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Numerical study of vibrations induced by horizontal-axis wind turbine on a steel building

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One of the most important acceptance issues regarding the implementation of small-scale wind turbines in urban environments is related to human comfort. Turbine-induced vibrations can have a large influence on small-scale turbine implementation for urban wind harvesting. A typical five-storey steel-framed office building with a roof-mounted, horizontal-axis wind turbine was numerically analysed. The aim of the work was to investigate the increase in vibration after the installation of the turbine. A comparison was made between floor vibrations caused by wind action on the building only and by wind action on the building and turbine combined. The results were then compared with requirements given in various design codes. The main outcome of the analysis is that the installation of the turbine did not compromise the building's serviceability.

1. Introduction

Demands for higher energy efficiency and use of renewable energy resources are constantly increasing. In addition to large onshore and offshore wind farms, the harvesting of wind energy as a renewable energy resource by the use of small-scale wind turbines (SSWTs) in urban built environments is attractive. Given that energy production takes place at the place of consumption, the installation of SSWTs in urban regions can result in higher energy efficiency.

According to Wineur (2005), small wind turbines are defined as those that are specially designed for the built environment and can be located on buildings or on the ground next to buildings. Wind turbines located in urban environments on the ground next to buildings have a capacity of a maximum of 100 kW (Wineur, 2005), while SSWTs mounted on buildings generally have capacity of 1–20 kW (Smith *et al.*, 2012).

Besides the high sensitivity of wind turbine efficiency to variations in wind conditions such as average wind speed, the three-dimensional wind speed profile and turbulent wind flow (Smith *et al.*, 2012), the implementation of wind turbines in urban environments is often compromised by public resistance. Some of the most important issues regarding the implementation of wind turbines in urban environments are related to noise pollution and increased building vibrations. Due to the lack of open free areas in urban settings, the installation of wind turbines on buildings is the most likely solution for urban areas. However, the installation of SSWTs on existing facilities can be compromised by increased structural vibrations. Human perceptions of floor vibrations and uncompromised serviceability of equipment in buildings are the two most important acceptability criteria when considering increased floor vibrations. Human response to floor motion is a very complex phenomenon and is often related to a combination of factors such as the magnitude of the motion, the surrounding environment and the type of human activity occurring at the moment of floor motion. Increased vibrations in a building structure due to the mounting of a SSWT can be overcome with the use of different types of dampers in the supporting structures. Polyurethane foam insulation, with open cells characterised by high elasticity and obtained by the use of specific types of catalysts and polyols, has specific dynamic properties and has been used successfully as an acoustic insulator in cases where it is necessary to protect the structure from so-called structure-borne or impact noise and vibration (Pavlović *et al.*, 2011). The use of different types of insulators is determined by limit values of vibration frequencies that should not be transferred to the supporting structure from a SSWT.

The mounting of SSWTs on existing buildings in urban environments is also limited by poor understanding of building interactions, which are closely related to the specific building and wind turbine construction. According to Smith *et al.* (2012), the main barriers for the implementation of SSWTs in urban environments can be classified into five key areas: safety, wind resource, turbine technology, building interactions and non-technical obstacles. Poor understanding of building interactions is mostly reflected through concerns regarding the excitation of resonance frequencies, increased vibrations in buildings and code compliance. To the best of the authors' knowledge, none of the current building design codes refers to loadings from building-mounted SSWTs. Furthermore, the major investigations concerning the implementation of SSWTs in urban environments are related to investigations of wind resources (Wineur, 2005), turbine technology and energy production (Encraft, 2009) and noise pollution.

An analysis of the relative increase in floor vibrations in a steel building after the installation of a SSWT on the building's roof is presented in this paper. A five-storey steel building with a steel–concrete composite floor deck structure was analysed with and without a 5 m rotor diameter horizontal-axis wind turbine (HAWT) considering a turbulent wind profile with average wind speeds of 9, 12 and 15 m/s. The highest analysed average wind speed of 15 m/s is approximately equal to the stop wind speed from SSWTs, according to Wineur (2005). The increases in floor vibrations were compared with the current design requirements for floor vibrations given in EN 1990 (CEN, 2010a), EN 1991-1-4 (CEN, 2010b), ISO 10137 (ISO, 2007) and BS 6472-1 (BSI, 2008).

2. Numerical models

Vibrations of the building were analysed by means of two independent numerical models. Firstly, an aero-elastic model of the small-scale HAWT was analysed using the multi-body analysis software package Ashes (Simis, 2013). This model was used to obtain the time-history load on the building arising from operation of the turbine. Secondly, a model of the steel building was analysed using Sofistik finite-element (FE) software (Sofistik, 2018) and time-histories of the forces and bending moments from the first aero-elastic model were applied at two different positions on the building's roof. Results of the building numerical analysis without a SSWT were compared with the results obtained from the building analysis with time-histories of forces and bending moments applied at the building's roof. In addition, the increases in the composite steel–concrete floor accelerations due to the mounting of the SSWT were compared with accelerations caused by pedestrian walking without wind action on the building.

2.1 Modelling of the small-scale HAWT

The model of the 5 m diameter HAWT with a 5 m hub height was developed using the multi-body analysis software package Ashes. Ashes software integrates finite-element analysis (FEA) and blade element momentum theory, providing a set of built-in templates of onshore and offshore wind turbines that can be further customised. The model of the SSWT which was used for the numerical analysis presented in this paper is not incorporated in the software. The small-scale HAWT model was thus developed by scaling a standard National Renewable Energy Laboratory (NREL) 5 MW onshore wind turbine with a 126 m rotor diameter, which is incorporated in the Ashes software package. Scaling of the NREL 5 MW wind turbine was performed by local scaling of the dimensions of different parts of the wind turbine using a scaling factor of 5/126 = 0·04. The rated power of the SSWT developed in Ashes was 2·4 kW and the rated wind speed was 9 m/s. However, scaling of a NREL 5 MW wind turbine by simple scaling of the dimensions of all parts of the wind turbine does not give always appropriate masses for different parts of the scaled wind turbine, and the scaled wind turbine needed to have a similar mass distribution to that in the default NREL 5 MW turbine in the Ashes software. The mass of the support tower and the rotor–nacelle assembly each comprise approximately 50% of the mass of whole wind turbine. Mass scaling models developed by the NREL (Fingersh *et al.*, 2006) show good agreement with the mass arrangement distribution of different parts of the NREL 5 MW wind turbine modelled in Ashes. Mass scaling of the blades, using a blade mass scaling relationship for LM Glasfiber blades, produced a difference of less than 1% in comparison with the NREL 5 MW wind turbine developed in Ashes, as shown in Figure 1.

Mass scaling models for SSWTs are not yet developed, as shown in Figure 1. Use of the scaling models presented in Figure 1 for SSWTs would therefore lead to inaccurate results and an irregular arrangement of masses, which does not reflect real conditions for SSWTs. Therefore, scaling of the NREL 5 MW default wind turbine was performed through dimension scaling with a scale factor of 0·04 and user-defined masses for different parts of the wind turbine. Based on the aerofoil geometry and blade length, the mass of one blade was estimated to be approximately 80 kg and the mass of the whole wind turbine was determined to be approximately 553 kg. The rotor mass adopted for the SSWT developed in the Ashes software showed good agreement with the rotor masses of wind turbines from different manufacturers with similar rated powers (Cace *et al.*, 2007).

The 2·4 kW SSWT modelled in Ashes is a direct-drive wind turbine without a pitch control system. Turbulent wind profile and aerodynamic load analysis of the turbine was performed using the multi-body analysis software package Ashes. The

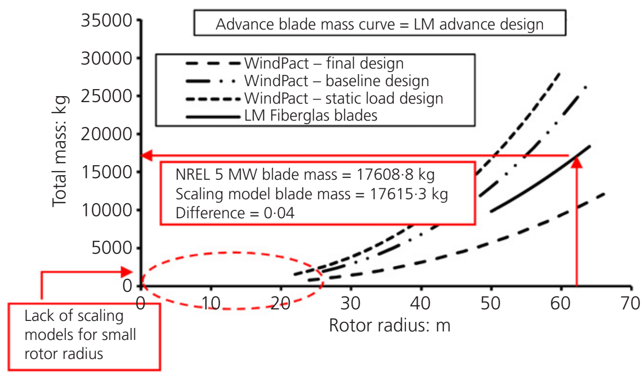

Figure 1. Blade mass scaling relationship (adapted from Fingersh *et al.*, 2006)

spatial distribution of a wind field was defined along with time-dependent wind in a certain number of grid points of the wind field. Turbulent wind time-history loading was generated by the TurboSim module integrated in the Ashes software for average wind speeds of 9, 12 and 15 m/s. The turbulent wind time-history loading generated for an average wind speed of 9 m/s is shown in Figure 2(a).

The results of the SSWT model in Ashes are loads arising from operation of the turbine in the form of time-histories of forces and bending moments calculated using spatial wind profile as loading with average wind speeds of 9, 12 and 15 m/s. The forces and bending moments calculated in the time-history analysis in Ashes showed significant increases during the first second of the time-history analysis, as shown in Figure 2(b). This increase in forces (up to four times greater than the axial force) and moments at the beginning of the time-history analysis are a consequence of the immediate start of the analysis with the rated wind speed and do not represent real conditions in urban built environments. In real environmental conditions, the start-up wind speed of the turbine is lower than the rated wind speed (which gives the rated power production). Therefore, in order to exclude an unrealistically high influence of the wind turbine on the building structure, the first 2 s of the time-histories of turbine forces and bending moments were excluded from the analysis of floor vibrations (Figure 2(b)).

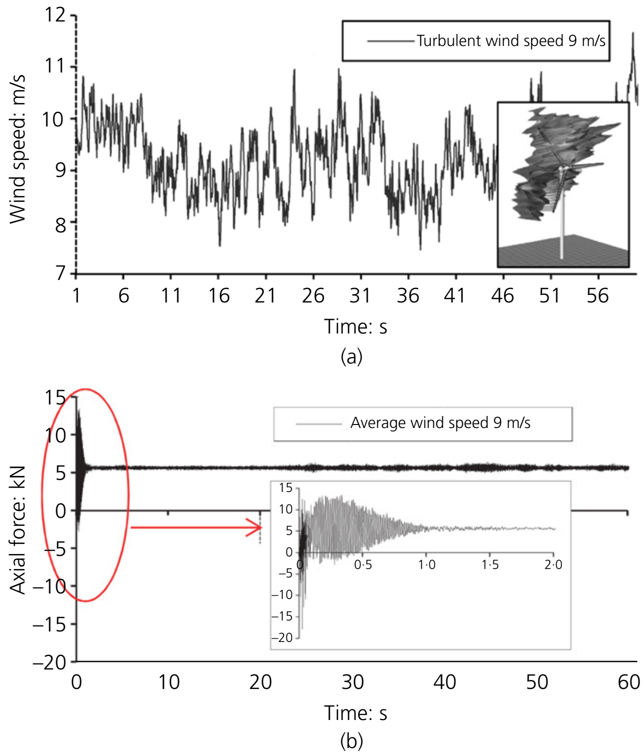

Figure 2. Development of SSWT in Ashes software package: (a) turbulent wind time-history generated by Ashes for 9 m/s average wind speed; (b) time-history of axial forces from wind turbine

The time-histories of axial forces, shear forces and bending moments for the strong and weak wind turbine axis for average wind speeds of 9, 12 and 15 m/s applied on the building's roof are shown in Figure 3. The upper and lower peak amplitudes of axial forces for the three average wind speeds are shown in Figure 3(a). The average axial force for wind speeds of 9, 12 and 15 m/s was 5·64, 5·70 and 5·74 kN, respectively, which are in close agreement with the total mass of the SSWT.

2.2 Numerical model of the steel building

The time-history response of the steel building to dynamic load excitation by wind action was analysed in Sofistik FE software, using the incorporated Dyna module for dynamic analysis (Sofistik, 2011). A typical steel building was chosen for the numerical study, but the results obtained in this framework will also be validated for other layouts of steel building structures. The building's base was 24×40 m and the total height of the building was 18 m. The steel–concrete floor decks were composed of 330 mm high steel I-beams (8 m span and 4 m distance), connected by shear connectors to the 160 mm full-depth concrete slab. Horizontal stability of the building in the longitudinal and transversal directions was achieved by moment-resisting frames and vertical bracing, respectively, as shown in Figure 4(a). The design of the case study building was according to the requirements given in EN 1994-1-1 (CEN, 2009b) and EN 1993-1-1 (CEN, 2009a) in order to provide a real case design and mass–stiffness properties of the structure. Installation of the wind turbine by clamping its tower at the roof was analysed in the positions shown in Figure 4(b): position 1 in the mid-span of a floor beam and position 2 immediately above an internal column.

Wind action on the building was analysed through external and internal pressure coefficients for the building's walls and roof according to the recommendations given in EN 1991-1-4 (CEN, 2010b). Wind action on the building in two orthogonal directions was applied as time-history loads at floor levels, obtained using the time-history wind speed obtained in Ashes (see Figure 2(a)) and pressure coefficients.

Dynamic loads induced by human activities can be classified as continuous dynamic loading and are known to be the most usual internal source of floor vertical vibrations (Gluhović *et al.*, 2016). A numerical model of a walking pedestrian was developed in the Sofistik FE software in order to compare the accelerations of the building floor caused by mounting of the SSWT with accelerations induced by a walking human.

3. Current design recommendations for floor vibrations

The increase in floor vibrations was compared with the current design requirements for floor vibrations given in EN 1990

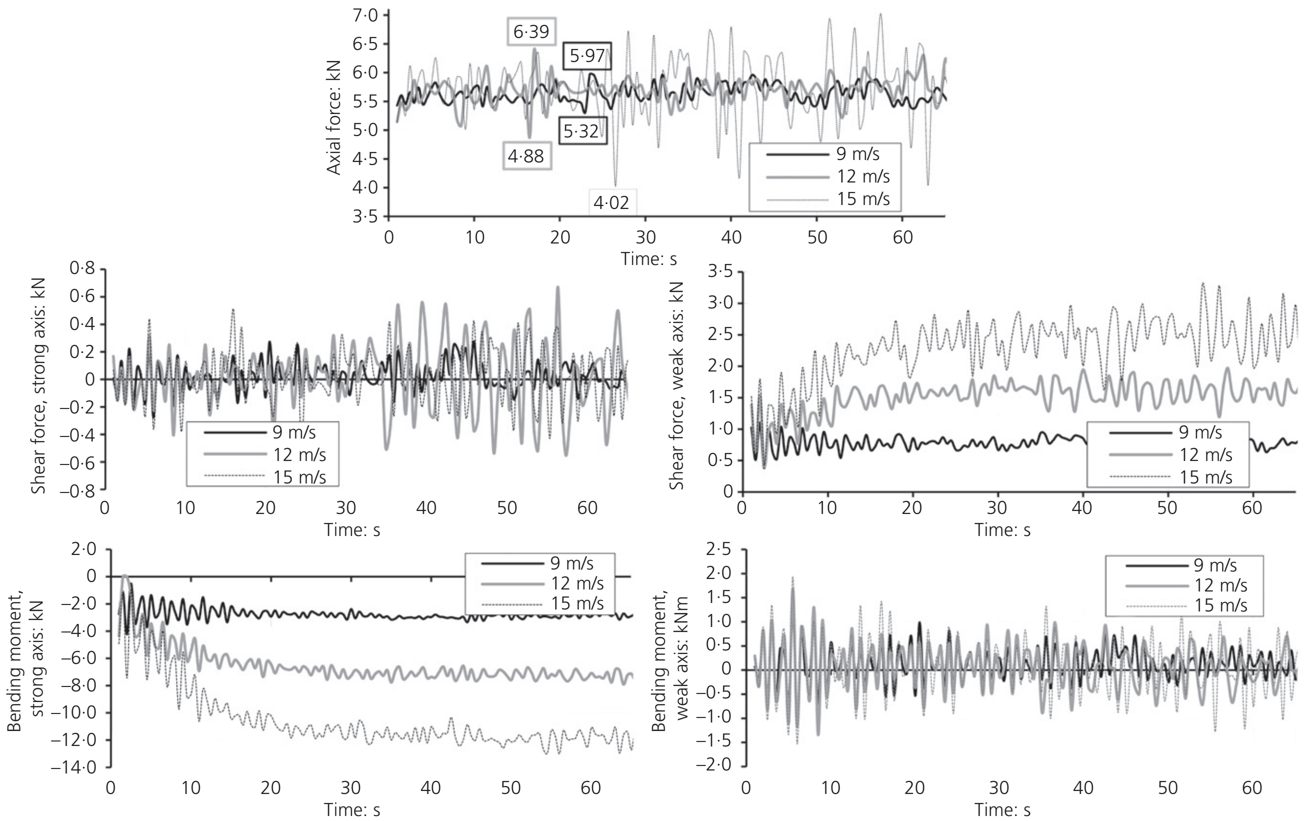

Figure 3. Time-history of forces and bending moments applied on the building's roof for average wind speeds of 9, 12 and 15 m/s

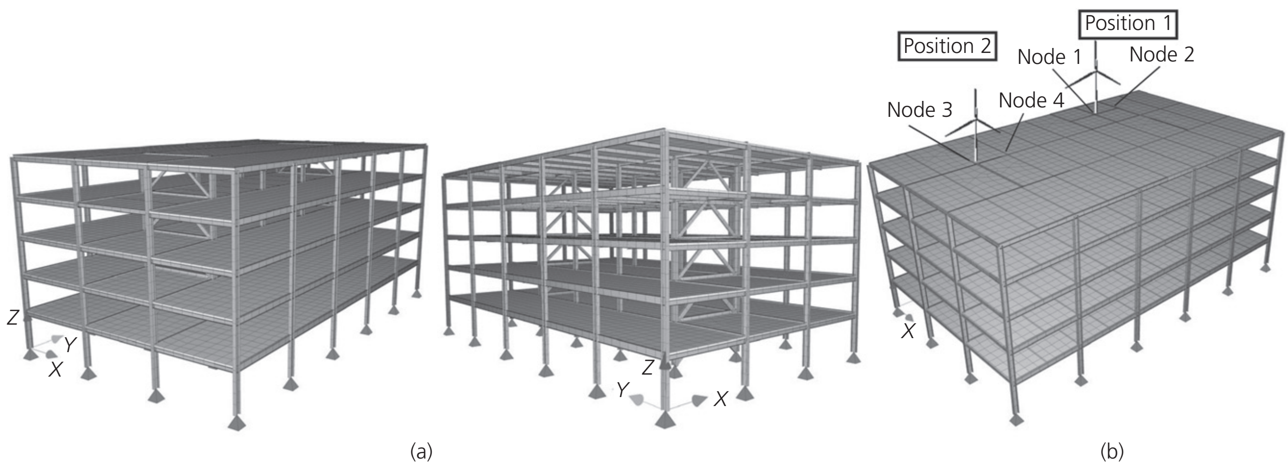

Figure 4. Numerical model of analysed steel building developed in Sofistik FE software: (a) steel building layout; (b) position of wind turbines

(CEN, 2010a), EN 1991-1-4 (CEN, 2010b), ISO 10137 (ISO, 2007) and BS 6472-1 (BSI, 2008).

EN 1990 (CEN, 2010a) gives basic requirements for structures exposed to dynamic loading. According to annex A2 of EN 1990, pedestrian comfort criteria should be defined by national annexes, but recommended maximum values of acceleration are given for vertical (0·7 m/s^2) and horizontal (0·2 m/s^2) vibrations. According to EN 1990 A1.4.4 (CEN, 2010a), considering the serviceability limit state of a structure or structural member, it is recommended that the natural frequency of vibrations of the structure or structural member should be kept above appropriate values. This standard also refers to EN 1991-1-4 (CEN, 2010b) and ISO 10137 (ISO, 2007).

EN 1991-1-4 (CEN, 2010b) provides two methods for the calculation of peak accelerations of a structure subjected to wind, but does not define the design limits for peak acceleration.

ISO 10137 (ISO, 2007) deals with the serviceability of structures against vibrations. Annex D of ISO 10137 provides guidance for human response to wind-induced motions in buildings with acceleration limits for residential areas and offices. Acceleration limits are provided in the form of peak acceleration–first natural frequency diagrams, and for residential buildings and hotels the maximum peak acceleration is $0{\cdot}04\ \mathrm{m/s^2}$. According to annex D of ISO 10137, frequencies of 1–2 Hz are the least favourable. While ISO 10137 gives diagrams for peak acceleration and offers hands-on calculations for horizontal x and y vibrations, BS 6472-1 (BSI, 2008) gives acceleration limits as a function of root mean square (RMS) accelerations. This standard covers many vibration environments in buildings and limits of satisfactory vibrations are given in relation to a frequency-weighted base curve and multiplying factors.

Considering the lack of design recommendations for the serviceability limit state of existing buildings due to the mounting of SSWTs, in this paper, horizontal peak floor accelerations induced by a wind turbine mounted on a building's roof are compared with the recommendations given in ISO 10137 (ISO, 2007) and vertical RMS accelerations are compared with the recommendations given in BS 6472-1 (BSI, 2008).

4. Results of numerical analysis

The considered positions of the SSWT on the building's roof are shown in Figure 4(b). Both wind turbine positions were analysed for wind in two orthogonal directions, x and y shown in Figure 4(b), considering different orientations of the wind turbine in order to ensure that the turbine's horizontal-axis was always in the direction of wind action. Different wind turbine positions were analysed to compare the increases in acceleration due to mounting of the wind turbine on different building elements. The possible increase in vibrations was analysed for one wind turbine at a time. Horizontal and vertical accelerations were obtained at roof nodes 1 and 2 for wind turbine position 1 and nodes 3 and 4 for wind turbine position 2 on the building's roof (Figure 4(b)). Accelerations were also obtained for the same vertical position of nodes at the fourth floor, denoted as nodes 5 and 6 for position 1 and nodes 7 and 8 for position 2. The nodes for obtaining accelerations were selected in order to analyse the increase in floor vibration at the position of wind turbine and the spreading of floor vibration in the surrounding zone, 4 m from the wind turbine. Horizontal and vertical accelerations were obtained for the three analysed average wind speeds.

For both positions of the wind turbine on the building's roof, up to double horizontal peak accelerations were obtained for wind action in the direction of the global x-axis in comparison with wind action in the direction of the global y-axis. In all the situations considered, the horizontal-axis of the wind turbine corresponded to the wind direction. The results presented in this paper correspond to wind action in the global x-direction of the building (see Figure 4). The horizontal and vertical vibrations of the roof for both wind turbine positions in the middle of the time-history analysis are shown in Figure 5. Figures 5(a) and 5(b) represent accelerations in the global x- and z-directions for wind turbine position 1 and

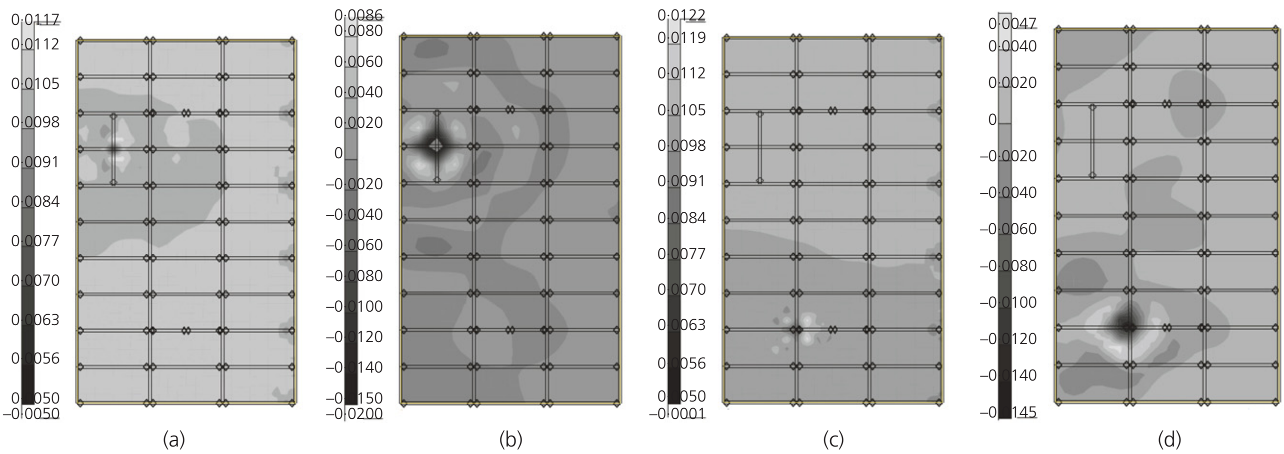

Figure 5. Roof vibrations for wind turbine positions 1 and 2: (a) horizontal acceleration in x-direction, position 1; (b) vertical acceleration in z-direction, position 1; (c) horizontal acceleration in x-direction, position 2; (d) vertical acceleration in z-direction, position 2

Figures 5(c) and 5(d) show the same for wind turbine position 2. The figure reveals the significant influence of the wind turbine on the node accelerations on the building's roof for both wind turbine positions. In addition, higher horizontal and vertical peak accelerations were obtained for higher average wind speeds, as shown in Figures 6 and 7.

As shown in Figure 6, higher vertical peak accelerations were obtained for wind turbine position 1 (node 1) than for wind turbine position 2 (node 3), which is due to the position of the SSWT at the intersection of roof beams in comparison to the mounting of the turbine at the top of a column. In the surrounding zone, 4 m from the wind turbine, the vertical peak accelerations were more than 50% smaller at the building's roof for both positions of wind turbine. Mounting of the SSWT on the roof had a negligible influence on peak accelerations at the fourth floor (nodes 5 and 6) for wind turbine position 1. A significant amount of vertical peak acceleration was transferred through the column from the roof to the fourth floor, as can be seen in Figure 6(b). Vertical peak accelerations at node 7 (position 2) were four times higher than those at node 5 (position 1) for an average wind speed of 15 m/s. The increase in the fourth-floor vertical accelerations for wind turbine position 2 was also negligible in the surrounding zone, as shown in Figure 6(b).

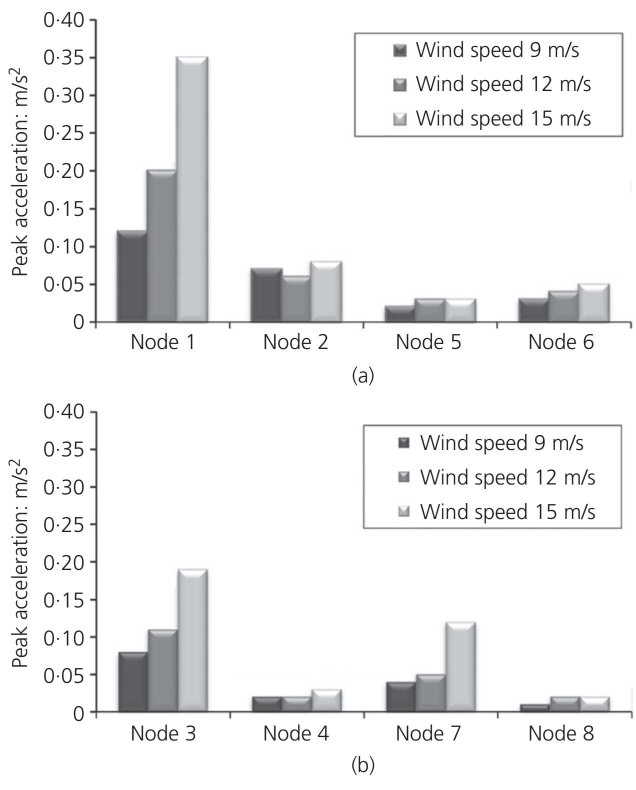

Figure 6. Peak vertical accelerations at roof and fourth floor: (a) wind turbine position 1; (b) wind turbine position 2

Increases in horizontal roof and fourth-floor accelerations were also obtained for higher average wind speeds, as shown in Figure 7. The horizontal peak accelerations had the same values at nodes 1 and 2 on the building's roof and at nodes 5 and 6 on the fourth floor (wind turbine position 1) for the three analysed average wind speeds (Figure 7(a)). Mounting of the SSWT on the top of the column incorporated in the internal vertical bracing (wind turbine position 2) resulted in higher horizontal peak accelerations at the mounting position (node 3 compared with node 1, Figure 7). The horizontal peak accelerations 4 m from the wind turbine mounted in this position were the same as the horizontal accelerations for position 1 (node 4 compared with node 2, Figure 7).

Figure 8 compares the vertical and horizontal peak accelerations of the analysed nodes on the building's roof and fourth floor after mounting the SSWT with the same accelerations without a wind turbine and with the vertical accelerations induced by a walking human. The comparison was only made for an average wind speed of 9 m/s, which represents usual wind conditions in urban built environments. Without a wind turbine mounted on the roof and for 9 m/s average wind

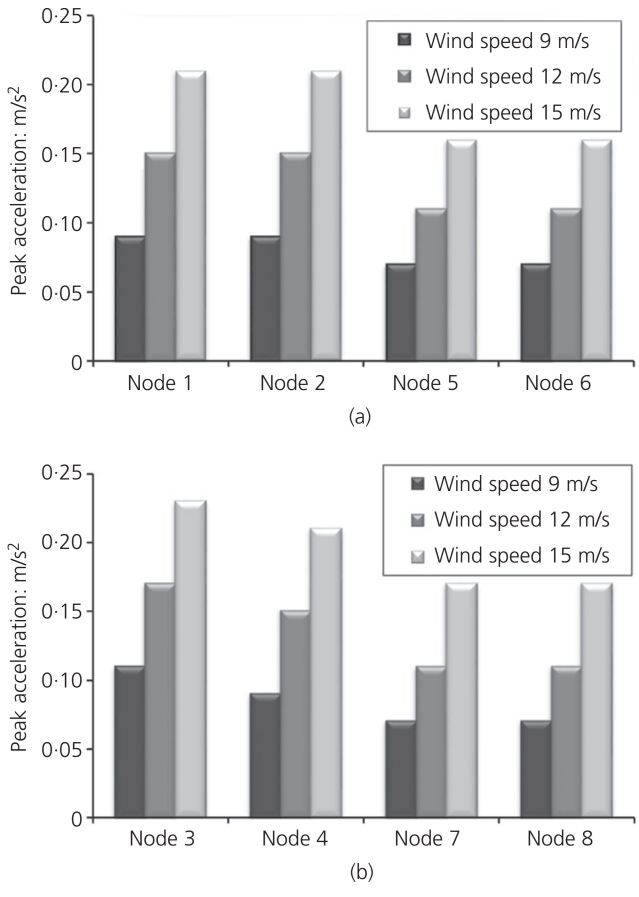

Figure 7. Peak horizontal accelerations at roof and fourth floor: (a) wind turbine position 1; (b) wind turbine position 2

speed, the vertical peak accelerations in node 1 and node 3 were 0·02 and 0·01 m/s^2, respectively. For the same average wind speed, the peak accelerations in nodes 1 and 3 after mounting of the wind turbine were 0·12 and 0·08 m/s^2, as shown in Figure 8. Vertical node accelerations on the fourth floor were significantly lower. The vertical peak accelerations in nodes 5 and 7 (same vertical position as nodes 1 and 3 respectively, but on the fourth floor) were 0·02 and 0·04 m/s^2, respectively (Figure 8). The vertical peak accelerations for nodes on the fourth floor with mounting of the SSWT were in the range of the vertical accelerations caused by a walking pedestrian (Figure 8). Similar fourth-floor horizontal and vertical accelerations were obtained for both wind turbine positions and also for wind action on the building without the SSWT mounted on the roof. Therefore, the influence of the wind turbine on the fourth-floor vibrations was very small, which is clearly favourable when considering human comfort and quality of life on the top floor of the considered building.

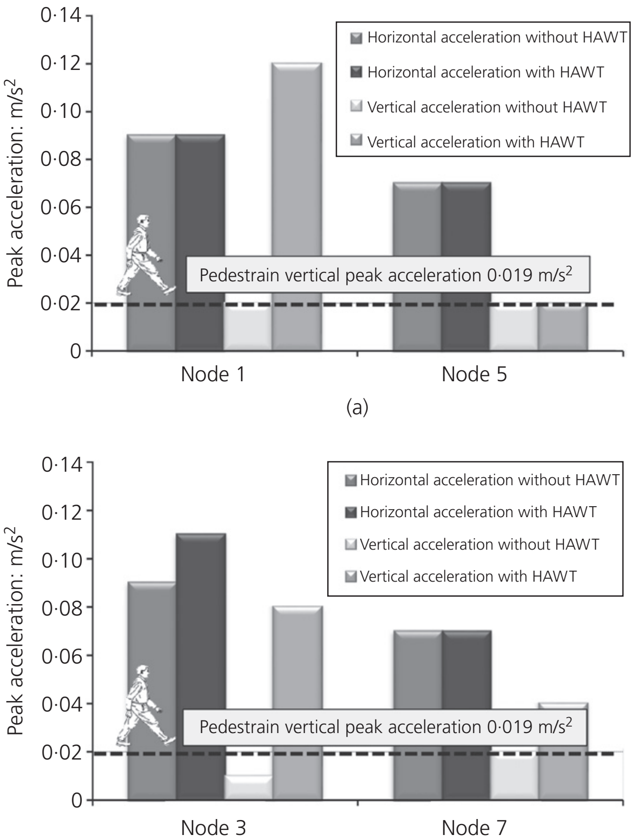

Figure 8. Comparison of roof and fourth-floor peak accelerations with and without a wind turbine for 9 m/s average wind speed: (a) wind turbine position 1; (b) wind turbine position 2

When analysing floor vibrations and human comfort accelerations, it is important to consider that the fourth floor of the analysed building (e.g. residences, offices, hospitals or schools) could be occupied for a certain purpose. The first natural frequency of the floor was calculated according to simple calculation methods (Feldmann *et al.*, 2009) and the mean of the first natural frequency of the considered steel–concrete composite floor was found to be 4·94 Hz.

The horizontal peak accelerations of nodes on the fourth floor were compared with the design recommendations given in ISO 10137 (ISO, 2007) and the results are shown in Figure 9(a). Considering high wind speeds (15 m/s) the horizontal accelerations of nodes on the top floor obtained from the time-history analysis did not satisfy the recommendations for buildings with offices and residential buildings. For a wind speed of 12 m/s, the horizontal accelerations did not satisfy the acceleration demands for residential buildings.

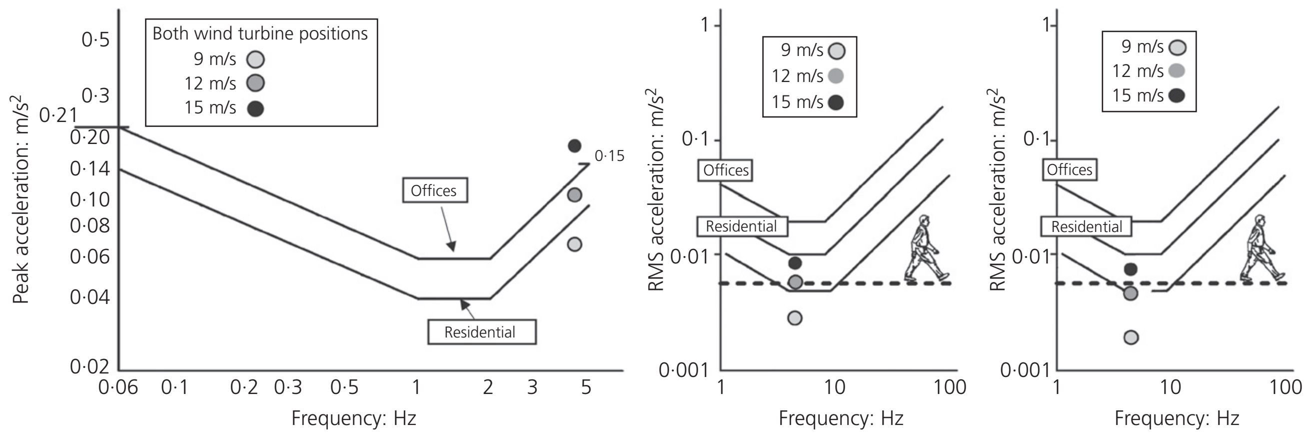

Figure 9. Comparison of results of numerical analysis with design recommendations: (a) wind-induced vibrations in horizontal direction compared with ISO 10137 (ISO, 2007); (b) vibrations in vertical direction for turbine position 1 compared with BS 6472-1 (BSI, 2008); (c) vibrations in vertical direction for turbine position 2 compared with BS 6472-1 (BSI, 2008)

Table 1. Vertical RMS accelerations of fourth floor of building

	Average wind speed						
	9 m/s		12 m/s		15 m/s		Pedestrian
Nodes of fourth floor	Node 5	Node 6	Node 5	Node 6	Node 5	Node 6	Node 5
RMS accelerations for wind turbine position 1: m/s^2	0·002	0·003	0·005	0·006	0·006	0·008	0·0053
Nodes of fourth floor	Node 7	Node 8	Node 7	Node 8	Node 7	Node 8	
RMS accelerations for wind turbine position 2: m/s^2	0·180	0·002	0·016	0·005	0·028	0·007	

The vertical RMS accelerations of nodes on the fourth floor (Table 1) were compared with the design recommendations given in BS 6472-1 (BSI, 2008) and the results are shown in Figure 9(b). Figure 9(b) shows base curves that, in comparison with the multiplying factors given in BS 6472-1 (BSI, 2008), present design recommendations for different building functions. The multiplying factor for residential buildings is 2 and that for a building with offices is 4. Therefore, the minimum value for RMS accelerations is 0·01 m/s^2 for residential buildings and 0·02 m/s^2 for offices. The vertical RMS accelerations at the analysed nodes on the fourth floor obtained from the time-history analyses satisfied these acceleration demands. The RMS vertical accelerations of node 7 of the fourth floor (Table 1) are the accelerations of the node at column and are significantly higher than the other accelerations because of vibration transfer through the column. Taking account of the fact that these values are not accelerations of the floor structure, they are not comparable with the design recommendations presented in Figure 9.

5. Conclusions

Increased vibrations in a typical five-storey steel-framed building due to the mounting of a small-scale HAWT on the building's roof were investigated by means of numerical analysis. Using the multi-body analysis software package Ashes, a numerical model of a small-scale HAWT was developed, similar to commercially available direct-drive SSWTs, with 5 m diameter and 2·4 kW rated power. Time-dependent wind histories were generated for wind speeds in the range 9–15 m/s. Numerical analyses were performed by combining the time-history FEA of the building with results obtained for the turbine in the Ashes software based on blade element momentum theory. The following conclusions were drawn from this study.

(*a*) For both wind turbine positions on the building's roof (at the mid-span of a floor beam and immediately above an internal column), the current limitations of horizontal floor vibrations in design code ISO 10137 (ISO, 2007) as criteria of human comfort for offices and residential buildings were fulfilled for average wind speeds of 9 m/s and 12 m/s. However, the design recommendations were not satisfied for an average wind speed of 15 m/s.

(*b*) Up to four times higher peak accelerations were obtained at certain 'singularity' points of the roof after installation of the small-scale HAWT.

(*c*) In both cases, with and without the wind turbine, the current limitations of vertical floor vibrations (criteria of human comfort) of the fourth floor in design code BS 6472-1 (BSI, 2008) were fulfilled.

(*d*) Mounting of a small-scale HAWT on a roof of a typical steel building in an urban environment results in increased vibrations only in the near surrounding zone of the wind turbine position on the roof. The influence of such a turbine on horizontal and vertical vibrations of the fourth floor was found to be negligible.

Further analyses of the implementation of SSWTs on roofs in urban environments should include more detailed assessments of different building layouts, wind conditions on the rooftop and further study of the optimal position and layout of the turbine tubular tower. The use of dampers in the supporting structure of the turbine as a possible way of reducing floor vibrations should also be analysed.

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