Risks and Vulnerabilities in the Design, Construction, and Operation of Offshore Wind Turbine Farms in Seismic Areas

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1 Introduction

The United Nations recently declared that we are facing a grave climate emergency, and this is one of the grand technological challenges in our times. Continuous ocean and atmospheric warming, heat waves, and rising sea levels are some of the most common manifestations of climate change. One of the pathways to reduce emission is to decarbonize energy sources. A practical way to achieve a net-zero target is to run the country mostly on electricity produced from renewable sources without burning much fossil fuel. Offshore Wind farms have evolved as one of the scalable technologies to produce power. These relatively new technologies are also being constructed in seismic areas like Taiwan, Japan, China, and the United States. There

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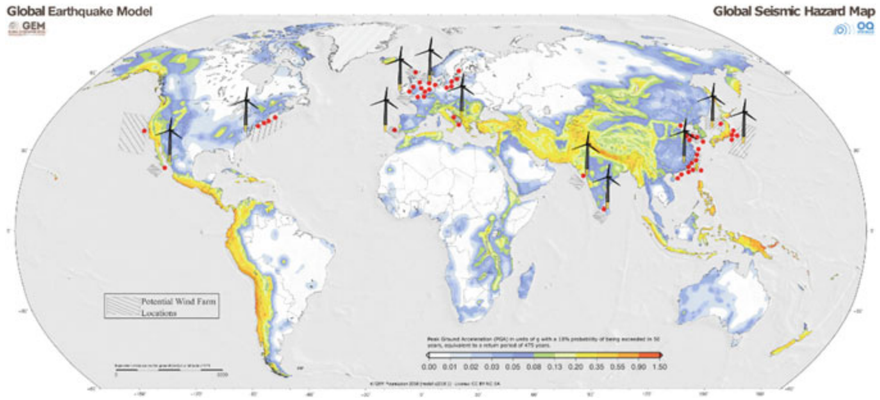


Fig. 1 Global Seismic Hazard Map showing the PGA and possible locations of offshore wind farm adapted from GEM project. **Base map source (GEM model)** (Bhattacharya et al. 2021a)

are plans to construct in other seismic countries such as India, Italy, Greece, Turkey, etc. Figure 1 shows a world map showing the potential locations of offshore wind along with the global seismic hazard map.

Offshore wind turbines are relatively new structures, and their construction in seismic areas is in its infancy. Therefore, codes of practice/best practice guidelines are not fully developed. As a result, the seismic design of offshore wind turbine (OWT) structure is uncertain, fragmentary, and often borrowed from methods adopted for Nuclear Power Plant design or building design. The aim of the keynote lecture at the conference is, therefore:

(A) To discuss the challenges in the analysis and design of these structures with emphasis on the foundations.
(B) To provide rational guidelines on the main issues concerning the risk and vulnerabilities of offshore wind farm.

1.1 Offshore Wind Farm

Offshore Wind farms are a collection of turbines with a substation and cables to transmit electricity to the onshore grid. Figure 2 shows a typical layout of an offshore wind farm for a grounded system where the different components are shown. Readers are referred to Chap. 1 of Bhattacharya (2019) for important details.

To de-risk an offshore wind farm for seismic conditions, we need to assess the vulnerability of all the main components:

(1) Offshore Wind Turbines structures,
(2) Inter array and export cables,
(3) Substation structure.

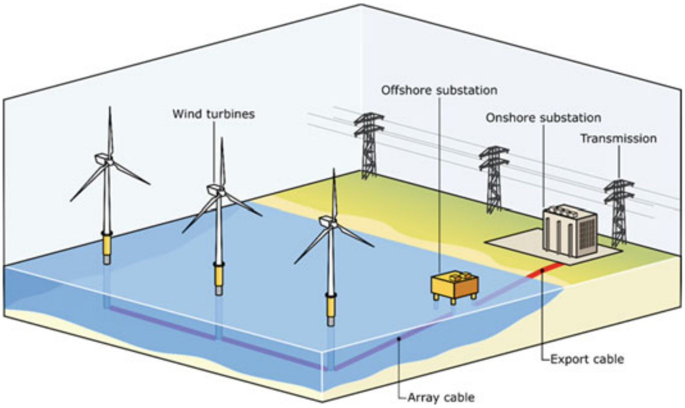


Fig. 2 Layout of an offshore windfarm (grounded system)

Figure 3 shows a range of offshore wind turbine structures currently used or planned to be used, and they are classified into grounded systems and floating systems. Typically, for water depth less than 60 m, it is expected grounded systems will be used and they are types 1–5 in the figure.

Offshore Wind Turbines are relatively new structures, and it is important to list the performance requirements for these systems for uninterrupted energy production. Table 1 lists the various requirements of offshore wind turbine systems keeping in mind the seismic hazards. It must be appreciated that offshore wind farms should

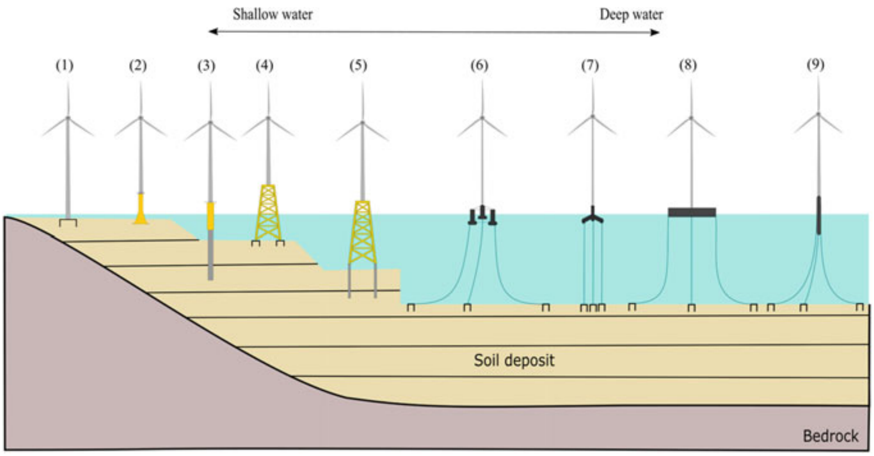


Fig. 3 Types of systems depending on water depths. Types of foundation [(1) Suction Bucket/Caisson, (2) Gravity-Based Foundation, (3) Monopile, (4) jacket on suction caisson, (5) jacket on monopile, (6) Semi-Submersible, (7) tension leg platform (TLP), (8) barge, (9) spar

Table 1 Examples of Offshore Wind Turbines with high consequences of failure where seismic design might need to be considered

Factor influencing the probability of exceedance	Typical example
Economic impact	(a) Permanent tilting of the whole wind turbines beyond repair (Fig. 6). The consequence of tilting is the loss of the investment. This is an example of ULS (Ultimate Limit State) (b) Tilt of the overall structure within the allowable limit (SLS criteria satisfied—e.g., tilt <0.5–0.75°) but the blade cracked. The blade needs a replacement and, therefore, a huge unplanned cost. In addition, energy production halted for a substantial amount of time (c) RNA acceleration exceeded the allowable limit damaging some components of the electronics. Repair would cost together with loss of energy production (d) Large-scale Wind farms in the coastal areas and with no power production will have a national economic impact
Impact on post-earthquake relief	Loss of power production could impact the rescue effort and recovery
Structural integrity	(a) Limit on blade deflection and not to hit the tower during earthquakes; see Fig. 5 (b) Tilting of the tower will enhance P-delta moment causing more fatigue damage leading to early end of life

remain operational even after a major earthquake so that rescue operations (if necessary) can be carried out.

It may be noted that Table 1 is by no means exhaustive, and further work is underway to describe these and bring out the criteria for seismic design. The majority of offshore wind turbines are supported on monopiles, and it is important to discuss the SLS criteria of this particular foundation. One of the important design aspects of monopile-supported wind turbines is the allowable tilt. The current allowable tilt is 0.5–0.75 degrees, and this requirement is still valid for both mainshocks and aftershocks. The possible reasons for stricter SLS are shown schematically in Fig. 4, and the readers are referred to Chap. 3 of Bhattacharya (2019) for further details. Increased tilt may result in reduced blade-tower collision, increased wear and tear of bearings, increased foundation loads (Fig. 5).

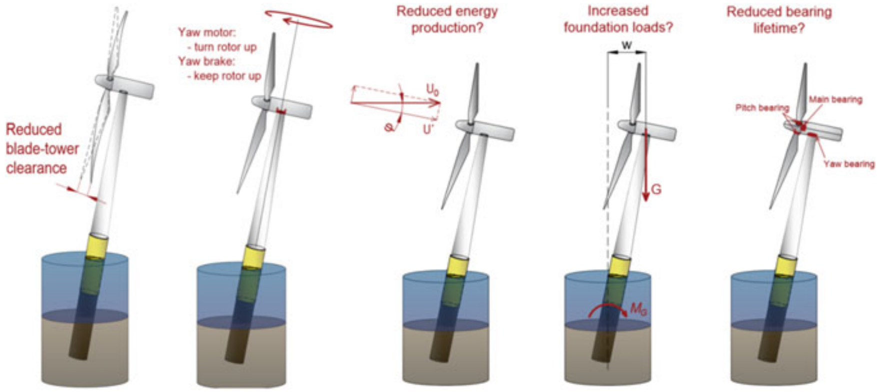


Fig. 4 Aspects governing the SLS requirements for monopile foundation

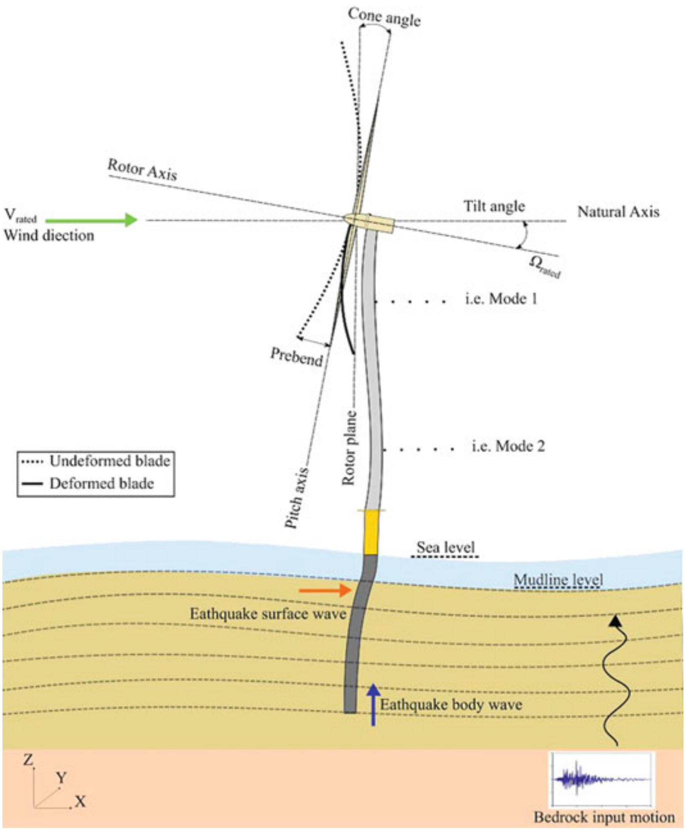


Fig. 5 Deflection of the blade and blade-tower interaction during seismic loading

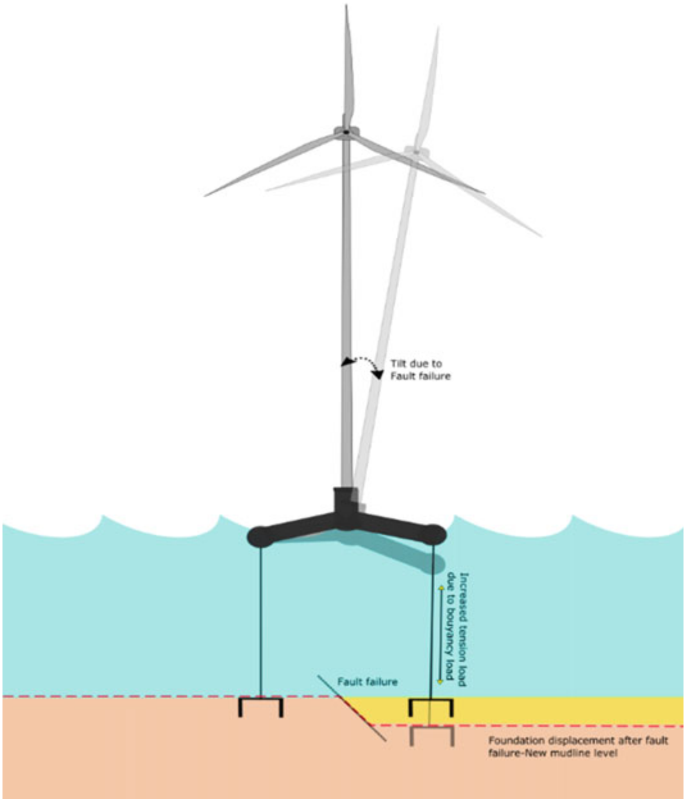


Fig. 6 Effect of support settlement for TLP foundations (Bhattacharya et al. 2021b)

1.2 Seismic Risks to an Offshore Wind Farm

Seismic hazards to an offshore wind farm can be numerous. Alati et al. (2015), Bhattacharya (2019), Bhattacharya et al. (2019), Kementzetzidis et al. (2019), Ali et al. (2020), Bhattacharya et al. (2021), and Bhattacharya et al. (2022) list necessary steps in a seismic risk evaluation:

(a) Identification of potential seismic hazards at the site and must include cascading events.
(b) Effect of large fault movements (i.e., subduction fault) can lead to rupture of the cables or embedded anchoring for floating systems. Figure 6 shows a schematic diagram explaining the situation taking into consideration a TLP system.
(c) Ground shaking with no-liquefaction of the subsurface. This includes inertial effects on the structure and will induce inertial bending moment on the foundation piles. Due to kinematic interaction, additional bending moments will be induced if the ground is layered with contrasting stiffness.

(d) Shaking of the ground together with liquefaction of the subsurface. Liquefaction may lead to a large unsupported pile length and will elongate the natural vibration period of the whole structure. One of the significant risks is the tilting of the foundation due to liquefaction. The ground may liquefy quickly or take time and is a function of the ground profile and type of input motion. In such scenarios, the transient effects of liquefaction need to be considered, as it will affect the bending moment in the piles.

(e) If there is a tsunami risk, the effect must be considered together with the ground shaking and liquefaction.

(f) Earthquakes may cause submarine landslides, and the potential impact must be considered.

(g) The effect of earthquake sequence such as Foreshock + Mainshock + Aftershock need to be evaluated.

1.3 Codes of Practices for Seismic Design of Offshore Wind Turbines

OWT consists of a long slender tower with a top-heavy fixed mass (Nacelle) and a heavy rotating mass. The structure is constantly exposed to variable environmental wind and wave loads. These relatively new structures can also be characterized as an inverted pendulum (with a substantial mass concentrated in the upper 3rd of the tower), and guidelines for designing such special structures are not explicitly mentioned in current codes of practices.

Eurocode 8 mainly focuses on buildings and bridges, and at present, it is of interest to review some clauses. Eurocode 8 (Part 1) (EC8, 2003) states that special structures such as offshore structures are beyond the scope. Clauses/Guidelines of EC8 are divided into principles (P) and application rules. Principles are identified by P after the clause number and cover items for which no alternative is permitted. Application rules are recommended methods of achieving the Principles, but alternative rules may also be used.

Other codes of practice for the seismic design, such as novel structures or guidelines for their certification (e.g., DNV/Risø, 2002; Germanischer Lloyd, 2005; DNV, 2014; IEC 2009) are not fully developed nor validated as the installation of offshore wind farms in earthquake-prone countries is in its relative infancy and is expanding rapidly. Often, specific requirements are borrowed from the guidelines developed for the petroleum and natural gas industries (ISO 19901-2:2017). Furthermore, the existing codes of practice on seismic design are mostly developed for conventional structures, and their applicability to offshore wind turbines needs to be verified and validated. However, analysis and design must be carried out to support the *energy-transition* initiatives to understand the vulnerability under seismic loading.

The required performance is to reduce operational expenditure (OPEX) following an earthquake and not to enhance capital expenditure (CAPEX), avoiding over-conservativism unnecessarily.

2 Challenges in Seismic Design

The analysis and design of foundations for offshore wind turbines is challenging due to complex load conditions arising from the environmental loads (i.e., wind, wave, currents). In seismic areas, there are additional loads due to the phenomena and processes discussed in the earlier section. Figure 7 shows a schematic diagram of the environmental loads acting on a typical offshore wind turbine, which must be carried by the foundations and transferred to the adjacent soil. There are four main environmental loads: wind, wave, 1P (rotor frequency), and 2P/3P (blade passing frequency) loads whose waveform is also shown in Fig. 7 for a monopile foundation. The salient characteristics of these loads are summarized as follows:

(a) Wind and wave result in a different offset of amplitude, frequency, and the number of cycles applied to the foundation. Figure 7 shows a schematic representation of the frequency of these loads together with the frequency intervals corresponding to the three possible design choices: Soft-Soft, Soft-Stiff, and Stiff-Stiff.
(b) Wind and the wave loads are random in both space and time and therefore are better described statistically through probability distributions, mean values, and standard deviation.
(c) Wave and wind load act in two different directions, which give rise to the so-called wind-wave misalignment.
(d) 1P loading is caused by mass and aerodynamic imbalances of the rotor, and hence the forcing frequency equals the rotational frequency of the rotor.

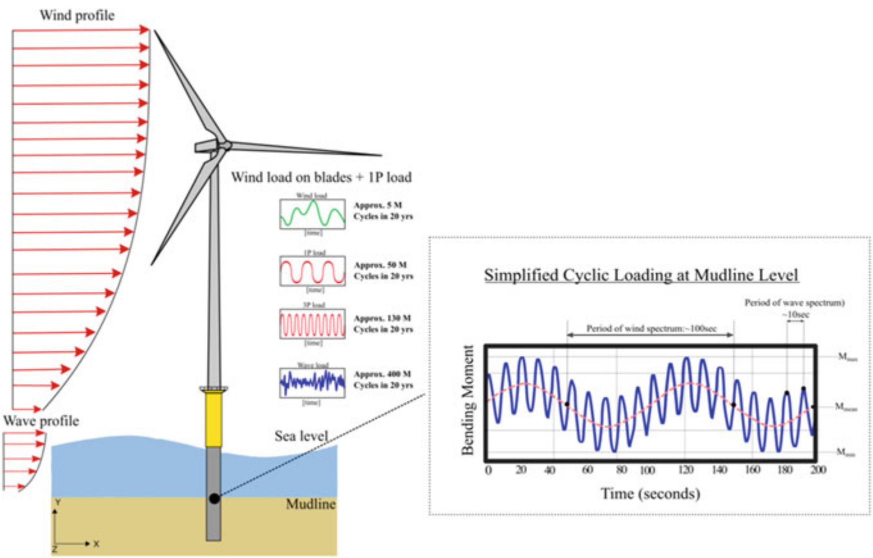


Fig. 7 Load complexity with an approximate number of cycles for 20 years assumed lifetime

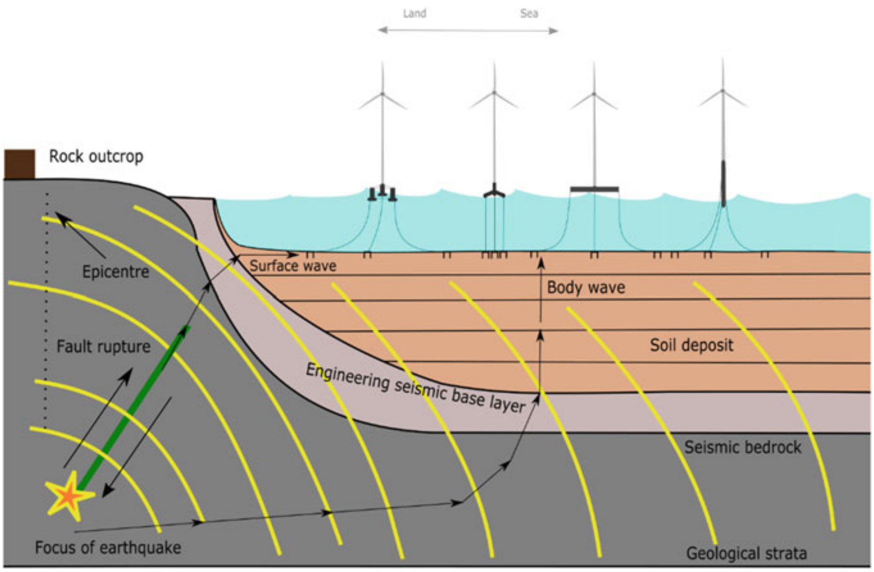


Fig. 8 Schematic of fault rupture to site response for Floating Offshore Turbines

(e) 2P/3P loading is caused by the blade shadowing effect and wind shear (i.e., the change in wind speed with height above the ground), and rotational sampling of turbulence. Its frequency is 2 or 3 times the 1P frequency for two and three-bladed turbines, respectively.

Figure 8 shows additional design considerations in seismic areas, and the different processes/mechanisms are schematically shown for floating systems. To assess the performance of wind turbines as laid out in Table 1, it is necessary to carry out a dynamic analysis for which time history of motions is required. From Fig. 8, the generation of input motion for a given site depends on the seismotectonics of the area. This includes faulting pattern, the site's distance from earthquake source, wave path, the geology of the area, etc. This can be done using either synthetic (artificially generated) or recorded ground motion from previous earthquakes (un-scaled records). Intuitively, it also appears that a grounded system will provide a higher response to RNA when compared with floating systems.

3 Issues in Seismic Analysis

The main issues that must be addressed in the design process of an offshore wind turbine are summarized as follows:

(a) Definition of return periods (T_R) for different hazard levels considering that the design lifetime for offshore wind turbines is typically 25–30 years.

(b) Assessment of the seismic hazard at the given site.
(c) Definition of the Design Response Spectra at different hazard levels.
(d) Selection of strong motions for time-history analysis.
(e) Definition of the load combination criteria considering wind, wave, earthquake (multi-directional), and the control system.
(f) Explicit performance requirements (limit states) at different hazard levels

This section of the paper discusses each of the points above.

3.1 Design Return Period

Large seismic events are low probability but high-risk for offshore wind farms, given their value (typically $0.75bN–$1.25bN for 500 MW). The typical Return Period (T_R) of large earthquakes is hundreds to thousands of years. Currently, offshore wind turbines are designed for a lifespan of 25–30 years, with a possible extension of up to 5 years. Therefore, it is imperative to quantify and mitigate seismic risk over their lifetime.

Most standards currently use the 475 years return period, corresponding to a 10% probability of exceedance in 50 years. The time window of 50 years refers to the lifespan of a typical structure/infrastructure. This calculation assumes the earthquake occurrence as Homogeneous Poisson Process (HPP), and, therefore, the time between seismic events is exponentially distributed. If the time window of 50 years is shortened to 25 years, an event with a 475-year return period has an approximately 5% probability of exceedance.

For an OWT, depending on client requirements (e.g., low OPEX cost), different limit states need to be considered. For example, due to the high replacement costs, the structural integrity of the blades should be guaranteed following an earthquake. Therefore, besides the essential requirements of collapse prevention (i.e., CO, the collapse of foundation, or structural failure of the tower), there must also be another criterion of the integrity of critical components (e.g., blades) based on the economic impact.

Codes of practice often consider the previous aspect through the analysis of the consequences of failure. EC8 [Part 1] recommends two levels, one preventing the ultimate limit state (ULS) and other the serviceability limit state (SLS) as follows: (a) No collapse (ULS) representing 10% exceedance probability in 50 years, i.e., 475-year return period; (b) Damage limitation – 10% exceedance probability in 10 years, i.e., 95-year return period.

In designing traditional structures, if a particular seismic code is adopted, it is inherently assumed that structures, during their lifetime, will be subjected to some form of damage under extreme events to dissipate energy to satisfy some performance criteria. In the case of inhabited structures, it will allow occupants/users a safe evacuation. In the case of offshore wind turbines, these limits cannot be accepted in their current form. Therefore, customized requirements for offshore wind farms may be necessary and must be agreed upon with the client in a contract. Table 1 provides a few examples of typical requirements but is by no means exhaustive.

Based on the lifetime of the structure, seismic hazard levels can be explicitly defined. The return periods for these hazard levels can then be obtained from Seismic Hazard Analysis (SHA).

3.2 Seismic Hazard Analysis (Ground-Motion Parameter and Fault Displacement)

3.2.1 DSHA and PSHA for Ground-Motion Parameters

A seismic hazard analysis (SHA) provides the probability of exceeding a certain ground-motion intensity parameter, typically peak-ground acceleration or spectral acceleration, or fault displacement in a given seismotectonic condition. There are two main types of SHA: (i) Probabilistic Seismic Hazard Analysis (PSHA) and (ii) Deterministic Seismic Hazard Assessment (DSHA). The two types of hazard assessment share similar inputs, namely, seismic catalog, seismic source, and ground-motion models. However, they also differ in some fundamental respects, most notably in the treatment of uncertainties and the characterization of the hazard. The main steps of a typical PSHA are illustrated in Fig. 9 and can be summarized as follows (Cornell, 1968; McGuire, 2004).

Step 1—Definition of seismic source models: It compiles an earthquake catalog that lists all known historical and instrumented earthquakes in the study region. The catalog is used to build the seismic source model that defines the spatial distribution of all the seismic sources that contribute to the hazard at the site.

Step 2—Definition of earthquake recurrence law: This step also relies on the earthquake catalog and defines the rate of earthquake occurrence for each seismic source defined in Step 1. The Gutenberg-Richter (GR) recurrence law is often adopted for the recurrence model. As the GR law may produce unrealistically large earthquakes, it is often truncated to the maximum possible magnitude that the seismic source can produce.

Step 3—Definition of ground-motion models: It consists of quantifying the intensity of the earthquake in terms of parameters of engineering interest, such as peak-ground acceleration (PGA), spectral accelerations, spectral velocities, etc. These are computed based on empirical ground-motion prediction equations (GMPEs), evaluated from a regression analysis of a large set of records. Although different GMPEs have been developed and are available for regions of different seismicity, all provide the distribution of a ground-motion parameter (Intensity measure) as a function of several independent variables such as the earthquake magnitude, the source-to-site distance, the faulting mechanism, and the geotechnical parameters that characterize the soil conditions at the site. Given the inherent randomness of the seismic process

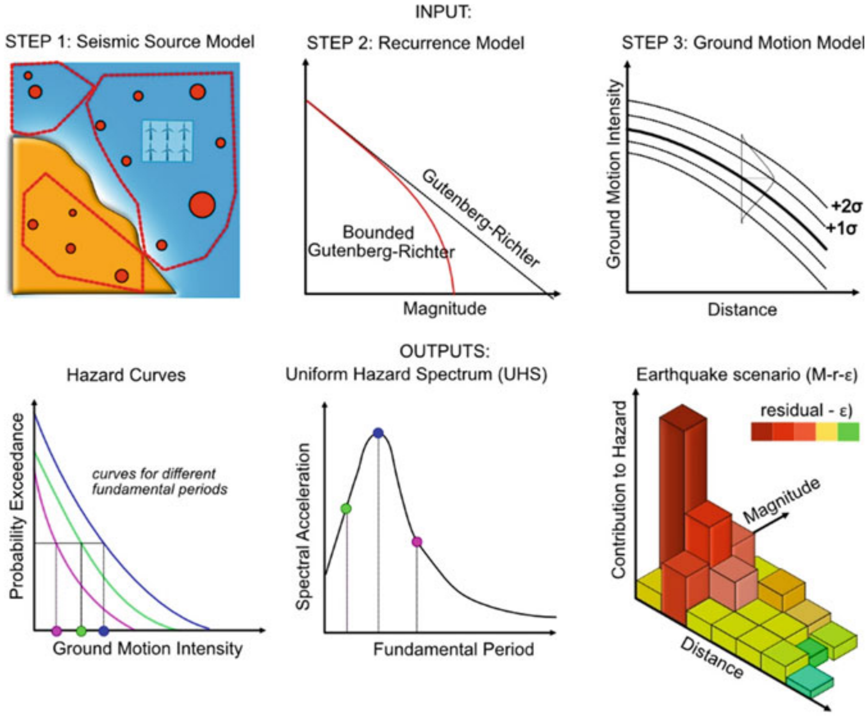


Fig. 9 3-step procedure for probabilistic seismic hazard assessment (PSHA) and typical outputs

and the epistemic uncertainty in the models, multiple GMPEs are usually adopted for SHA using a logic tree with appropriate weights.

The output of a typical PSHA is often presented using a suite of curves, known as seismic hazard curves, which represent the average annual rate of exceedance of a given ground-motion intensity measure for different vibration periods. Since the earthquake occurrence is modeled as a Poisson process, the average annual rate, λ, can be expressed in terms of the probability of exceedance, P, and time, t, such that

$$\lambda = \frac{-\ln(1 - P)}{t} \tag{1}$$

from which it follows that a probability of exceedance of 10% (P = 0.1) in 50 years (T = 50) corresponds to an average annual rate of 0.002 or return period (which is its inverse, i.e., 1/λ) of approximately 475 years.

The PSHA results can be used to plot the spatial distribution of the hazard, such as in hazard maps, or compute the ordinates of the Uniform Hazard Spectrum (UHS). Since the PSHA "aggregates" different earthquake scenarios, the resulting hazard cannot be associated with any real earthquake scenarios. The disaggregation analysis enables identifying a "fictitious" seismic scenario, expressed in terms of magnitude-distance-residual, which provides the greatest contribution to the hazard. This scenario is often used for the selection of ground-motion records compatible with the estimated hazard.

The deterministic approach, DSHA, can be seen as a special case of the PSHA, where only the most dangerous scenario is considered. This is the so-called worst-case scenario, defined in magnitude and source-to-site distance regardless of its likelihood of occurrence. It is worth noting that both methodologies present limitations based on the simplifying assumption they rely upon and the degree of subjectivity involved in the process.

Deterministic seismic hazard analysis can be easily performed and may be suitable in the early stages of an offshore wind farm project (e.g., feasibility study, preliminary design) and if the wind farm is to be built in a low- to-medium-seismicity area. From the catalog of historical seismic events, it is possible to identify the maximum magnitude and the minimum distance of the farm location from the potential seismic sources. Subsequently, using ground-motion prediction equations (GMPEs) suitable for the specific case study. Using the probability of exceedance agreed with the Wind Farm Developer, it is possible to define the hazard that is suitable for design purposes.

It is important to state that PSHA is critical for Nuclear Power Plants due to other far-reaching consequences, and in low-seismic wind farm locations, such type of analysis may have a lower cost-benefit ratio. The readers are referred to Yawson and Lombardi (2018) for an example of PSHA for a low-seismic country such as the UK.

3.2.2 Probabilistic Fault Displacement Hazard Assessment

Similar to the PSHA described in Sect. 3.2.1, one can compute the displacement induced by the fault rupture at the surface. It is worth noting that this is different from the displacement generated by the propagation of seismic waves through the sediments as it is directly caused by the fault rupture. Hence, the probabilistic approach needs to be modified in order to include the probability of slip exceedance given that an earthquake of strikes the site. Figure 10 presents a flow chart for the probabilistic fault displacement hazard assessment for an offshore site. The steps are as follows:

(1) The first step is to identify the site and determine the mean annual occurrence rate λ_m and distribution of magnitude occurrence f(m) from parameter of the Gutenberg-Richter law (GR law). The distribution f(m) is normally truncated to a maximum magnitude M_{max}.
(2) The second step requires the definition of fault displacement prediction equation that substitute the ground-motion prediction equation used in the conventional PSHA. The fault displacement prediction equation depends on the style of faulting, e.g., reverse, normal and strike-slip. As not all fault rupture will propagate to the surface, it is required to determine the probability of occurrence of a slip given a magnitude m; this is expressed by the conditional probability function $P(slip|m)$.
(3) Then, the probability of exceedance of a given level of displacement $P(D > d|m)$ can be computed as the product of $P(slip|m)$ and the convolution of the

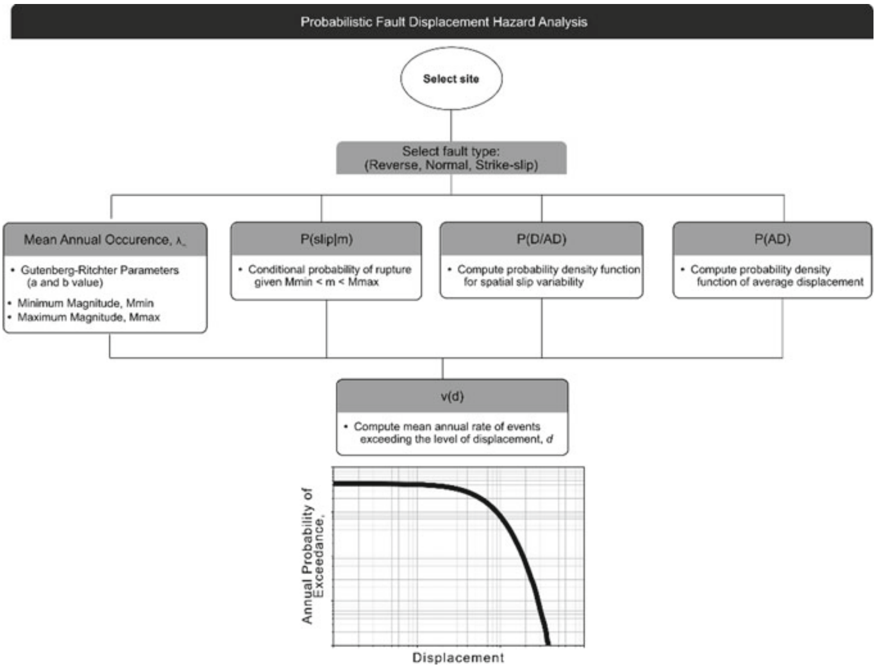


Fig. 10 Probabilistic fault rupture hazard analysis

probability density function of the fault displacement *D*, normalized by the average displacement AD, i.e., P(*D/AD*), and probability density function of the average displacement P(*AD*).

(4) Finally, the annual rate of events exceeding a given fault displacement $\nu(d)$ can be computed from the integral:

$$\nu(d) = \lambda_m \int_{M_{min}}^{M_{max}} f(m) P(D > d|m) dm$$

3.3 Choosing the Response Spectra

The dynamic modal analysis with response spectrum is an accepted procedure used to evaluate the structural response of many structural typologies (Zhao et al., 2020). In the context of Offshore Wind turbine design, there are broadly three types of response spectrum that can be used:

(a) The response spectrum of a single record. It shows the maximum response acceleration of a family of single degree of freedom (SDOF) structures with different periods and prescribed damping.
(b) Uniform Hazard Spectra (UHS) is the main product of the PSHA and can be calculated for different return periods. This is a horizontal spectrum and not directional dependent. Vertical UHS can also be produced. UHS is site-specific and does not take into account the energy dissipation due to allowable structural damage.
(c) Code-based standard Response Spectrum is readily available in most codes of practice (for example, EC8 or IBC). The code-based response spectrum is generally just a functional smooth form (Malhotra, 2006) and can be completely defined if its parameters have been calculated using PSHA. Code-based response spectra are available for both horizontal and vertical directions. The spectrum can be customized to incorporate the response reduction factor (R) to reflect the extent of energy dissipation and ductility. Traditionally, these code-based response spectra have only been defined for onshore or near-shore environments and cannot be readily used for an offshore site.

It must be mentioned that the code-specified elastic spectrum is just a normalization of the uniform hazard spectrum (UHS) that is obtained from the PSHA. A response spectrum can also be derived from a DSHA, and normalization can also be done on this spectrum. Figure 11 shows an example from a site to illustrate the above description.

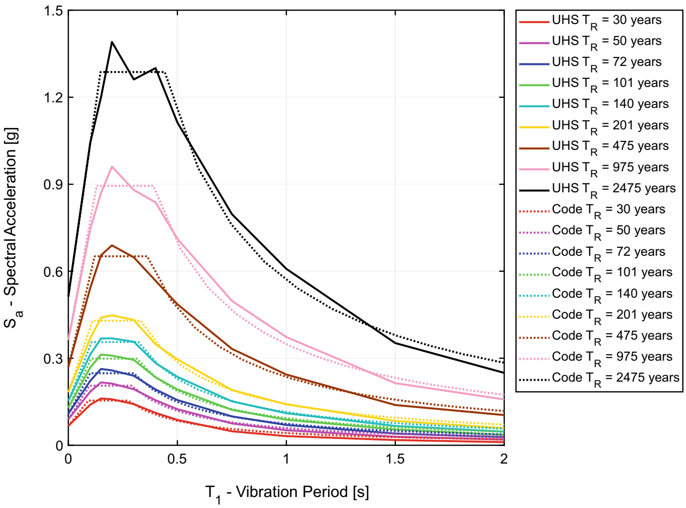


Fig. 11 Example of UHS and its regularization for several return periods

3.4 Ground-Motion Selection for Time-History Analyses

Two methods are generally used for selecting ground motions: (a) Scenario-based methods and (b) Response-spectrum-matching-based methods.

Earthquake scenarios are defined by the seismotectonic features such as magnitude, distance, local site conditions, typology of the fault mechanisms. All these parameters may influence the spectral content of the ground-motion records. Two potential approaches are possible for scenario-based selection. If DSHA is used, it is required to define a design critical earthquake scenario for a given site considering the characteristics of the earthquake rupture of the identified fault. On the other hand, if PSHA analysis is performed, it must utilize the seismic disaggregation results from the PSHA. If multiple scenarios have significant contributions to the hazard, multiple scenarios should be examined (De Risi et al., 2018).

On the other hand, Response spectrum matching methods are based on matching the amplitude of spectral ordinates, and therefore the method attempts to match both the ground-motion intensity and frequency content. The target response spectrum is often the design code spectrum (Iervolino et al., 2010). This selection method is based on the comparison of a candidate response spectrum with the target response spectrum. The matching is usually calculated using as a reference the differences between the spectral ordinates of the reference spectrum and the spectrum of the candidate ground motion. Such a difference is usually evaluated over a vibration period range. This period range should ideally cover the relevant vibration periods of the offshore wind turbine structure under scrutiny. In this regard, Eurocode 8 suggests a range of 0.2 times to 2 times the first vibration period. Furthermore, EC8 suggests that the average spectrum of 7 records needs to be larger than 90% of the target spectrum, which avoids underestimation. A further upper-bound criterion can also be implemented to avoid dispersion of the results.

Ideally, the target spectrum should be site-specific, and, therefore, the uniform hazard spectrum is desirable. It may be noted that different earthquake scenarios govern different parts of the uniform hazard spectrum. For example, moderate events at short distances tend to be dominant at shorter vibration periods, whereas large events at far distances tend to be more critical for longer vibration periods. Furthermore, when UHS is used as the target spectrum, candidate records having similar spectral ordinates for the entire period range tend to be extreme. Thus, forcing an input motion to match the UHS may result in excessively conservative and unrealistic ground motions.

To resolve this issue, a different approach for obtaining hazard consistent ground motions utilizing the Conditional Mean Spectrum (CMS) (Baker, 2011) was proposed. The CMS approach is a combination of scenario-based and spectral-matching methods. In this method, a suite of ground motions is scaled to match the CMS, the mean response spectrum conditioned at a target spectral acceleration value at the period of the structure. To control the dispersion, confidence intervals are generally adopted around the conditional mean spectrum.

Practically, it is hard to find natural records that can match a specific target spectrum. There are a couple of possible solutions in such cases: (a) Natural records from real events can be scaled to reach the matching. This scaling factor should not be excessively high, otherwise unrealistic, combinations of amplitude and frequency contents may be obtained (Luco & Bazzurro, 2007); (b) Time-histories can be simulated to obtain stochastic ground motions matching the hazard spectrum.

More recently, as highlighted by Zhang and Zheng (2020), strong motion records at offshore sites may have different spectral signatures compared to similar motion recorded onshore, particularly at longer periods. The differences are most significant in the vertical component of motion attributed to the high-frequency suppression due to the overlying ocean. Therefore, further work is required to ensure if conventual attenuation relationships developed using data largely recorded at onshore sites are applicable to their offshore counterparts.

3.5 Combination of Seismic Actions with Wind and Wave

As presented in Fig. 12, different loads may act on a monopile-supported wind turbine system. There will be an overturning moment for a monopile type of foundation due to the combination of wind and wave load, which is generally asymmetric and can be one way. Seismic action will increase the lateral load and add the operational load due to normal or emergency braking. If the ground is liquefiable, lateral load-carrying capacity will be lost, leading to a permanent tilt, and is discussed later in the paper. Figure 12 identifies different stages so that engineering calculations can be carried out.

(a) Stage 1 represents the standard calculations necessary for non-seismic locations. There will be minimum and maximum moment and will depend on turbine size, water depth, wind, and wave characteristics. Further details can be found in Jalbi et al. (2019).
(b) Stage 2 represents the arrival of the seismic waves and the onset of the control mechanism of the turbine to reduce overall damage or OPEX cost. It is likely that a normal or emergency brake may be applied depending on whether the turbine is idling (not connected to the grid) or parked or in power generation mode. The loading in this stage will comprise of inertia load together with the braking load. To obtain a conservative estimate of the lateral load and moment at Stage 2, the braking and inertia loads may be added to the load in Stage 1.
(c) In liquefiable deposits, as the earthquake progresses, the ground would progressively liquefy in a top-down fashion, and the moment carrying capacity of the foundation may reduce drastically. The enhanced unsupported length of monopile due to liquefaction coupled with seismic and other operational loads may lead to the potential failure of OWT structure based on ULS (Ultimate Limit State) or SLS (Serviceability Limit State).

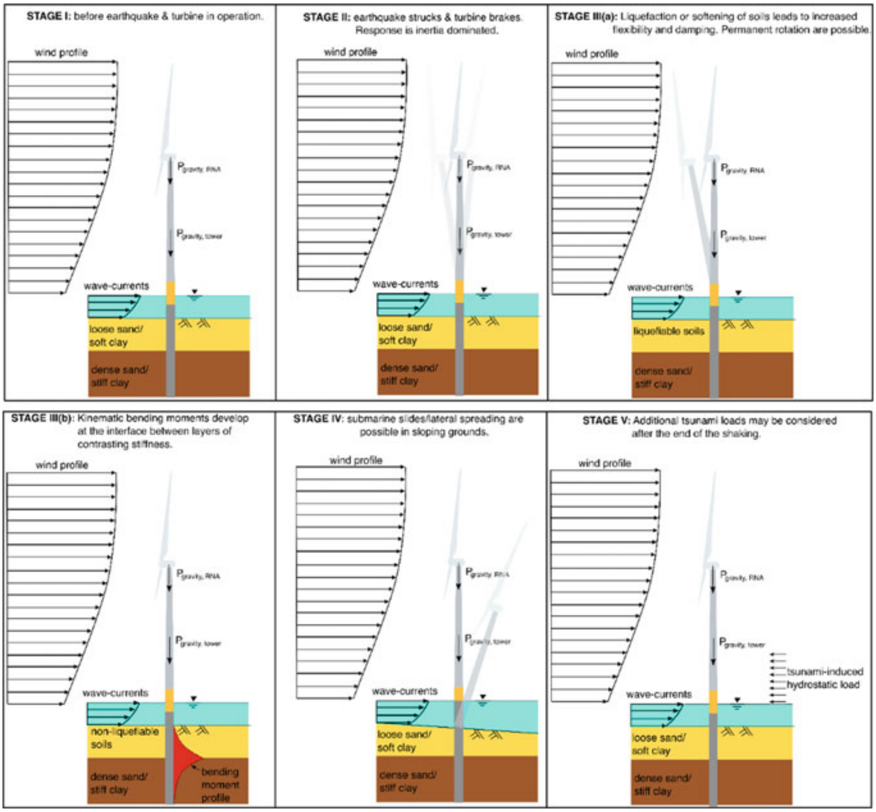


Fig. 12 Schematic diagram of the load cases

(d) If the ground is non-liquefiable, in layered deposits, there may be high kinematic bending moments.
(e) If there are submarine landslides, extra lateral loads may be applied to the foundation.
(f) In Tsunami risk areas, there may be additional loads due to hydrodynamic loads.

4 Effect of Soil Liquefaction on Monopile: Reduction in Capacity and Permanent Tilt

Offshore wind farms are increasingly being constructed in areas of potentially liquefiable soil (Bhattacharya and Goda, 2016). Moreover, young offshore deposits are particularly vulnerable to soil liquefaction, during strong shaking. Soil liquefaction results in stages where the supporting ground behaves as a heavy fluid, resulting in a loss of lateral and vertical resistance. Further, upward artesian flow due to excess pore pressure generation can result in high buoyant forces that can result in cable floatation, if not accounted for in the design. Figure 13 presents a simplified sketch to plot the capacity of the foundation in two stages: No-Liquefaction (Pre-liquefaction) and Maximum Liquefaction (Post-Liquefaction). In the same plot, the load cases can also be shown. The effect of soil liquefaction is the loss of lateral and moment resisting capacity of the foundation as shown in Amani et al. (2022). Such simplified estimates of capacity using pre- and post-liquefaction properties can help in the preliminary sizing of foundations for offshore wind farms. Once, resistance (capacity) is estimated, the action (demand) on the foundation using several load cases can be used to schematically estimate the margin of safety according to Aleem et al. (2022).

During earthquakes, soil deposits often liquefy top-down. The upper layers lose strength, and the liquefaction front progressively travels to deeper layers as highlighted by Scott (1986). Therefore, as presented in Fig. 14, it is expected that with progressive liquefaction, the foundation capacity will reduce in stages.

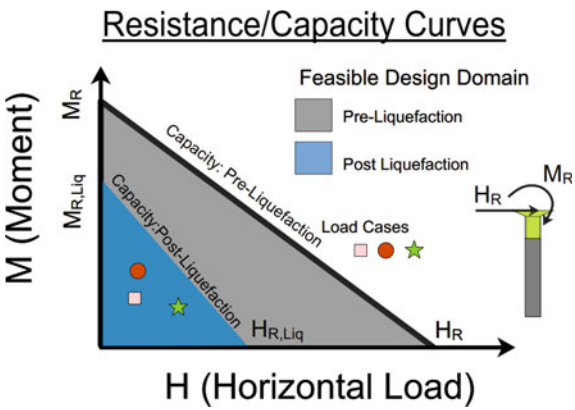


Fig. 13 Schematic diagram showing the demand and capacity

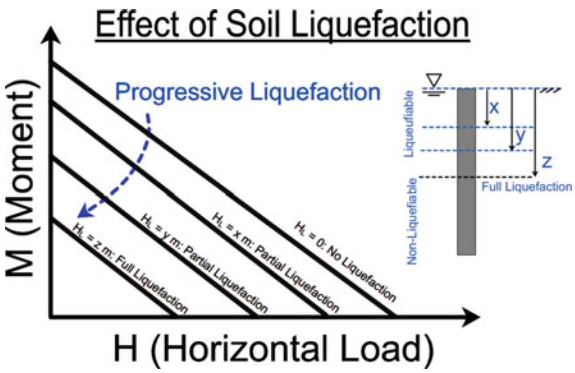


Fig. 14 Capacity envelopes following progressive liquefaction

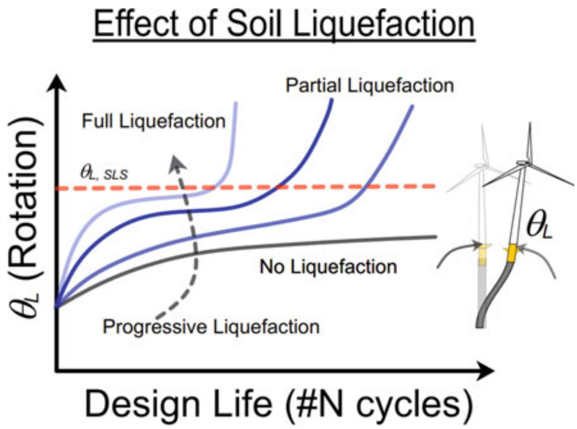


Fig. 15 Accumulation of tilt at pile head during progressive liquefaction

Therefore, considering the design life of the wind farm and tolerable risk, a probabilistic assessment can be performed for liquefaction triggering potential, thereby reducing the requirement for expensive ground remediation measures.

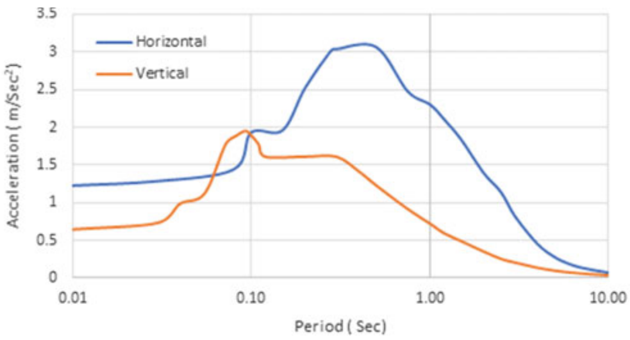
An additional hazard due to loss of foundation capacity is the accumulated tilt of the system. As shown in Fig. 15, pre-Liquefaction, the ground offers sufficient resistance (capacity) to prevent excessive tilt of the superstructure. However, strong shaking can result in high inertial demands at the hub level, resulting in large moments on the mudline. These demands can lead to tilting of the foundation which can be exacerbated by soil liquefaction and additional long-term loading, if not corrected. It is noted that appropriate soil constitutive models such as Lombardi and Bhattacharya (2014, 2016), Lombardi et al. (2017), and Dash et al. (2017) must be used to model soil-pile interaction after liquefaction.

5 Offshore Substations

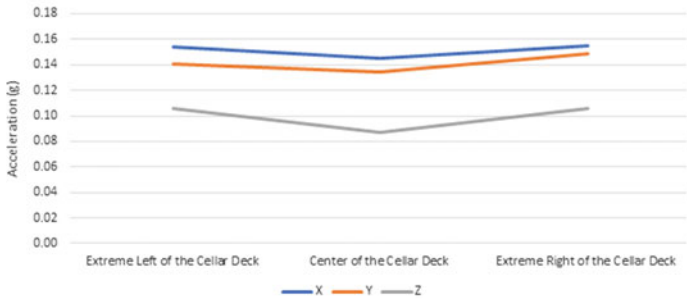
For a wind farm to be operable and producing energy after a seismic event, all the essential components (cables, turbines, and offshore substation) must also be operating. Offshore substations systems serve to collect and transmit the power generated from multiple turbines in the wind farm. Electricity generated from the turbines is transported through submarine cables and collected at a common substation. The substation is then connected to the grid, transferring the generated power onshore, see Fig. 2 for the layout. Therefore, characterization of the seismic resilience of offshore substations is crucial, when designing windfarms. In general, offshore substations are top-heavy structures similar to offshore rigs from the oil and natural gas industry. However, substations have acceleration-sensitive non-structural components which require detailed seismic design. Figure 16 shows two photos of offshore substations supported on monopile and jacket. The weight of the top side of the substation depends on the type of transformers. DC-type transformers are normally heavier than AC transformers. Figure 17 details an analysis of a 600 MW offshore substation structure, with an approximate weight of 40MN. A water depth of 70 m was assumed with a 50-year wave height of 15 m at a period of 10 s.

Fig. 16 Photographs of offshore substations. *Source* http://www.trianel-borkum.de/en/kraftwerk/converter-platform/ and https://commons.wikimedia.org/wiki/File:Offshore-132kv-Substation.jpg

Fig. 17 Analysis of offshore substation structure



(a)



(b)

Fig. 18 **a** Typical offshore substation structure. **b** Acceleration response at deck of the platform

The structure is analyzed using the response spectrum method, using the horizontal and vertical spectra presented in Fig. 18a. The analysis results are presented in Fig. 18b. Further detailed characterization requires time-history analyses with appropriate constitutive models to account for material non-linearity within the structural elements.

However, it must be noted that the design of each component of the wind farm: Turbines, offshore substation, and cables are done separately and therefore, the design may not be risk consistent, i.e., each structure may not be explicitly designed using the same set of ground motions selected at a specific hazard level.

6 Performance-Based Design and Risk Modeling

Conventional design (Load and Resistance Factor Design) of offshore turbines is performance-based, however with limited consideration to the explicit performance of the system. Performance-based design frameworks, such as that by the Pacific

Earthquake Engineering Research Center (PEER), supersede existing design methodologies, explicitly characterizing system performance, risk, and associated loss of function.

Offshore wind farms serve as lifeline structures, necessitating their need to remain functional post-seismic events. Further, costs incurred during times of zero output can significantly affect the agencies involved. Therefore, the operation of the group of wind turbines, and associated structures, including power stations and underwater cables, is critical. Further, explicit considerations toward satisfying performance limits enable greater transparency, bringing together stakeholders in the design stage. Figure 19 presents a preliminary workflow drawn from the PEER PBEE methodology, designed to compute the associated risk for wind farms post-earthquakes. Each step has been explained in the previous sections. Such a framework allows designers to estimate potential downtimes at different hazard levels systematically.

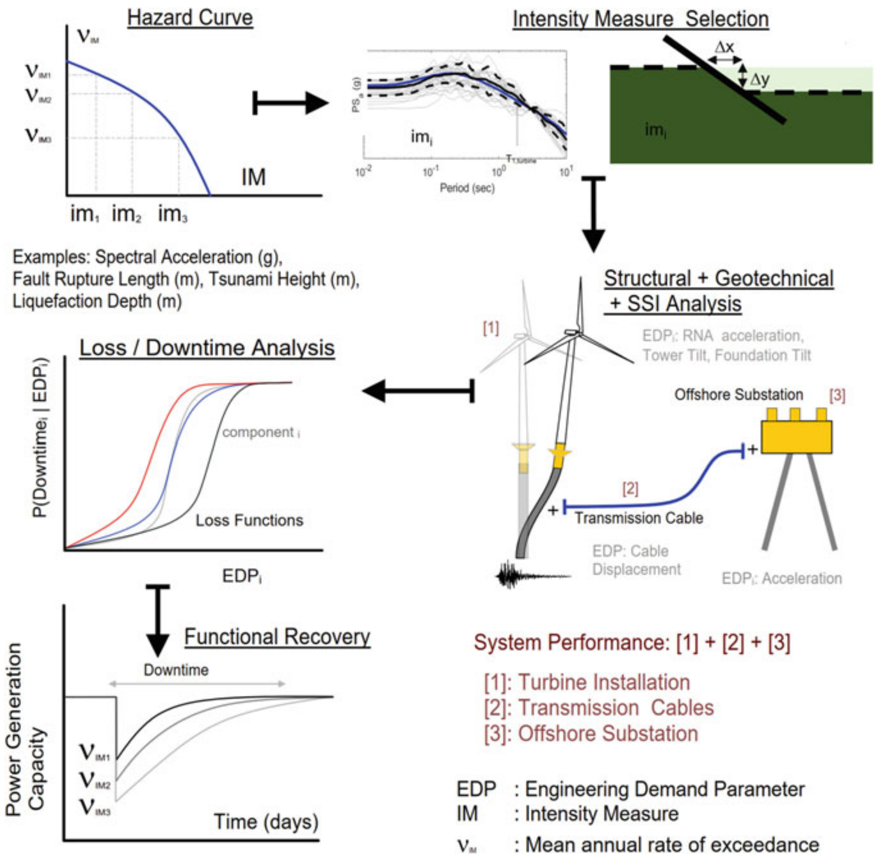


Fig. 19 Workflow for functional recovery analysis

The sensitivity of non-structural components in wind farms necessitates performance-based design. Further, the proposed framework can also look at other seismic hazards apart from ground shaking, including fault rupture, soil liquefaction, and seismically induced landslides.

EERI (2009) highlights the need to shift the existing prescriptive design paradigm toward functional recovery to obtain “better than code” seismic designs, where explicit considerations are made toward the loss of functionality of the structure post-earthquakes. Currently, the existing PBEE framework is predominantly used to study buildings and bridge structures. Therefore, more work is necessary for each step of this process while adopting the framework for the design of offshore wind farms, particularly given their short lifespan. Further work in the area should look at estimates of the loss and fragility functions for turbines, power station, and cables.

6.1 Approaches to RISK Modeling for Offshore Wind Farms

In catastrophe risk modeling, the seismic risk can be computed by convoluting seismic hazard, exposure, and vulnerability. For a typical offshore wind farm, as mentioned earlier, the seismic hazard includes:

1. Ground shaking: effect induced by propagation of body waves.
2. Liquefaction: loss of bearing capacity of soil caused by excess pore water pressure development.
3. Fault displacement: propagation of fault rupture at the surface. This is usually classified in primary and distributed faulting.
4. Submarine landslide: loss of stability of sloping grounds.
5. Tsunami: sudden large surges of water, reaching heights above 30 m.

Exposure components

The exposure component includes information related to the different assets operating in an offshore wind plant; these can be classified into generation assets (e.g., turbine) and transmission assets (e.g., cables, substations). The information included in the exposure model can be diverse but they normally include: geographical location (i.e., Latitude and Longitude), structural and non-structural characteristics, and economic data, including replacement costs, insured costs, etc.

The vulnerability model comprises fragility functions and loss functions. The fragility functions define the probability of exceeding a set of damage states given an intensity measure. One of the intellectual tasks is the relevant damage states that need to be identified for both generation and transmission assets. The loss functions define the probability distribution of loss for each damage state. The seismic risk assessment can be performed following one of the following approaches:

Method–1: Intensity-based approach: Hazard is computed in terms of an earthquake intensity measure. This can be expressed in terms of either spectral accelerations or displacement, or suite of accelerograms that have been selected and scaled for consistency with the design spectrum. The output is expressed in terms of annual probability of loss for a given intensity measure.

Method–2: Scenario-based assessment: This provides intensity parameters for one or more earthquake scenario, each of which is defined by the pair magnitude-distance. The chosen earthquake scenario often corresponds to the so-called worst-case scenario, although this may not be appropriate for offshore wind farms whose assets are largely unmanned. The output is expressed in terms of annual probability of loss for a given scenario (i.e., pair magnitude-distance).

Method–3: Time-based assessment: Wherein the ground-shaking hazard is determined through probabilistic seismic hazard analysis where all possible earthquake scenarios that might affect the study area are considered within a probabilistic framework. The time-based assessment is normally adopted by cat models used in the (re)insurance industry. The output is expressed in terms of annual probability of loss for a selected asset and/or a portfolio of assets.

7 Discussion and Conclusions

The available methods for the design of offshore Wind Turbine structures in seismically active areas are largely based on codes intended for ordinary buildings and critical facilities, such as Nuclear Power Plants. However, there are important differences between wind turbines and these structures as the former are designed for a significantly shorter life span and are predominantly unmanned. Therefore, it is questionable whether the available seismic provisions should be extended to design wind farms in seismic areas. It is argued that the entire design process is driven by the overall performance of the turbine, and the safety of the individual components (e.g., blades, gearboxes, etc.), whose failure may lead to prolonged downtimes and expensive reparation costs. Considering the relatively lifespan for which offshore wind turbines are typically designed, it is questionable whether a detailed PSHA is required to define the seismic hazard at the site. Furthermore, the paucity of recorded strong-ground-motion data at offshore sites introduces additional challenges and uncertainties in the seismic hazard estimates. The paper summarizes the various analysis and design issues.

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