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Influence of Foundation Damping on Offshore Wind Turbine Monopile Design Loads

Modeling hysteretic material damping from soil-foundation interaction

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Abstract

The dynamic behavior of offshore wind turbines (OWTs) must be designed considering stochastic load amplitudes and frequencies from waves and mechanical loads associated with the spinning rotor during power production. The proximity of the OWT natural frequency to excitation frequencies combined with low damping necessitates a thorough analysis of sources of damping; of these sources of damping, least is known about the contributions of damping from soil-structure interaction (foundation damping). This paper studies the influence of foundation damping on cyclic load demand for monopile-supported OWTs considering the design situations of power production, emergency shutdown, and parked conditions. The NREL 5MW Reference Turbine was modeled using the aero-hydro-elastic software FAST and included equivalent linear foundation stiffness and damping matrices. These matrices were determined using an iterative approach with FAST mudline loads as input to a soil-pile finite element software which calculates hysteretic material damping. Accounting for foundation damping in time history analysis can reduce cyclic foundation moment demand by as much as 30% during parked conditions, 25-33% during emergency shutdown, but only 2-3% reduction during power production without wave and wind misalignment. The calculated foundation damping from the emergency shutdown cases agreed with experimental testing performed in similar site conditions.

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25 **Nomenclature**

26	DE	Delaware
27	DLC	Design load case
28	DNV	Det Norske Veritas
29	ESS	Extreme Sea State
30	ETM	Extreme Turbulence Model
31	EWB	Extreme Wave Height
32	EWM	Extreme Wind Model
33	EWS	Extreme Wind Shear
34	HSS	High-speed shaft
35	IEC	International Electrotechnical Commission
36	NGI	Norwegian Geotechnical Institute
37	NOAA	National Ocean and Atmospheric Administration
38	NREL	National Renewable Energy Laboratory
39	NSS	Normal Sea State
40	NTM	Normal Turbulence Model
41	OWT	Offshore wind turbine
42	RWH	Reduced Wave Height
43	RWM	Reduced Wind Model
44	SSS	Severe Sea State
45	SWH	Severe Wave Height
46	TI	Turbulence intensity
47	ULS	Ultimate limit state
48	c_{mud}	Mudline damping matrix
49	$c_{\phi\phi}$	Mudline rotational dashpot
50	f	Natural frequency
51	g	Acceleration due to gravity
52	k_{mud}	Mudline stiffness matrix
53	k_{xx}, k_{yy}	Horizontal translational stiffness
54	$k_{x\phi}$	Coupled stiffness term
55	$k_{\phi\phi}$	Rotational stiffness
56	s_u	Undrained shear strength
57	u	Cyclic amplitude of mudline displacement
58	$v_{in}, v_{rated}, v_{out}$	Cut-in, rated, cut-out wind speed
59	x	Horizontal degree of freedom in fore-aft direction
60	y	Horizontal degree of freedom in side-to-side direction
61	z	Vertical degree of freedom
62	E	Modulus of elasticity
63	E_h	Hysteretic energy loss
64	$E[\cdot]$	Expected value
65	G_0	Shear modulus at small strains
66	H	Wave height
67	H_s	Significant wave height
68	H_{N-yr}	N -year wave height
69	F_x	Cyclic amplitude of horizontal mudline force
70	M_ϕ	Cyclic amplitude of mudline moment
71	T_p	Peak spectral period
72	$U_{10, hub}$	10-minute hub height wind speed
73	U_{hub}	Hub height wind speed
74	ϕ	Rotational degree of freedom
75	θ	Cyclic amplitude of mudline rotation
76	ν	Poisson's ratio
77	ρ	Density

78	σ	Standard deviation
79	ψ	Wave height reduction factor

80 **1 Introduction**

81 Approximately 15-20% of the capital cost of offshore wind farms can be attributed to the foundation
82 and support structure of offshore wind turbines (OWTs) [1–4]. OWT support structures are lightly
83 damped and must withstand highly uncertain offshore wind and wave loads with stochastic load
84 frequency and amplitude in addition to stochastic mechanical loads associated with the spinning rotor
85 during power production. OWTs are typically designed in a so-called “soft-stiff” frequency design
86 regime, wherein the first natural frequency is designed to lie between the 1P and 3P blade rotation
87 frequency bands. Because a stiffer structure implies higher costs (due to increased structural material
88 requirements), it is desirable for the first natural frequency to be near, but safely above the 1P frequency
89 band (DNV suggests a clearance of $\pm 10\%$ of blade rotation frequency bands [5]). The close proximity
90 to excitation frequencies combined with the low amount of damping present in the support structure
91 necessitates a thorough analysis of not only the stiffness, but also the various sources of damping within
92 the OWT system: structural, hydrodynamic, aerodynamic, soil-foundation interaction (foundation), and
93 sometimes tuned mass damper. More damping reduces structural demand, and therefore the material
94 costs.

95 DNV (2014) recommends soil damping to be considered in the design phase, but no recommended
96 practice for estimating soil damping is suggested; therefore, foundation damping is typically included
97 in the overall structural Rayleigh damping in OWT design [5–7]. While geotechnical finite element
98 models are the most realistic way of assessing foundation damping, they are computationally expensive
99 for structural time history analysis [8]; consequently, other (simplified, less time consuming) ways of
100 including foundation damping in OWT design include the use of a dashpot [9] or by using a macro-
101 element which uses kinematic hardening [10].

102 Foundation damping is probably the least understood and most complex of all the sources contributing
103 to the OWT system damping, and there is no consensus on its importance in an OWT design context
104 with respect to the other sources of damping [6,11–13]. Previous work [6] indicated that for a monopile-

105 supported OWT subjected to extreme storm loads, cyclic design loads for the pile foundation can be
106 reduced by as much as 10% when foundation damping is incorporated into OWT modeling.

107 Because over 80% of installed OWTs are supported by monopile foundations [14], this paper evaluates
108 the relative impact of hysteretic foundation damping on monopile foundation loads for power
109 production, emergency stop, and parked storm conditions. The purpose of the paper is not to propose a
110 specific methodology, rather its intention was to better understand how foundation damping affects
111 foundation loads. The methodology for accounting for foundation damping continues to develop and
112 the intention of the paper is to show potential for foundation load reduction for various load cases and
113 not to identify specific reduction quantities (which may vary based on subsurface conditions, metocean
114 conditions, as well as turbine model and size). While studies have shown that radiation damping is small
115 for frequencies below 1 Hz [11,12,15] further research is needed to understand how radiation damping
116 contributes to the foundation damping. This paper, however, only considers the contribution of
117 hysteretic material damping from pile-soil interaction.

118 The NREL 5MW Reference Turbine (“NREL 5MW”) [16] was analyzed using the open-source
119 aeroelastic simulation program FAST (v7, [17]), considering the IEC 61400-3 design load cases (DLCs)
120 [18] to dictate wind, wave, and turbine conditions. Soil-structure interaction was modeled in FAST via
121 mudline stiffness and damping matrices which were calculated using the results from the soil-pile
122 software INFIDEL (INFInite Domain Elements) [19,20]. The NREL 5MW was analyzed considering
123 a North Sea site described by [6] and due to the lack of available metocean data for that location, the
124 approximately equivalent environmental site conditions from the National Ocean and Atmospheric
125 Administration (NOAA) buoy sited off the coast of Delaware in the U.S. Atlantic Ocean were used to
126 determine input wave heights and wind speeds for the DLCs.

127 Section 2 illustrates the analysis process used to determine the influence of foundation damping on
128 cyclic mudline demand, with further discussion of how foundation stiffness and damping matrices were
129 calculated. Section 3 describes the OWT model used to determine monopile loads and the calculation
130 of foundation stiffness. The DLCs selected for analysis, and how each design situation (power

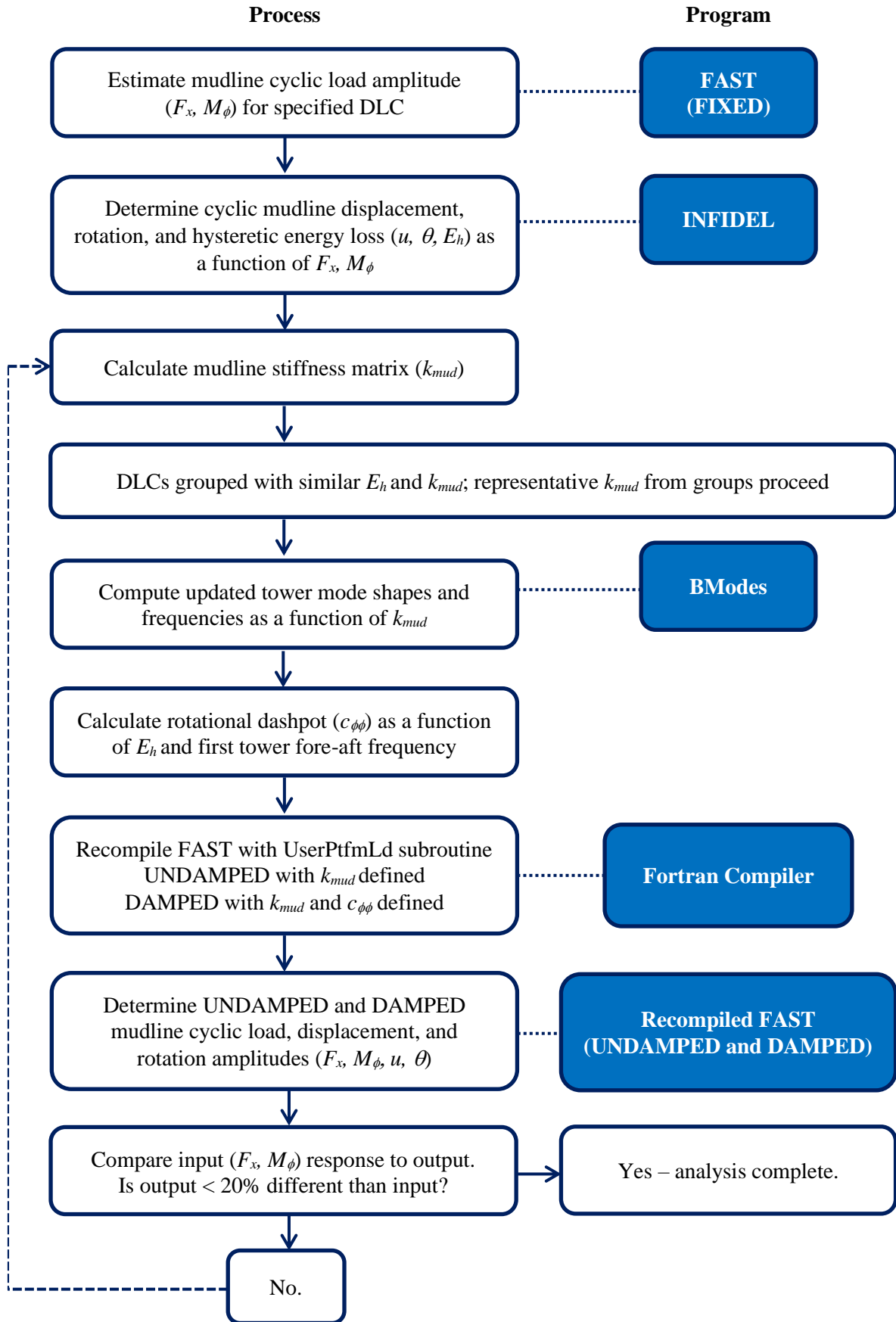
131 production, emergency shutdown, and parked) was modeled in FAST (Figure 1) is described in Section
132 4. The results are given in Section 5 and a summary and conclusions in Section 6.

133 **2 Methodology**

134 **2.1 Offshore Wind Turbine Analysis Procedure**

135 Several different methods and numerical tools are used in the analysis process of this paper (Figure 1)
136 to define the impact of foundation damping on cyclic foundation demand. The approach described is
137 not recommended as a methodology for design per se, but as an ad hoc approach for evaluating the
138 importance of foundation damping for OWTs in various design situations. Each Design Load Case
139 (DLC, described in Section 4) was initially analyzed using the aeroelastic offshore wind turbine
140 simulation code FAST [17] (further described in Section 4.4) assuming a fixed connection of the
141 substructure at the mudline to estimate cyclic foundation load demand. The cyclic foundation demand
142 was then used as input to INFIDEL [19] (described below) to compute the foundation response
143 (displacement, rotation, and hysteretic energy loss, described in Section 2.2) and the corresponding
144 equivalent linear foundation stiffness and damping matrices. Given the mudline stiffness matrix, new
145 tower mode shapes and frequencies were calculated using the NREL-distributed program BModes [21]
146 and the FAST analysis was repeated with foundation stiffness and damping matrices. Two versions of
147 the OWT model were analyzed for each DLC, one version including the mudline dashpot (“DAMPED”
148 in Figure 1) and one without (“UNDAMPED”), to determine the amplitude of cyclic mudline loads,
149 displacements, and rotations. The impact of foundation damping was assessed by the difference in
150 cyclic foundation demand between the undamped and damped models.

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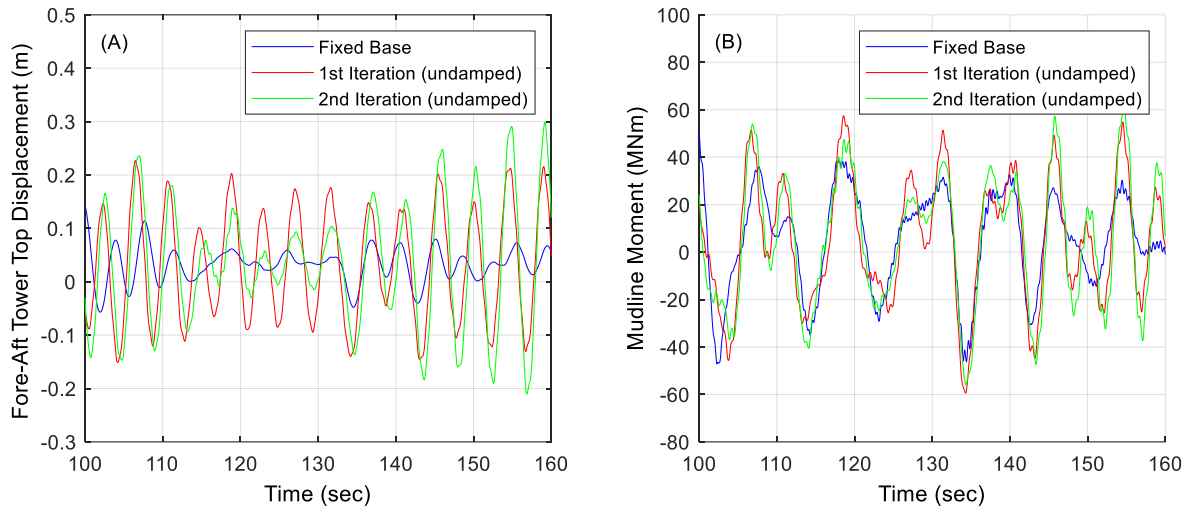


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153

Figure 1 Flowchart of foundation damping analysis process

154 Because the analysis process (Figure 1) used to define the impact of foundation damping on cyclic
155 OWT monopile design loads is relatively time-consuming, the DLCs were grouped according to similar
156 hysteretic energy loss and mudline stiffness matrices. If the mudline cyclic load amplitudes from the
157 fixed-base FAST analysis and the flexible-base FAST analysis differed by more than 20%, the mudline
158 stiffness matrix was updated and the flexible-base FAST analysis was repeated to ensure the soil-pile
159 properties were approximately compatible with the cyclic load amplitude. The 20% difference in
160 foundation response was chosen as rational ad-hoc criteria, giving an error in the foundation stiffness
161 smaller than 20% (for the purposes of design, it is recommended that the difference in foundation
162 response be less than 20%; inclusion of foundation flexibility in the initial run would reduce iterations).
163 The average difference in response after the first iteration (using fixed-base mudline response to
164 estimate mudline stiffness parameters) was approximately 11%; for cases which exceeded the 20%
165 criteria, a second iteration decreased the average difference in response to approximately 1%. It should
166 be noted that for load cases which use random seeding, the variation in mudline response based on
167 seeding alone can be at least 3%. The resulting small error in eigenfrequency using the criteria cited
168 above was considered negligible for our analysis (it should be noted that the natural frequency from the
169 representative mudline stiffness matrices was on average 0.23 Hz and varied by less than 0.01 Hz). The
170 relative change in natural frequency can be seen in one example time history of tower top displacement
171 (stochastic load case DLC 6.1a, refer to Section 4 and Figure 2A), where there is a significant phase
172 and amplitude difference between the fixed base and first iteration flexible foundation case; with a
173 second iteration, the difference in frequency and amplitude is substantially minimized. Methodology
174 for the analysis herein relied on differences in foundation response: the difference in cyclic mudline
175 moment amplitude from the fixed base to flexible base case was 39% but differed by only 2% after the
176 second iteration (Figure 2B).



177

178

Figure 2 Effect of methodology iteration on (A) tower top displacement and (B) mudline response

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In a proper design, further iteration would be required to reduce the difference in mudline response.

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The aero-hydro-elastic simulation code FAST uses Blade Element Momentum (BEM) theory to calculate wind loads on OWT blades and includes the effects of the spinning rotor on the support structure dynamics. Time histories of the dynamic behavior of the support structure is computed by superposition of the first and second fore-aft and side-to-side mode shapes. These mode shapes were determined using the NREL-distributed software BModes and are defined by sixth-order polynomial coefficients in the FAST tower property file.

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Depending on the requirements of the DLC, wind can be defined as either steady or turbulent and waves as regular or irregular. Turbulent wind conditions were modeled in FAST using the Kaimal spectrum. Linear wave theory was used to generate wave conditions using the JONSWAP spectrum and Wheeler stretching. The effects of breaking waves were neglected.

190

191

2.2 Cyclic Load Amplitude for Computing Foundation Stiffness and Damping

192

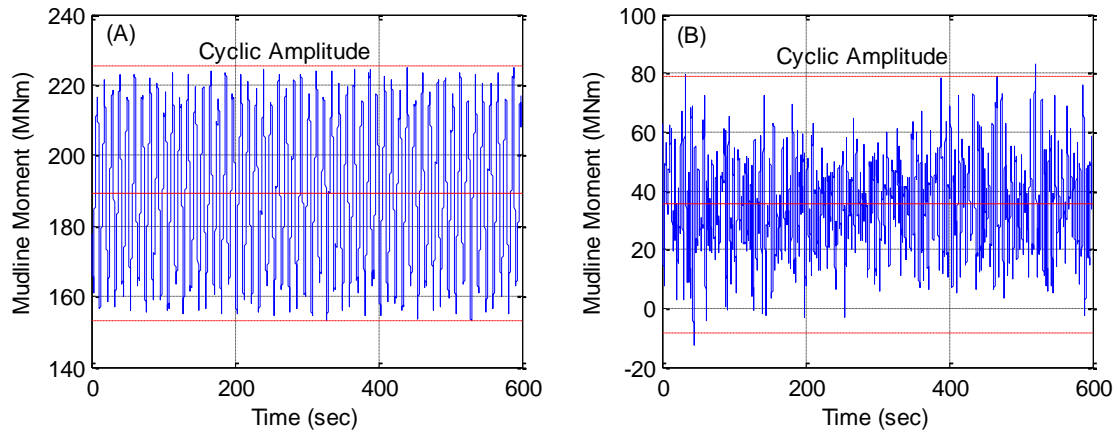
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Figure 3 show examples of a periodic and stochastic time histories of foundation moment loading. The loading can be visualized as a harmonic loading with an average component and a cyclic amplitude

194 component [22]. The definition of the cyclic amplitude influences the calculations of stiffness and
195 damping – higher cyclic amplitudes lead to higher damping but lower stiffness.

196 The behavior of cyclically loaded clays is intricate, e.g. the stiffness is more influenced by cyclic load
197 amplitude and number of cycles rather than maximum response [23], as long as the maximum response
198 is much lower than the response corresponding to the ultimate capacity. The ultimate peak load a
199 foundation can sustain is affected by the preceding cyclic loading; thus in an ultimate state load case,
200 the cyclic load amplitude has to be accounted for in addition to the peak load. The cyclic stiffness of
201 soil is also affected by the average load component and there are methods developed to account for this
202 (e.g. [23,24]). While there are procedures for establishing cyclic stress-strain curves accounting for the
203 average load components, more research is needed to understand the effect of average load components
204 and number of load cycles on soil material damping [25].

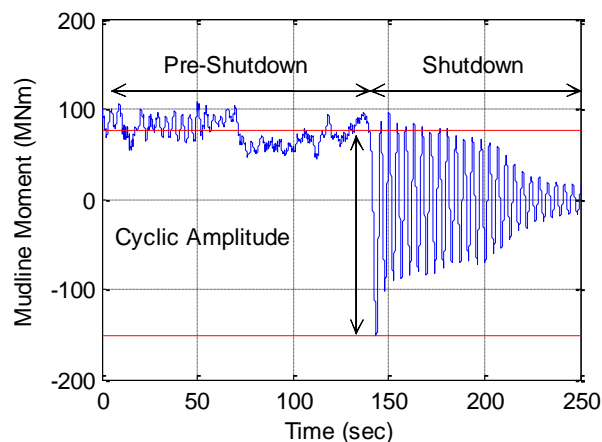
205 For regular wave train and steady wind DLCs, estimating cyclic load amplitude was straightforward
206 (due to the periodic nature of the time history output, half of the difference between maximum and
207 minimum response, Figure 3A); for stochastic time histories (with irregular wave trains or turbulent
208 wind fields), these loads were estimated as three times the standard deviation of the response (3σ , Figure
209 3B [6]). Using three times the standard deviation can be partially justified by the fact that the average
210 load component was not considered (shown with red solid lines Figure 3) which would decrease the
211 foundation stiffness, and the objective of the paper was to understand the order of magnitude of
212 foundation damping has on the foundation loads. In practice, the cyclic and average load amplitude
213 variation in a time history is determined by load cycle counting [26] to estimate the soil degradation
214 and corresponding foundation stiffness and damping.



215

216 **Figure 3 Example time histories indicating load amplitude for (A) regular wave train/steady wind (DLCs 1.5, 6.1c,**
 217 **6.2b) and (B) stochastic time histories of mudline moment (DLCs 1.1, 1.3, 1.6a, 1.6b, 6.1a, 6.2a)**

218 The emergency shutdown design situation required a somewhat different approach due to the
 219 nonstationary nature of the response. In this case, the cyclic amplitude of concern was taken to be the
 220 difference between the mean pre-shutdown response and the absolute minimum response (Figure 4).
 221 Using mudline stiffness and damping parameters determined for the shutdown cyclic amplitude also for
 222 the pre-shutdown portion of the analysis is assumed to have limited influence on our results. Using the
 223 maximum amplitude in the beginning of the shutdown process likely overestimates the foundation
 224 damping to some extent. Since the focus was to evaluate the effect of damping on the maximum mudline
 225 moment occurring at shutdown, it was chosen as simple approach to obtain a first order estimate of
 226 foundation damping. Further refinement of the foundation damping estimates is recommended in a real
 227 design.



228

229 **Figure 4 Example emergency shutdown time history of mudline moment during rated wind speeds (DLC 5.1)**

230 2.3 Foundation Stiffness and Damping

231 The NGI-developed INFIDEL software has been used to compute foundation stiffness and damping
 232 matrices for use with FAST [6,19,20]. To the authors' knowledge INFIDEL is the only finite element
 233 code which can account for the variation of soil damping with the shear strain distribution around the
 234 pile and integrate it to a foundation damping value. The method of computation is very efficient, making
 235 it practical for offshore wind turbine design where very many load cases with different load
 236 combinations between moment, horizontal and vertical load need to be analyzed. INFIDEL defines an
 237 axisymmetric three-dimensional soil-pile space with infinite extents. It models cyclic soil behavior with
 238 a nonlinear constitutive model based on user defined stress-strain curves and soil damping curves [27].
 239 The INFIDEL soil-foundation analysis is an equivalent linear approach based on monotonic stress-
 240 strain curve which represents the cyclic stress-strain response of the soil. For each load level an
 241 equivalent cyclic strain is computed which gives the stress and the damping in each soil element. The
 242 software is not iterative and does not include kinematic hardening. Linear elastic pile behavior was
 243 assumed. Figure 6 shows the input shear modulus at small strains G_0 , undrained shear strength s_u , and
 244 Poisson's ratio ν used in the INFIDEL model. The Poisson's ratio for the pile was 0.3. The soil-pile
 245 model and methodology of INFIDEL are described in detail by Carswell et al.[6].

246 The cyclic foundation load amplitudes (H , M) from FAST were used as input to INFIDEL to obtain
 247 mudline displacement u and rotation θ . In order to compute the equivalent linear stiffness elements (k_{xx} ,
 248 $k_{x\phi}$, $k_{\phi\phi}$) comprising k_{mud} , accounting for soil non-linearity, two runs of INFIDEL were required:

- 249 1) Using cyclic mudline load amplitudes H and M (denoted $F_{x,1}$ and $M_{\phi,1}$ in Eq. 2) to obtain cyclic
 250 mudline displacement and rotation amplitudes u and θ (denoted u_1 and θ_1 in Eq. 2), and
- 251 2) Using just the horizontal mudline shear amplitude H ($F_{x,2}$ in Eq. 2) but setting $M = 0$ to obtain
 252 a second set of displacement and rotation amplitudes (u_2 and θ_2).

253 The displacement and rotation results were then used in conjunction with the input loads to determine
 254 k_{mud} , calculated per Zaaijer [28] using

$$\begin{pmatrix} k_{xx} \\ k_{x\phi} \\ k_{\phi\phi} \end{pmatrix} = \begin{pmatrix} u_1 & \theta_1 & 0 \\ 0 & u_1 & \theta_1 \\ u_2 & \theta_2 & 0 \end{pmatrix}^{-1} \begin{pmatrix} F_{x,1} \\ M_{\phi,1} \\ F_{x,2} \end{pmatrix} \quad (1)$$

255 where k_{xx} is the horizontal translational stiffness, $k_{\phi\phi}$ is the rotational stiffness, $k_{x\phi}$ is the cross-term of
 256 k_{mud} , and assuming that k_{mud} is symmetric.

257 The hysteretic energy loss in each soil element, corresponding to the area of one load-strain cycle
 258 (hysteresis loop) is summed over all the whole soil volume to compute a total hysteretic energy loss for
 259 the foundation E_h , which is converted with the method described in Carswell et al.[6] into a viscous
 260 rotation dashpot value $c_{\phi\phi}$ by

$$c_{\phi\phi} = \frac{E_h}{2\theta^2 \pi^2 f} \quad (2)$$

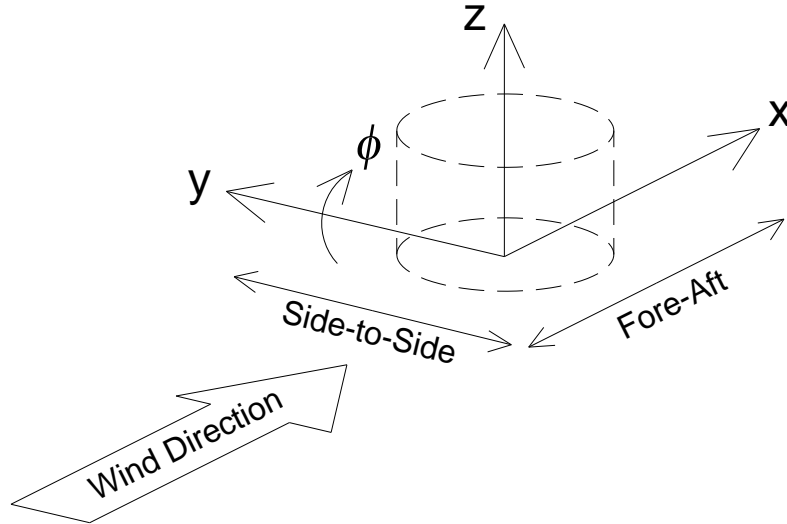
261 where θ is the mudline rotation amplitude in rad, f is the loading frequency in Hz, taken here to be the
 262 first (fore-aft) natural frequency of the NREL 5MW.

263 The choice of frequency used for converting the hysteretic foundation damping into a viscous dashpot
 264 is based on the assumption that largest contribution to the cyclic moment at mudline is due to vibration
 265 at first natural frequency. This assumption likely leads to foundation damping that is overestimated for
 266 the frequencies near the selected frequency, while respectively underestimated at frequencies above and
 267 beneath the selected frequency. The stiffness and damping characteristics of soil, simultaneously
 268 subjected to cyclic loads of different amplitudes, frequencies and directions, is complex [29] and further
 269 work is needed to understand this in an OWT design context.

270 There is no standardized procedure for decomposing foundation damping into different degrees of
 271 freedom. While foundation damping is assumed to occur as a function of combined translation and
 272 rotation, the difficult of proportioning of converting foundation damping into multiple degrees of
 273 freedom is unknown. For the cases analyzed for this particular turbine and foundation, foundation
 274 rotation contributes on average 13 times more to the horizontal response at the nacelle than the
 275 foundation horizontal displacement, therefore the hysteretic energy loss was modelled with a rotational

276 dashpot to capture this significant influence of the rotational response of the foundation when compared
 277 to horizontal displacement.

278 The elements of k_{mud} and c_{mud} are then used as input to the user defined subroutine *UserPtfmLd* in FAST,
 279 which calculates “platform” loads (in this case, loads at the mudline). Perfect fixity was assumed in the
 280 vertical z -direction as well as in torsion (rotation about the z -axis, Figure 5).



281

282

Figure 5 Degrees of freedom in FAST user subroutine

283 Due to the sign conventions inherent in FAST, the stiffness matrix defined in *UserPtfmLd* is defined as

$$k_{mud} = \begin{pmatrix} k_{xx} & 0 & 0 & 0 & -k_{x\phi} & 0 \\ 0 & k_{xx} & 0 & k_{x\phi} & 0 & 0 \\ 0 & 0 & 0 & 0 & 0 & 0 \\ 0 & k_{x\phi} & 0 & k_{\phi\phi} & 0 & 0 \\ -k_{x\phi} & 0 & 0 & 0 & k_{\phi\phi} & 0 \\ 0 & 0 & 0 & 0 & 0 & 0 \end{pmatrix} \quad (3)$$

284 Wherein $k_{xx} = k_{yy}$ and $k_{x\phi} = k_{\phi x}$ due to radial symmetry; the mudline damping matrix is defined as

$$c_{mud} = \begin{pmatrix} 0 & 0 & 0 & 0 & 0 & 0 \\ 0 & 0 & 0 & 0 & 0 & 0 \\ 0 & 0 & 0 & 0 & 0 & 0 \\ 0 & 0 & 0 & c_{\phi\phi} & 0 & 0 \\ 0 & 0 & 0 & 0 & c_{\phi\phi} & 0 \\ 0 & 0 & 0 & 0 & 0 & 0 \end{pmatrix}. \quad (4)$$

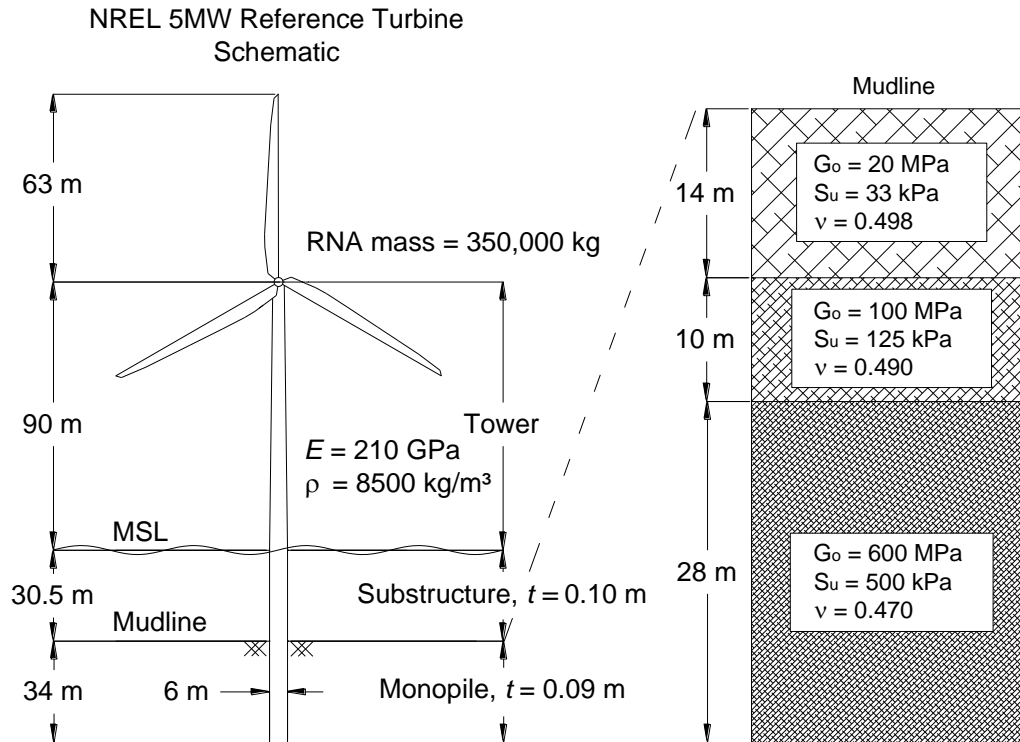
285 To reduce the number of analyses we did not define different matrices for the stiffness, k_{mud} and damping
 286 c_{mud} for every different yaw angle or wind speed bin within a DLC, rather a representative k_{mud} (and c_{mud} ,
 287 as applicable) were selected such that individual analysis cases within a DLC could be analyzed using
 288 one k_{mud} and therefore also one corresponding compiled version of FAST. FAST has to be recompiled
 289 each time one changes the foundation stiffness.

290 3 NREL 5MW Reference Turbine and Foundation

291 The NREL 5MW Reference Turbine (NREL 5MW) was used for the analysis, assuming the
 292 substructure, foundation, and soil (clay) properties shown in Figure 6 and Table 1. While much of the
 293 recently published research focuses on larger turbine models, the NREL 5MW was used in this study
 294 because it is more representative of currently installed OWTs. The analysis procedure in this study is
 295 similar to the prior study of foundation damping [9] but considers different environmental site
 296 parameters. Because metocean data were unavailable for the North Sea site used in this analysis [9,30]
 297 (for which the authors had soil information such as undrained shear strength s_u , Poisson's ratio ν , and
 298 shear modulus at small strains G_0), metocean data from the Delaware data buoy were used to determine
 299 input wave heights and wind speeds for DLC analysis in FAST. The authors believed that using a
 300 specific North Sea offshore site was critical because it facilitated comparison with the damping studies
 301 performed in literature which are primarily in clayey soils [9,12,13,15,31,32].

302 **Table 1 Structural properties of the NREL 5MW Reference Turbine, substructure, and foundation assuming linearly**
 303 **tapering properties**

Location on Support Structure	Diameter, Thickness
Tower top	3.87 m, 0.019 m
Tower base (MSL)	6 m, 0.027 m
Substructure	6 m, 0.10 m
Monopile	6 m, 0.09 m



304

305

Figure 6 NREL 5MW Reference Turbine Site

306 4 Offshore Wind Turbine Design Load Cases

307 The design load cases (DLCs) described in the OWT design standard IEC 61400-3 [18] are used for the
 308 vast majority of OWT designs [5,33]. The DLCs for power production, emergency shutdown, and
 309 parked conditions (extreme storm loading) were used to assess the effects of foundation damping on
 310 the design loads (Table 2). In an effort to reduce the computational expense of the process described in
 311 Figure 1, many DLCs were omitted from this study, with scope limited as follows: the fatigue limit state
 312 and examination of fatigue life were not considered; due to the higher wind speeds and braking in
 313 shutdown DLCs which impact the support structure more, startup DLCs were not considered; normal
 314 shutdown DLCs were not included because they do not include wind turbulence (which is considered
 315 in emergency shutdown DLCs). Foundation damping is typically much lower than aerodynamic
 316 damping for operational OWT conditions [13,34]; therefore, normal power production DLCs are
 317 adequate for assessing the influence of foundation damping on the operational OWT system and power

318 production with fault occurrence was excluded. DLCs for transport, assembly, maintenance and repair,
 319 were also excluded.

320 **Table 2 IEC offshore wind turbine design load cases analyzed**

Design Situation	Load Case	Wind Speed	Wave Height	Yaw Misalignment	Limit State
1) Power production	1.1	NTM $v_{in} < U_{10,hub} < v_{out}$ TI = 11%	NSS $H_s = E[H_s U_{10,hub}]$	0°	ULS
	1.3	ETM $v_{in} < U_{10,hub} < v_{out}$ TI = 16%	NSS $H_s = E[H_s U_{10,hub}]$	0°	ULS
	1.5	EWS $v_{in} < U_{10,hub} < v_{out}$	NSS $E[H_s U_{10,hub}]$	0°	ULS
	1.6a	NTM $v_{in} < U_{10,hub} < v_{out}$ TI = 11%	SSS $H_s = H_{s,50-yr} U_{10,hub}$	0°	ULS
	1.6b	NTM $v_{in} < U_{10,hub} < v_{out}$ TI = 11%	SWH $H = H_{50-yr}$	0°	ULS
5) Emergency Shut Down	5.1	NTM $v_{rated}, v_{out} \pm 2\text{m/s}$ TI = 11%	NSS $E[H_s U_{10,hub}]$	0°	ULS
6) Parked Conditions	6.1a	EWM $U_{hub} = U_{10,50-yr}$ TI = 11%	ESS $H_s = H_{s,50-yr}$	$\pm 8^\circ$	ULS
	6.1c	RWM $U_{hub} = 1.1U_{10,50-yr}$	EWH $H = H_{50-yr}$	$\pm 15^\circ$	ULS
	6.2a	EWM $U_{hub} = U_{10,50-yr}$ TI = 11%	ESS $H_s = H_{s,50-yr}$	$\pm 180^\circ$	ULS Abnormal
	6.2b	EWM $U_{hub} = 1.4U_{10,50-yr}$	RWH $H = \psi H_{50-yr}$	$\pm 180^\circ$	ULS Abnormal

KEY: NTM = Normal Turbulence Model; ETM = Extreme Turbulence Model; Extreme Wind Shear; RWM = Reduced Wind Model; EWM = Extreme Wind Model; NSS = Normal Sea State; SSS = Severe Sea State; SWH = Severe Wave Height; ESS = Extreme Sea State; EWH = Extreme Wave Height; RWH = Reduced Wave Height; TI = Turbulence Intensity; ULS = Ultimate Limit State; v_{in} = cut-in wind speed; v_{out} = cut-out wind speed; $U_{10,hub}$ = hub height wind speed (10-min average); v_{rated} = rated wind speed; U_{hub} = hub height wind speed; $U_{10,50-yr}$ = 50-year hub height wind speed (10-min average); H_s = significant wave height; $H_{s,50-yr}$ = 50-year significant wave height; H = wave height; ψ = wave height reduction factor.

321 The details of how the DLCs shown in Table 2 were implemented in FAST are described below. Wind-
 322 wave misalignment (as studied by Tarp-Johansen et al. [12]) was excluded since it was considered more
 323 significant for OWT fatigue life assessment than for foundation ULS design loads; consequently, wind
 324 and waves were modeled co-directionally in one direction for all DLCs.

325

326 **4.1 Power Production**

327 Power production DLCs are relevant for wind speeds within the cut-in and cut-out wind speeds (3 m/s
328 and 25 m/s for the NREL 5MW, respectively); only the rated wind speed (11.4 m/s) and cut-out wind
329 speed (25 m/s) cases were examined in this paper [16].

330 The ULS DLC 1.4 case was omitted, as we understand that extreme wind direction change primarily
331 tests the OWT controls and not the integrity of the support structure.

332 Power production DLCs were run in FAST using the simple pitch control and variable speed control
333 provided in the user-defined subroutines. The Thevenin-equivalent, 3-phase induction generator model
334 built into FAST was used.

335 With the exception of DLC 1.5, all power production DLCs use the Normal Turbulence Model (NTM).

336 The Extreme Wind Shear (EWS) in DLC 1.5 was used with a steady (non-turbulent) wind input file in
337 FAST, considering only vertical wind shear. FAST does not define horizontal wind shear in the steady
338 wind input files and was thus neglected. Because the steady wind input file is only capable of modeling
339 linear or power law wind shear, the power law wind shear exponent defined for EWS was taken as the
340 average estimated power law exponent over the rotor disk for each second of the 12 second transient
341 EWS event [5].

342 **4.2 Emergency Shutdown**

343 An emergency shutdown occurs when a safety supervisor system turns off the OWT to prevent damage.
344 We model the emergency shutdown with a simplified version of the comprehensive procedure described
345 by [35] as follows:

- 346 • The generator was turned off at $t = 200$ s into the time history simulation.
- 347 • Pitch control was overridden at time = 200 s and the blade pitch was set to 90° (feathered blades
348 for the NREL 5MW) at the rated limit of $8^\circ/\text{sec}$ [16].
- 349 • The simple high-speed shaft (HSS) brake was then applied 0.6 s after the blade pitch reached
350 90° , which is the time it takes the NREL 5MW brake to fully engage after deployment [16].

351 The emergency shutdown case used the same wind field and wave trains as DLC 1.1.

352 4.3 Parked Conditions

353 Several DLCs were omitted from the parked conditions due to similarity:

- 354 • DLC 6.2b has the same environmental conditions as DLC 6.1b, but considers loss of electrical
355 connection; DLC 6.1b was omitted, as it was assumed that the loss of electrical connection
356 (blade pitch less than 90° , $\pm 180^\circ$ yaw misalignment) would cause a more critical load condition.
- 357 • DLC 6.3a and 6.3b were similar to DLC 6.2a and 6.2b (respectively), but with a smaller wave
358 height and smaller range of yaw misalignment; DLCs 6.3a and 6.3b were omitted.

359 The DLCs in the parked condition design situation were all modeled considering parked (i.e.,
360 nonrotating) blades. In DLCs 6.1a and 6.1b the blade pitch is 90° (feathered blade position), and in
361 DLCs 6.2a and 6.2b the blade pitch is 0° due to loss of electrical network connection (and assumedly
362 therefore loss of pitch control).

363 4.4 Site Specific Environmental Loads

364 The site specific input to the DLCs are based on the environmental site conditions (summarized in Table
365 3) taken from the NOAA data buoy 44009 [36] located off the coast of Delaware (DE). The DE buoy
366 data include the 1-hr average wind speed at 5 m above sea level and 1-hr average significant wave
367 height H_s from 1986-2014. Wind speed at hub height was calculated using the power law for vertical
368 wind shear, with an exponent of 0.14 per DNV [5].

369 **Table 3 Wave height and wind speed at particular mean return periods for the Delaware data buoy site used for**
370 **parked design situation**

Site Condition	Value
5-year Significant Wave Height, $H_{s,5-yr}$	7.1 m
50-year Significant Wave Height, $H_{s,50-yr}$	8.1 m
50-year Wind Speed at Hub Height (1-hr average), $U_{1hr,50-yr}$	37 m/s

371 Wind speeds and significant wave heights used for the DLCs in the parked design situation were
372 calculated using a Generalized Extreme Value (GEV) distribution fit to the maximum annual wind
373 speed and wave height from 1986-2014. This approach is conservative, as the maximum wind speed

374 and maximum wave height are not necessarily simultaneous. The 5-year significant wave height $H_{s,5-yr}$
 375 was used in the Reduced Wave Height model (DLC 6.2b) to reduce the 50-year wave height $H_{s,50-yr}$ by
 376 the factor ψ , which is a ratio of $H_{s,5-yr}/H_{s,50-yr}$.

377 Peak spectral period T_p was calculated similarly to Valamanesh et al. [37], where

$$T_p = 1.05 \left(11.1 \sqrt{\frac{H_s}{g}} \right) \quad (5)$$

378 where H_s is the significant wave height and g is the acceleration due to gravity.

379 The sea states in the power production DLCs use significant wave height conditional on 10-min average
 380 hub height wind speed ($H_s/U_{10,hub}$). Wind speeds from the DE data buoy were separated into 2 m/s bins
 381 ranging from 3 m/s (cut-in wind speed) to 25 m/s (cut-out wind speed), and the expected mean and 50-
 382 yr (98th percentile) significant wave heights were calculated as a function of a Weibull probability
 383 density function [5] fit to the wave data associated with the wind data within each bin. The mean and
 384 50-yr wave heights conditional on wind speed (Table 4) were used to model Normal Sea State (NSS)
 385 and Severe Sea State (SSS), respectively. It was assumed for the purposes of this study that the
 386 relationship between 1-hr wind speed (from the DE buoy data) and wave height was similar to 10-
 387 minute hub height wind speed ($U_{10,hub}$) and wave height.

388 **Table 4 Significant wave height values conditional on wind speed**

Mean Wind Speed, $U_{10,hub}$ (m/s)	Expected Value Conditional on $U_{10,hub}$		50-yr Value Conditional on $U_{10,hub}$	
	Significant Wave Height, H_s (m)	Peak Spectral Period, T_p (sec)	Significant Wave Height, H_s (m)	Peak Spectral Period, T_p (sec)
4	0.87	3.5	1.8	5.0
6	0.89	3.5	1.9	5.1
8	0.95	3.6	2.0	5.2
10	1.1	3.9	2.1	5.4
12	1.3	4.2	2.4	5.8
14	1.5	4.6	2.8	6.2
16	1.8	5.0	3.2	6.7
18	2.1	5.4	3.6	7.1
20	2.4	5.7	4.1	7.5
22	2.8	6.2	4.9	8.3
24	3.2	6.7	5.7	8.9

389 The rated wind speed for the NREL 5MW is 11.4 m/s and cut-out is 25 m/s; for power production
390 DLCs, the mean (turbulent) wind speed bins of 12 m/s and 24 m/s (ranging from 11-13 m/s and 23-25
391 m/s) were used for rated and cut-out conditions.

392 **5 Results and Discussion**

393 **5.1 Validation of computed foundation damping**

394 To validate this approach for computing foundation damping, the contribution of foundation damping
395 to the overall OWT system damping was calculated using numerical free vibration analysis and
396 compared with damping values found in the literature. The free vibration analysis was used for
397 numerical purposes (i.e., as a method to determine OWT system damping for various mudline damping
398 conditions) and is not meant to represent a true physical condition or DLC. In the free vibration analysis,
399 a static initial tower top displacement was imposed at hub height in the fore-aft direction and then the
400 support structure was permitted to vibrate freely in conditions with no wind or waves, considering
401 parked blades with a blade pitch of 90° (feathered blades). The logarithmic decrement method [6] was
402 then used on the resulting time history using a best-fit of a series of amplitudes. Two free vibration
403 analyses were carried out with the foundation stiffness and damping for each of the representative cases
404 – first including structural damping in the tower property input file (1.0%) and then excluding it
405 (structural damping = 0%).

406 The foundation damping contribution to the OWT system damping was calculated by taking the
407 difference between these two cases (Table 5) assuming that damping for OWTs can be modeled
408 independently and combined linearly, and that aerodynamic and hydrodynamic damping can be
409 neglected in this case [6,12,13,32]. This approach was used due to the manner in which structural
410 damping is accounted for in FAST, and the sensitivity of the percent critical structural damping to input
411 load level [6,38].

412

413

414 **Table 5 Percent critical damping of OWT system for all representative mudline stiffness and damping cases**

Foundation Stiffness and Damping from Representative Case	Contribution to Percent Critical OWT System Damping by Foundation Damping [%]	Contribution to Percent Critical System Damping by Structural Damping [%]	Percent Critical Total Damping (Foundation + Structural) [%]
DLC 1.1 v_{rated}	0.28	0.28	0.56
DLC 1.6a v_{out}	0.49	0.30	0.79
DLC 5.1 v_{rated}	0.65	0.31	0.96
DLC 5.1 v_{out}	0.58	0.30	0.88
DLC 6.2a Yaw = 60°	0.64	0.31	0.95
DLC 6.2b Yaw = 0°	0.65	0.32	0.97

415 The foundation damping calculated here (ranging from 0.28% to 0.65%) is within the range found in
416 the literature [6,12,13,15,31,32]. Most notably, the contribution by foundation damping to the percent
417 critical damping of OWT system calculated for emergency shutdown cases (0.58%) is very similar (also
418 0.58%) to the foundation damping contribution to the OWT system damping estimated by Damgaard
419 et al. [32] from a field rotor-stop test of a wind turbine located at a site with a soil profile dominated by
420 very stiff to very hard clay similar to the one analyzed in this paper. These results offer at least one
421 instance of experimental validation of the approach used in this paper to estimate foundation damping.

422 The variation in structural damping in Table 5 can likely be attributed to the manner in which structural
423 damping is accounted for in FAST, which is effectively Rayleigh damping with the mass-proportional
424 coefficient set to zero [38]. Consequently, while 1.0% damping was defined in the tower property input
425 file for the first and second fore-aft and side-to-side modes for all DLCs (defining the structural damping
426 for the support structure between mudline and hub height), the net resulting damping attributed to the
427 structure was approximately 0.3%.

428 **5.2 Nonlinear foundation stiffness**

429 The DLCs were grouped based on k_{mud} and E_h , and representative k_{mud} matrices were selected to
430 represent each group (Table 6).

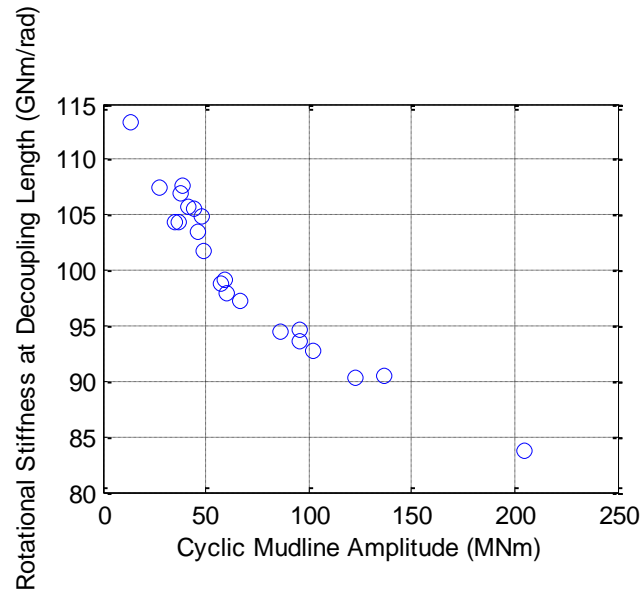
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Table 6 Representative mudline stiffness matrices for design load case groups

k_{xx} $\left(\frac{\text{GN}}{\text{m}}\right)$	$k_{x\phi}$ $\left(\frac{\text{GN}}{\text{rad}}\right)$	$k_{\phi\phi}$ $\left(\frac{\text{GNm}}{\text{rad}}\right)$	Decoupling Length (m)	Natural Freq. (Hz)	$c_{\phi\phi}$ $\left(\frac{\text{GNs}}{\text{rad}}\right)$	Design Load Cases
2.6	20	270	7.7	0.23	2.3	1.1, 1.5, 6.1c
2.6	22	290	8.5	0.23	3.7	1.3, 1.6a, 1.6b
3.3	32	400	9.7	0.23	3.4	5.1 (v_{rated})
3.0	28	350	9.3	0.23	3.5	5.1 (v_{out})
2.8	26	340	9.3	0.23	4.0	6.1a, 6.2a
2.6	24	310	9.2	0.23	4.6	6.2b

433 The values of k_{xx} , $k_{x\phi}$, and $k_{\phi\phi}$ in the mudline stiffness matrices using the method from Zaaijer [28] above
434 show a counter-intuitive trend: the stiffness values appear to increase with loading conditions (F_x , M_ϕ);
435 however, despite the increase in individual stiffness matrix values, the resulting foundation response (u ,
436 θ) and frequency trend appropriately – i.e., with increasing moment amplitude, displacement and
437 rotation increase and frequency decreases, representing a softening in soil-pile response. Prior work on
438 foundation damping by Carswell et al. [9] utilized horizontal and rotational springs at the end of a rigid
439 decoupling length to decouple the 2x2 stiffness matrix model of soil-foundation interaction. Using the
440 rotational spring stiffness as defined by Carswell et al. [9], Figure 7 shows that the rotational stiffness
441 of the soil-pile system decreases with increasing moment as expected, even though the magnitude of
442 individual components of the stiffness matrix may increase. The depth down to so called decoupling
443 point, the horizontal and moment increases with decreasing soil stiffness due to increasing moment
444 amplitude, which is in agreement with other results for monopiles from NGI [39]. The increase in
445 decoupling length with increasing load amplitude causes the mudline rotational stiffness values (given
446 in Table 6) to increase with loading for some of the DLC groups.



447

448

Figure 7 Rotational stiffness at the end of a rigid decoupling length vs. mudline moment

449

5.3 Response with and without foundation damping

450

Using the representative mudline stiffness matrices from Table 6, aero-hydro-elastic analyses were

451

performed in FAST including foundation damping (“damped”, Table 7) and considering no foundation

452

damping (“undamped”). Cyclic amplitudes for mudline loads, displacements, and rotations decreased

453

for all DLCs when mudline foundation damping was included in the analysis. Broadly speaking,

454

mudline moment amplitude (M_{ϕ}) was reduced more than mudline horizontal force amplitude (F_x) with

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the inclusion of foundation damping, which is similar to the results found by Carswell et al. [6] (Figure

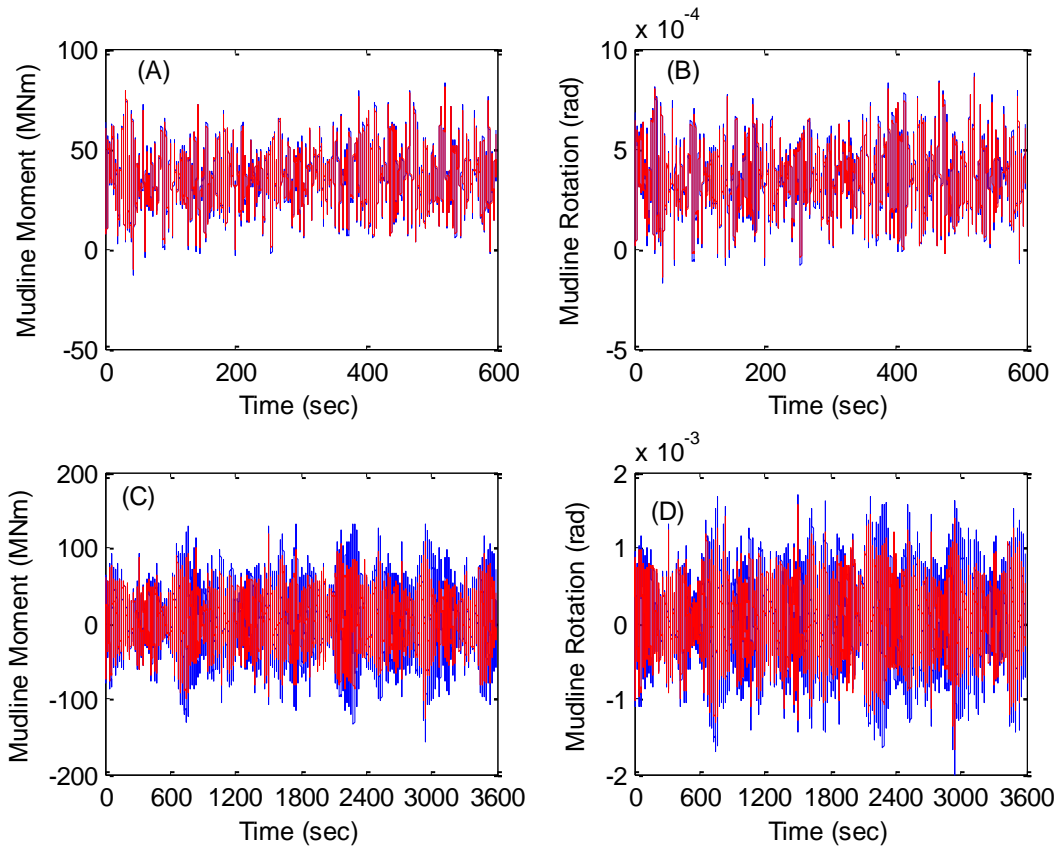
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8).

457
458**Table 7 Mudline response comparison between the damped and undamped analyses in FAST. Damped analyses included mudline foundation damping in the form of a viscous rotational dashpot.**

<i>Load Case</i>	<i>Condition</i>	UNDAMPED				DAMPED			
		F_x (MN)	M_ϕ (MNm)	u_x (mm)	θ_ϕ (10^{-3} rad)	F_x (MN)	M_ϕ (MNm)	u_x (mm)	θ_ϕ (10^{-3} rad)
1.1	v_{rated}	0.59	42	3.5	0.41	0.57	41	3.4	0.41
	v_{out}	1.3	44	4.3	0.48	1.2	43	4.2	0.47
1.3	v_{rated}	0.66	46	4.5	0.50	0.61	46	4.4	0.49
	v_{out}	1.4	53	5.7	0.61	1.3	51	5.5	0.59
1.5	v_{rated}	0.47	17	1.6	0.19	0.46	17	1.6	0.18
	v_{out}	1.2	34	3.6	0.40	1.2	34	3.6	0.40
1.6a	v_{rated}	0.98	46	4.8	0.52	0.96	45	4.7	0.51
	v_{out}	2.1	56	6.8	0.71	2.1	55	6.7	0.69
1.6b	v_{rated}	1.4	52	5.8	0.61	1.4	51	5.7	0.61
	v_{out}	2.9	65	8.6	0.87	2.9	64	8.5	0.86
5.1	v_{rated}	0.87	88	11	1.1	0.57	56	7.3	0.73
	v_{out}	1.0	72	8.4	0.88	0.91	52	6.3	0.65
6.1a	Yaw = 0°	2.9	98	13	1.3	2.8	81	11	1.1
	Yaw = 8°	2.9	99	13	2.3	2.8	83	12	1.1
6.1c	Yaw = 0°	1.8	38	5.7	0.55	1.8	37	5.6	0.54
	Yaw = 15°	1.8	37	5.6	0.54	1.8	36	5.5	0.53
6.2a	Yaw = 0°	2.9	87	12	1.2	2.9	85	12	1.2
	Yaw = 60°	2.8	100	12	1.2	2.8	82	11	1.1
	Yaw = 90°	3.0	130	16	1.6	2.8	90	11	1.1
6.2b	Yaw = 0°	3.3	61	9.6	0.92	3.3	61	9.5	0.91
	Yaw = 90°	3.3	70	10	0.99	3.3	63	9.7	0.93
	Yaw = 180°	3.4	64	9.9	0.95	3.3	63	9.8	0.94

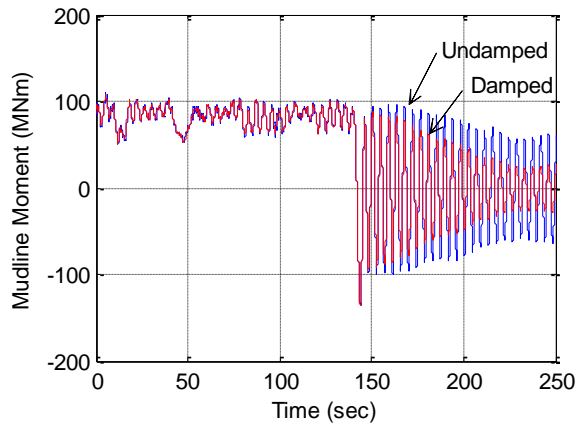
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Figure 8 Example time histories of undamped (blue) vs. damped (red). (A) and (B) show mudline moment and rotation for DLC 1.1 at cut-out wind speed; (C) and (D) show mudline moment and rotation for DLC 6.2a at Yaw = 90°. The OWT is in a power-production state for DLC 1.1 (with spinning rotor blades) and parked for DLC 6.2a.



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465
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Figure 9 Example time history of undamped vs. damped response for emergency shutdown DLC 5.1 at rated wind speed

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468
469**Table 8 Percent reduction in mudline response (horizontal mudline shear, moment, displacement and rotation) due to of foundation damping considering Power Production, Emergency Shutdown, and Parked DLC conditions**

Design Situation	Load Case	Condition	F_x [%]	M_ϕ [%]	u_x [%]	θ_ϕ [%]
1) Power Production	1.1	v_{rated}	4.1	1.1	1.5	1.3
		v_{out}	2.7	1.8	2.2	2.0
	1.3	v_{rated}	8.2	1.6	2.4	2.1
		v_{out}	7.3	3.3	4.5	4.2
	1.5	v_{rated}	1.7	2.8	2.8	2.7
		v_{out}	0.4	0.8	0.7	0.8
	1.6a	v_{rated}	2.2	1.1	1.5	1.4
v_{out}		1.4	1.8	1.8	1.8	
1.6b	v_{rated}	1.0	0.6	0.8	0.8	
	v_{out}	0.6	0.9	0.9	0.9	
Average			3.0	1.6	1.9	1.8
5) Emergency Shutdown	5.1	v_{rated}	34	36	36	36
		v_{out}	15	30	29	29
	Average			25	33	32
6) Parked Conditions	6.1a	Yaw = 0°	1.9	17	12	13
		Yaw = 8°	2.0	17	12	13
	6.1c	Yaw = 0°	1.9	2.8	2.6	2.6
		Yaw = 15°	2.0	1.5	1.4	1.4
	6.2a	Yaw = 0°	0.8	2.3	1.9	2.0
		Yaw = 60°	2.1	19	6.6	9.2
		Yaw = 90°	4.4	30	28	30
	6.2b	Yaw = 0°	0.1	1.1	0.5	0.6
		Yaw = 90°	0.5	9.2	4.6	5.5
		Yaw = 180°	0.8	1.6	1.3	1.4
Average			1.6	10	7.1	7.9

470

471 The emergency shutdown case (DLC 5.1, defined in Section 4.4) displayed the most significant
472 reduction in cyclic mudline demand when foundation damping was included in the analysis (Table 8,
473 Figure 9). The peak response at shutdown is reduced by approximately 5-10%, however, using two
474 times the standard deviation (2σ) of the post-shutdown response (i.e., after $t = 140$ s in Figure 9) as a
475 comparative metric, the reduction in mudline load and response amplitude after emergency shutdown
476 due to foundation damping is approximately 25-33%. Furthermore, the number of load cycles applied
477 to the foundation reduces by approximately 30%. Using the maximum amplitude in the beginning of
478 the shutdown process (chosen as a simple approach to obtain a first order estimate of the importance of
479 foundation damping for this DLC) likely overestimates the foundation damping for subsequent load

480 cycles; however, accounting for foundation damping the combined reduction in load amplitude and
481 number of cycles could lead to a considerable reduced demand on the foundation, since smaller
482 amplitude load cycles subsequent to large amplitude load cycles may control the foundation capacity
483 [24]. Power production cases were not as significantly affected by foundation damping as emergency
484 shutdown and the parked cases were (Table 8, Figure 8). With the exception of approximately 8%
485 reduction in F_x and 2-5% reduction in u_x and q_ϕ for DLC 1.3, the majority of the reductions in F_x and
486 M_ϕ for power production cases ranged from approximately 1-4% and for u_x and θ_ϕ the reductions were
487 approximately 1-2%.

488 The parked DLCs showed larger reduction in mudline response compared to the power production cases
489 for which the aerodynamic damping is much more significant than foundation damping [13,34].

490 The reductions in response are much greater for the turbulent wind, irregular wave cases (DLCs 6.1a
491 and 6.2a) than the steady wind, regular wave cases (DLCs 6.1c and 6.2b): the largest reduction in M_ϕ
492 for the steady wind/regular wave cases was 9.2% (DLC 6.2b, 90° yaw case) while for the turbulent
493 wind/irregular wave cases, the largest reduction in M_ϕ was nearly 30% (DLC 6.2a, 90° yaw case). The
494 difference in reduction is likely due in part to the larger mudline cyclic amplitude for the turbulent
495 wind/irregular wave cases used to determine mudline foundation damping (for which M_ϕ are on average
496 approximately twice that of the steady wind/regular wave cases), but may also underscore the
497 importance of including foundation damping in stochastic analyses (note that while the components of
498 the stiffness matrices for DLCs 6.2a and 6.2b differ by less than or equal to 10%, the dashpot value
499 $c_{\phi\phi}$ is actually higher for DLC 6.2b than for DLC 6.2a).

500 **6 Summary and Conclusions**

501 This paper analyzed the influence of foundation damping on the behavior of a monopile-supported
502 offshore wind turbine (OWT) considering the design situations of power production, emergency
503 shutdown, and parked conditions. These design situations were modeled in FAST [17] according to the
504 design standard IEC 61400-3 [18], considering the NREL 5MW Reference Turbine [16] and the soil

505 conditions of a site in the North Sea. Because metocean data was unavailable at the North Sea Site
506 analyzed in this paper, environmental conditions from a data buoy in the U.S. Atlantic Ocean were used
507 to determine input wave heights and wind speeds for analysis in FAST.

508 Foundation damping was modeled using viscous rotational dashpots at the mudline. The dashpot
509 coefficient was calculated as a function of hysteretic energy loss from the soil-pile system and mudline
510 rotation amplitude using the NGI-developed program INFIDEL [19,20] and the first fore-aft natural
511 frequency of the NREL 5MW. The rotational dashpots were used in conjunction with a mudline
512 foundation stiffness matrix to model soil-monopile interaction for the OWT modeled in FAST.

513 Foundation damping played a more significant role in the emergency shutdown and parked design
514 conditions than power production. For power production cases, the average reduction in cyclic demand
515 (amplitude of mudline loads) due to the inclusion of foundation damping was approximately 3% for
516 horizontal mudline force and 1.6% for mudline moment. Comparatively, the cyclic moment demand
517 was reduced by 10% on average for the parked conditions and by as much as 30% in some cases. The
518 emergency shutdown cases experienced the largest reduction in cyclic mudline demand (25-33%). In
519 all cases, the selection of cyclic load amplitude influences foundation damping results: larger cyclic
520 load amplitudes lead to lower foundation stiffness and more foundation damping and vice versa (Table
521 6, Figure 7).

522 The percent critical damping of the OWT system in the free vibration study range was 0.3-0.7% and
523 were in good agreement with those found in literature [6,12,13,15,31,32], particularly with the
524 experimentally-derived estimate of foundation damping from an instrumented emergency shutdown test
525 of an OWT in similar clay soil from Damgaard et al. [32] This validates the general approach used
526 hereinto compute and incorporate foundation damping in an OWT analysis and demonstrates the
527 benefits of including foundation damping into time history analysis. While this paper focused on a
528 specific example, the principles illustrated in this paper are generally applicable to OWTs supported by
529 monopile foundations and it can be expected that similar reductions in cyclic amplitude may be seen in

530 other cases (as the required stiffness of the foundation to meet natural frequency and fatigue criteria
531 would likely result in foundations of similar stiffness).

532 It may be concluded from this paper that the role of foundation damping in parked and emergency
533 shutdown conditions is significant. For power production cases, the assumption that the first natural
534 frequency is the dominant frequency of the cyclic response may not be accurate, and that in some
535 instances the frequency dominating the cyclic response may be peak wave frequency; consequently, in
536 that instance using the dominant wave frequency in the formulation for foundation damping may
537 increase the accuracy and significance of foundation damping under power production design
538 situations. Additionally, further research on the significance of foundation damping during situations
539 of wind-wave misalignment [12] would also show an increase in the significance of foundation damping
540 during power production design situations.

541 Future work on foundation damping and fatigue analysis should be performed to better understand how
542 foundation damping influences OWT support structure design: the steel of the support structure may
543 benefit from low amplitude cycle reduction (due to the large number of low amplitude cycles
544 experienced throughout the life of the turbine) while the pile-soil interaction may benefit from high
545 amplitude cycle reduction. Additionally, using harmonic assumptions in the formation of mudline
546 foundation stiffness and damping for highly stochastic time histories does not capture the full
547 complexity of the dynamic behavior of these systems; to better understand the influence of this
548 assumption, a foundation model which can re-calculate mudline foundation stiffness and damping at
549 each load step is required.

550 The influence of soil profile on foundation damping should be investigated in future work, particularly
551 with regard to soil type – the majority of existing work on foundation damping has focused on clayey
552 soils, with limited information regarding how much damping may be contributed by a monopile in sand
553 and how it may be compared to the amount of damping from clays [6].

554 There is a need for design guidelines on how to quantify and account for foundation damping in
555 modelling. Current design often includes foundation damping as an increase in overall damping;

556 however, if foundation damping is accounted for by either a dashpot at the mudline or a macro element
 557 approach, the contribution of foundation damping should not also be included in the overall structural
 558 damping.

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