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# Application of state-of-the-art in measurement and data analysis techniques for vibration evaluation of a tall building

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Recent advancements in sensing and data acquisition technology have made monitoring of structures and infrastructure more affordable and, at the same time, more comprehensive. Examples of such advancements are application of wireless technology for communication, the utilisation of fully automated systems for long-term monitoring and the remote control of the sensing system over Internet. Although each of these technologies has been used in different structural health monitoring projects in the recent years, inclusion of an all-in-one sensing system represents the state-of-the-art in measurement techniques. This paper presents the integration of all of the above-mentioned advanced monitoring approaches in one sensing system for forensic quantification of an in-service tall building. The inclusive measurement and monitoring system along with advanced data analysis techniques enabled extraction of detailed information about dynamic characteristics of the building structure and development of reliable conclusions regarding its performance. It is shown that the performance of the investigated structural components is satisfactory in terms of strength demand. However, the level of vibration in some portions of the structure does not meet the limits of human comfort. In addition, wind-speed spectrum, acceleration response spectrum and the modes of lateral vibration are extracted to assist with evaluation of the structure's performance.

**Keywords:** remote sensing; vibration; data processing; evaluation; monitoring; remote control



## Introduction

In the past few decades, the philosophy of structural design has been complemented by including a focus on the in-service performance of structures. This change is rooted in the concept of performance-based design which is explained, in brief, as 'practice of thinking and working in terms of ends rather than means' (Gibson, 1982). A fundamental step in this practice is the validation and verification of the resulted design against the objective performance. This step, in design of new structures, is usually achieved by means of modelling and simulation. In existing structures, however, reliability is achieved through instrumentation, measurements and assessment of the constructed structure. The goal of instrumentation and measurement goes beyond validation of design when there are concerns over the performance of a structure or when an evaluation is needed to develop maintenance strategies. Although the literature shows significant research efforts in the advancement of instrumentation techniques for the evaluation of in-service structural systems (Lynch & Loh, 2006; Wong, 2007), the existing applications still face practical difficulties in many cases.

These difficulties usually result from either limited budget designated to instrumentation or restrictions imposed by the operation of in-service structures (e.g. limited monitoring duration or limited access to structural elements). For example, instrumentation of Tsing Ma Bridge in Hong Kong (with more than 350 sensing channels) is one of the largest monitoring projects, which has provided significant amount of information concerning the performance of the bridge (Ko & Ni, 2003). However the instrumentation cost is reported to be approximately $8 million (Farrar, 2001) which is prohibitively high. Celebi (2002) also presents a survey on the instrumentation and monitoring of building structures for seismic studies and estimates $4K per sensing channel (where $2K is allocated to labour and cabling) for monitoring of tall buildings which is not affordable for many building's owners and administrators.

Recent advancements in sensing technology have helped alleviate a lot of challenges in both the economical and practical aspects. For example, a distinguished improvement, introduced to the monitoring systems, is the deployment of wireless technology for data communication and powering sensing networks. Wireless sensor

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networks (WSNs) were first introduced to structural health monitoring (SHM) applications in the late 1990s (Straser & Kiremidjian, 1998), and showed an inherent potential to improve the monitoring techniques in terms of the total cost and deployment by eliminating cables for communication and power (Ghazanfari, Pamukcu, Yoon, Suleiman, & Cheng, 2012; Jang et al., 2010; Lynch et al., 2004; Pakzad, Fenves, Kim, & Culler, 2008; Wang, Lynch, & Law, 2007).

Current structural monitoring techniques utilise either the traditional wired sensor network or the relatively new WSN. This paper, however, presents the application of combined wired and WSNs in order to provide a cost- and performance-effective sensing and acquisition solution for SHM. Each sensor network has its advantages and disadvantages. For example, wireless sensors are less expensive, less labour-intensive and more portable. However, wireless sensors are powered by finite-life batteries and also need line-of-sight for communication which limits the application in long-term monitoring of a building structure. The combination of both wired and wireless networks provided an opportunity to confirm the adequacy of each while producing the necessary short- and long-term data for the performance evaluation of an in-service 16-storey building structure suffering from perceptible vibration in different directions.

The monitoring system employed in this study includes a broad range of sensors, measuring acceleration, displacement, strain (from which stress can be directly calculated), wind speed and wind direction. Using the portability advantage of wireless sensors, vibrations response from various locations is obtained with minimal effort. Wired sensors, on the other hand, are installed on fixed locations to perform automated long-term monitoring. The applicability of this sensing system is validated, and its performance is demonstrated through the evaluation of the measured data. Collecting effective and reliable data from the structure's response at various locations during different modes of excitation enable the extraction of fundamental dynamic characteristics of the building. This information is used in evaluating potential mitigation strategies for maintenance and serviceability of the building in its future operation.

The vibration issues under investigation in this paper are those caused by wind and human–structure interaction. In this work, the structure is investigated on two fronts. The first is the vibration-induced demand (i.e. the stresses in critical elements of the structure) in comparison to the structural capacity of the building. This effort entails an assessment as to whether the demand could lead to potential structural failure in the building system. The second is the effect of the demands on the comfort of the occupants of the building. This effort results in an assessment of the vibration in the structure as compared to the acceptability limits [Murray, Allen, & Ungar (1997) for vertical vibration and Irwin (1978) for lateral vibration].

## Sensing system

The monitoring system in this work is designed to measure acceleration, displacement, strain, wind speed and wind direction. Utilising a sensing system with different types of sensors and with high spatial and temporal sensing resolution assures that the response of the instrumented structure is captured, and therefore, a reliable evaluation of the structural performance can be achieved. Wireless sensors are used because of the portability and ease of installation in various locations. A network of wireless sensors integrates a base station and remote sensors, has a substantial computational capability, and can provide the platform for the sensor nodes to communicate with each other through wireless radio signals. Portability makes WSNs very effective tools when there are limitations in access to structural elements and in cases where data from a large number of different locations on the structure are required.

These sensors are particularly beneficial for monitoring in-service structures since they eliminate the need for installing wires in an occupied facility and thus, the sensor network can be set up quickly and inexpensively. Integrating the on-board computational capability of the sensing nodes enables filtering and conditioning of the collected data before transmitting it to the base station, thus eliminating the need for a central data logger and a conditioner. The wireless sensor platform of this sensing system is Imote2, developed by Intel (2005), which comprises a low-power PXA271 XScale CPU. Imote2 is integrated with an external antenna (Linderman et al., 2010) and when necessary, the external antenna can be connected through an extension cable and directed to a clear line-of-sight with the base station to enhance the wireless communication.

The other component of the wireless sensor hardware platform is the utilised sensor board. The sensor board of this platform is SHM-A developed by Rice and Spencer (2009). The tri-axial LIS3L02AS4 (2005) analog accelerometer manufactured by ST Microelectronics is used on SHM-A which is a low-cost, high-sensitivity analog accelerometer with 50 $\mu g/\sqrt{Hz}$ noise density. A low-pass filter, a gain difference amplifier and the Quickfilter 16-bit ADC are other components integrated on this sensor board. Figure 1 shows the hardware platform of Imote2 and SHM-A sensor board.

While WSNs are effective tools for the response monitoring of structures, maintaining the power supply by batteries with finite-life makes the application somewhat more challenging for long term monitoring. Additionally, wireless sensors need line-of-sight for the communication which imposes a difficulty in the application particularly in building structures due to the presence of obstacles (e.g. interior walls and ceilings). Thus, a traditional wired sensor network is used in conjunction with the wireless sensors for performing automated long-term monitoring.

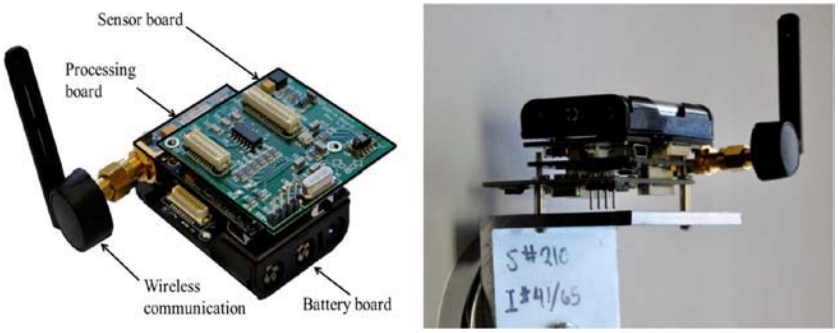


Figure 1. Wireless hardware platform.

The wired network includes strain gauges, accelerometers, displacement transducers and an anemometer: strain gauges are uniaxial weldable resistance-type gauges with 350 Ω resistance and 10 V excitation; wired accelerometers are uniaxial, capacitive with ±3 g acceleration range and 3 μg/√Hz noise density (PCB Piezotronics, Inc. [2005] – model 3701G3FA3G); displacement transducers are linear variable displacement transducer (LVDT) which measure the displacement response in the range of ±25.4 mm. with sensitivity of 0.2 mV/μm and 16-bit ADC resolution; the anemometer is a product of R. M. Young Inc. (Model 05103), and is used to record wind speed and wind direction at the roof level.

Figure 2 presents the different utilised sensors in the wired network. The data acquisition system for the wired network is a Campbell Scientific CR9000 (1995). This acquisition system is configured with digital and analog filters to remove noise from the collected data. Remote communications with the data logger are established using a dedicated wireless broadband modem. Utilising remote communications, the data acquisition system can be controlled over the network remotely and data can be downloaded automatically via any server with broadband access. The combination of wired and wireless networks with different types of sensors is utilised in the monitoring of a test-bed structure with the goal of measuring its vibration performance under in-service loads.

## Building description

The building selected for this study is a 16-storey steel structure with four floors of parking below grade located in Pennsylvania. The structure is supported on a combination of spread footings and caissons socketed into rock. The main lateral-force-resisting system for the building is steel braced frames in both primary directions. Two braced frames are used for each transverse axis of the building. The frames in the North–South axis of the building continue to the roof. The frames in the East–West direction, however, terminate at the 15th floor. While the braced frames in the East–West direction are located on the two outermost lines (north and south sides) in the plan, the braced frames in the North–South direction are closely spaced and located near the building centreline in plan (Figure 3).

The building is clad with masonry and has concrete floors supported by steel beams and girders. The structure has a consistent plan up to the 13th floor where the building face steps back. At this level, slender columns support the East face of the upper floors. These columns are supported on transfer girders at the 13th floor. There have been concerns over the building's vibrations which are induced by activities in an exercise room, located on the 16th floor, and also by high speed wind gusts. Building occupants housed next to, and down to several floors below the exercise room have complained of excessive

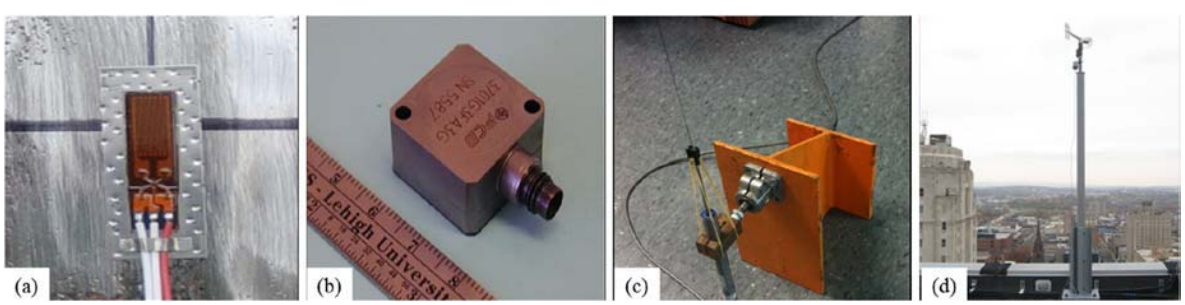


Figure 2. Wired instrumentation: (a) strain gauge, (b) accelerometer, (c) LVDT and (d) anemometer.


Figure 3. (a) East face of the testbed structure, (b) a view of 16th floor exercise room and (c) typical plan of the building.

vibration when the aerobics class takes place. Placement of offices around and immediately adjacent to the exercise room leads to user conflict as the vertical accelerations transmit from the activity room to the adjacent portions of the structure where active motions do not take place.

For example, while individuals in the aerobics class would not notice the movement, individuals sitting at a desk in other portions of the building are very sensitive to the motion. Objections also exist with regard to the lateral vibration when the wind speed is high. These perceptible movements together with some diagonal cracks on the building claddings have intensified concerns of the building occupants and owner regarding the general performance of the building. As a result, the vibration evaluation and assessment of the building are the first step for rectifying the issues. Figure 3 shows the East face of the structure, a view of the exercise room and the typical plan of the building.

## Background on vibration and human perception

Accurate vibration evaluation is critical in assessing a building's performance under the effects of dynamic excitations. As a serviceability consideration, a proper design of a building should assure that the structure's vibrations do not exceed the level of comfort of its occupants. A few different factors which affect the perception and tolerance level of the human are (i) the magnitude of vibration, (ii) the frequency of vibration, (iii) the occupant's activity, (iv) the occupant's body orientation and (v) the duration of motion. Although the effect of magnitude on human perception is evident, the effects of other parameters may need more elaboration.

Frequency of vibration is a critical factor as, depending on the natural frequency of a human body, the effects of the vibration can be intensified. Fundamental organs of a body have natural frequencies in the range of 5–8 Hz (Murray, 1991). Therefore, vibrations with major frequency contents within this range would have significantly more uncomfortable effects on the body, as compared to vibration with frequencies outside of this range. Perception also varies significantly depending on the type of activity of the individual (Irwin, 1978; Murray et al., 1997). For example, an individual sitting at a desk would be much more sensitive to floor vibrations than an individual walking around an office or one who is performing exercise.

Tolerance is also dependent on the direction of the acceleration and the orientation of the body. High wind gusts induce horizontal and torsional motions in a building, whereas fast walking and aerobics activities induce vertical accelerations. Depending on the frequency, the direction of each of these vibrations may result in different perception levels [e.g. in lower frequencies, the lateral vibration is more perceptible than vertical vibration (Irwin, 1978)]. Finally, the duration of the motion has considerable influence on the tolerance of people. It is stated in AISC LRFD Commentary (1986) that 'Generally,

occupants of a building find sustained vibrations more objectionable than transient vibrations'. While this reference does not provide an exact duration as a threshold, Irwin (1978) and Boggs (1997) suggest the evaluation of vibration over approximately 10 and 20–60 min duration, respectively, for the purpose of human comfort assessment.

Determination of level of comfort and perception for a human body subjected to vibration are a well-researched area of study. Although a large portion of these investigations are devoted to human response to vibrations in moving automobiles, ships or airplanes, there are many established studies for the evaluation of motions in building structures: Irwin (1978) investigated human response to dynamic motion of a structure and presented suggestions for maximum acceptable magnitudes of storm-induced horizontal motion for buildings; International Standard Organization (1989) discussed many building vibration environments and presented acceleration limits for mechanical vibrations as a function of exposure time and frequency, for both longitudinal and transverse directions of persons in standing, sitting and lying positions; Allen and Murray (1993) proposed criteria for design of gymnasiums subject to rhythmic loads; Murray et al. (1997) summarised recommendations and design criteria for floor vibration due to human activity in AISC Design Guide Series 11.

In the building investigated in this study, vibrations are induced due to both wind and human-structure interaction which is caused by rhythmic actions. Recommended tolerances are reproduced from the study of Murray et al. (1997) and Irwin (1978) and are shown in Figure 4. In this figure, the recommended limits are presented in terms of the frequency of vibration. The limits also depend on the occupancy of the building (e.g. office areas or exercise area). The recommended values in this plot are used later in the paper for the evaluation of the building's performance. Note that while the limits of vertical vibration are in terms of peak acceleration, limits of lateral vibrations are in terms of root-mean-square (RMS) acceleration.

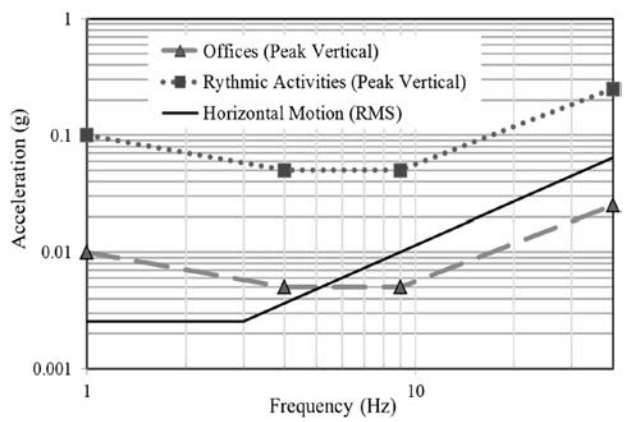


Figure 4. Recommended limits on acceleration tolerance.

## Instrumentation details

Monitoring of the building took place during a three-month period (November 2011–January 2012). This duration allowed for the measurement of a large number of high wind-speed events and monitoring of vibration response during 15 aerobic classes, which was enough to accurately assess the performance of the structure under the two types of load effects. As mentioned earlier, the combination of wired and wireless sensors provides a cost- and performance-effective sensing system. As an advantage, wireless sensors are significantly less expensive than wired sensors since they do not require wires or wire installation, or a central data logger (each re-usable wireless sensing unit costs less than $350, whereas the data logger for wired sensors costs more than $20K. In addition, considering the expense for wiring and the extensive labour demand, the total cost per each channel is more than $2K).

However, because of the long-term monitoring and also limitation in having obstacle-free radio communication between different floors in the building, use of wired sensors was necessary. Thus, the objective was to minimise the number of channels of wired sensors while ensuring that all necessary structural responses are measured. The onsite monitoring includes the following:

- Measurement of the floor vibration from multiple locations on the 16th, 15th, 13th, 12th and 7th floors. Wireless sensors were used to capture the vibration from multiple locations quickly and with minimal effort.
- Measurement of the lateral vibration of the building using wireless accelerometers installed on the roof, 13th and 3rd floor, with two sensors at each elevation. The sensors were connected to the external antennas and communicated with the base station, which was located outside of the building, through a line-of-sight radio link. The collected data from those six wireless sensors are used for the extraction of response spectra and modal parameters of the building.
- Measurement of displacement response at the mid-span of two beams along column lines 9 and C during aerobics classes, using LVDTs and wired sensing system.

The remote long-term monitoring using wired sensors includes the following:

- Measurement of strains from seven beam elements on the 16th floor (along column lines 9 and C) and from lateral bracing members at different elevations and directions for the evaluation of performance in terms of lateral strength demand.
- Measurement of floor acceleration response from multiple locations on the 16th floor for the assessment of the aerobics-induced vibrations and the maximum-induced vibrations during the long-term monitoring phase of data collection.
- Recording of wind speed and wind direction using an anemometer installed on the roof level. The recorded data are used to estimate wind spectra.

The automated remote long-term monitoring system, controlled by a CR9000 data logger, is programmed to record data under a number of different conditions:

(1) *Wind history:* For the duration of monitoring, the wind data are recorded on five-minute intervals. For each five-minute interval, the average wind speed and direction as well as the maximum wind speed are recorded. These data collection continued throughout the three-month monitoring period, and the data are used for lateral vibration evaluation.
(2) *High wind speed events:* To capture the high wind-speed events, the data acquisition system is programmed to record data when a predefined trigger condition (wind gust speed higher than 14 m/s) is reached. Upon detecting the trigger event, a predefined quantity of data is recorded, both before (by a data buffer) and after. These data are collected for lateral vibration evaluation.
(3) *Aerobics class monitoring:* Continuous high-speed data are collected from each wired sensor during all aerobics classes in the three-month monitoring period, and the collected data are used for vertical vibration evaluation.

Table 1 presents the quantity and location of all the sensors that are used for monitoring of the building.

## Measurements and data processing

The vibration issues of the building are classified into the aerobic-induced floor vertical vibration and lateral wind-induced vibrations. The data processing related to each source is presented seperately.

### *Floor vibration*

The first step in the evaluation of vibration is to inspect the collected data in terms of magnitude. Instrumented wired sensors on the 16th floor are shown in Figure 5. The time-history plot of acceleration, displacement and stresses (from strain gauges) at a beam supporting the aerobics class floor is presented for an hour-long class in Figure 6. The beam and location of sensors are selected such that the worst case scenario can be represented. The responses that are plotted in Figure 6 are consistent with the events that occurred in the class: very low vibrations and low stresses which start to increase as people arrive at the exercise class and start to warm-up. At the end of the class, the magnitudes decrease as people leave (this is particularly evident in plots of displacement and stress versus time).

To provide a more reliable evaluation and conclusion, the magnitudes are investigated over a longer duration. The maximum stresses and accelerations of different aerobic classes during the monitoring period are plotted in Figure 7. In addition to the estimation of strength demand, the acceleration response is investigated for serviceability concerns. For this reason, the acceleration response is inspected in terms of both magnitude and frequency, and is compared to the recommended criteria for human comfort (as presented in Figure 4). Additionally, since the occupancy of different portions of the building is different (offices and gym area), the acceleration response of

Table 1. Instrumented sensors for monitoring of the testbed structure.

| Floor | Sensor type | Quantity | Location(s) |
|---|---|---|---|
| 1st floor | Strain gauges | 4 | Lateral Bracings along axes 8 and A |
| 3rd floor | Accelerometers | 2 | On NE and SE corners |
| 7th floor | Accelerometers | 4 | On the floor- under exercise room |
| 12th floor | Accelerometers | 4 | On the floor- under exercise room |
| 13th floor | Accelerometers | 4 | On NE and SE corners |
| 15th floor | Accelerometers | 4 | On the floor – under exercise room |
| | Strain gauges | 2 | Lateral Bracings along axis 8 |
| 16th floor | Accelerometers | 7 | On the floor – gym and office areas |
| | Strain gauges | 7 | On beam flanges, along axes 9 and C |
| | LVDTs (Disp. sensors) | 2 | Mid-span of beams along axes 9 and C in gym area |
| Roof | Accelerometers | 4 (wired and wireless) | On NE and SE corners |
| | Anemometer | 1 | On a steel pole on the penthouse roof framing |

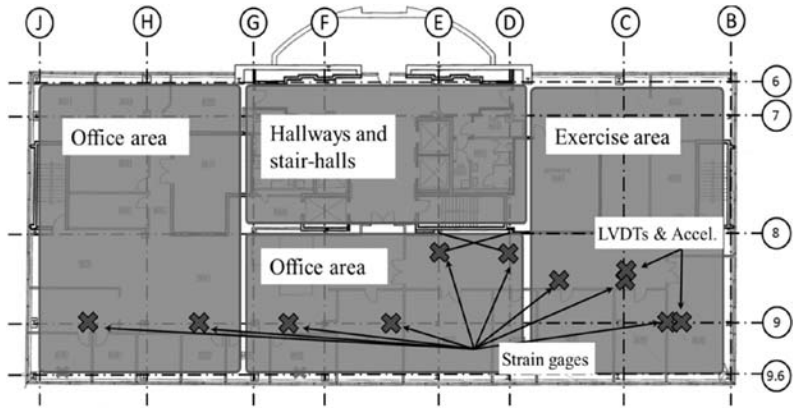


Figure 5. Sixteenth floor plan, different occupancies and wired sensor locations.

different locations needed to be investigated seperately. The portable WSN is used to collect data from various locations on different floors during the onsite monitoring.

Figure 8 illustrates the maximum acceleration amplitude in different locations in the building. Note that the closest location to the aerobics class, where vibration is measured is 5.4 m away from the centre of the class. The highest vibrations (in Figure 8(a),(b)) correspond to the vibration at this location. These two plots illustrate how the amplitude of vertical vibration attenuates on the floor and in the vertical direction with increasing distance from the aerobics class. To evaluate the effects of vibration on human perception, it is necessary to find the frequency range of vibration. Figure 9 shows as an example the

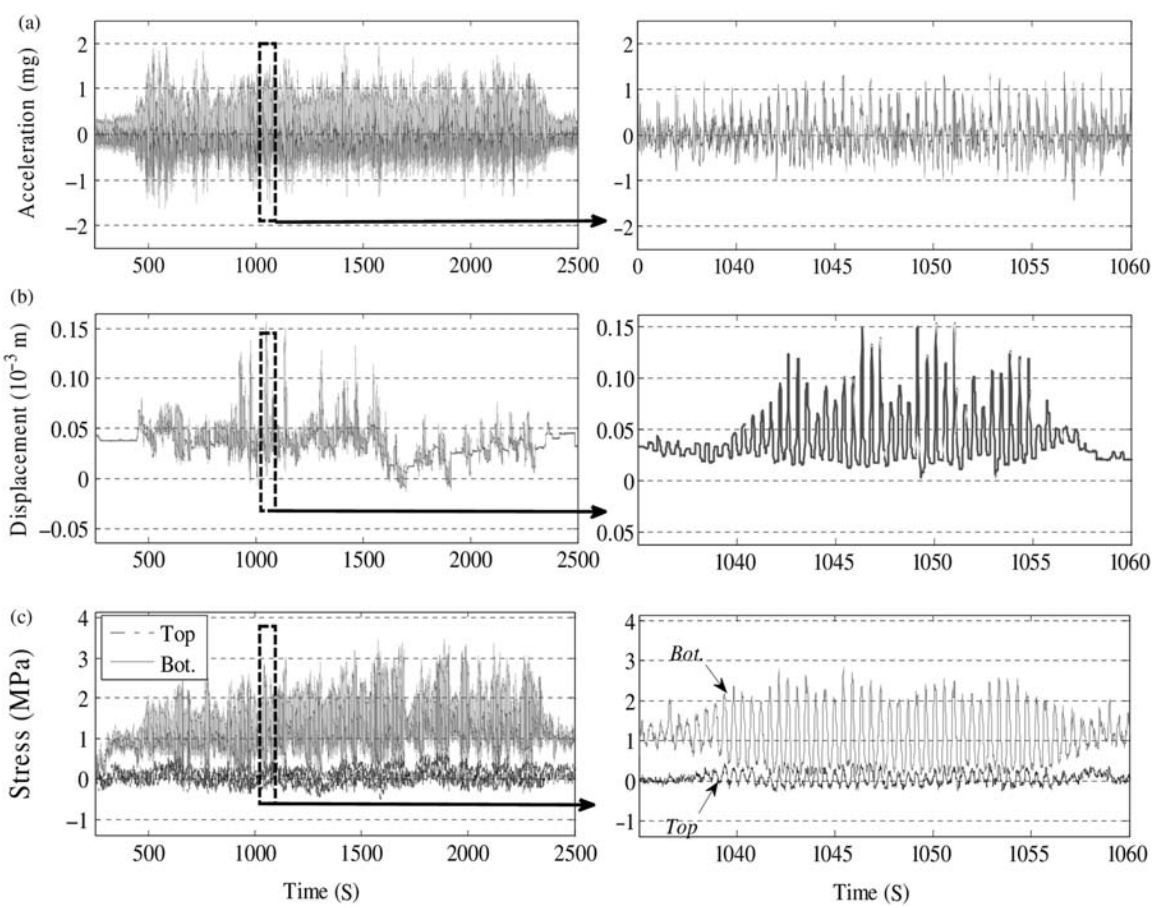


Figure 6. Sample floor responses (acceleration, displacement and stress) to aerobics class.

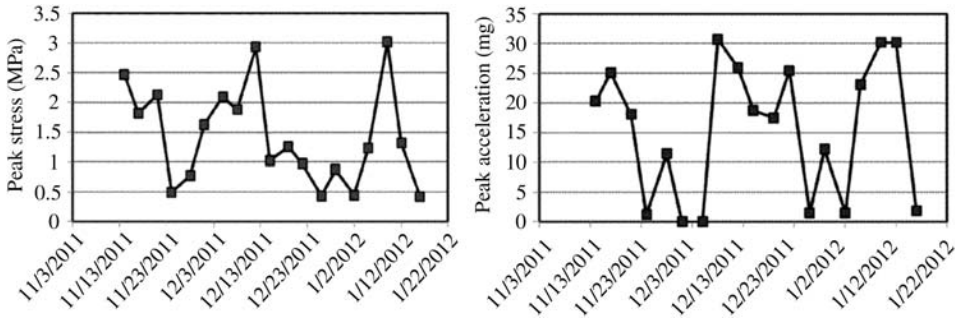


Figure 7. Peak stress and accelerations measured under aerobics floor.

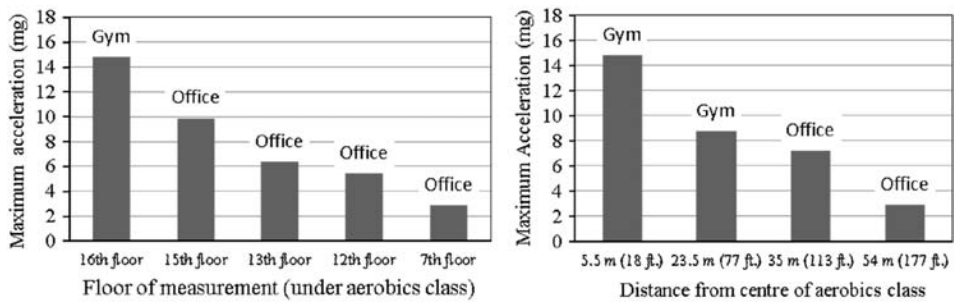


Figure 8. Maximum vertical aerobics-induced acceleration at different locations through the structure.

vertical acceleration response in frequency domain, collected from a beam supporting the aerobics class floor. This plot presents some peaks (2.1, 3.2, 4.3, 6.4 and 8.5 Hz) which are the dominant frequencies of the measured vertical acceleration response. These frequencies are consistent in all data collected from different locations during the aerobics class. It is clear that the acceleration response of the floor to the rhythmic exercise forces is not a single harmonic function, rather it has a range of dominant frequencies (2–8.5 Hz). This is particularly important when assessing the amplitudes based on the limits of human comforts.

While the main goal of monitoring was to evaluate the serviceablity of the structure, its performance in terms of strength demand is also evaluated to ensure that the structural system under the applied loads is without any concern. Therefore, the performance evaluation of the floor system in this building has two aspects: (i) strength demand versus capacity and (ii) vibration levels versus acceptable comfort limits. With regard to the strength demand, the induced stresses during the applied rhythmic forces are examined. As Figure 7 shows, the magnitudes of stress induced by the applied forces in the beam are below 5 MPa. Although this stress presents the effect of live loads

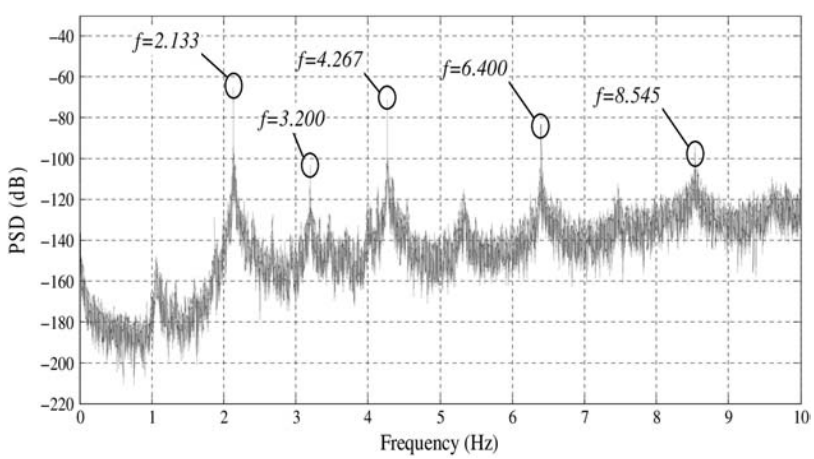


Figure 9. Typical frequency content of measured vertical acceleration.

only, it is significantly lower than the strength limits of a steel beam.The yield strength of the steel is 400 MPa, and therefore, there is no concern over the performance of the floor beams in terms of strength.

For the evaluation of performance of the floor system in terms of serviceability, the criteria suggested in AISC Steel Design Guide Series 11 are used. As the first step, the magnitudes of the peak accelerations in different occupancies are compared to the recommended limits. These limits (as shown in Figure 4) are 50 mg for vertical vibration in areas with rhythmic activities and 5 mg for offices and residential areas, when the vibration frequency is in the range of 4–8 Hz. For the vibration frequencies outside the 4–8 Hz range, the limit increases (as shown in Figure 4). To be conservative, the minimum limits corresponding to 4–8 Hz conservatively are selected for this evaluation.

Additionally, according to AISC Steel Design Guide Series 11, there are two other controls for the evaluation of floor vibration, which are related to the fundamental natural frequency of the floor. (i) For floors with a fundamental natural frequency above 8 Hz, to account approximately for footstep impulse vibration, the acceleration limit is not increased after 8 Hz, as it would be if the acceleration limit as shown in Figure 4 is used. In other words, the horizontal portion of the curves between 4 and 8 Hz as shown in Figure 4 is extended to the right beyond 8 Hz. (ii) If the natural frequency is greater than 9–10 Hz, to account for motion due to varying static deflection, a minimum static stiffness of 1 kN/mm under concentrated load is introduced as an additional check.

To evaluate the floor performance and to check whether additional considerations are needed, the natural frequency of the floor is estimated using the ambient vibration data collected from the floor and also using the equivalent beam method (EBM) which is recommended in the AISC Steel Design Guide Series 11. Inspecting the ambient vibration response in the frequency domain (e.g. power spectral density of the response), the first fundamental natural frequency is observed as 8.53 Hz. Also, using Equation (1) (from EBM), the first natural frequency of the composite beams can be estimated as:

$$f_{\mathrm{n}} = \frac{\pi}{2}\sqrt{\frac{gEI_{\mathrm{t}}}{wl^4}}, \tag{1}$$

where $f_{\mathrm{n}}$ is the fundamental natural frequency in Hz, $g$ is the gravity acceleration (9.81 m/s$^2$), $E$ is the steel modulus of elasticity, $I_{\mathrm{t}}$ is the transformed moment of inertia which is computed by having the steel beam and concrete slab specifications (the slab thickness, effective slab width and concrete modulus of elasticity [$E_{\mathrm{c}}$]), $w$ is the uniformly distributed weight per unit length and $l$ is the member span. Applying this equation on a W16 × 31 section with a 150 mm composite concrete slab depth and a 5880 mm$^2$ reinforcement area, the first fundamental natural frequency is estimated to be 8.19 Hz. This estimated frequency is quite consistent with that obtained from the measurement.

Considering the estimated natural frequency, the acceleration limit should not be increased after 8 Hz when using the recommendations of AISC Steel Design Guide Series 11. Additionally the static stiffness of the composite beams is checked, though the design guide does not impose it for natural frequencies less than 9 Hz. Having a W16 × 31 with a 150 mm. composite concrete slab depth and a 5880 mm$^2$ reinforcement area, the static stiffness under concentrated load ($48EI/l^3$ for simply supported beam) is 9.5 kN/mm, which is well beyond the minimum of 1 kN/mm limit required by AISC Steel Design Guide Series 11. Thus, to evaluate the vibration performance of the floor, only the amplitudes of vibration and frequencies are assessed.

Based on the range of frequencies observed in the collected data (2–8.5 Hz as shown in Figure 9), it is reasonable to consider the horizontal portion of the recommended limitations plot (50 mg for rhythmic activities and 5 mg for offices and residential areas). In the aerobics class area on 16th floor, the maximum measured vertical acceleration (Figure 8) is below 0.035 g. Consequently, the peak acceleration of floor vibration in the aerobics class area is below the recommended limit for aerobics activity. However, in the office areas in some portions of the building, the peak values, as presented in Figure 8, exceed the threshold of 0.005 g for perceptibility at elevations as low as the 12th floor. Therefore, it is concluded that the performance of the structural components in some portions of the building is poor in terms of serviceability, though these structural components meet the measured strength demands.

### *Lateral wind-induced vibration*

#### *Wind load characteristics*

Wind force is one of the important loading factors in the design of building structures. The effects of wind are classified into static and dynamic effects. In order to ensure the structural safety of the building, both the static and dynamic effects of wind force should be carefully investigated. Some important characteristics of wind are wind speed, wind direction and wind fluctuation. During the entire monitoring period, the average wind speed and direction and the maximum wind speed in five-minute intervals are recorded using the wired sensor network. Additionally, during high wind-speed events (defined as a gust speed beyond 14 m/s), the continuous high-speed data are collected from all wired sensors for a period of approximately 5 min. Having collected data, the wind load and the building's response are analysed.

In order to evaluate the structure's response to wind, as the first step, the wind characteristics are inspected. In Figure 10(a) the time history of the maximum and average wind speed for the duration of the monitoring is presented. It can be seen that the wind speed reached a maximum of 32 m/s, and exceeded 30 m/s just a few times. Figure 10(b) shows a wind rose plot for the same data, where the direction of 0° represents wind out of the north. The total bar height at each direction indicates the frequency of occurrence of wind from that direction. A discretisation of 10° was used. Within each bar, the colour variation provides the makeup of the wind speeds that occurred in each direction bin. It can be seen that the wind was predominantly out of the west-northwest and the south.

While the wind speed and direction are the main factors in the static loading, the wind-speed fluctuation is the more important factor in the vibration evaluation. The basic statistical characteristics of the wind-speed fluctuation can be respresented by a turbulance spectrum. The energy of turbulance fluctuation is distributed over a frequency range which is described by the turbulance frequency spectrum density function $S(f)$. Evaluation of this spectrum and frequencies, in which the energy is mainly distributed, is important when assessing the effect of the wind on a structure's vibration. The turbulance spectrum, $S(f)$, is defined as the Fourier transform of the correlation function of wind speed $R(\tau)$. The relationship between turbulance spectrum and wind-speed variance is given as follows (Davenport & Novak, 2002):

$$\sigma_v^2 = \int_0^\infty S(f)\,\mathrm{d}f. \tag{2}$$

It is customary to use another form of the spectrum, known as the *logarithmic spectrum*, which is dimensionless and preserves the relative contribution to the variance at different frequencies (Davenport & Novak, 2002). The logarithmic spectrum is defined as $f \cdot (S(f)/\sigma_v^2)$, and its integral is:

$$\int_0^\infty f \cdot \frac{S(f)}{\sigma_v^2 \mathrm{d}[\ln(f)]} = 1. \tag{3}$$

The logarithmic spectrum of the wind data is extracted using the measurements of the wind speed and wind direction. The wind spectrum can be obtained either for the along-wind direction or for a specific direction. In this work, as the objective is to assess the wind effects on the building structure, the wind speed is projected on each direction and the spectrum is extracted accordingly (from 0° to 360° with 10° intervals). The logarithmic spectrum of wind fluctuation is presented in Figure 11. In this figure, the spectrum is plotted for 360° in a three-dimensional plot as well as the projected spectrum plot for North–South and East–West directions. The wind spectrum is used only for the determination of the main fluctuation frequency contents and the comparison with the fundamental frequencies of the building's response. From Figure 11, it is clear that the major frequency contents of the wind fluctuation spectrum are in a very low frequency range (0.01–0.1 Hz). This range is far below the fundamental natural frequency of this building, as estimated and discussed later in the paper.

As the characteristics of the wind load are investigated, the structure's response and its vibration are also evaluated. Similar to the evaluation of human-induced vertical vibrations, the wind-induced lateral vibrations are investigated in terms of both strength demand and the effects on human comfort. Both wired and wireless sensors are used in this evaluation. The wired sensors are shown to be effective for the long-term strain/stress monitoring and the wireless sensors for the lateral acceleration response measurement at different locations.

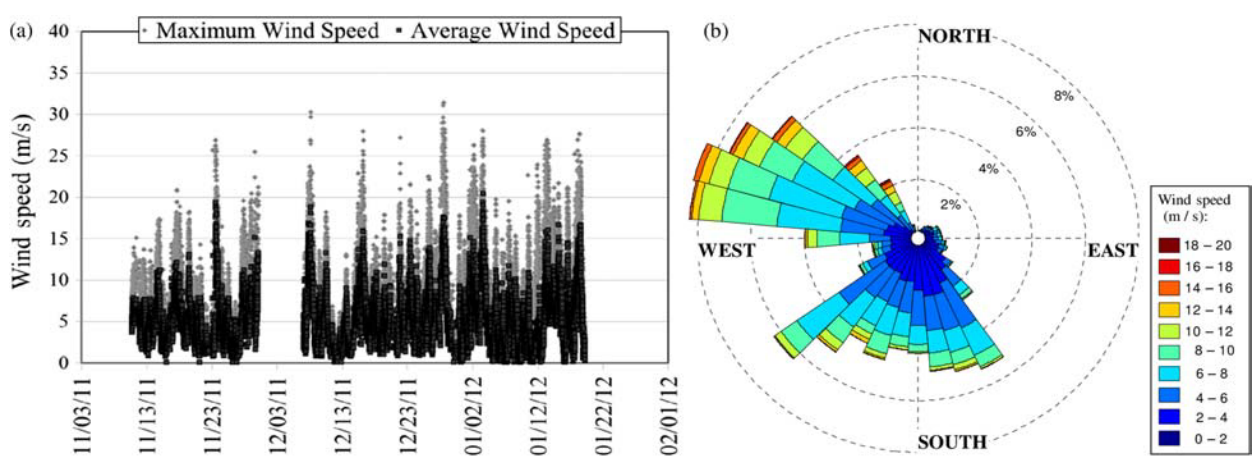


Figure 10. (a) Maximum and average wind speed and (b) wind rose presenting wind direction, wind speed average and the frequency of occurrence.

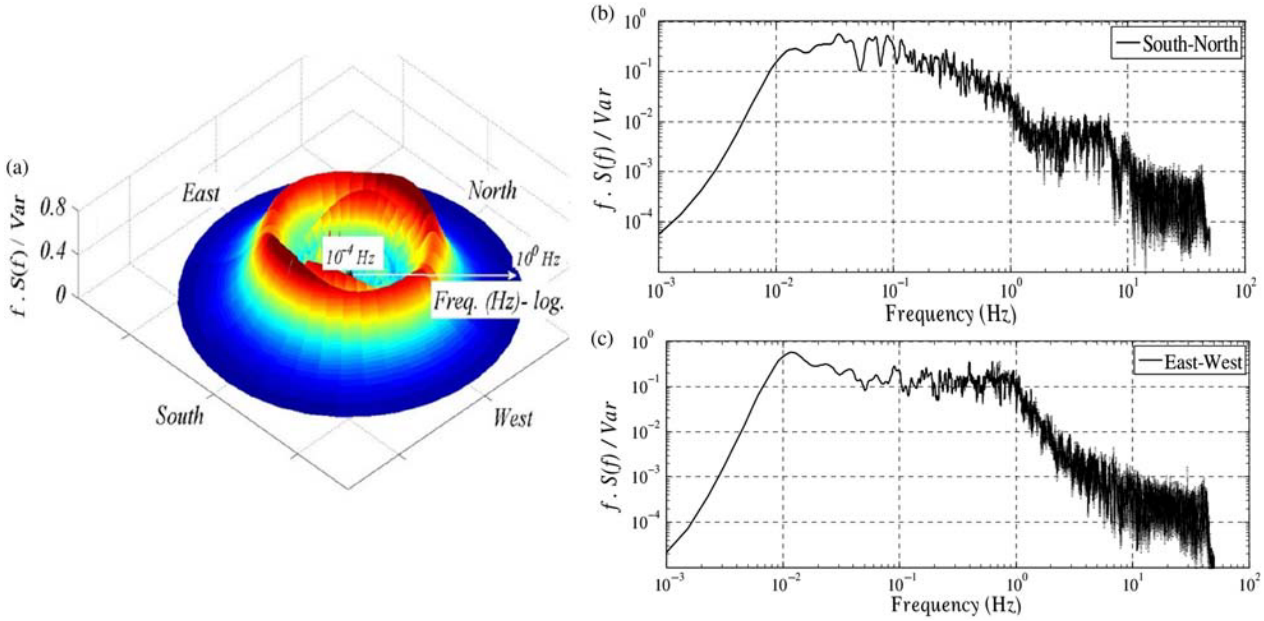


Figure 11. (a) Logarithmic spectrum of wind speed projected on different directions, (b) and (c) logarithmic spectrum of wind speed in South–North and East–West directions, respectively.



*Structural response – strength demands*

Data from the strain gauges installed on the selected wind-bracing members on the 1st and 15th floors are collected during the high wind-speed events. Stresses are presented in the time-history plots for the 1st and 15th floor bracing members together with the wind-speed time history for an example of high wind-speed duration in Figure 12. The wind-speed time history is presented in this figure to enable the comparison of the load and response. Each plot contains the raw data and a 50 point moving average to aid in the identification of lower frequency trends.

It can be seen that at the 15th floor (Figure 12(b)), there does not appear to be a strong correlation with the wind history as shown in Figure 12(a), since there is not a well-defined peak in stress history corresponding to the time of the large wind gust. At the 1st floor bracing, the stress history shown in Figure 12(c) exhibits a more identifiable trend expected for the response of a tall building to dynamic wind loading. In Figure 12(c) in particular, there is a spike in the response inducing tension in the brace element caused by the large wind gust. However, the measured stresses were again low, less than 1.00 MPa for this event. This low level stress demand is less than 0.3% of the lower bound yield strength of the steel members. Therefore, there is no concern over the performance of the lateral load resisting system in terms of strength demand.

*Structural response – vibration performance*

To evaluate the performance of the building in terms of lateral vibrations, the acceleration response due to the wind load is inspected. Figure 13(a)–(c) shows an example of the wind speed, wind direction and acceleration response time-history data collected from the roof level. As can be seen in this figure, the peak accelerations are between 3 and 4 mg. For cross-verification of the measurement results, one wired accelerometer was located on the roof level (the same location as one of the wireless sensors) to measure the lateral vibration and verify the measurements of wireless sensors. Having time stamps in the collected data from the wired and wireless sensors makes the comparison of measurements possible.

Figure 14 depicts the power acceleration response from the two sensing systems. Since the two sensors have different noise characteristics and different sampling rates, it is more reasonable to compare measurements in the frequency domain rather than the time domain. It can be seen that both sensors show similar power spectra and have agreement about the dominant frequencies of the lateral vibration. This consistency confirms the reliability of the two sensing systems while each has certain advantages over the other one (e.g. the wireless sensors are easier to install and more portable, whereas the wired sensors are more controllable for long-term monitoring).

The peak frequencies in the power spectrum plot are important in understanding of the lateral vibration as well as in the comparison of peak accelerations with the recommended limits (Figure 4). As can be seen in Figure 14, there are several dominant frequencies distributed in the range of 0–15 Hz, starting from 0.62 Hz which is the first identified natural frequency of the building. The vibration modes at each of these

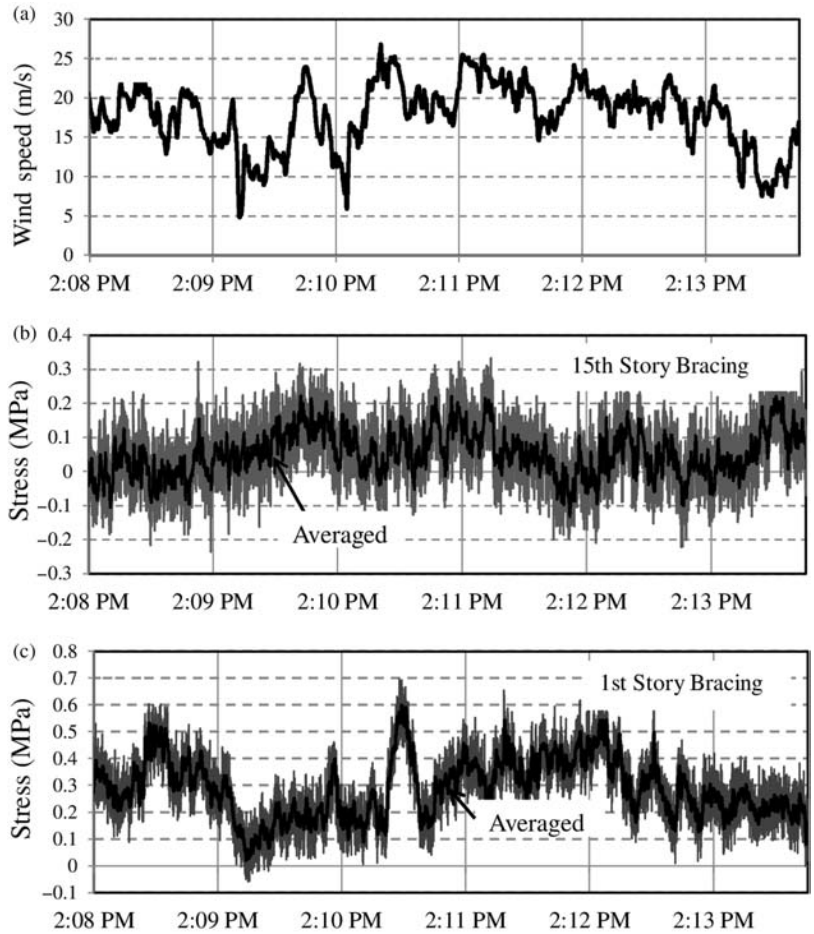


Figure 12. (a) Wind speed, (b) and (c) stresses in bracing at 15th and 1st floor, respectively.

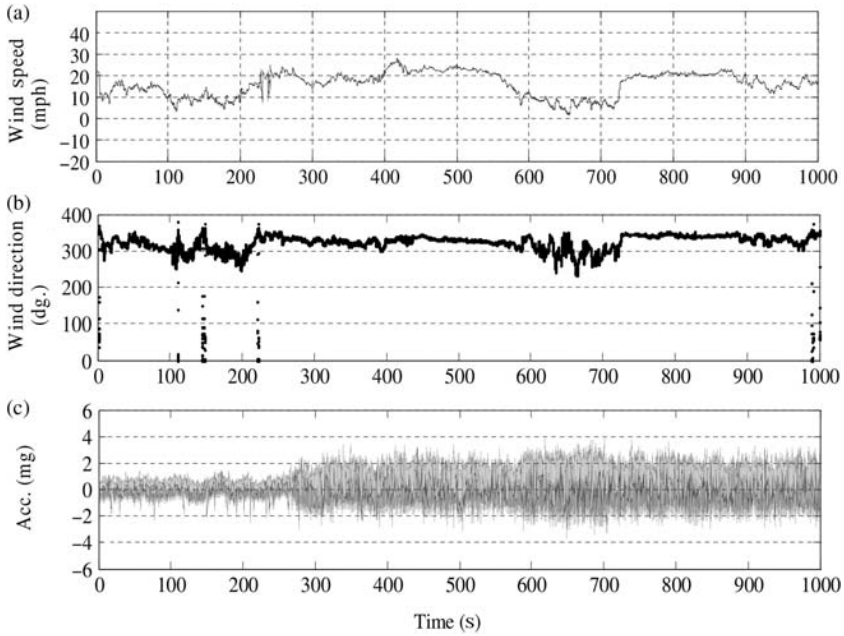


Figure 13. Sample building response to wind: (a) wind speed, (b) wind direction and (c) lateral acceleration at roof.

frequencies are presented later in the next section. For the evaluation of vibration amplitude, the limits are conservatively considered to be the same as the horizontal portion of the plot in Figure 4 despite the lower frequencies of less than 2 Hz.

Since the limit for lateral vibration is in terms of RMS acceleration, the RMS of vibration is inspected. To identify the RMS acceleration amplitude, a 100 s window is moved over the acceleration data, collected during high speed wind events, and the RMS of each window is calculated. The length of window is selected as 100 s since this length is believed to be a reasonable duration of steady vibration which may disturb human comfort (100 s is a conservative assumption as literature [Boggs, 1997] suggests even longer durations [20–60 min]). The maximum RMS obtained from the 100 s windows moved over the entire acceleration data is 1.2 mg which is well below the limits of Figure 4 for all vibration frequencies. It shows that the lateral vibration issues are not as severe as that of the vertical vibrations. To further investigate the lateral vibration of the structure and the different modes of vibration, the acceleration data collected from wireless sensors on different floors are used for modal parameter identification in the next section.

### *Modal parameter identification*

Modal identification is a process in which the modal parameters (natural frequencies, damping ratios and mode shapes) of a system are identified, using the structure's excitation and dynamic response. For modal identification of this structure, the Auto Regressive (AR) time-domain algorithm (Ljung, 1999) is selected. The AR model can be presented as:

$$\bar{y}(n) = -\sum_{i=1}^{p} \alpha_i y(n-i) + e(n), \tag{4}$$

where $y = [y_1(n)\ y_2(n) \cdots y_m(n)]$ is the output (response) matrix, $\alpha_i$'s are the AR coefficients, $e(n)$ represents the noise and measurement error and $p$ is the AR model order. To extract the modal parameters of the system, the AR model can be rearranged to the state space representation which is presented as follows:

$$x(n+1) = A_d x(n) + B_d u(n), \tag{5a}$$

$$y(n) = Cx(n) + Du(n), \tag{5b}$$

where $x(n)$ is the state vector, $y(n)$ is the observation vector and $u(n)$ is the input vector at time step $n$; $A_d$ is the state and $B_d$ is the input matrices in discrete format, $C$ is the observation matrix and $D$ is the transmission matrix.

Choosing the state vector in the controller form, the state and observation matrices can be expressed as:

$$A_d = \begin{bmatrix} 0 & I & \cdots & 0 \\ \vdots & \vdots & \ddots & \vdots \\ 0 & 0 & \cdots & I \\ -\alpha_p & -\alpha_{p-1} & \cdots & -\alpha_1 \end{bmatrix} \quad \text{and} \tag{6}$$

$$C = \begin{bmatrix} I & 0 & . & . & 0 & 0 \end{bmatrix},$$

where $[I]$ and $[0]$ are identity and zero matrices with appropriate dimensions, respectively. Eigenvalue decomposition of the state matrix ($A_d$) results in the matrices of eigenvalues ($\lambda_i$'s) and eigenvectors ($\psi_i$'s) from which the natural frequencies, damping ratios and mode shapes of the system can be obtained (Ljung, 1999).

Some of the estimated mode shapes are presented graphically in Figure 15. Since the sensors were located only on three elevations of the building (3rd, 13th floor and roof), the modal displacements of the intermediate floors are not available. This limitation on sensor placement was imposed by the building's accessibility. Therefore, modal displacement of only the six points (two N–E and S–E corners of each elevation) of the structure is shown in the mode shape plots. The lateral vibration modes of the structure can be categorised into the flexural transverse

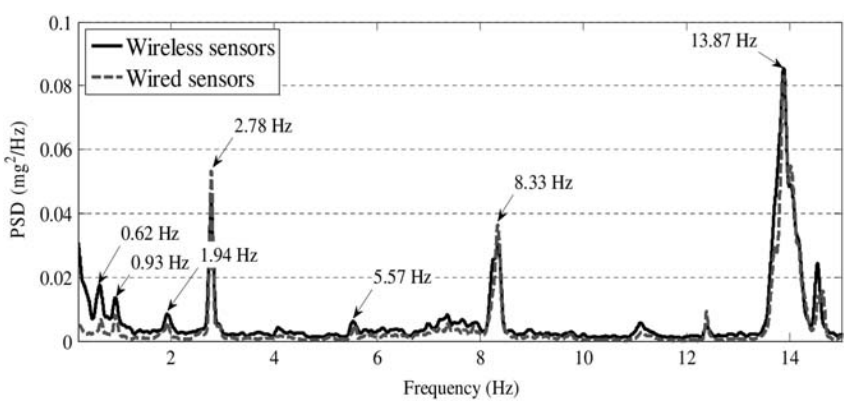


Figure 14. Power spectrum of the lateral acceleration response at roof from wired and wireless sensors.

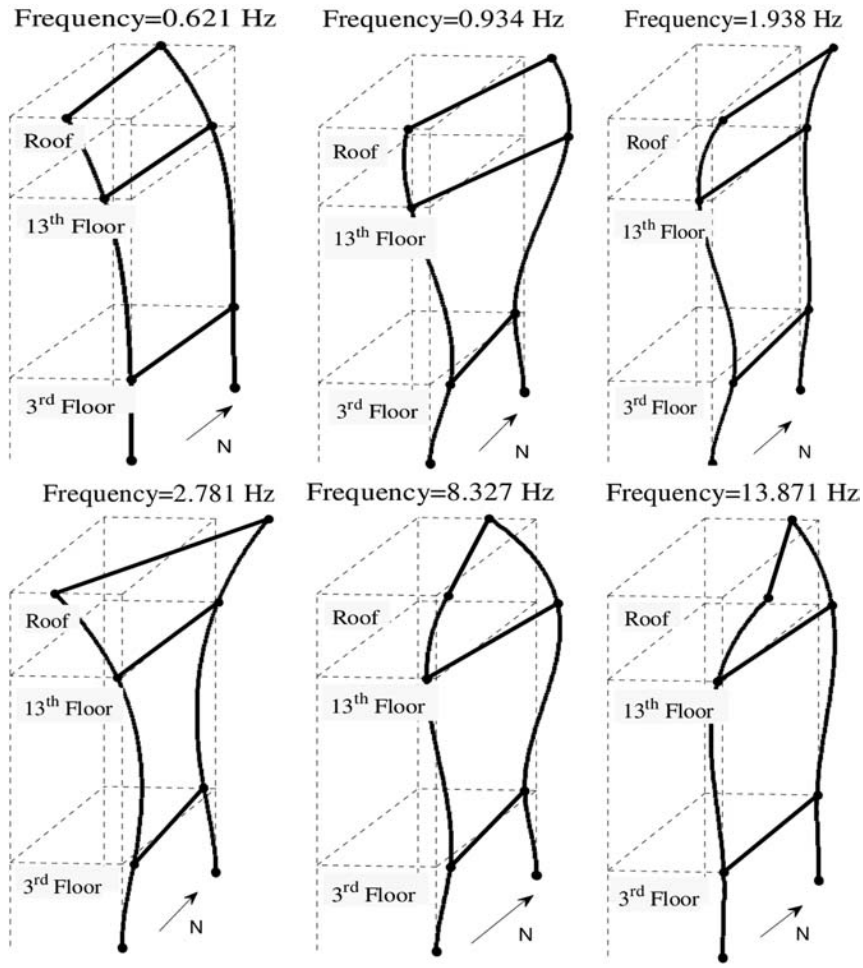


Figure 15. Identified modes of vibration.

and torsional modes based on the in-phase or out-of-phase modal displacements of sensors in the two corners of the building. Using the results of modal identification, the type of vibration at each dominant frequency can be identified. It should be noted that the mode shapes identified through modal identification process are those that were properly excited by the ambient wind loads.

By observing Figure 15, it can be seen that most of the excited mode shapes are torsional modes. These identified torsional modes correspond to the peaks in the response power spectrum (0.93, 2.78, 5.567 and 14.53 Hz), presented in Figure 14. Considering the dominant frequency contents, it can be concluded that the perceived lateral vibrations on higher floors of the building are rooted mostly in torsional modes. This hypothesis is consistent with diagonal cracks that have been discovered on the exterior claddings of the building. A possible reason for such behaviour can be the configuration of the lateral load-resisting systems (braced frames) in the structure's plan. As explained in the building's description, the braced frames in the East–West direction are located on the two outermost column lines (north and south sides) in the plan and the braced frames in the North–South direction are located adjacent to the building centreline in the plan (this can also be observed from Figure 3(c)). This stiffness distribution is not efficient in terms of torsional resistance of the structural system.

The above evaluation examined different aspects of the structural performance. Many of these results are obtained with only a limited measurement effort through the use of mobile wireless accelerometers. The combination of wired and wireless sensors was the key point which made a comprehensive sensing system for short-term and long-term vibration evaluation of the building structure.

## Conclusions

The paper presented the integration and deployment of a comprehensive sensing system for the evaluation of vibration of a building. The sensing system included two networks of wired and wireless sensors providing an

effective tool for capturing different structural responses. Integrated in the sensing system were accelerometers, strain gauges, displacement sensors and an anemometer. Utilising different sensor types with relatively high spatial and temporal resolution enabled a reliable evaluation of the building's performance.

It is shown that the application of the wireless accelerometer sensors significantly facilitates the measurement of structural response from different locations. The ease of installation due to the absence of wiring was a major advantage in wireless sensors as valuable information about the structure's performance is obtained with minimal efforts. Deployment of this sensing system demonstrated that wireless mobile accelerometer networks are effective tools for forensic quantification of building vibrations. However, some challenges associated with the use of wireless sensors, such as limitation due to finite battery life and the communication problem in the presence of obstacles, make the application difficult for long-term monitoring of building structures, requiring the deployment of wired sensors for this purpose.

Complementing the wireless sensors by the traditional wired sensors further enhanced the capability of the sensing system in capturing the structural response. While wireless sensors were relocated multiple times for measuring acceleration response from various locations, the wired sensors in the fixed locations were monitored for a long duration. The wired sensors were programmed specifically for automatic data collection during different events such as the occurrence of a high-speed wind or an aerobics class in the building. Reliable data characterising the structure's response to different dynamic excitations provided the needed information for the assessment of the structure's performance.

The collected data were analysed with different data processing techniques (e.g. time- and frequency-domain data analysis, spectrum analysis and modal identification) to extract different characteristics of the loads and the structural response. Dynamics of floor vibrations, the structural demands under various loading conditions, the wind spectra and response spectra and the modal parameters of the structure are important results obtained using the collected data. Applying the available standards together with engineering insights, the vibration performance of the building structure is evaluated. The evaluation showed that the performance of the structure under human induced vibration effects is not satisfactory in terms of serviceability, as the vibration amplitudes in some locations of the building exceed the maximum comfort limits. However, the strengths of structural components are well above the demands by the measured stresses. The evaluation of the structural response to wind showed that the lateral vibration performance of the building meets the limits in terms of strength demand as well as serviceability. Moreover, inspection of the response spectrum along with the results of modal identification showed that the torsional modes of vibration dominate the response, as compared to flexural modes.

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