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Effect of soil-foundation-structure interaction on the seismic response of wind turbines

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ABSTRACT

Soil-foundation-structure interaction can affect the seismic response of wind turbines. This paper studies the effects of soil-foundation-structure interaction on the seismic response of 65 kW, 1 MW, and 2 MW horizontal-axis wind turbines with truncated cone steel towers. Four types of foundations with frequency-based design were analyzed, including spread foundation, mono pile, pile group with cap, and anchored spread foundation. Soil is modeled both implicitly (subgrade reaction modulus) and explicitly. The finite element model developed using the ANSYS program was first validated using experimental data. Numerical models are then analyzed in both frequency and time domains using the Block Lanczos and generalized HHT-α formulations. Recommendations were given to simplify the soil-foundation-structure interaction analysis of wind turbines subjected to seismic loading.

1. Introduction

Wind turbines are the world's fastest-growing source of renewable energy across America and around the globe. In 2015, the US wind industry installed a total of 8598 Megawatts (MW) of new power capacity, a 77% increase over 2014 [1]. Decreasing number of prime sites with high wind availability and good access, coupled with increasing demand for higher power output has increased the need to use taller towers with longer blades especially in less windy sites [2]. In seismic regions, taller wind turbines develop large seismic forces that are sometimes bigger than the wind forces [3]. In such cases, an inaccurate estimate of the seismic force can result in either structural failure or uneconomic design. An important factor in estimating the seismic forces on wind turbines is the soil-foundation-structure interaction, which is affected by different parameters including turbine size, foundation type, and soil properties.

This paper analyzes the soil-foundation-structure interaction effects on the seismic response of wind turbines. Three wind turbine capacities are selected for the study, namely, 65-kW (similar to the experimental model), and 1-MW and 5-MW (representing the current lower and upper threshold of utility scale sizes). In this study, horizontal-axis turbines with truncated cone steel towers were used. Foundation types are spread foundation, mono pile, pile group with cap, and anchored spread foundation. Soil effects are included using modulus of subgrade reaction and also explicit model. The finite element model developed using the ANSYS program was first validated using experimental data. Natural frequencies of numerical models are then examined using the Block Lanczos method with ANSYS program. Next, time history analysis is performed using the records from the 1992 Landers Earthquake and the generalized HHT-α formulation. Recommendations are provided to simplify the soil-foundation-structure interaction analysis of wind turbines subjected to seismic loading.

2. Literature review

Prowell et al. [4] analyzed a full soil-structure system with a 5-MW wind turbine with a hub height of 90 m and a rotor diameter of 126-m. A detailed finite element model of the turbine was created, including a full three-dimensional (3-D) soil mesh to study the influence of soil-structure interaction (SSI) on the dynamic properties and response. The turbine was modeled on 3- to 15-m (9.8- to 49-ft) thick soil profiles with varying stiffness and subjected to a 1994 Northridge Earthquake record. Their investigation found that for these conditions the SSI influence on the first and second longitudinal bending modal parameters was relatively

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minor, while the SSI influence on the maximum moment and shear demand distributions along the tower height was more significant. Prowell et al. recommended the selection of a range of carefully chosen ground motions to match the anticipated shaking for the proposed site in SSI analyses.

Hongwang [5] analyzed the seismic response of two 1.65-MW and 3-MW wind turbine models, including the SSI and P-Δ effects under horizontal and vertical components of six historical earthquake time histories. The SSI was modeled by connecting the turbine base to a rigid support mounted on translational and rotational springs and dampers. The results showed that the SSI caused a 7% decrease in the first natural frequency, 10% decrease in the horizontal acceleration at the top of the tower, 10–12% decrease in the tower base moment, and 5–6% decrease in the tower base shear force. The SSI had no significant effect on the vertical acceleration and axial force of the towers, but the P-Δ effect increased the tower base moment slightly.

Kourkoulis et al. [6] performed a parametric seismic analysis on two wind turbines with 2 MW and 3.5 MW capacities supported on suction caisson foundations under static cyclic and earthquake loads. The analysis included non-linear SSI caused by sliding between the caisson skirt and the soil and gap formation. The model included 3-D soil elements with shell elements representing the interface, beam elements for tower, and a concentrated mass representing the rotor blades and nacelle. The results showed that interface failure could reduce the capacity of suction caisson foundations especially in foundations with deep caissons. It was also shown that foundation rotation caused by interface problems could cause irrecoverable displacement on the nacelle level. Increasing the caisson diameter was found to be a better solution compared to increasing the depth of embedment.

Kjørlaug et al. [7] studied the dynamic response of a wind turbine supported on mono pile foundations under horizontal and vertical earthquake excitations. A non-homogeneous, deep-soil stratum was considered. Their analyses showed an acceleration amplification factor of 2 from the ground surface to the top of the tower. Vertical earthquake excitations were found to be critical in low-to-moderate seismic areas.

Cheng and Lien et al. [8] evaluated the load bearing characteristics of the jacket foundation pile for offshore wind turbines on the west coast of Taiwan. Effective stress analysis, with consideration of pore pressure generation and soil/liquid coupled analysis, was conducted. A numerical procedure to evaluate the design of offshore wind turbine foundation piles in the sand and clay interlayered soil was developed.

Loubser et al. [9] analyzed a 3D finite element model of wind turbine and foundation with fully non-linear material and discreet reinforcement using DIANA 3-D software. It was found that a 30% material saving can be achieved using PLAXIS model.

Katsanos et al. [10] presented a comparative survey of the published research relevant to the seismic analysis, design and assessment of wind turbines. The use of full FE models, including the nacelle and the rotor blades, the supporting tower as well as the soil-foundation system, along with time domain analysis was recommended. It was also shown that due consideration should be paid to the SSI phenomena, since the soil compliance and the earthquake-induced inertial interaction between the superstructure and the soil foundation system may significantly modify the dynamic characteristics of a wind turbine and its seismic response. It was also found that current foundations systems of wind turbines with gradually increasing size in areas of high seismicity may be vulnerable. It was suggested that advanced techniques of modeling and analysis should be adopted to scrutinize the demanding foundation structures and the soil underneath.

3. Methodology

A parametric study is performed in both time and frequency domains. A series of wind turbines with different sizes and capacities with different foundation types and two types of soil model are analyzed using the Block Lanczos method for modal analysis and generalized HHT-α method for transient analysis. Block Lanczos is a frequency domain method used by the ANSYS program. In this method, eigenvalue solver uses the Lanczos algorithm where the Lanczos recursion is performed with a block of vectors. Block Lanczos uses the sparse matrix solver and is especially powerful when searching for eigenfrequencies in the eigenvalue spectrum of a given system. The convergence rate of the eigenfrequencies, when extracting modes in the mid-range and higher end of the spectrum, will be about the same as when extracting the lowest modes. This method is recommended to find many modes of large models and it can handle poorly shaped solid and shell elements [11]. The generalized HHT-α method is an implicit time scheme similar to the Newmark method in which the structural stiffness matrix is factorized to solve for $\{u_{n+1}\}$ at time t_{n+1}. Systems are assumed to have frequency based design.

3.1. Frequency-based design

In the analysis and design of wind turbines, tower design is usually controlled by its frequency limits to prevent interference with turbine operational frequencies [12]. Fig. 1 shows the allowable frequency range in a typical frequency design problem. Natural frequencies (f_{n1}, f_{n2}, etc.) should be separated from operational frequencies (f_{op1}, f_{op2}, etc.) with a safety margin. Considering that the operational frequencies of utility scale wind turbines typically range from 0.1 Hz for larger turbines to 0.5 Hz for smaller ones, the natural frequency of these turbines should be above this range to prevent resonance. In other words, the ratio of natural to operational frequency must be greater than 1, preferably with a 10% safety margin. The recommended values for this factor of safety are between 1.1 and 2. If the safety margin is not big enough, the effect of soil-foundation-structure interaction can shift the natural frequencies of the structure too close to operational frequencies and dynamic amplification can occur. Therefore, assuming a fixed tower base in the design may not be conservative; and it may be necessary to analyze the soil-foundation-structure interaction. In other words, unlike other structures, the design of wind turbine foundations may not be governed by soil bearing capacity alone and can be significantly affected by the dynamic properties of the wind turbine.

3.2. Seismic load

Selected seismic load should have frequencies close to the frequency of the turbine so that they can excite the natural modes of the turbine. The East-West and vertical components of the Landers Earthquake (June 28th, 1992) record have Peak Ground

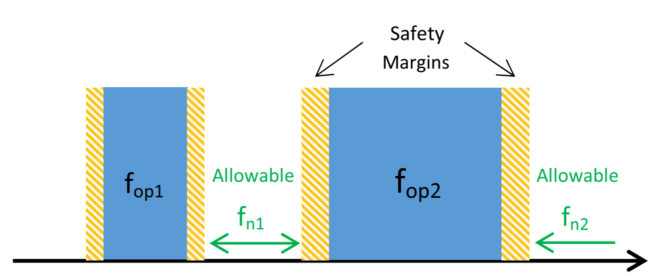

Fig. 1. Allowable frequency range in frequency-based design.

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Accelerations (PGA) of 0.15 g and 0.17 g, respectively recorded at Desert Hot Springs station (DHS), with moment magnitude of 7.3. Located on deep alluvium, DHS is 23 km (14.3 miles) far from the Landers Earthquake fault. The ground is classified as stiff soil, i.e., site class D with an average shear wave velocity within the upper 30 meters of the ground, V_{s30} = 345 m/s (1132 ft/s) [13]. The dominant frequencies of the record are 1.06 Hz and 6.74 Hz. Fig. 2 shows the input seismic acceleration records used in the study.

3.3. Assumptions and Considerations in modeling and analyses

- X direction is parallel to the rotor's axis, Y direction is perpendicular to the rotor axis, and Z direction is parallel to the tower.
- The wind turbine is assumed to be parked, which means the blades are locked to prevent excessive force on the mechanical parts.
- The tower and nacelle connection is bonded in all degrees of freedom (DOFs).
- Global and local buckling modes of towers are neglected assuming they are designed to resist buckling. In practice, it is usually achieved by using stiffeners along the tower length. Preventing local buckling without actually modeling the stiffeners reduces the number of the nodes and elements and increases the analysis speed drastically.
- In reference models with no foundations, towers are fixed at the bottom in all translational and rotational DOFs.
- In all numerical models, all parts are flexible. These include foundations, tower, rotor blades, and nacelle.
- The duration of transient analysis is chosen to be longer than the duration of the earthquake load so that the free-vibration phase is captured.
- Acceleration responses are given as a fraction of gravitational acceleration (g).
- From the finite element analysis, the determined stiffness is higher than the experimental values. Increasing the number of elements reduces the stiffness and mesh size-stiffness curvature is asymptotic to experimental stiffness.
- In order to isolate the effects of foundations, the load and foundation size variations due to soil effects are ignored. Therefore, foundation types investigated in this study are suitable for certain types of soil conditions that maybe different from the ones utilized in this parametric study. Also considering the specific characteristic of the soil at the recorded site of the input earthquake load, the numerical values obtained from these analyses should not be used in actual design cases.

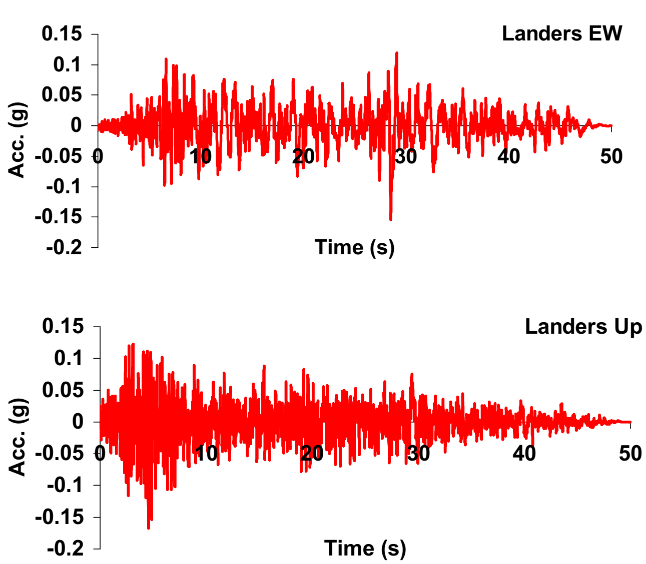

Fig. 2. Input seismic acceleration records.

4. Parametric study

4.1. Validation of numerical method

Like any numerical models, finite element models should be validated before any application. This is done to ensure the accuracy of material models, element formulations, and mathematical calculations. In this study, the experimental data obtained from testing a full-scale wind turbine on a shaking table [14] is used to validate the models. The test was performed on an industrial scale 65-kW wind turbine with 23 m (75 ft) height and 10,700 kg (1616 slug) mass. Earthquake load is a uniaxial horizontal excitation perpendicular to the rotor's axis, applied to the base of the tower through a 7.6 m $\times$ 12.2 m (25 ft $\times$ 40 ft) outdoor shaking table with a stroke of ±0.75 m (±29.5 in.). The table can exert a peak horizontal velocity of 1.8 m/s (3.9 ft/s), a horizontal force of 6.8 MN (1.53×10^6 lbf), and a vertical force of 20 MN (4.5×10^6 lbf). The shaking table is capable of simulating frequencies of 0–33 Hz. The input acceleration is the East-West component of Landers 1992 earthquake. The input acceleration is filtered for DC offset and high-frequency noise using a 0.05–25 Hz band-pass filter. The accelerometer located at the top of the nacelle recorded a peak acceleration response of 0.28 g at t = 30.48 s. The observed first and second natural frequencies are 1.7 Hz and 12 Hz, respectively. Mode shapes are constructed using an average of the amplitude and phase of the transfer function and are depicted in Fig. 3 (a). Acceleration transfer function is also shown in Fig. 4. Equivalent viscous damping at the first natural frequency is found to be 0.86% (see Fig. 5).

The 3-D numerical model consists of tower, nacelle, rotor blades, and hub. Nacelle and hub are modeled as solid elements and the tower as shell elements with a uniform thickness of 60 mm (2.36 in.) along the length. Simplifying the blade geometry will not cause a problem as long as the mass distribution is not altered due to the fact that local modes of rotor blades are very different from tower modes. Correct mass distribution is accounted for by adjusting the blade width along the length of the tower. The mass of miscellaneous tower parts (flanges, bolts, etc.) is 1929 kg (132.2 slug) is added to the tower as a distributed mass along the length of the tower. Two different materials are used, i.e., composite material (fiberglass and carbon fibers) [15] for the rotor, and structural steel for tower, nacelle, and hub. Because the experimental nacelle is lighter than a solid steel box with the same volume, an equivalent lower density is used for the nacelle model. Table 1 summarizes the material properties used in the numerical model. All materials are assumed to be linear. Tower and blades are meshed using shell181 elements. The nacelle and hub are meshed with solid186 elements.

Modal analysis is performed using the Block Lanczos method. The analysis includes the first 100 modes. The effective mass of these frequencies is found to include more than 90% of the total mass. The calculated first and second natural frequencies are 1.65 Hz and 9.14 Hz, respectively. To evaluate the correlation of mode shapes, modal assurance criterion (MAC) is also calculated. Table 2 compares the MAC values between the experimental and numerical mode shapes. Numerical mode shapes are shown in Fig. 3(b). Transient analysis is performed using the generalized HHT-α method. A time step of 0.02 s is found to be sufficient. Using the damping value from experimental test, the peak numerical acceleration occurs at t = 28.7 s and is equal to 0.287 g. The numerical analysis results shows that the first and second mode shapes are similar to the experimental mode shapes. In the time domain,

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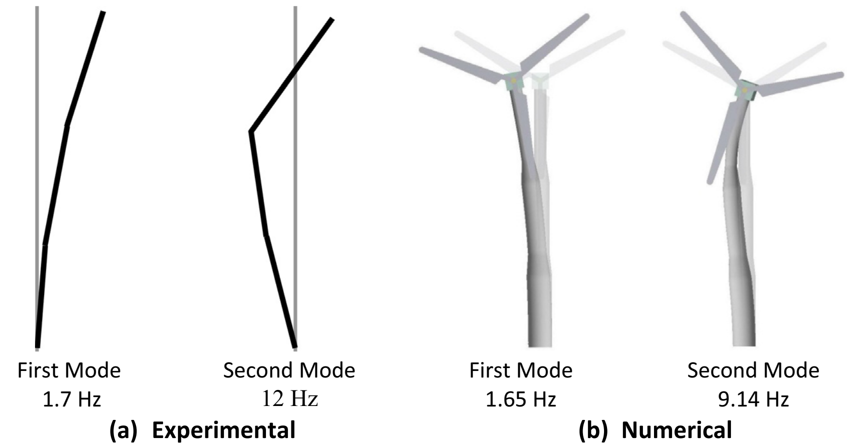

Fig. 3. Natural modes of the 65-kW turbine [3].

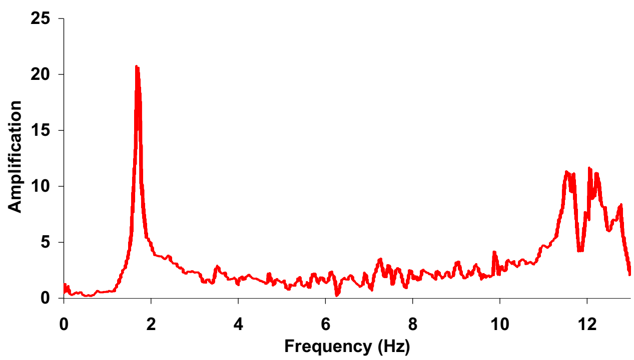

Fig. 4. Acceleration transfer function from the tower base to the top of the nacelle [3].

the computed peak acceleration response is approximately 2.5% higher than the experimental value. The numerical errors are, therefore, small and the finite element model is validated.

4.2. Numerical soil models

To investigate the effects of soil-foundation-structure interaction, the effect of soil can be included implicitly or explicitly. In implicit methods, the effects of the soil are added to the analysis using springs and dampers without modeling the soil itself. Different implicit analysis techniques use different assumptions and are suitable for specific problems. In an explicit analysis method, however, the soil itself is modeled with finite elements. The soil body should be large enough to be accurate and, therefore, it's more time-consuming compared to the implicit method. Implicit method is usually used in critical problems. Two common implicit techniques are linear soil pressure distribution and K-model [16]:

4.2.1. Linear soil pressure distribution model

In this method, the soil pressure is assumed to be distributed linearly under the foundation. This soil pressure depends on the foundation forces only and nonlinear reactions cannot be modeled. Linear soil pressure distribution model is a good approximation for rigid foundations like column footings; however, it is conservative for flexible foundations.

4.2.2. K-Model

This implicit method simulates soil behavior by a series of elastic springs under the foundation and results in a nonlinear soil pressure distribution proportional to the foundation settlement. The stiffness of K-model springs are referred to as K or modulus of subgrade reaction. The K-model is often used to analyze footings under single concentrated load. In the K-model, K is a combination of soil and structure stiffness and, therefore, in design situations it should be determined by trial and error. Fig. 6 shows a soil pressure distribution in K-model.

4.2.3. Explicit model

This method is the most accurate way to analyze the soil-structure interaction. Soil body is modeled fully or partially and damping can be added to the structure, which results in a more realistic and economical design [12]. Depending on the size and complexity of the soil body, explicit model can be time-consuming and, therefore, costly.

In this study, only Implicit K-model and Explicit Soil model are considered. Linear model is not used since it ignores the effects of foundation flexibility.

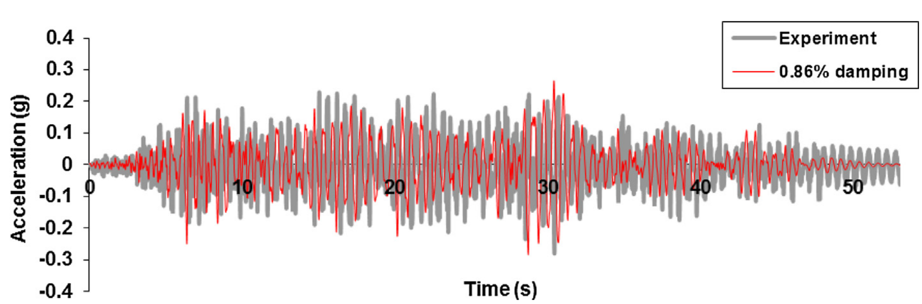

Fig. 5. Experimental and numerical transient results with 0.86% damping.

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Table 1
Material properties used in the finite element model.

Property	Fiberglass and carbon fibers composite	Steel
Density	648 kg/m^3 (1.26 slug/ft^3)	7860 kg/m^3 (15.25 slug/ft^3)
Young's modulus	235,000 MPa (34,084 ksi)	200,000 MPa (29,000 ksi)
Poisson's ratio	0.3	0.3
Tensile yield strength	3920 MPa (569 ksi)	250 MPa (36,000 psi)
Tensile ultimate strength	3920 MPa (569 ksi)	460 MPa (66,700 psi)

Table 2
Modal assurance criterion values between the experimental and numerical modes.

MAC Values		Experimental	
		Mode 1	Mode 2
Numerical	Mode 1	0.95	0.13
	Mode 2	0.11	0.87

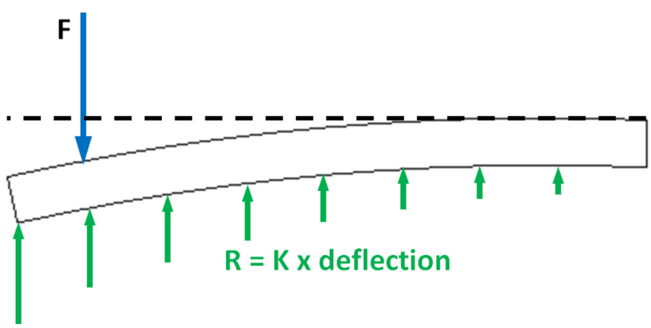

Fig. 6. Soil pressure distribution in K-model.

4.3. Foundation types

Based on turbine properties and soil conditions, wind turbine foundations can have different design and configurations. These designs can be classified into four major categories, namely, spread foundations, mono piles, pile groups with cap, and anchored spread foundations [17,18].

4.3.1. Spread foundation

Spread foundations are the cheapest and easiest types of foundations to build. If soil has enough bearing capacity, spread foundation is the first design choice. Spread foundations are usually rectangular, circular, or octagonal and made of reinforced concrete and/or steel. Overturning resistance usually comes from a combination of weight of the foundation and the backfill soil on the top. Fig. 7 shows a spread foundation with pedestal.

Table 3
Physical properties of wind turbines.

Property	65-kW Turbine	1-MW Turbine	5-MW Turbine
Hub diameter, length	0.4, 0.25 m 1.31, 0.82 (ft)	1.6, 0.5 m 5.25, 1.64 (ft)	2.2, 0.5 m 7.22, 1.64 (ft)
Hub height	22.6 m (74.1 ft)	61.14 m (200.6 ft)	90 m (295.3 ft)
Rotor blades diameter	16 m (105 ft)	60.62 m (198.8 ft)	126 m (413.4 ft)
Rotor blades mass	6400 kg (439 slug)	42,000 kg (2878 slug)	110,000 kg (7537 slug)
Rotor blades thickness	60 mm (2.36 in.)	480 mm (18.9 in.)	550 mm (21.65in.)
Nacelle width, height, length	1.45, 1.4, 3.28 m 4.76, 4.59, 10.76 (ft)	3.93, 3.93, 10.09 m 12.89, 12.89, 33.1 (ft)	3.93, 3.93, 10.09 m 12.89, 12.89, 33.1 (ft)
Nacelle mass	2400 kg (164 slug)	53,700 kg (3680 slug)	240,000 kg (16,445 slug)
Tower diameter-outer, bottom	2.02 m (6.6 ft)	3.875 m (12.7 ft)	6 m (19.7 ft)
Tower diameter-outer, top	1.06 m (3.5 ft)	2.45 m (8 ft)	3.87 m (12.7 ft)
Tower length	21.9 m (71.8 ft)	57.19 m (187.6 ft)	88.5 m (290.3 ft)
Tower mass	1900 kg (130 slug)	78,600 kg (5386 slug)	347,460 kg (23,809 slug)
Tower thickness	5.3 mm (0.21 in.)	18 mm (0.71 in.)	27 mm (1.06 in.)

4.3.2. Mono pile

In some cases, the top soil cannot provide sufficient bearing capacity and using a pile can be a viable option. Mono piles may or may not bear on the bedrock and they transfer the wind turbine loads through a combination of bearing and frictional resistance. Mono piles are usually made of reinforced concrete with or without steel pipe and the length can be 1/3 to 2/3 of the tower height [19]. Overturning resistance in mono piles is provided by axial and bending strength of the pile.

4.3.3. Pile group & cap

Depending on the soil condition, it may be necessary to use two or more piles in a group configuration. Usually, all piles in a pile group are similar and connected with a cap. The wind turbine loads are applied on the cap and distributed to individual piles. Depending on the spacing of the piles, the capacity of the pile group can be equal or less than the combination of individual piles due to overlapping stress zone around the piles.

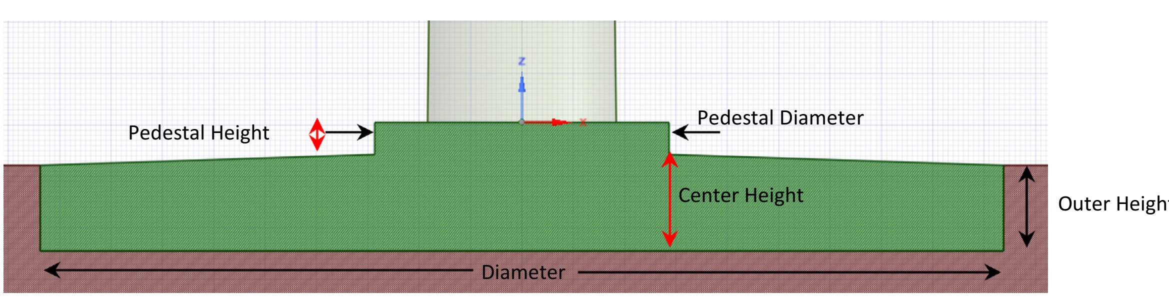

Fig. 7. Spread foundation and pedestal.

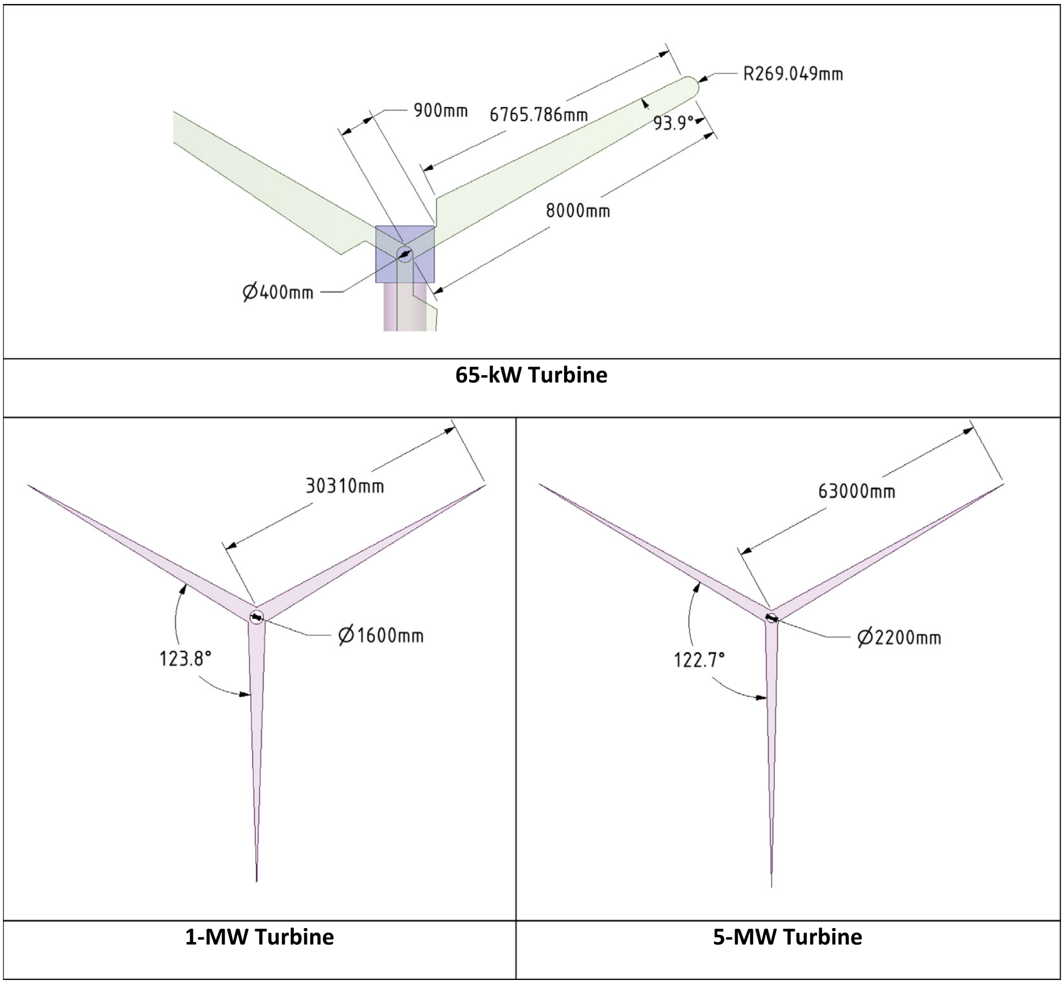

Fig. 8. Blade dimensions.

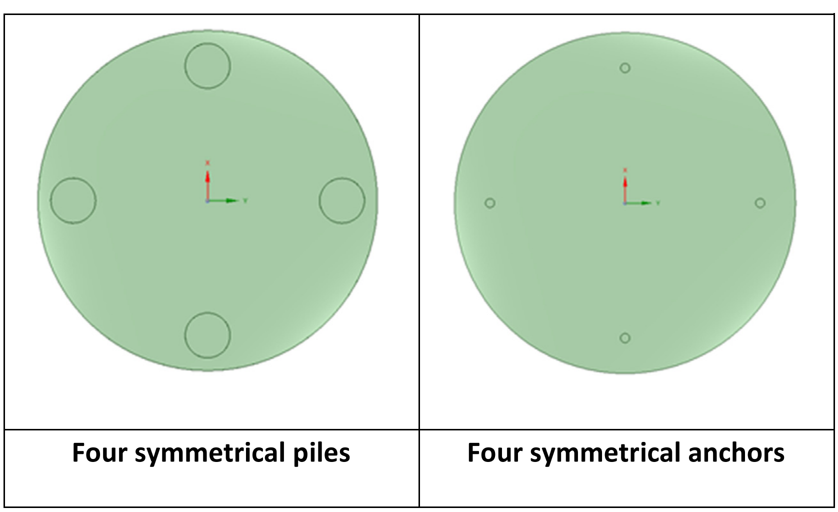

Fig. 9. Placement of piles and anchors.

4.3.4. Anchored spread foundation

In cases where soil doesn't have enough bearing capacity and bedrock is easily accessible, the spread foundations can be anchored to the bedrock. In this case, the spread section is usually made of reinforced concrete. Anchors can be steel cables, helical steel shaft, or steel tendons [20]. Anchored spread foundations offer minimal footprint areas and are ideal for rocky sites where high bearing capacities are available.

4.4. Parametric models

4.4.1. Geometry

Three turbine sizes are selected; a 1-MW and a 5-MW utility scale turbine, and the 65-kW industrial scale turbine from the experimental study described above. The towers are truncated steel cones with constant thickness through the height and increased diameter at the base. Rotor blades in all turbines are

Table 4
Dimensions of the soil bodies and foundations.

Part	65-kW Turbine	1-MW Turbine	5-MW Turbine
Spread footing pedestal height	0.253 m (0.83 ft)	0.658 m (2.16 ft)	1.012 m (3.32 ft)
Spread footing pedestal diameter	2.314 m (7.59 ft)	6.016 m (19.74 ft)	9.256 m (30.37 ft)
Spread footing center height	0.758 m (2.49 ft)	1.971 m (6.47 ft)	3.032 m (9.95 ft)
Spread footing outer height	0.673 m (2.21 ft)	1.75 m (5.74 ft)	2.692 m (8.83 ft)
Spread footing diameter	7.576 m (24.86 ft)	19.698 m (64.62 ft)	30.304 m (99.42 ft)
Mono pile cap height	1.011 m (3.32 ft)	2.629 m (8.62 ft)	4.044 m (13.27 ft)
Mono pile cap diameter	2.314 m (7.59 ft)	6.016 m (19.74 ft)	9.256 m (30.37 ft)
Mono pile height	10 m (32.81 ft)	26 m (85.3 ft)	40 m (131.23 ft)
Mono pile diameter	2.02 m (6.63 ft)	5.252 m (17.23 ft)	8.08 m (26.51 ft)
Pile group height	10 m (32.81 ft)	26 m (85.3 ft)	40 m (131.23 ft)
Pile group diameter (each)	1 m (3.28 ft)	2.6 m (8.53 ft)	4 m (13.12 ft)
Anchor height	10 m (32.81 ft)	26 m (85.3 ft)	40 m (131.23 ft)
Anchor diameter	0.2 m (0.66 ft)	0.52 m (1.71 ft)	0.8 m (2.62 ft)
Pile/Anchor distance to cap center	3 m (9.843 ft)	15.6 m (51.181 ft)	24 m (78.74 m)
Soil depth	10.673 m (35.02 ft)	27.75 m (91.04 ft)	42.692 m (140.07 ft)
Soil square width	20 m (65.62 ft)	52 m (170.6 ft)	80 m (262.47 ft)

three-bladed cantilevered and is made of carbon fiber-reinforced with epoxy. Table 3 summarizes the physical properties of the three wind turbines used in this study. Detailed dimensions of the blades is given in Fig. 8. Explicit soil bodies are cuboid with square areas.

Four types of foundations are investigated, in addition to a fixed-base model without foundation. Spread foundations are circular slabs with pedestal, with varying thicknesses along the radius as shown in Fig. 7. Mono piles also have a pedestal on top. Pile groups and anchors are in groups of four with each pile or anchor placed symmetrically relative to the center of the cap as shown in Fig. 9. Dimensions of the soil bodies and foundations are given in Table 4.

4.4.2. Material properties

As mentioned, design of wind turbine foundations is often controlled by turbine operational and natural frequencies, in addition to the bearing capacity of the soil. Assuming a frequency-based design for foundations being investigated, the soil-foundation properties should be first adjusted to achieve similar first natural frequencies. The response of the structure is then analyzed to evaluate the effect of soil-foundation-structure interaction on the seismic response of the structure. To achieve this, the Young's modulus (E) in explicit models and foundation properties are first selected. In K-models, K values are determined using trial and error. Next, displacement at the top of the nacelle is recorded for each system. For example, for a 1-MW tower on a spread foundation, it is determined that a soil with $K = 30 \times 10^6\ N/m^3$ ($110.52\ lbf/in^3$) results in an equal first mode to the same structure on the selected soil. The unit weight of soil (γ) is $25{,}000\ N/m^3$ ($159\ lbf/ft^3$). Other material properties are similar to validation model given in Table 1.

4.4.3. Meshing

Turbines are analyzed with detailed numerical models including the tower, rotor blades, and nacelle. Modeling tower details compared to an idealized model helps with taking into account

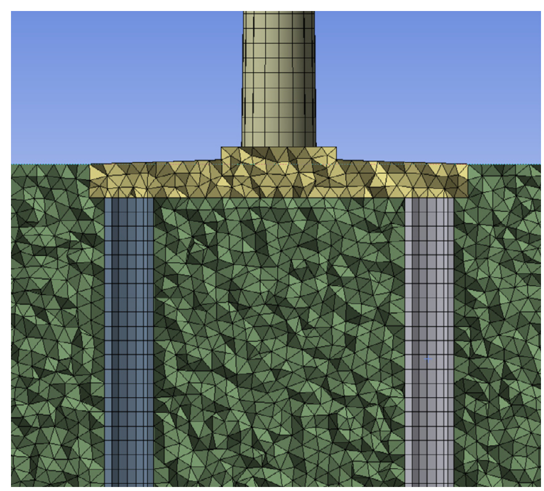

Fig. 11. Mesh details for the 1 MW turbine with pile group foundation and explicit soil.

Fig. 10. Finite element model of 1-MW wind turbine with different foundation types.

Table 5
Number of nodes and element for the parametric model parts.

Part	Element Type	65-kW Turbine		1-MW Turbine		5-MW Turbine	
		Elements	Nodes	Elements	Nodes	Elements	Nodes
Tower	Shell181	2174	2195	1751	1768	1734	1751
Blades	Shell181	445	573	335	488	432	632
Nacelle & hub	Solid186	6606	10054	7592	11516	1816	2942
Spread foundation	Solid186	13634	58355	23560	99474	19026	80488
Mono pile	Solid186	28506	41871	29393	43122	27588	40527
Pile group & cap	Solid186	51533	80059	15984	71913	76876	49371
Anchored spread	Solid186	35132	160335	40941	184603	36407	165617

Table 6
Natural frequencies and mode shapes of the systems.

Foundation Type	Mode	Frequency (Hz)						Mode Shape
		65-kW Turbine		1-MW Turbine		5-MW Turbine		
		K-Model	Explicit	K-Model	Explicit	K-Model	Explicit	
Spread	1	1.55	1.55	0.42	0.42	0.23	0.22	1st Translational X
	2	1.59	1.59	0.42	0.42	0.23	0.22	1st Translational Y
	3	7.96	[a]3.74	3.10	3.10	1.43	1.41	2nd Translational Y
Mono-Pile	1	1.53	1.56	0.42	0.42	0.23	0.22	1st Translational X
	2	1.64	1.60	0.42	0.42	0.23	0.23	1st Translational Y
	3	8.42	6.73	3.09	3.11	1.42	1.39	2nd Translational Y
Pile-Group & Cap	1	1.54	1.58	0.42	0.42	0.23	0.22	1st Translational X
	2	1.64	1.62	0.42	0.42	0.23	0.23	1st Translational Y
	3	8.38	4.08	3.10	2.80	1.42	1.36	2nd Translational Y
Anchored Spread	1	1.52	1.56	0.42	0.42	0.22	0.22	1st Translational X
	2	1.61	1.60	0.42	0.42	0.23	0.23	1st Translational Y
	3	8.91	4.96	3.10	3.11	1.32	1.32	2nd Translational Y
None	1	1.65						1st Translational X
	2	1.65						1st Translational Y
	3	9.14						2nd Translational Y

[a] 1st translational Z.

Table 7
Peak acceleration and deformation response at top of the nacelle and maximum von Mises stress at tower base for 1 MW system with K soil and different foundation types.

Foundation type	a_{max} (g)		δ_{max} (mm)		σ_{max} (MPa)	
	X	Y	X	Y	X	Y
None (Fixed-base)	0.203	0.199	119	119	47	48
Spread	0.225	0.216	126	126	29	28
Mono pile	0.229	0.219	124	125	30	28
Pile group & cap	0.226	0.217	129	127	30	28
Anchored spread	0.226	0.219	131	128	31	28

the effect of stress concentration in the connections and also stress distribution in the tapered sections. A detailed model also increases the accuracy of analysis by realistically distributing the mass across the body. Tower and blades are meshed using shell181 elements. Nacelle, hub, soil, and foundations are meshed with solid186 elements. Resulting finite element model of the 1 MW wind turbine with and without foundations is shown in Fig. 10. Cross section of the pile group & cap foundation with explicit soil model is shown in Fig. 11. Meshing summary for various parts of numerical models is given in Table 5.

4.5. Parametric analysis

4.5.1. Modal analysis

Parametric modal analysis is performed using the Block Lanczos method. The analysis includes 100 modes. The effective mass of these frequencies is found to include more than 90% of the total mass. The first three natural modes of the systems and their mode shapes are given in Table 6. Frequencies are given for both K and explicit soil models. Frequency of model with no soil and foundation is also given as a reference.

4.5.2. Transient analysis

Parametric transient analysis is performed using the generalized HHT-α method with a time step size of 0.02 s. The analyses are performed using the horizontal component of 1992 Landers Earthquake record with a damping value of 1.0%. In all analyses, the measured response is in the direction of the earthquake load component. The horizontal component of the seismic record is first applied in the X direction and the acceleration response at the top of the nacelle is measured. The analysis is then repeated for the Y direction.

Table 7 summarizes the peak acceleration and deformation response at the top of the nacelle and also the maximum von Mises stress at tower base for all models.

5. Results

The results of modal analyses presented in Table 6 show that adding soil and foundation has decreased the first and second natural frequencies of the model with 65-kW turbine. In the model with 1-MW and 5-MW turbines, this change is small. Adding the soil and foundation is found to have more effect on the third natural frequency for all turbine sizes. This effect, however, depends on the type of soil model and foundation used in the analysis. For 65-KW turbines, adding a spread foundation with explicit soil, causes the mode shape of third natural frequency to shift from second translational mode in the Y direction to first translational mode in the Z direction. This shift in the mode shapes wasn't seen in other analyses. It's also seen that K-models have lower first frequencies compared to the explicit soil models but differences are small enough to assume a frequency based design. For the 1-MW and 5-MW systems, the first natural frequencies are similar for all foundation types and soil models. It is seen that K-models have higher second natural frequencies compared to explicit models. Among different types of foundations, pile group and cap have the highest second natural frequency and spread foundations have the lowest. The maximum overall difference between second natural frequencies is only 3.6%. In case of third natural frequency, however, soil model and foundation types have a significant effect. It is seen that for the 65-KW and 1-MW systems with K soil model, the third natural frequency is consistent for all foundation types. For the 5-MW system with K soil model, however, the third natural frequency of anchored spread foundation is 7–8% lower compared to other foundation types and 13% lower compared to system with no foundation. In the 65-kW system with explicit soil, the third natural frequencies vary for different foundation types. For the 1-MW system with explicit soil, except for pile group and cap foundation, all foundation types have similar frequencies. For the 5-MW turbine system with explicit soil, the third natural frequency of anchored spread foundation is 13% lower compared to system with no foundation and other frequencies vary for different foundation types.

The results of transient analyses presented in Table 7 show that adding the effects of soil and foundation, has caused 8–13% increase in horizontal acceleration at the top of the nacelle. The horizontal displacement at the top of the nacelle are also increased 4–11%. The increase is caused by rigid rotation of the foundation and except for response in the X direction of the system with mono pile, is slightly higher in the X direction.

6. Conclusions

1. The natural frequencies obtained from the finite element model compare well with the experimental frequencies obtained from the literature for the 65-kW turbine. Hence, numerical analysis is a valid tool for the seismic analysis of wind turbine and their foundations.
2. For the specific cases studied in this research, the natural frequencies of the soil-foundation-wind turbine systems with frequency-based design are comparable for both K-model and explicit soil model. Therefore, soil can be modeled by K-model, instead of using explicit soil model which is more complicated and requires more analysis time.
3. For the specific cases studied in this research, the effect of soil-foundation-structure interaction on the seismic response of wind turbines is negligibly minor. Therefore, seismic analysis of the wind turbine towers in these cases can be simplified by assuming them fixed at the base.

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