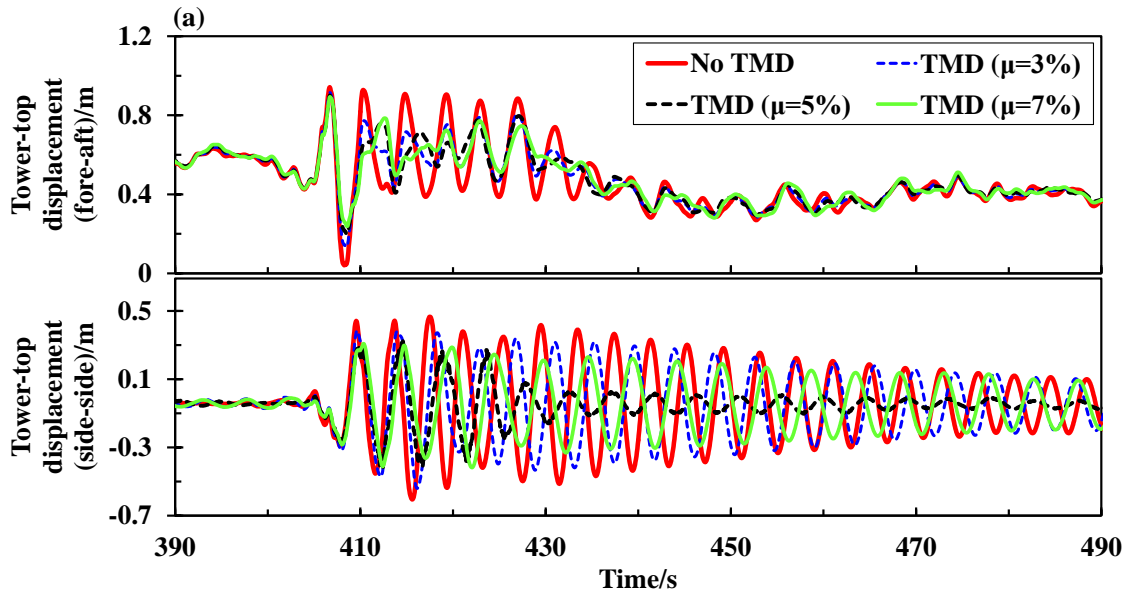
534 tower-top displacement by 39%, implying that TMD has much better effect on reducing the
 535 tower displacement for the earthquake-only condition. The reason is that the TMD is effective
 536 in mitigating tower vibration caused by an earthquake, and it is not capable of reducing the
 537 tower elastic deformation which is the major contribution of tower displacement for the
 538 coupled-loading condition.



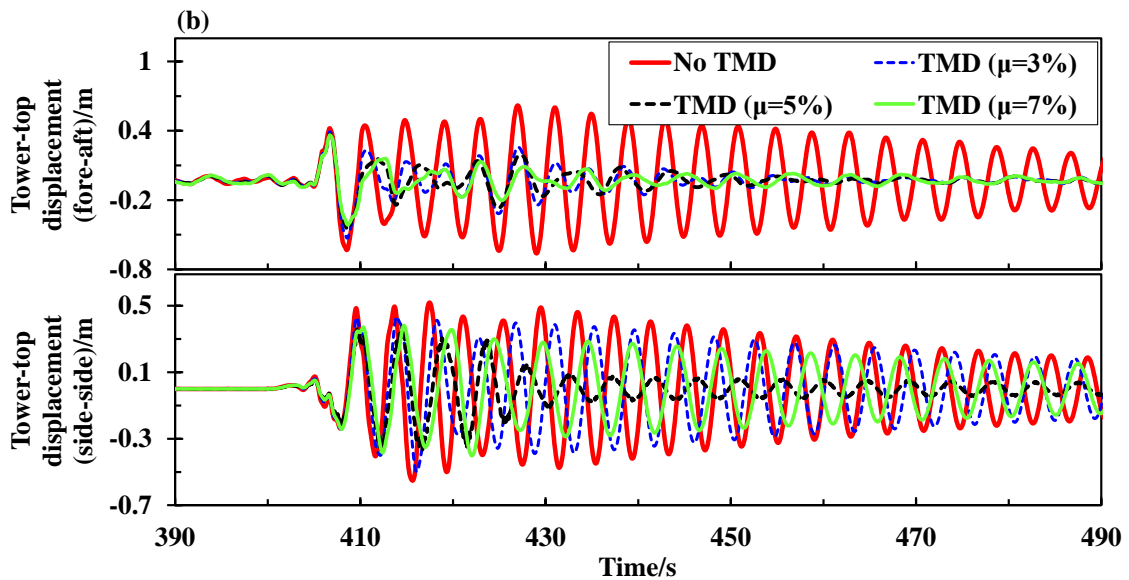
539
 540 Fig. 19: Optimal control parameters of the TMDs with different mass ratios. (a)

541 coupled-loading and (b) earthquake-only condition

542
 543 Fig. 20 presents the tower-top displacement variations of the OWT under the control of
 544 different optimal TMDs. Although the TMD with a mass ratio of 0.07 leads to a smaller
 545 resultant tower-top displacement, the TMD with a mass ratio of 0.05 can better alleviate the
 546 vibration in the side-side direction for both examined loading cases. The results indicate that
 547 the TMD with a lower mass ratio could be a better option.



548



549

550 Fig. 20: Comparison between tower-top displacements controlled by different TMDs for (a)

551 coupled-loading and (b) earthquake-only scenarios

552

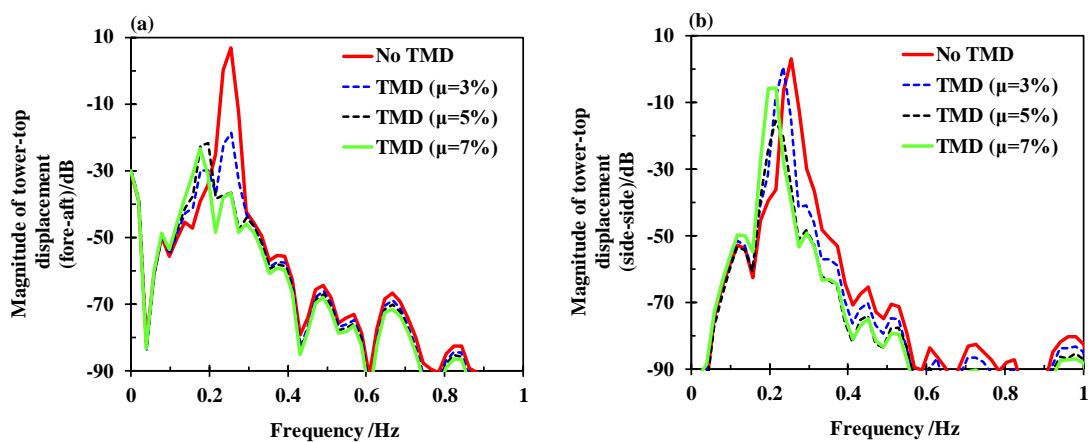
553 Fig. 21 presents the spectral curves of the tower-top displacement of the 10 MW OWT

554 for different optimal TMDs under the earthquake-only condition. The magnitude of the

555 fore-aft tower-top displacement at the 1st-order natural frequency (0.254 Hz) has been reduced

556 significantly by the TMDs. It is also noted that the vibration frequency decreases with the

557 increase in the TMD mass ratio. This further confirms the conclusion that a heavier TMD
 558 requires lower tuned frequency for the best mitigation effect. The TMDs have comparatively
 559 insignificant effect in mitigating the vibration of the side-side direction. Nonetheless, the peak
 560 magnitude of the TMD with a mass ratio of 0.05 is reduced significantly as the fore-aft
 561 displacement. This implies that the 0.05 mass ratio TMD is the optimum configuration for use
 562 in the mitigation control of the 10 MW OWT in earthquake-prone areas.



563
 564 Fig. 21: Magnitude of tower-top displacements of the (a) fore-aft and (b) side-side directions
 565 in frequency domain for the earthquake-only condition
 566

567 6 Discussions

568 The results of the coupled-loading condition indicate that the earthquake excitation
 569 increases the tower-top displacements in the fore-aft and side-side directions. An interesting
 570 observation is that the spectral magnitude of the fore-aft tower-top displacement at the
 571 first-order natural frequency under the coupled-loading condition is much smaller than that of
 572 the earthquake-only condition. It means that aerodynamic load is effective in mitigating the
 573 earthquake-induced vibration. The explanation to this result is that the interaction between

574 rotor and wind dissipates the energy from the earthquake excitation. The presence of
575 aerodynamic damping has positive effect in alleviating the tower vibration under an
576 earthquake event. Moreover, it is noted that the monopile beneath the seabed suffers much
577 stronger loads compared to the portion above the ground when the earthquake excitation is
578 examined. However, this observation was not present in the wind-wave condition. This is
579 attributed to the fact that there is only soil reaction force acting on the monopile beneath the
580 seabed in the wind-wave-only condition. But when the earthquake loading is considered, the
581 earthquake excitation makes significant contribution to the loads on the embedded monopile.
582 As a result, the slope of the bending moment varying with the monopile elevation is
583 significantly increased.

584 The results of the structural control analysis indicate that a TMD with rational
585 parameters is efficient in mitigating the tower vibration. The coupling between the dynamics
586 of the TMD and tower results in smaller tower responses, since the TMD dissipates energy
587 from the external excitations. The results also show that a heavier TMD with a smaller tuned
588 frequency is capable of achieving a larger mitigation on the tower responses. An explanation
589 to this observation is that a heavier TMD can dissipate more energy from the wind turbine
590 system. It is noted that the vibration mitigation is achieved only when the tuned frequency of
591 the TMD is close to the natural frequency of the wind turbine system which decreases with
592 the increase of TMD mass. This explanation is further confirmed by the spectral results of the
593 tower-top responses where the peak spectral magnitude corresponds to a lower frequency for
594 a heavier TMD.

595

596 **7 Conclusions**

597 This study investigates the use of TMD for the mitigation of the coupled responses of a
598 10 MW monopile OWT due to wind, wave and earthquake loadings. A generic tool, SCASCA,
599 has been developed to examine the coupling effects of multiple loadings. The comparisons of
600 the tool's outputs against Bladed and NREL-Seismic have validated the accuracy and
601 capability of SCASCA in performing fully coupled seismic analysis of OWTs. Furthermore,
602 SCASCA is also capable of performing structural control analysis using TMDs. The effect of
603 TMDs in mitigating the dynamic responses of the 10 MW monopile OWT subjected to a
604 scaled ground motion is investigated. This study offers the following conclusions:

605 (1) Comparisons of SCASCA's results against Bladed and NREL-Seismic have validated
606 its accuracy and capability in performing fully coupled seismic analysis. The generic
607 SCASCA is independent of the user's experience compared to FAST-Seismic for design of
608 different wind turbines. In addition, SCASCA addresses the limitation of FAST-Seismic in
609 considering the SSI effect and the limitation of Bladed in examining the vertical earthquake
610 excitation.

611 (2) Coupled responses of the 10 MW OWT due to wind, wave and earthquake loadings
612 are investigated while the SSI effect is examined using the nonlinear p - y curves. The
613 earthquake-induced vibration in the fore-aft direction is mitigated by the wind and wave
614 loadings due to the energy dissipation by aerodynamic and hydrodynamic damping. The
615 spectral magnitude at the first-order natural frequency of the fore-aft tower-top displacement
616 is mitigated from 22.6 dB to -20.4 dB. In addition, the tower vibration is dominated by the
617 earthquake as indicated by the Welch spectral results.

618 (3) The tower-top displacement and pile-cap bending moment increase, respectively, by
619 47.6% and 95.1% due to the earthquake loading mainly. The bending moment along the
620 embedded pile increases significantly with the soil depth, suggesting that the structural
621 strength of the embedded portion shall be enhanced against earthquake events.

622 (4) The TMD with appropriate control parameters is effective in mitigating the tower-top
623 displacements for both the coupled-loading and earthquake-only conditions. The standard
624 deviations of the fore-aft and side-side tower-top displacements are reduced by 70.4% and
625 56.8 % respectively for earthquake-only conditions. The large fluctuations caused by an
626 earthquake can be eliminated efficiently by the TMDs when the design parameters are
627 appropriately selected.

628 (5) Rational control parameters corresponding to different mass ratios of the TMD are
629 obtained by conducting a sensitivity analysis. It is noted that a heavier TMD requires a lower
630 tuned frequency to achieve a larger mitigation. The 0.05 mass ratio TMD mitigates the
631 maximum tower-top displacement by 13.7% and 39.0% for the coupled-loading and
632 earthquake-only conditions, respectively. The vibration magnitude corresponding to the
633 1st-order natural frequency is reduced significantly for both of the fore-aft and side-side
634 directions. The 0.05 mass ratio TMD is the recommended configuration for use in the
635 mitigation control of the 10 MW monopile OWT in earthquake-prone areas.

636

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645 **Nomenclature**

DOF	Degree of Freedom
DTU	Technical University of Denmark
EUR	Currency of the European Union
IEA	International Energy Agency
IRENA	International Renewable Energy Agency
LCoE	Levelised Cost of Electricity
NREL	National Renewable Energy Laboratory
OWT	Offshore Wind Turbine
PGA	Peak of Ground Acceleration
RNA	Rotor-Nacelle-Assembly
SCASCA	Seismic Coupled Analysis and Structural Control Architecture
SDG	Sustainable Development Goal
SSI	Soil-Structure Interaction
TMD	Tuned Mass Damper
a_{eq}	Earthquake acceleration
C_s	Soil damping
c_T	TMD damping
D_m	Diameter of the monopile
d_s	Structure displacement for soil force calculation
f_T	Tuned frequency of the TMD
f_{WT}	Dominant vibration frequency of the wind turbine
$F_{aero,i}$	Generalized aerodynamic loads of the i^{th} mode
$F_{eq,RNA}$	Seismic load of the RNA
$F_{hydro,i}$	Generalized hydrodynamic loads of the i^{th} mode
$F_{gra,i}$	Generalized gravity loads of the i^{th} mode
F_s	Soil force
F_{TMD}	TMD force
G_s	Soil shear modulus
H	Length of the support structure
m_i	Modal mass associated with the i^{th} mode

m_{RNA}	Total mass of RNA
m_{T}	TMD mass
m_{WT}	Wind turbine mass
k_{T}	TMD stiffness
k_{s}	Soil stiffness
q_i	Modal displacement of the i^{th} mode
\dot{q}_i	Modal velocity of the i^{th} mode
\ddot{q}_i	Modal acceleration of the i^{th} mode
v_s	Structure velocity for soil force calculation
x_{TMD}	TMD displacement
\dot{x}_{TMD}	TMD velocity
\ddot{x}_{TMD}	TMD acceleration
\ddot{x}_{N}	Nacelle acceleration
α_{N}	Nacelle angular velocity
β_s	Hysteresis damping ratio of the soil
λ	Tuned frequency ratio
μ	Tuned mass ratio
ω_m	First-order natural angular frequency of the support structure
ω_{N}	Nacelle translational velocity
ω_i	Angular frequency of the i^{th} mode
ρ_s	Soil density
$\rho(h)$	Mass density of the support structure at the height of h
$\phi_i(h)$	Normalized modal shape of the i^{th} mode of the support structure.
ξ_i	Damping ratio of the i^{th} mode
γ_i	Earthquake participation factor associated with the i^{th} mode

646

647 **References**

- 648 [1] Det Norske Veritas - Germanischer Lloyd, Energy transition outlook 2019: A global and
649 regional forecast to 2050. *DNV-GL report*, 2019
- 650 [2] Liang, Y., Yu, B., & Wang, L. (2019). Costs and benefits of renewable energy
651 development in China's power industry. *Renewable Energy*, 131, 700-712.
- 652 [3] People, planet and prosperity: Raising climate ambition through renewables.
653 <https://www.irena.org/publications/2019/Jul/People-Planet-and-Prosperity> [accessed 24
654 April 2020].
- 655 [4] International Energy Agency, Offshore wind outlook 2019. World Energy Outlook
656 Special Report. *IEA report*, 2020

- 657 [5] Ishihara, T., & Sarwar, M. W. (2008). Numerical and theoretical study on seismic
658 response of wind turbines. *European wind energy conference and exhibition 2008*,
659 Brussels, Belgium.
- 660 [6] Butt, U. A. & Ishihara, T. (2012). Seismic load evaluation of wind turbine support
661 structures considering low structural damping and soil structure interaction. *European*
662 *wind energy association annual event 2012*, Copenhagen, Denmark.
- 663 [7] Prowell, I., & Veers, P. (2009). Assessment of wind turbine seismic risk: Existing
664 literature and simple study of tower moment demand. *Sandia National Laboratories*
665 *Report*, Albuquerque, NM, USA.
- 666 [8] Haciefendioğlu, K. (2012). Stochastic seismic response analysis of offshore wind turbine
667 including fluid- structure- soil interaction. *The Structural Design of Tall and Special*
668 *Buildings*, 21(12), 867-878.
- 669 [9] Haenler, M., Ritschel, U., & Warnke, I. (2006). Systematic modelling of wind turbine
670 dynamics and earthquake loads on wind turbines. *European Wind Energy Conference &*
671 *Exhibition 2006*, Athens, Greece
- 672 [10]Zhao, X., & Maisser, P. (2006). Seismic response analysis of wind turbine towers
673 including soil-structure interaction. *Proceedings of the Institution of Mechanical*
674 *Engineers, Part K: Journal of Multi-body Dynamics*, 220(1), 53-61.
- 675 [11]Ritschel, U., Warnke, I., Kirchner, J., & Meussen, B. (2003). Wind turbines and
676 earthquakes. *World Wind Energy Conference 2003*, Cape Town, South Africa.
- 677 [12]Witcher, D. (2005). Seismic analysis of wind turbines in the time domain. *Wind Energy*,
678 8(1), 81-91.
- 679 [13]Santangelo, F., Failla, G., Santini, A. & Arena, F. (2016). Time-domain uncoupled
680 analyses for seismic assessment of land-based wind turbines. *Engineering Structures*, 123,
681 275-299.
- 682 [14]Asareh, M. A. & Prowell, I. (2011). Seismic loading for FAST. *National Renewable*
683 *Energy Laboratory, Technical Reports No. NREL/SR-5000-53872*, Golden, Colorado,
684 USA.
- 685 [15]Asareh, M. A., Prowell, I., Volz, J., & Schonberg, W. (2016). A computational platform
686 for considering the effects of aerodynamic and seismic load combination for utility scale
687 horizontal axis wind turbines. *Earthquake Engineering and Engineering Vibration*, 15(1),
688 91-102.
- 689 [16]Prowell, I., Elgamal, A., Uang, C., & Jonkman, J. (2010). Estimation of seismic load
690 demand for a wind turbine in the time domain. *National Renewable Energy Laboratory*
691 *(NREL), Technical Report No. NREL/CP-500-47536*, Golden, Colorado, USA.

- 692 [17]Prowell, I., Elgamal, A. W. M., & Jonkman, J. M. (2010). FAST simulation of wind
693 turbine seismic response. *National Renewable Energy Laboratory, Technical Report No.*
694 *NREL/CP-500-46225*, Golden, Colorado, USA.
- 695 [18]Asareh, M. A., Schonberg, W. & Volz, J. (2016). Effects of seismic and aerodynamic load
696 interaction on structural dynamic response of multi-megawatt utility scale horizontal axis
697 wind turbines. *Renewable energy*, 86, 49-58.
- 698 [19]Yang, Y., Ye, K., Li, C., Michailides, C. & Zhang, W. (2018). Dynamic behaviour of wind
699 turbines influenced by aerodynamic damping and earthquake intensity. *Wind Energy*, 21
700 (5), 303-319.
- 701 [20]Kim, D. H., Lee, S. G. & Lee, I. K. (2014). Seismic fragility analysis of 5 MW offshore
702 wind turbine. *Renewable Energy*, 65, 250-256.
- 703 [21]Mo, R., Kang, H., Li, M. & Zhao, X. (2017). Seismic fragility analysis of monopile
704 offshore wind turbines under different operational conditions. *Energies*, 10(7), 1037.
- 705 [22]Alati, N., Failla, G. & Arena, F. (2015). Seismic analysis of offshore wind turbines on
706 bottom-fixed support structures. *Philosophical Transactions of the Royal Society A*, 373
707 (2035), 20140086.
- 708 [23]Yang, Y., Li, C., Bashir, M., Wang, J., & Yang, C. (2019). Investigation on the sensitivity
709 of flexible foundation models of an offshore wind turbine under earthquake loadings.
710 *Engineering Structures*, 183, 756-769.
- 711 [24]Yang, Y., Bashir, M., Li, C., & Wang, J. (2019). Analysis of seismic behaviour of an
712 offshore wind turbine with a flexible foundation. *Ocean Engineering*, 178, 215-228.
- 713 [25]Jonkman, J. M., & Buhl Jr, M. L. (2005). FAST user's guide. *National Renewable Energy*
714 *Laboratory, Technical Report No. NREL/EL-500-38230*, Golden, Colorado, USA.
- 715 [26]Lackner, M. A., & Rotea, M. A. (2011). Passive structural control of offshore wind
716 turbines. *Wind energy*, 14(3), 373-388.
- 717 [27]Velarde, J. (2016). Design of monopile foundations to support the DTU 10 MW offshore
718 wind turbine, *Master thesis*, Norwegian University of Science and Technology.
- 719 [28]Jonkman, J. M. (2003). Modeling of the UAE wind turbine for refinement of FAST_AD.
720 *National Renewable Energy Laboratory, Technical Report No. NREL/TP-500-34755*,
721 Golden, Colorado, USA.
- 722 [29]C. Bak, R. Bitsche, A. Yde, T. Kim, M. H. Hansen, F. Zahle, et al. Light Rotor: the
723 10-MW reference wind turbine, *Proceedings of the European Wind Energy Association*
724 *(EWEA) Annual Event*, Copenhagen, Demark.
- 725 [30]Makris, N., & Gazetas, G. (1992). Dynamic pile-soil-pile interaction. Part II: Lateral and
726 seismic response. *Earthquake engineering & structural dynamics*, 21(2), 145-162.

- 727 [31]Ministry of Housing and Urban-Rural Development of China.(2010). Code for seismic
728 design of buildings. *Chinese Standard, GB 50011-2010*.
- 729 [32]Al Atik, L. & Abrahamson, N. (2010). An improved method for nonstationary spectral
730 matching. *Earthquake Spectra*, 26(3), 601-617.
- 731 [33]Jonkman, B. J. (2009). TurbSim user's guide: Version 1.50. *National Renewable Energy*
732 *Laboratory Technical Report No. NREL/TP-500-4619* , Golden, Colorado, USA.
- 733 [34]Mader, C. L. (2004). *Numerical modeling of water waves*. CRC press, Florida, USA.