

RESEARCH ARTICLE

Structural damping estimation from live monitoring of a tall modular building

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Abstract

The damping ratio is a key indicator of an individual structure's susceptibility to dynamic loads, including the level of discomfort experienced by the occupants of a tall building subjected to wind loading. While computational models, laboratory studies and empirical data can provide estimates of structural damping, the most reliable way to evaluate true damping ratio values is through modal identification using data from field tests on full-scale finished structures. As an innovative form of construction, high-rise modular buildings have not been the subject of previous vibration monitoring investigations, implying an absence of essential structural dynamics information. This paper assesses the reliability of four modal identification methods for estimating the damping ratio of a structure using ambient acceleration response data recorded from the world's tallest modular structure, the *Ten Degrees* building in Croydon, South London. The methods considered are two implementations of the Bayesian fast Fourier transform (BFFT), the random decrement technique (RDT), and a hybrid of the RDT which first decomposes the ambient data into sub-signals using analytical mode decomposition (AMD-RDT). Each method is applied to response data collected during 10 significant wind loading events to evaluate the inherent modal properties of the structure, with the computed damping ratio values compared between methods and events. By reporting the first measured damping ratios for a tall modular structure, the paper makes an important contribution to knowledge about the vibration properties of an emerging form of construction.

KEYWORDS

ambient vibration monitoring, damping ratio, modal identification methods, tall buildings, tall volumetric modular construction, wind-induced acceleration response

1 | INTRODUCTION

In recent years, there has been many advancements in structural engineering including the development of new materials, construction methods, and computational power which has enabled the construction of super-tall, slender, and sometimes striking structures.^{1–4} These structures are often inherently lightweight with lower natural frequencies and damping ratios making them more likely to be excited by wind loading than conventional structures.^{2,3} The wind-induced motion of these structures can be perceptible to humans and cause fear and alarm along with symptoms such as inability to focus, dizziness, nausea, and motion sickness.^{2,3,5} The damping ratio has been recognized as a key parameter in the

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dynamic analysis of structures to assess their vulnerability to excessive wind-induced response.^{1,6} Therefore, determining the damping ratio is imperative in assessing the habitability of a structure.

Over the last two decades, modular buildings have experienced increased interest due to their reduced environmental impact, improved quality and accuracy, and speed of construction.^{7–10} Volumetric modular construction typically involves the off-site manufacture of individual modules in a controlled factory environment. The modules are then transported to site where they are constructed around an in situ lateral stability element, such as a reinforced concrete core, to complete a finished building. The construction of the modules in a factory means that a significant amount of construction time is saved on site, less labor is required and there is more accuracy, less injuries, and less waste in the construction process.^{11,12} While modular construction is predominantly used in low to medium rise construction projects such as multi-unit residential accommodation, it is a relatively new concept for taller buildings.^{7,11,13} Modular construction continues to increase in height due to economic drivers, with building heights of over 130 m now realized.^{14,15} However, as with other structural forms, habitability requirements associated with excessive acceleration response can become the governing design criterion as building heights increase. From a dynamic response viewpoint, modular structures are often required to meet the more stringent vibration criteria imposed for residential buildings, typically limiting response acceleration to less than 5 milli-g (50 mm/s²). Hence, it is crucial for the further development of modular construction that the wind-induced acceleration response of this form of construction is better understood and characterized. While there is a small amount of literature discussing the potential of high-rise modular construction,^{15,16} there is little or no work reporting results of field measurements or modal properties for these type of buildings.

Damping does not relate to a unique physical phenomenon like mass or stiffness.¹⁷ Therefore, there is no theoretical calculation process which can be performed to determine damping values for an individual structure.¹⁸ Damping estimation of a structure relies on empirical data from previous full-scale monitoring campaigns of buildings with many design codes offering values based on type of construction and building materials.^{17,18} For this reason, despite advances in technology allowing for better modeling and scaled wind tunnel tests, the only accurate method for obtaining the damping ratio of a structure is from full-scale monitoring of the completed structure.^{19–21} To the best of the authors' knowledge, damping ratios of tall modular buildings have not been previously reported in literature, meaning designers are forced to use estimates based on other forms of construction. For example, the European standard design code, Eurocode 1991-1-4,²² recommends a value of 0.08 for the logarithmic decrement of damping, equivalent to 1.27% of critical damping, for mixed concrete and steel structures. For comparison, the Eurocode 1991-1-4 recommended values for critical damping for steel and concrete buildings are 0.8% and 1.59%, respectively. Furthermore, the American standard design code, ASCE 7-10,²³ recommends a critical damping value of 1% for steel structures and 1.5% for concrete structures. Similarly, for structures more than 80 m tall, the International Organization for Standardization, ISO 4354:2009,²⁴ recommends values of 1% for steel structures but recommends a lower value than Eurocode 1991-1-4 and ASCE 7-10 of 1.2% for concrete structures.

Tamura and Yoshida²⁵ proposed equations for estimating the critical damping ratio of office buildings based on the Japanese Damping Database for structures with a maximum ratio of tip displacement to height, $\frac{x_H}{H}$, of 2×10^{-5} . Similarly, Smith et al.²⁶ proposed equations for the values of damping ratios derived from a database given by Satake et al.²⁷ which contains the intrinsic damping in the fundamental mode obtained from full-scale monitoring of a number of tall buildings. These expressions are presented in Table 1, where the predicted damping values are compared for a 95-m-tall building.

In terms of results from full-scale monitoring, Ohkuma et al.²⁸ reported the full-scale monitoring of a 68-m-tall predominately steel structure and found the actual damping ratios of the structure to be 0.95% in the first along-wind mode. This value falls within the range of values recommended by Eurocode 1991-1-4, ASCE 7-10, and ISO 4354:2009 and the estimated values given by the equations in Table 1. Similarly, Gonzalez-Fernandez et al.²⁹ performed full-scale monitoring of a 150-m-tall concrete structure and found the actual damping ratio to be 0.80% in the first mode; again, this value is smaller than the aforementioned recommended values. Data from the Chicago Full-Scale Monitoring Programme were analyzed by Kijewski-Correa et al.²¹ who used the random decrement method described in Section 3.2 to estimate the damping ratio of the in situ structures. In total, three buildings were instrumented for full-scale monitoring, all over 60 stories tall. The first building comprised a steel tube system with exterior columns and stiffening elements and was found to have a damping ratio of 0.87% in the first mode. The second building was a reinforced concrete building with shear walls and was found to have a damping ratio of 1.42% in the first mode using the

TABLE 1 Expressions for structural damping reported in literature.

Building type	Expression	95-m building	Reference
Reinforced concrete (10 m < H < 100 m)	$\zeta = \frac{0.75}{H} + 380 \frac{x_H}{H} - 0.0014$	$\zeta = 1.41\%$	Tamura and Yoshida ²⁵
Steel (30 m < H < 200 m)	$\zeta = \frac{0.52}{H} + 320 \frac{x_H}{H} + 0.0023$	$\zeta = 1.40\%$	Tamura and Yoshida ²⁵
Steel	$\zeta = 7.20H^{-0.3887}$	$\zeta = 1.23\%$	Smith et al. ²⁶
Steel and reinforced concrete	$\zeta = 21.2H^{0.5708}$	$\zeta = 1.58\%$	Smith et al. ²⁶
Reinforced concrete	$\zeta = 28.25H^{-0.6393}$	$\zeta = 1.54\%$	Smith et al. ²⁶

RDT method. The third building was a steel moment-connected, framed tubular system and was found to have a damping ratio of 1.04% in the first mode.

The results from the above full-scale monitoring campaigns of in situ steel and concrete structures show large variability in the actual critical damping ratios of full-scale structures. In many cases, the critical damping ratios recommended by Eurocode 1991-1-4, ASCE 7-10, and ISO 4354:2009 are higher than the observed results from full-scale monitoring as described above. Similarly, there is variation in the predicted critical damping ratios using equations derived by Tamura and Yoshida and Smith et al.

It is clear that there is significant uncertainty in the critical damping ratio of structures and while design codes and equations provide estimates of critical damping ratios based on empirical data, the variability between these estimates is evident. Given the novelty of modular construction for high-rise buildings, it is important to investigate the damping ratio of this new form of structure. There are a much larger number of movement joints in modular structures than in conventional steel or concrete buildings. Therefore, it is not unreasonable to hypothesize that there is much greater potential for frictional energy dissipation and modular structures may have higher damping ratios than their more conventional counterparts.

The damping ratio is the key parameter for modal identification chosen in this study as it is the modal parameter which is hardest to predict accurately without full-scale testing and is significantly influential in the dynamic response of a structure. Reporting values of damping ratios obtained from live monitoring is an important advancement for modular construction and, ultimately, will allow for more informed designs in the future.

2 | IN SITU TESTING

2.1 | Description of instrumented building

Modal identification was performed to analyze the ambient acceleration response of a full-scale high-rise volumetric modular building for a series of 10 sets of measurement data. The monitored building considered in the study is the *Ten Degrees* building, in Croydon, South London, UK. Upon completion, this became the tallest modular building in the world¹⁴ and recently won the Council of Tall Buildings and Urban Habitat's 2022 Structural Engineering Award.³⁰ The full structure, location shown and pictured in Figure 1, is a 44-story, 135-m-tall residential building. The structure consists of two slip formed concrete cores, a transfer slab at Level 4 and two adjoining towers of 37 and 44 stories that consist of volumetric corner post modules stacked around and connected to the two concrete cores. The concrete cores are approximately 8×8 m in plan and have walls which range from 300 to 450 mm thick. The landing slabs within the core are 300 mm deep. The concrete cores act as the main element for lateral load resistance. The structure has a slenderness ratio (height/breadth) of 8.



FIGURE 1 View of the *Ten Degrees* Tower and location.

The volumetric modules are typically 2.875 m tall; are limited in length and width to 13 and 6 m, respectively, due to transportation; and have a typical self-weight of 7 kN/m. Each module consists of a 150-mm-thick concrete slab encased in steel perimeter channels, four square hollow sections acting as corner post columns, four square hollow sections as beams, and two bracing members, one on each of the longer faces of the module. The roof of the module is a suspended plaster ceiling, supported by the square hollow section beams, which conceals services and inter-module connections. The member sizes within the modules are highly optimized and change from story to story to enhance efficiency and reduce costs.

Connections are an imperative feature of modular construction. The nature of modular buildings leads to a larger number of connections compared to traditional forms of construction. This is due to the need to attach both the individual members within the modules and the modules to one another to form the finished structure. There are also significantly more members in a modular structure, further adding to the number of connections. Generally, there are considered to be two categories of module connections: intra-module connections and inter-module connections.³¹ Intra-module connections refer to the internal connections between individual members of the module such as connections between beams, columns, and bracing members. This form of connection is consistent with conventional forms of construction, and members are welded to a high tolerance level in a factory setting prior to transportation to site.

Inter-module connections concern the connections between individual modules and are unique to modular construction. In total, three types of inter-module connections were employed within the investigated building: connections between adjacent vertical members and connections between adjacent horizontal members and vertical ties. These connections use a combination of welded plates, brackets, and dywidag bars. The two towers of the structure are joined together at the modules using these bespoke connections, providing continuity to the structure and creating diaphragm action across the floors of the multi-story building. Hence, the stiffness of these connections influences the overall dynamic response of the structure.

The presence of an additional lateral stability element in this building results in another type of connection: connections from the modules back to the RC core. These connections facilitate the transfer of lateral wind loads from the exterior modules to the interior cores. The connections between the RC core and modules utilize a combination of embedded steel plates, brackets, and welds. The stiffness of this type of connection is significant in the dynamic response of the overall structure as it will impact the overall stiffness contribution of the modules.

The complexity of a modular system and bespoke connections adds difficulty in quantifying the stiffness of the structure and assuming a damping ratio. In reality, the only easily quantifiable inherent property of the structure is mass. The full-scale monitoring campaign is required to assess the other inherent dynamic properties.

2.2 | Instrumentation overview

Two three-axis MEMS accelerometers (CX1 *Structural Response Monitor*, resolution 0.00001 g, range ± 1.5 g, frequency range DC–200 Hz) were directly mounted approximately 1.84 m above the floor slab at Level 43, that is, just below roof level in the taller core, as pictured in Figure 2. The

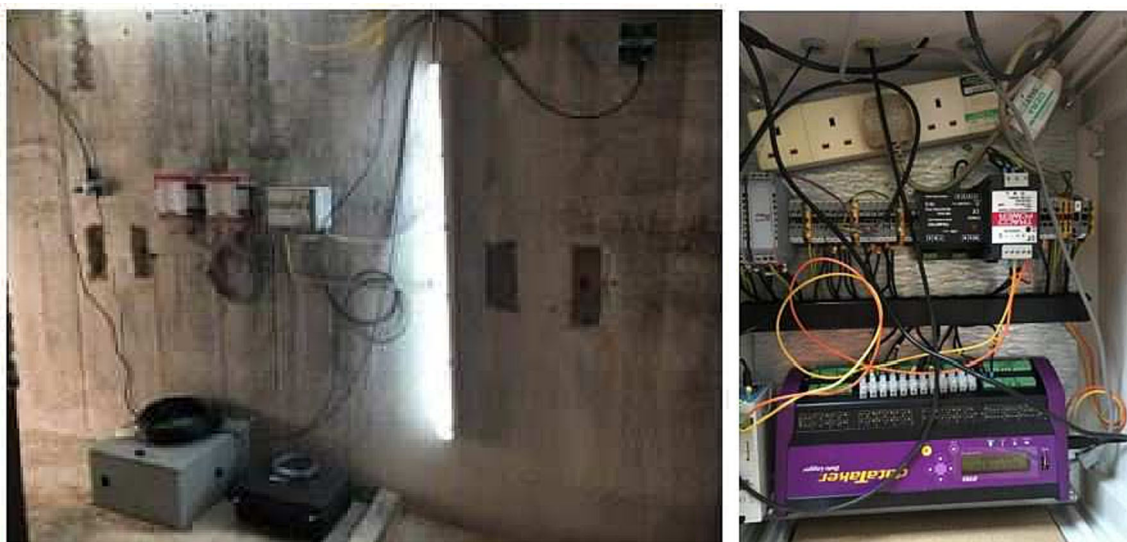


FIGURE 2 Picture of accelerometer and data logger used in monitoring campaign.

sampling rates for the accelerometers was 20 Hz. The accelerometers were installed off center to capture lateral and torsional vibrations as shown in Figure 3. On the roof of the core, a weather station was installed to record 10-min averaging wind speed and direction, maximum/minimum wind speed and direction within each 10-min window, temperature, humidity, atmospheric pressure, rain, and its duration. A 3G router was also installed to allow for remote access to all data. Data from the weather station were continuously monitored.

Construction of the cores to full height was completed in late 2018, prior to installation of any modules. As pictured in Figure 4, modules were then installed around the cores. Monitoring of the structure was undertaken over a 2-month period, beginning in late August 2019 and ending in late October 2019. Not all modules were installed when the monitoring campaign began. Therefore, initial measurements were taken on a partly completed structure.

In total, 10 sets of measurement data were recorded. A measurement set comprises of 12 h of acceleration and wind speed data from before and after an instance in which the recorded acceleration exceeded a predetermined threshold value for at least 10 s.

As mentioned, the module installation was not complete when monitoring commenced and continued over the period when measurements were taken, resulting in monitoring being performed on an evolving structure. In this paper, the measurement data are presented in chronological order with the structure becoming progressively closer to completion as the sets of measurements were taken and being fully complete by Measurement 8 with all floors of modules installed. Table 2 shows the dates and number of stories of modules installed for each measurement taken. Figure 5 presents a graphical representation of the stories of modules installed when each set of measurement data was recorded. Comparing Figures 4 and 5, it can be appreciated how initial measurements were performed on a structure approximately corresponding to the right-hand picture in Figure 4. Furthermore, from Figure 5, it can be seen that there was a significant increase in modules installed between Measurements 5 and 6.

The evolution of the structure as measurements were taken results in damping ratios and natural frequencies that differ between events and reflect the change in the inherent properties of the structure. As modules are added to the upper levels of the structure, the mass of the structure increases. This is expected to effect the dynamic response significantly as the mass increase occurs at the top of the structure where it has most influence. The increase in the number of modules will also influence overall lateral stiffness. However, in contrast to the additional mass, the stiffness may not increase significantly due to the modules being added at the top of the structure, further from the point of most influence at the base. Therefore, a decrease in the natural frequency estimated by the modal analysis performed on each consecutive set of measurement data is expected, which will affect response amplitude observed in each measurement, as presented later.

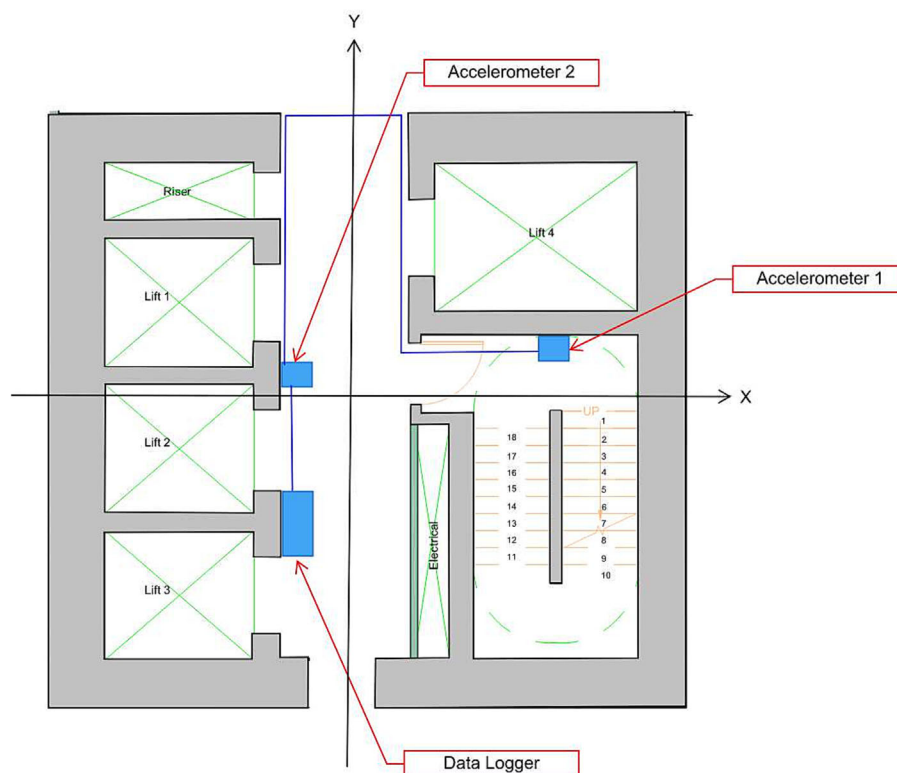


FIGURE 3 Location of accelerometers and data logger.



FIGURE 4 The *Ten Degrees* building in Croydon, London, pictured at two points in time during module installation. It can be seen how the structure consists of volumetric modules stacked around two RC cores.³²

TABLE 2 Dates in situ measurement data were recorded and the number of floors of modules installed.

Measurement	Date	Number of stories completed	
		Tower A	Tower B
1	31/08/2019	32	32
2	02/09/2019	33	32
3	06/09/2019	34	34
4	09/09/2019	35	34
5	11/09/2019	36	34
6	27/09/2019	41	34
7	29/09/2019	42	34
8	11/10/2019	44	37
9	13/10/2019	44	37
10	26/10/2019	44	37

3 | DAMPING ESTIMATION—STATE OF THE ART

As discussed in Section 1, estimating the damping ratio of a structure relies on empirical data from monitoring a full-scale building. Full-scale monitoring of a structure can take the form of forced vibration experiments or ambient data collection. Forced vibration experiments involve applying a known input loading to the structure and recording the output. Often, forced vibration experiments require an impractically large loading, cannot be performed while the structure is operational, and are uneconomical. For this reason, ambient vibration tests have become increasingly popular in both structural health monitoring and modal identification technique development.^{19,33} The tests typically involve the placement of sensors, such as accelerometers, along the geometrical planes of the structure. A data acquisition system is employed to record and save the data from the sensors. The structure will naturally be excited by wind loading and other dynamic loads as it is operational during these field measurements. The loading the structure experiences is unquantified; however, it is assumed to be statistically random.³³ The acceleration response of

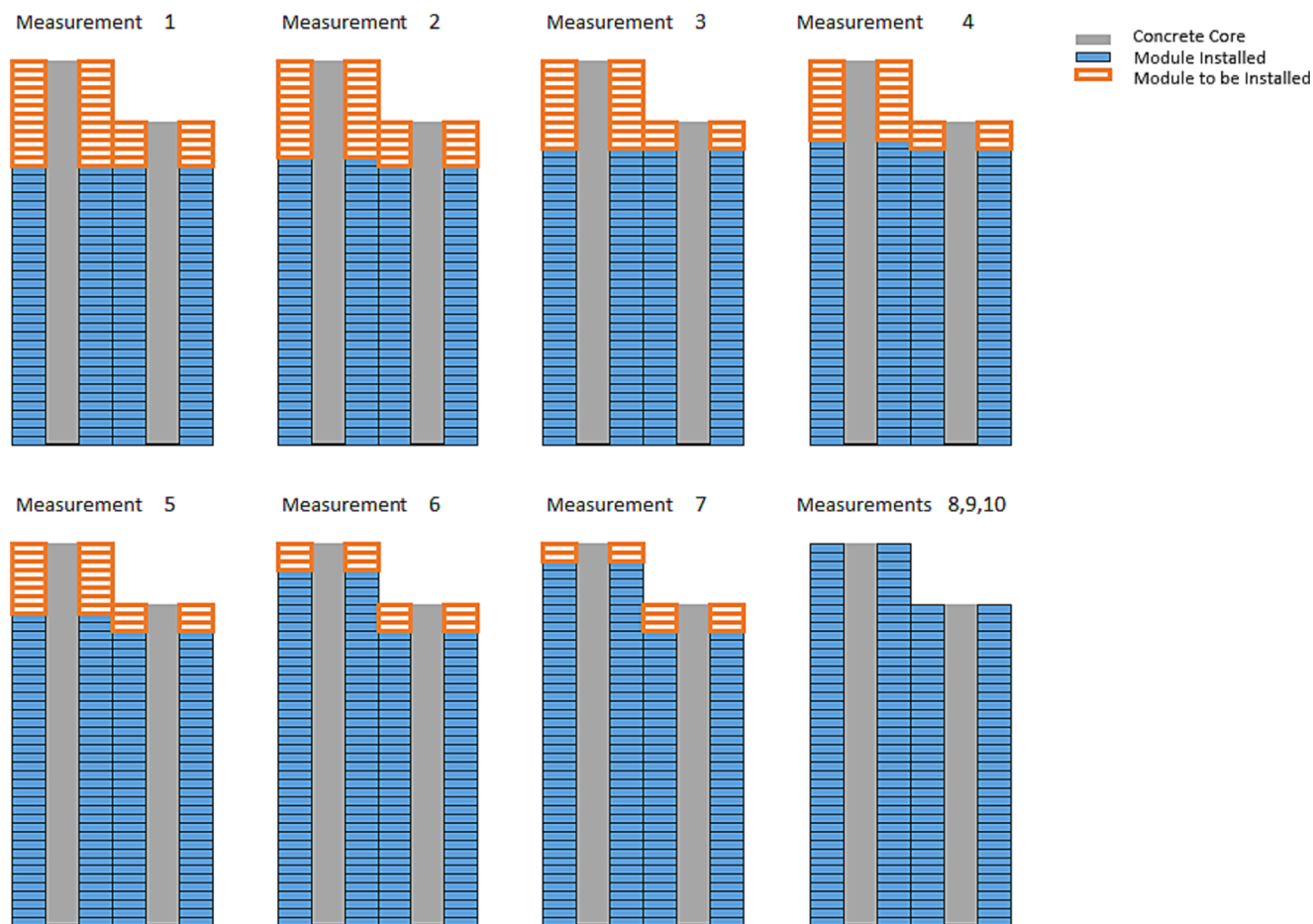


FIGURE 5 Graphical representation of the number of levels of modules installed for each set of measurement data.

the structure is recorded by the accelerometers and is considered ambient response data. These ambient data can then be processed using modal identification techniques to identify the key parameters of the structure such as natural frequencies, damping ratios, and mode shapes.

Many different modal identification techniques are proposed in literature, some of which operate in the time domain, while others operate in the frequency domain. This paper considers three modal identification methods: the Bayesian fast Fourier transform (BFFT) which operates in the frequency domain, the random decrement technique (RDT) which operates in the time domain and a hybrid of the RDT which involves first decomposing the response signal into sub-signals using analytical mode decomposition (AMD). The third method is here on referred to as the AMD-RDT.

3.1 | BFFT

The first method employed to estimate the damping ratio of the structure is the BFFT. The BFFT is a method commonly used in modal identification to predict the modal properties of a structure from ambient data and has been investigated extensively.^{33–41} Ambient data are frequently described as data in which the loading is unknown but is assumed to be statistically random and stationary.^{33,34,36} The principle of the BFFT is that both the real and imaginary parts of frequency domain data, obtained from a fast Fourier transform (FFT) of the recorded time-domain acceleration response of a structure experiencing broad-band excitation will have a joint Gaussian distribution that can be described analytically by a set of modal parameters, θ . The modal parameters contained in θ are the natural frequency f , damping ratio ζ , mode shape ϕ , entries of the force spectral density matrix $\{S_{ij}\}$, and the spectral density of the prediction error σ^2 , for any given mode. Frequency domain data are used to maximize the posterior probability density function (PDF) of the modal parameters and hence find the most probable value (MPV) of each of the modal properties given the measured response.

The measured acceleration can be modeled as $\hat{\mathbf{x}}_j = \mathbf{x}_j(\boldsymbol{\theta}) + \epsilon_j$ where $\mathbf{x}_j(\boldsymbol{\theta})$ is the acceleration response of the structural system calculated from fundamental structural dynamics theory given a set of modal parameters, $\boldsymbol{\theta}$, and ϵ_j is the measured error in the acceleration response, that is, the difference between the actual and modeled response of the structural system.

The output of an FFT at a given frequency k is a complex number Z_k , with a real part F_k and imaginary part G_k . It has been shown that for long duration data, the real and imaginary components of Z_k are jointly Gaussian with zero mean and a covariance matrix defined as

$$\mathbf{C}_k = \frac{1}{2} \begin{bmatrix} \boldsymbol{\Phi}(\text{Re}\mathbf{H}_k)\boldsymbol{\Phi}^T & -\boldsymbol{\Phi}i(\text{Im}\mathbf{H}_k)\boldsymbol{\Phi}^T \\ \boldsymbol{\Phi}(\text{Im}\mathbf{H}_k)\boldsymbol{\Phi}^T & \boldsymbol{\Phi}(\text{Re}\mathbf{H}_k)\boldsymbol{\Phi}^T \end{bmatrix} + \frac{\sigma^2}{2} \mathbf{I}_{2n}, \quad (1)$$

where $\boldsymbol{\Phi} \in \mathbb{R}^{n \times m}$ is the mode shape matrix of the measured DOFs, σ^2 is the constant spectral density level of the prediction error, $\mathbf{I}_{2n}$ denotes the $2n \times 2n$ identity matrix, $\mathbf{H}_k$ is the spectral density matrix of the model response, and its (i, j) entry is given by

$$\mathbf{H}_k(i, j) = S_{ij} [(\beta_{jk}^2 - 1) + i(2\zeta_i \beta_{jk})]^{-1} [(\beta_{jk}^2 - 1) + i(2\zeta_j \beta_{jk})]^{-1}, \quad (2)$$

where $\beta_{jk} = f^{(i)}/f_k$ is the frequency ratio; $f^{(i)}$ and ζ_i are natural frequency and damping ratio of the i th mode, respectively; and S_{ij} is the force cross-spectral density between the i th and j th modal excitation. The modal identification is accounted for by the first term in Equation (1), and the prediction error is accounted for by the second term.

Using Bayes theorem, the posterior PDF of $\boldsymbol{\theta}$ given the FFT data is proportional to the likelihood function $p(\{\mathbf{Z}_k\}|\boldsymbol{\theta})$ under the assumption that the prior distribution is uniform, sometimes termed an uninformative prior^{33,36,37}:

$$p(\boldsymbol{\theta}|\{\mathbf{Z}_k\}) \propto p(\{\mathbf{Z}_k\}|\boldsymbol{\theta}). \quad (3)$$

It can be shown that the distributions of $\mathbf{Z}_k$ at different frequencies are independent. Therefore, the distribution considering all frequencies in an FFT is given by the product of the distribution at each individual frequency. Thus, the posterior PDF, which is to be maximized, is proportional to the product of likelihood functions such that

$$p(\boldsymbol{\theta}|\{\mathbf{Z}\}) \propto p(\{\mathbf{Z}\}|\boldsymbol{\theta}) = (2\pi)^{-(N_q-1)/2} \left[\prod_{k=2}^{N_q} \det \mathbf{C}_k(\boldsymbol{\theta}) \right]^{-1/2} \times \exp \left[-(1/2) \sum_{k=2}^{N_q} \mathbf{Z}_k^T \mathbf{C}_k(\boldsymbol{\theta})^{-1} \mathbf{Z}_k \right]. \quad (4)$$

The dependence of $\mathbf{C}_k$ on $\boldsymbol{\theta}$ is highlighted in the equation, so it is possible to calculate and therefore maximize, $p(\mathbf{Z}|\boldsymbol{\theta})$ for a given $\boldsymbol{\theta}$. While the posterior distribution $p(\boldsymbol{\theta}|\mathbf{Z})$ is unknown, thanks to the proportionality defined in Equation (4), it can be maximized by maximizing the known $p(\mathbf{Z}|\boldsymbol{\theta})$ distribution. This process is made more efficient if Equation (4) is written in terms of the log-likelihood function $L(\boldsymbol{\theta})$:

$$p(\boldsymbol{\theta}|\{\mathbf{Z}_k\}) \propto \exp[-L(\boldsymbol{\theta})], \quad (5)$$

with $L(\boldsymbol{\theta})$ defined as in Equation (6):

$$L(\boldsymbol{\theta}) = (1/2) \sum_{k=2}^{N_q} \left[\ln \det \mathbf{C}_k(\boldsymbol{\theta}) + \mathbf{Z}_k^T \mathbf{C}_k(\boldsymbol{\theta})^{-1} \mathbf{Z}_k \right]. \quad (6)$$

It can be shown that the posterior PDF can be approximated by a Gaussian PDF for a sufficiently large data set.³³ To do this, the second-order approximation of $L(\boldsymbol{\theta})$ is taken by letting $\hat{\boldsymbol{\theta}}$ be the MPV that minimizes L . By expanding as a second-order Taylor series about $\hat{\boldsymbol{\theta}}$ with the first-order term vanishing due to optimality of $\hat{\boldsymbol{\theta}}$.

$$L(\boldsymbol{\theta}) \approx L(\hat{\boldsymbol{\theta}}) + \frac{1}{2} (\boldsymbol{\theta} - \hat{\boldsymbol{\theta}})^T \mathbf{H}_L(\hat{\boldsymbol{\theta}}) (\boldsymbol{\theta} - \hat{\boldsymbol{\theta}}). \quad (7)$$

The Hessian of L at the MPV is $\mathbf{H}_L(\hat{\boldsymbol{\theta}})$. The posterior PDF becomes a Gaussian PDF by substituting into Equation (5).

$$p(\boldsymbol{\theta}|\{\mathbf{Z}_k\}) \propto \exp \left[-(1/2) (\boldsymbol{\theta} - \hat{\boldsymbol{\theta}})^T \hat{\mathbf{C}}^{-1} (\boldsymbol{\theta} - \hat{\boldsymbol{\theta}}) \right], \quad (8)$$

where $\hat{\mathbf{C}}$ is the posterior covariance matrix

$$\hat{\mathbf{C}} = H_L(\hat{\theta})^{-1}. \quad (9)$$

The calculation of both the MPV and covariance matrix are required for the computation of the Gaussian PDF. Therefore, the goal of the BFFT method is to estimate these two variables so that $L(\theta)$ is minimized.

Computationally, this can be challenging, both in terms of time and achieving convergence. Au³³ proposed a more efficient method to perform the minimization that avoids the need to calculate the determinant and inverse of $C_k(\theta)$, as required in Equation (6). Within this work, two separate approaches were proposed. Firstly, a “general” approach, that makes no assumptions about the quality of the measured data, is presented. It is then shown that if it is assumed that the signal-to-noise ratio is high, an approximate, or “asymptotic,” approach can reduce the dimensionality of the minimization problem and further improve computational speed and convergence. In this paper, damping is estimated using both minimization schemes proposed, that is, the asymptotic and general approaches, meaning that two separate sets of modal properties, referred to as the $BFFT_{asm}$ and $BFFT_{gen}$ results, respectively, are obtained from the BFFT technique.

The ambient acceleration signals collected in this monitoring campaign contained 12 h of data. In order to apply the BFFT, the data set was broken down into non-concurrent 12-min sub-signals and processed separately. The estimated natural frequency and damping ratio from each sub-signal were then averaged to obtain the estimates from this method. The purpose of the signal decomposition is to improve efficiency in processing time and account for possible outliers resulting from numerical issues related to convergence of the minimization problem.

3.2 | RDT

The second approach employed to estimate the damping ratio of the structure is the RDT. This was first proposed for use in the aerospace industry by Cole⁴² and has been applied extensively throughout the fields of Civil and Structural Engineering for damping estimation of structures.^{18,19,43,44} The RDT is based on the basic idea that white noise excitation of a single degree of freedom (SDOF) system will result in response, $x(t)$, which consists of the response due to initial displacement x_{x_0} , the response due to the initial velocity $x_{\dot{x}_0}$, and forced response due to random excitation x_F such that

$$x_{ni}(t) = x_{x_0} + x_{\dot{x}_0} + x_F. \quad (10)$$

The RDT can be used to estimate the damping experienced by a linear structural system examining a “signature” obtained from averaging segments of the response.^{18,45} These segments are sub-sections of the time history of the acceleration response which follow a threshold condition,^{18,21,42,44} typically a triggering value, x_p , which satisfies both amplitude and slope criteria. By setting x_p at an appropriate level to intercept the time series of the data $x(t)$, multiple sub-signals $x_{ni}(t)$ can be obtained. Assuming the forced vibration and the initial velocity of the structure are zero-mean stationary Gaussian variables, the expectation of the sub-excitation $E[x_F]$ and of the sub-response $E[\dot{x}_0]$ are zero. Therefore, by averaging a large number of sub-signals with identical triggering conditions, the ensemble average of the initial velocity and forced vibration responses reduce to zero, leaving only the response due to the initial displacement:

$$E[x_{ni}(t)] = E[x_{x_0}] + E[\dot{x}_0] + E[x_F] = E[x_{x_0}]. \quad (11)$$

This means that essentially the random component of the response is removed leaving a signal comprising only the free decay response of the structure. Due to the limited duration of data measurement in practice, the expectation of the sub-response is taken as the arithmetic mean across each sub-signal extracted. Therefore, the random decrement signature, $\delta(t)$, is defined as

$$\delta(t) = \frac{1}{N} \sum_{k=1}^N x_{ni}(t_k + \tau), \quad (12)$$

where N is the number of subsamples and $\tau = t - t_i$.

Once the free vibrational response has been determined, the Hilbert transform is applied to the random decrement signature $\delta(t)$ to approximate the free decay response and determine the damping ratio ζ_i .^{44,46} For multi-degree of freedom (MDOF) systems, simply assuming an SDOF system and applying the RDT, termed the “traditional RDT” by Tamura et al⁴⁷ can efficiently evaluate the natural frequency and damping ratio if the modes are well separated.^{47,48}

3.3 | AMD

While the traditional RDT can work for MDOF systems if the modes are well separated, if there are closely located predominant frequency components, a beating phenomenon is observed in the random decrement signature and the RDT cannot be used for evaluation of damping ratio.^{43,46,47} To overcome this issue, the RDT is commonly combined with a signal decomposition method. A conceptually straight-forward decomposition approach is to employ a band-pass filter, as in Zhou and Li¹⁸ for example. However, it is difficult to design a filter for a complete separation of the two frequency components as the spacing of the two frequencies becomes very small.⁴⁹ Empirical mode decomposition (EMD) is one method that has been shown to yield good results when combined with the RDT.^{19,46} However, EMD has many shortfalls, the most problematic of which is that it does not accurately decompose signals with closely spaced frequencies or beating which are common in structural systems.^{43,49–51} Others have examined the use of AMD for signal decomposition and found it superior to EMD and to be highly effective for signals with highly coupled modes.^{49–51} In this paper, the AMD is combined with the RDT for modal identification from ambient data as presented by Wen et al.⁴³ and Chen and Wang.⁵¹ The AMD decomposes a signal into two components, separated by a bisecting frequency ω . Each sub-signal can then be analyzed using the RDT outlined in Section 3.2 to extract the free decay response of the structure and determine its damping ratio.

Let $\mathbf{x}(t)$ denote the measured acceleration data containing a number of frequency components $(\omega_1, \omega_2, \dots, \omega_n)$ where n is the number of sub-signals in which the data are to be decomposed. The sub-signals, $\mathbf{x}_i(t) (i = 1, 2, \dots, n)$, have Fourier spectra $\tilde{\mathbf{X}}(\omega)$ which cover n mutually exclusive frequency ranges such that $(|\omega| < \omega_{b1}), (\omega_{b1} < |\omega| < \omega_{b2}), \dots, (\omega_{(bn-2)} < |\omega| < \omega_{(bn-1)})$ and $(\omega_{(bn-1)} < |\omega|)$. $\omega_{bi} \in (\omega_i, \omega_{i+1}) (i = 1, 2, \dots, n-1)$ are the bisecting frequencies. Therefore,

$$\mathbf{x}(t) = \sum_{i=1}^n \mathbf{x}_i(t). \quad (13)$$

Each of the modal responses has a narrow bandwidth in the frequency domain and can be determined by

$$\mathbf{x}_i(t) = \mathbf{s}_i(t) - \mathbf{s}_{i-1}(t), \dots, \mathbf{x}_n(t) = \mathbf{x}(t) - \mathbf{s}_{n-1}(t), \quad (14)$$

$$\mathbf{s}_i(t) = \sin(\omega_{bi}t)H[\mathbf{x}(t)\cos(\omega_{bi}t)] - \cos(\omega_{bi}t)H[\mathbf{x}(t)\sin(\omega_{bi}t)], \quad (15)$$

where $H[\cdot]$ represents the Hilbert transform. The AMD method provides an alternative to the band-pass filtering technique which, as mentioned, performs poorly when modes become very close.⁴⁹ Other than possible end effects due to the finite length of a structural systems response, the AMD separates the frequency contents of a time series exactly. It has been shown that the selection of the bisecting frequencies does not influence the response decomposition.^{49,51} However, typically, bisecting frequencies, ω_{bi} , are selected around the average of the two adjacent natural frequencies.^{43,49–51}

After application of the AMD method to create sub-signals of the measured acceleration response, the RDT is applied to obtain the free vibrational response of the structural system and identify the damping ratio. In this paper, damping is calculated using the RDT with and without AMD.

4 | ANALYSIS OF DATA FOR MODAL IDENTIFICATION

4.1 | Analysis of raw measurement data

Prior to applying the three modal identification techniques, the raw data were first assessed in order to identify differences across the 10 measurements. As the data recorded are all ambient and this is not a controlled test, the wind exciting the structure and therefore the accelerations experienced vary across measurements. Qualitative assessment of the raw acceleration responses of the structure can also identify any significant noise in the acceleration signals recorded.

Figure 6 shows the average wind speed for each 10-min period over the course of each of the 10 measurement sets. This allows the variation in average wind speed both within and between measurements to be examined. Average wind speeds were more significant for later measurements (after Measurement 6). This may be due to these measurements occurring later in September and October when storms and strong winds are typically more common. Another possible explanation for this is that more modules were installed in this structure for the later measurements, the structure was close to complete/completed, and therefore, there was more significant surface area for winds to be deflected upwards and towards the weather station. However, this is difficult to prove without completing CFD or wind tunnel tests for the structure at different stages of completion.

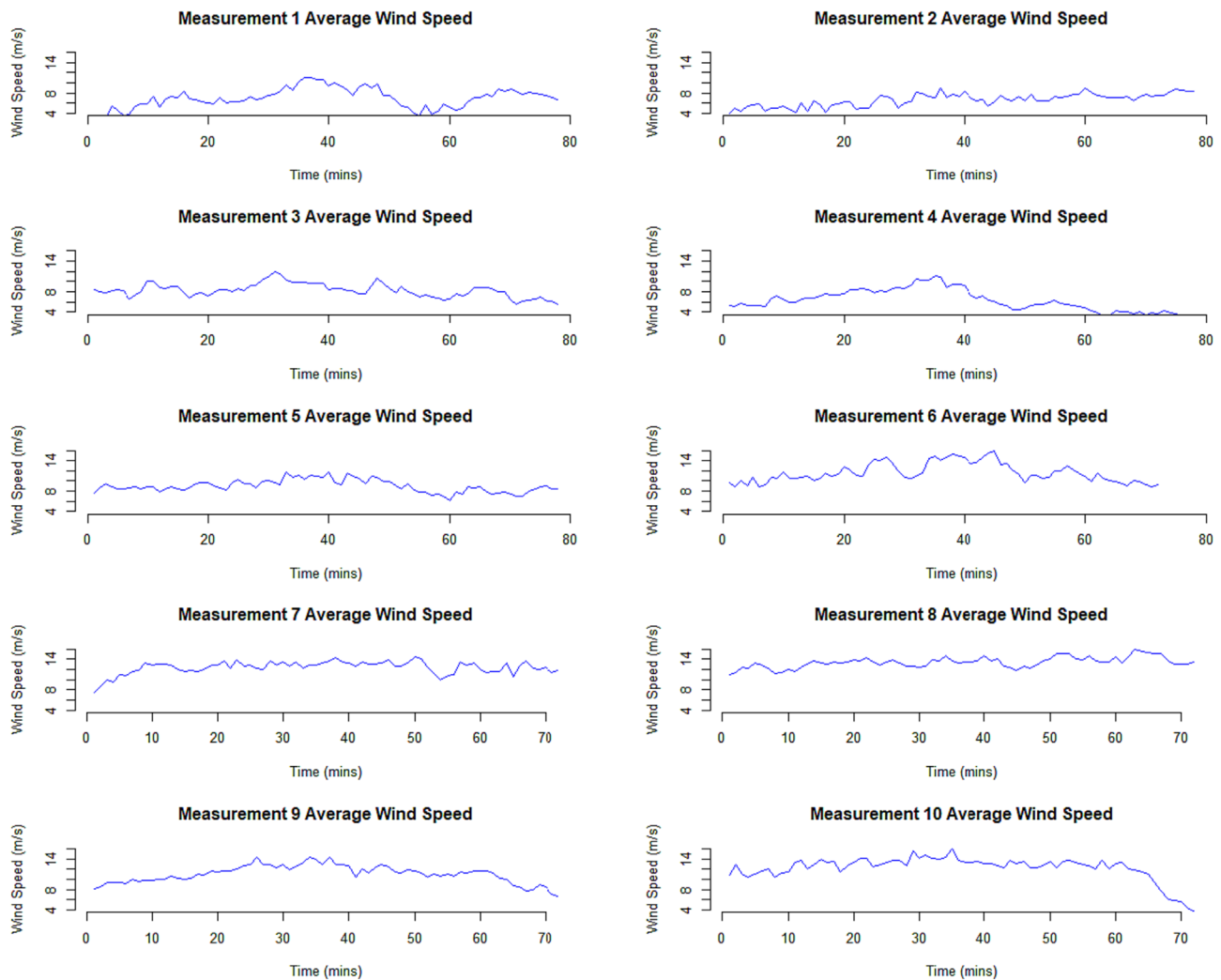


FIGURE 6 Average wind speed.

TABLE 3 Wind speeds for each in situ measurement of response.

Measurement	Average wind speed (m/s)	Max wind speed (m/s)
1	7.00	15.93
2	6.66	14.04
3	8.23	16.83
4	6.28	13.78
5	8.99	17.50
6	11.65	23.92
7	12.35	20.20
8	13.40	23.23
9	10.99	20.47
10	12.22	25.54

The average and maximum wind speed for each measurement are given in Table 3. Again, it can be seen that both average and max wind speeds have increased for Measurement 6 onward. For context, it is worth highlighting that the basic 10-min average 50-year wind speed at 10 m is approximately 21.5 m/s and the 1-year 10-min average wind speed at 10 m used to assess serviceability limit state performance is approximately 17 m/s.

Figures 7 and 8 show the acceleration response in the x (roughly East–West) in red, y (roughly North–South) in blue, and z (roughly vertical) in green directions for both accelerometers overlaid by the average wind speeds. The z-direction accelerations have had acceleration due to gravity removed. For Measurements 1, 6, and 8, both accelerometers display time segments with significant accelerations in the z direction. This can be identified as noise due to construction equipment being used within the structure. It is possible that the signals from these measurements will be significantly distorted and may not yield useful results during modal analysis. Measurements 7–10 show accelerations that appear to match the profile of the average wind speeds quite well. However, larger accelerations can be seen at less significant wind speeds for Measurements 1–6; this is likely due to the structure not yet being complete.

Tables 4 and 5 show the maximum and RMS accelerations for each accelerometer, in the two primary directions (EW being x and NS being y) for each measurement. The large values seen for Measurement 8 can be attributed to construction noise as previously discussed.

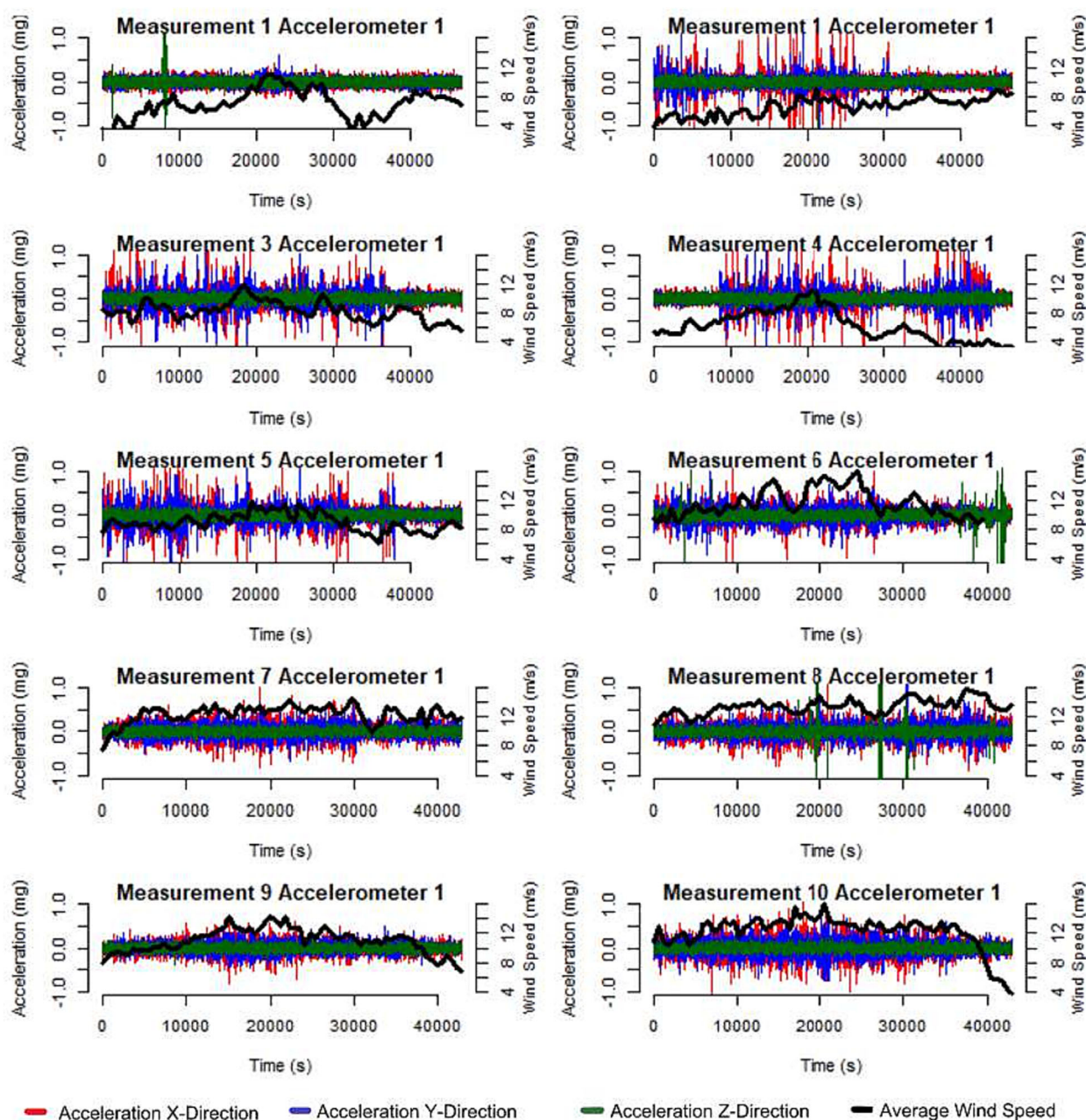


FIGURE 7 Acceleration and wind time histories for Accelerometer 1.

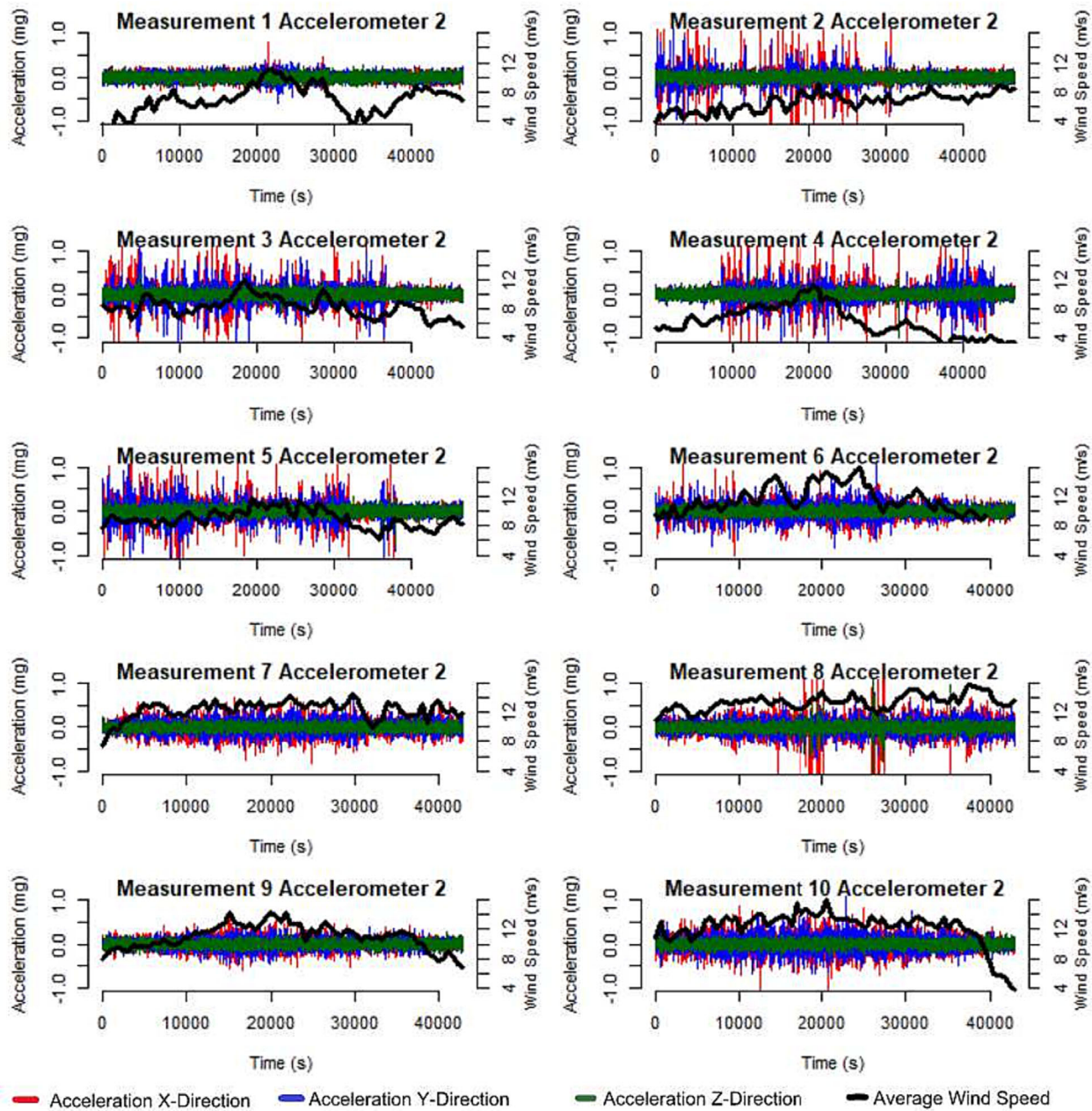


FIGURE 8 Acceleration and wind time histories for Accelerometer 2.

4.2 | Applying modal identification techniques for estimation of natural frequencies

Each of the three modal identification techniques discussed in this paper were applied to the raw data for each of the measurements. Figure 9 shows the FFT of the acceleration data from Accelerometer 1 for Measurement 10 in both x and y directions. Close inspection suggests two separate fundamental bending modes are found near 0.31 and 0.36 Hz. The BFFT was applied first in order to find the first natural frequencies in both the x (EW) and y (NS) directions. Table 6 shows the natural frequencies calculated from both accelerometers, in both directions, for each measurement. It can be concluded that the first natural frequency of the measurements taken from the completed structure (Measurements 8–10) in the first mode in the EW direction is approximately 0.316 Hz and in the NS direction is approximately 0.371 Hz.

In general, it can be seen that the values are in close agreement between the two accelerometers and do not vary significantly across measurements. The natural frequencies are slightly higher for earlier measurements, as is evident in Figure 10. This can be explained by the structure not yet being complete and therefore having a lower modal mass. For earlier measurements, there are less modules and therefore less mass and stiffness. As modules are added, the mass of the structure increases, as does the stiffness. However, since the natural frequency reduces as more

TABLE 4 Maximum accelerations for each measurement.

Measurement	Accelerometer 1		Accelerometer 2	
	Maximum acceleration EW (milli-g)	Maximum acceleration NS (milli-g)	Maximum acceleration EW (milli-g)	Maximum acceleration NS (milli-g)
1	0.868	15.586	2.185	0.058
2	3.005	4.756	2.770	2.493
3	3.512	1.744	12.847	1.964
4	3.262	3.329	15.157	3.351
5	3.159	2.753	3.024	2.831
6	2.658	2.089	2.520	1.644
7	1.322	0.854	1.023	0.805
8	15.577	74.869	61.829	7.281
9	0.967	16.309	1.258	0.751
10	1.709	1.154	2.002	1.315

TABLE 5 RMS accelerations for each measurement.

Measurement	Accelerometer 1		Accelerometer 2	
	RMS acceleration EW (milli-g)	RMS acceleration NS (milli-g)	RMS acceleration EW (milli-g)	RMS acceleration NS (milli-g)
1	0.103	0.092	0.091	0.089
2	0.205	0.138	0.198	0.137
3	0.228	0.177	0.224	0.177
4	0.223	0.178	0.221	0.177
5	0.226	0.183	0.219	0.184
6	0.179	0.150	0.173	0.155
7	0.178	0.142	0.170	0.139
8	0.210	0.834	0.630	0.160
9	0.148	0.124	0.136	0.115
10	0.211	0.172	0.205	0.170

modules are added, it can be concluded that the impact on natural frequency of any increase in the stiffness of the structure due to the additional modules is not as significant as the impact due to increased mass.

4.3 | Applying modal identification techniques for estimation of critical damping ratios

Each of the modal identification methods was applied to the ambient data for the 10 sets of measurements. Figure 11 shows the damping ratios calculated using the two described BFFT methods for each of the 10 measurements. The damping ratios estimated by the BFFT_{asm} method can be seen to be quite large in some cases and often unrealistic. The BFFT_{gen} method gives more appropriate estimates of the damping ratio; however, there is close agreement between the methods in some cases in the EW direction.

Figure 12 shows the calculated damping ratio for the two described RDT methods for each of the 10 measurements. It can be seen that both methods are in close agreement for most sets of measurement data. However, the AMD-RDT method presents more consistent estimates, particularly for Measurements 4 and 8.

Figure 13 compares the calculated damping ratio for the BFFT_{gen} and AMD-RDT methods for each of the 10 measurements. There is variation in the estimates from each method. The AMD-RDT method shows lower estimates in the EW direction for Events 6–10 but generally higher estimates for the earlier measurements in the NS direction. However, there does appear to be some convergence on values for the later events in both directions. In general, it can be observed that the AMD-RDT method provides more conservative estimates of the critical damping ratio of the completed structure.

Table 7 presents the average damping ratios estimated using each method for Measurements 7–10.

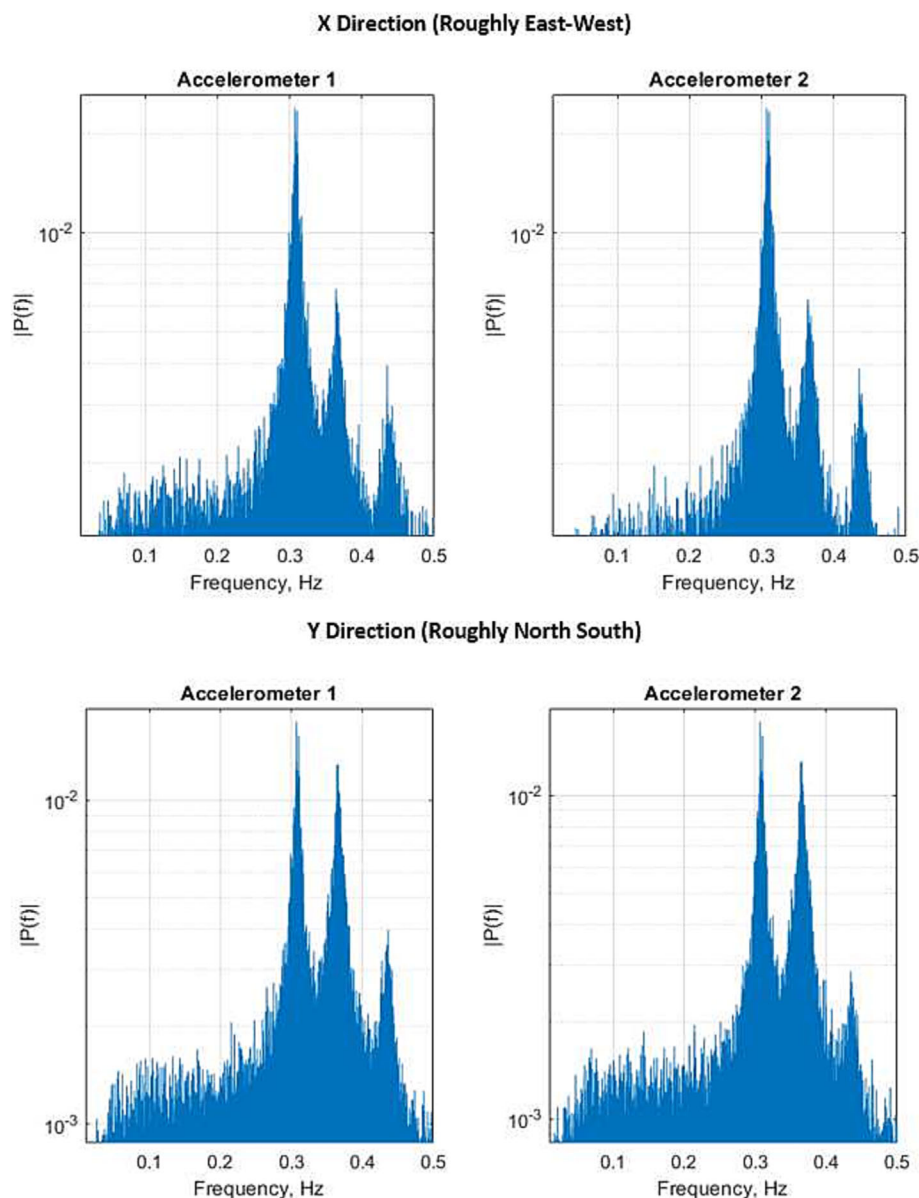


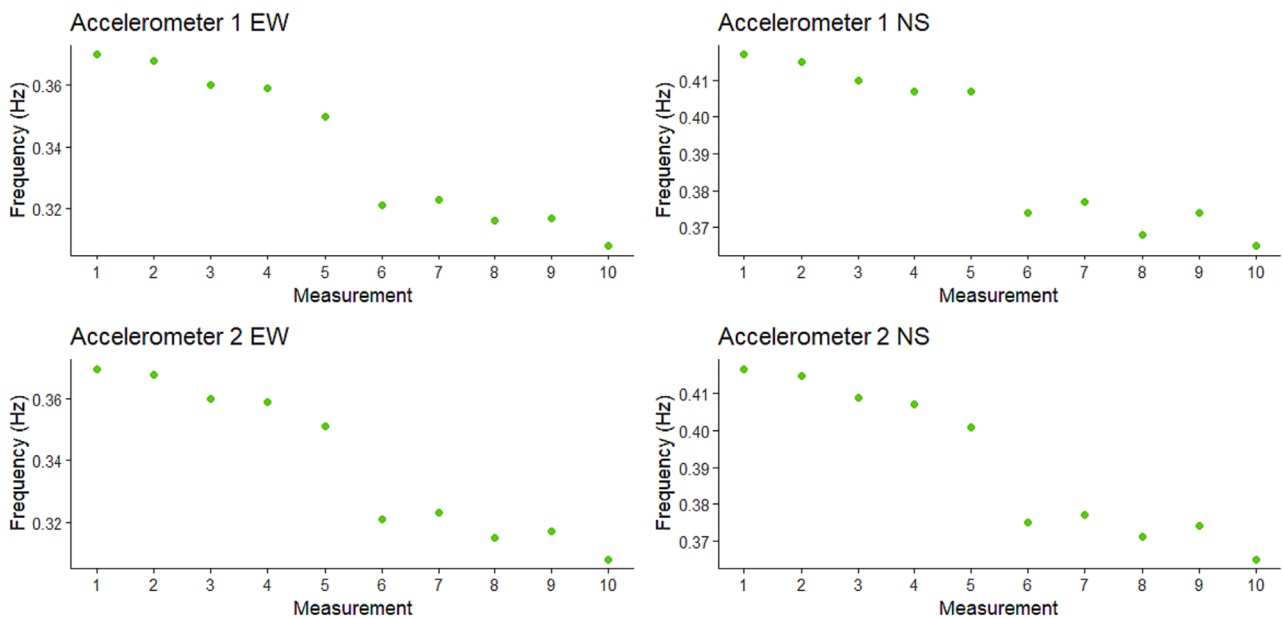
FIGURE 9 Fast Fourier transform of Measurement 10.

It can be seen from Table 7 that the damping ratio values given by the $BFFT_{gen}$, RDT, and AMD-RDT methods all agree quite well. The values obtained from the $BFFT_{asm}$, which as discussed earlier assumes a low signal-to-noise ratio, seem unrealistically high, and it appears reasonable to conclude that results from this calculation approach are unreliable. Therefore, from these results it can be concluded that the true damping ratio of the measured data in the EW direction likely lies between 1% and 1.1% and in the NS direction between 1% and 1.3%.

There are a number of points worth making about these values. Firstly, to the best of the authors' knowledge, this is the first time damping values for tall modular structures have been reported in literature. Secondly, it is interesting to note that the value of damping ratio obtained is very similar to the value of 1.27% recommended in Eurocode 1991-1-4 for mixed steel and concrete structures. The estimated damping ratios also lie within the range of values recommended by ASCE 7-10 of 1% for steel and 1.5% for concrete structures and only slightly higher than the range of 1% to 1.2% recommended by ISO 4354:2009 in the NS direction. This contradicts the hypothesis that damping ratios may be higher for modular structures due to the greater potential for frictional energy dissipation in joints. At least for the levels of excitation examined during this monitoring campaign, there appears to be no evidence of damping being higher than in other forms of construction. While damping values are highly specific to individual structures, the results in Table 7 obtained here appear to suggest that it is reasonable to apply the values proposed by codes for mixed buildings to modular structures. However, results from further monitoring of tall modular structures, particularly under higher levels of excitation, are needed to fully validate this conclusion.

TABLE 6 Modal identification natural frequency results.

Measurement	Accelerometer 1		Accelerometer 2	
	Natural frequency EW (Hz)	Natural frequency NS (Hz)	Natural frequency EW (Hz)	Natural frequency NS (Hz)
1	0.370	0.417	0.370	0.417
2	0.368	0.415	0.368	0.415
3	0.360	0.410	0.360	0.410
4	0.359	0.407	0.359	0.407
5	0.350	0.406	0.351	0.401
6	0.321	0.375	0.321	0.375
7	0.323	0.377	0.323	0.377
8	0.316	0.368	0.315	0.371
9	0.317	0.374	0.317	0.374
10	0.308	0.365	0.307	0.365
Average	0.337	0.390	0.337	0.390
Average (Measurements 7–10)	0.316	0.371	0.316	0.372

**FIGURE 10** Natural frequencies for each measurement.

Additionally, it is also relevant to note that the estimated damping ratio, irrespective of the calculation approach, appears to decrease over the course of the monitoring campaign. In other words, as extra modules, and therefore extra inter-module movement joints, were added, the damping ratio decreased. Again, this appears to contradict the hypothesis that the additional joints in modular construction may provide additional opportunities for frictional energy dissipation. The RMS accelerations for later events were lower than for some of the earlier events, which could be a key factor in the lower damping ratios estimated for the later events.

Figures 14 and 15 show the relationship between RMS acceleration and the natural frequencies and damping ratios estimated from the measurements. There does not appear to be a strong trend linking natural frequency to acceleration. However, there is a weak trend that appears to show damping ratio increasing as RMS acceleration increases. Generally speaking, higher damping ratios were observed when RMS accelerations were higher. At lower RMS accelerations damping does not drop below 1% and at higher RMS accelerations the range of observed damping values increases with values above 2% determined in some events, depending on the processing method employed. Previous studies on other types of buildings have shown similar results; that damping tends to increase with acceleration response (e.g., Smith et al.²⁶ and Fu et al.⁵²). While this same relationship can be observed from the measurement data recorded during this study, it does not appear to be an exceptionally strong.

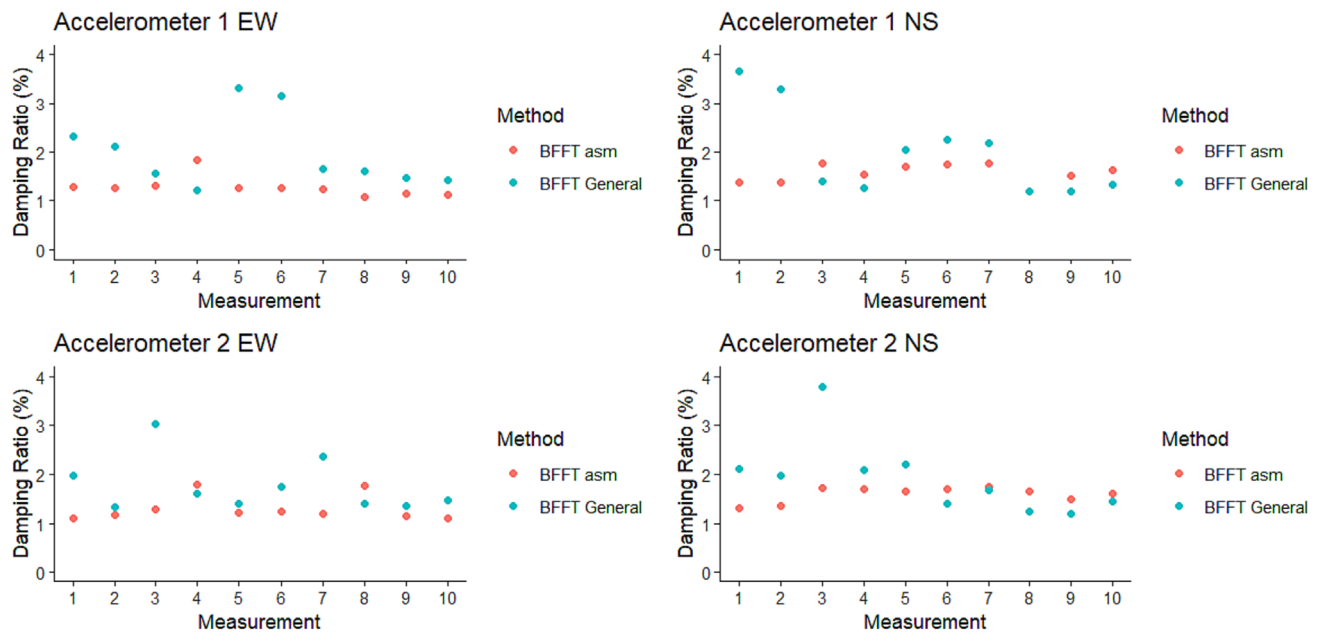


FIGURE 11 Calculated damping ratio for each measurement using BFFT methods.

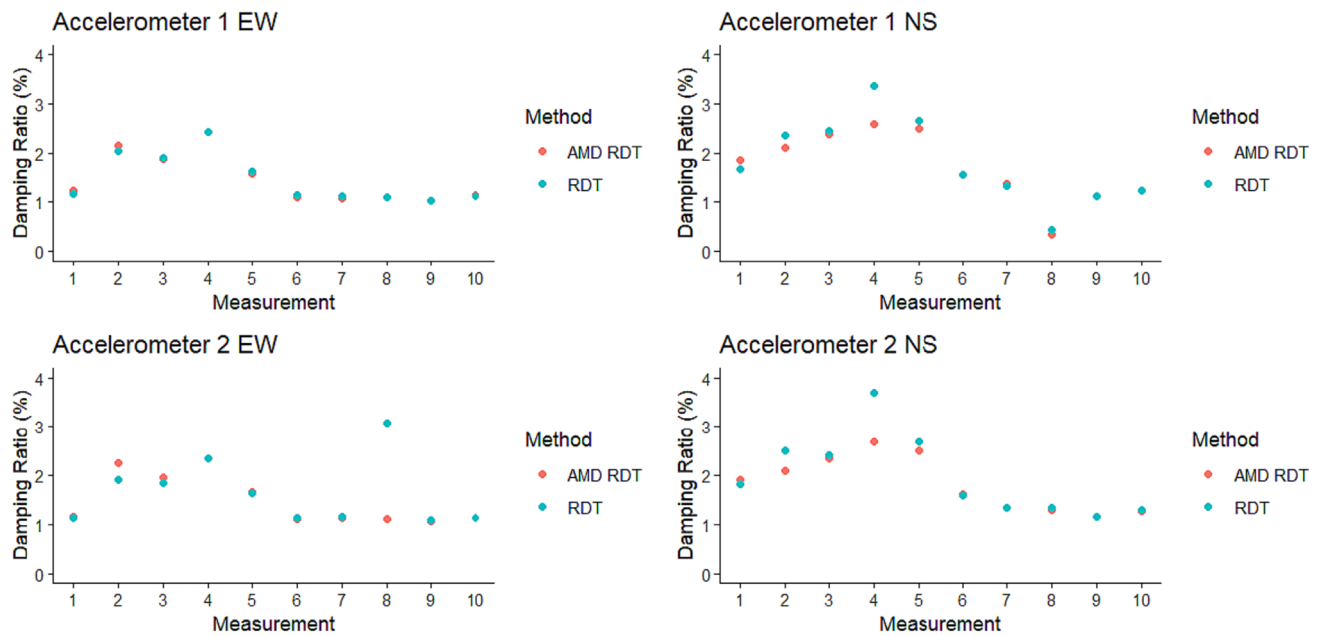


FIGURE 12 Calculated damping ratio for each measurement using RDT methods.

However, it is worth noting that the range of acceleration amplitudes observed in this monitoring campaign is not very large. The wind speeds reached when measurements were taken were low, far from inciting a structural response that would give rise to habitability issues. As the data recorded are ambient, there was no control over how significant the wind speed was, and in reality, the measured wind speed was far less than the 1 in 50-year wind speed in which the structure is designed for. A measurement of acceleration response from a period of high wind loading may offer more insight into the variation in acceleration with in natural frequency or damping ratio.

There does not appear to be a strong relationship between the natural frequencies and damping ratios estimated from the measured data. Figure 16 shows how the estimates vary with one another. It can be seen that the highest damping levels occur at higher estimates of natural frequency. However, this also reflects higher RMS accelerations during early events.

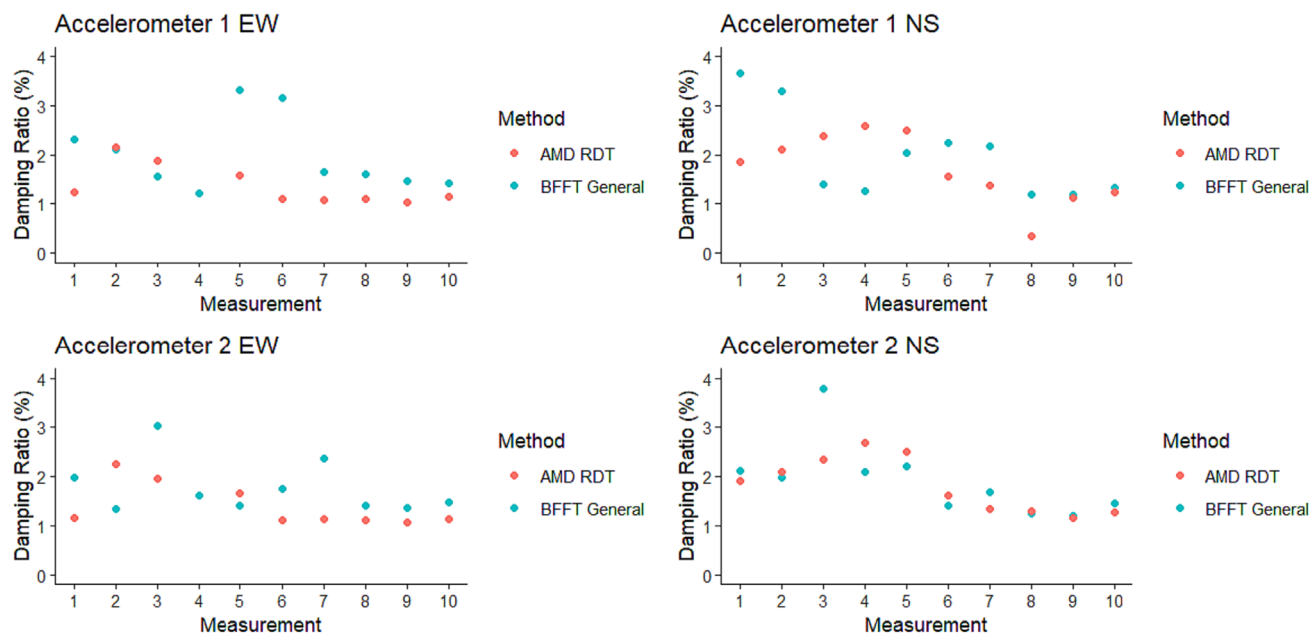


FIGURE 13 Comparison of calculated damping ratios from AMD-RDT and BFFT methods.

TABLE 7 Modal identification damping ratio average for Measurements 7–10.

Method	Accelerometer 1		Accelerometer 2	
	EW	NS	EW	NS
BFFT asm	1.38 %	12.64%	2.17%	4.00%
BFFT General	0.95%	1.25%	0.96%	1.25%
RDT	1.09%	1.03%	1.61 %	1.28%
AMD-RDT	1.09%	1.02 %	1.10 %	1.26%

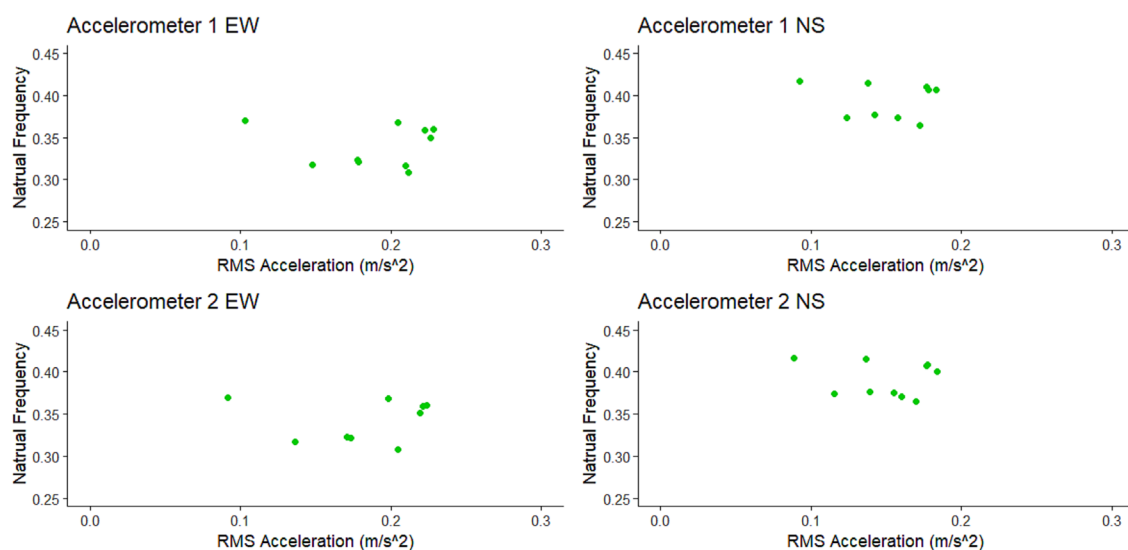


FIGURE 14 Estimated natural frequency versus RMS acceleration.

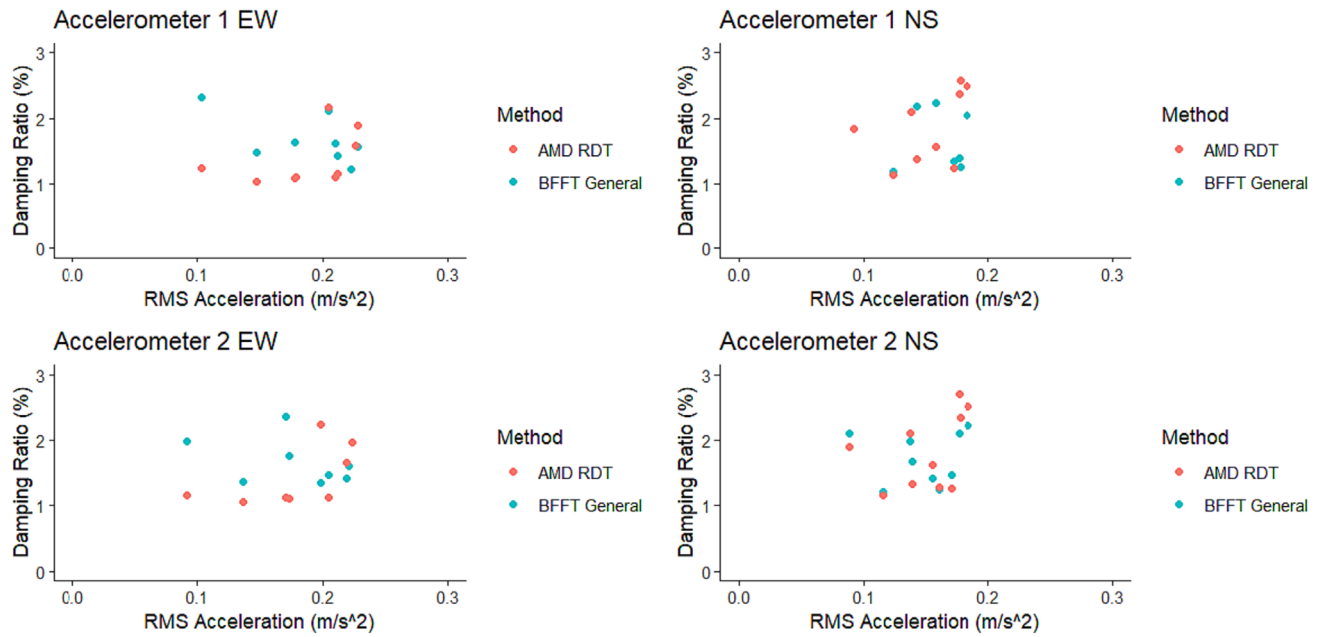


FIGURE 15 Estimated damping ratio versus RMS acceleration.

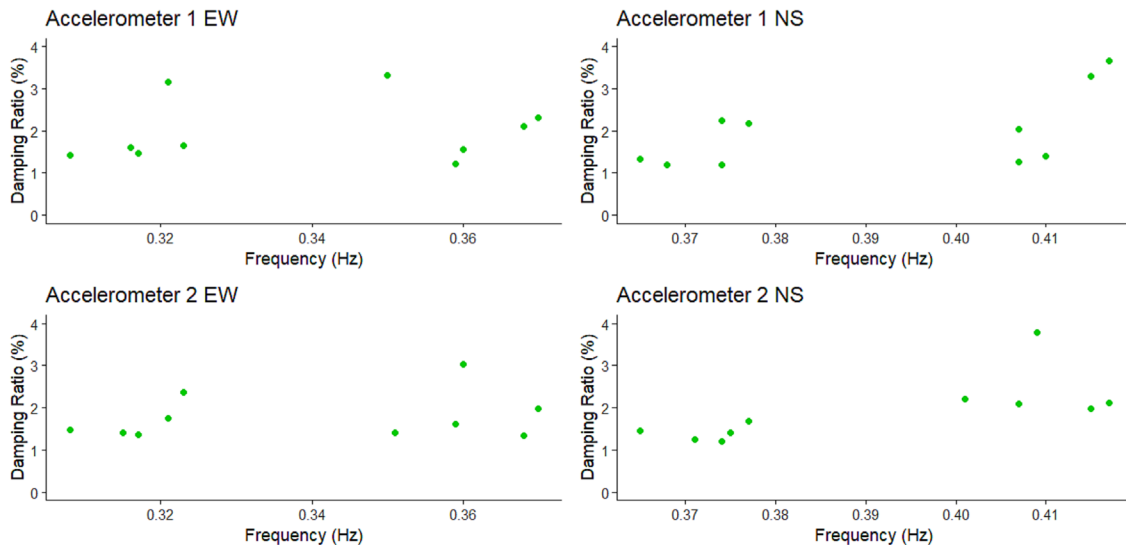


FIGURE 16 Estimated damping ratio versus estimated natural frequency.

5 | CONCLUSION

The application of modular construction to buildings of increasing height requires specific information on the vibration properties of this relatively new structural form. This paper has addressed the need for data on the in situ damping ratios of tall modular buildings subjected to wind loading. Four modal identification techniques have been applied to ambient acceleration data measured in a vibration monitoring campaign on a 135-m-tall structure comprising a RC core connected to 44 stories of steel-framed volumetric modules. The four modal identification techniques considered were two implementations of the BFFT and two implementations of the RDT. The first two natural frequencies of the structure were identified as 0.316 and 0.371 Hz, and the ability of the different modal identification techniques to provide consistent estimates of modal damping ratios for such closely spaced modal frequencies was assessed. The robustness of the modal damping ratio estimates was enhanced by analyzing acceleration data obtained at different stages of construction when varying relative stiffness contributions were provided by the core and modular elements of the structure.

For the completed structure, the modal damping ratios were estimated as 1.0% to 1.1% in the EW direction and 1.0% to 1.3% in the NS direction. Importantly, these values are close to the value of 1.27% recommended for use in mixed steel and concrete structures by Eurocode 1991-1-4. They are also close to the values set out by ASCE 7-10 and ISO 4354:2009. This is essential information for the optimal design and vibration control of future tall modular buildings to meet habitability requirements. This study found that despite the potential presence of many more friction movement joints, there is no evidence that damping ratios are higher in modular buildings than in their more conventional counterparts. The maximum response accelerations observed during the monitored wind events were generally in the range 3–4 m/s², somewhat below the typical habitability limit value range of 5–7 m/s². As the modal property results indicate that the observed damping increased with response amplitude, slightly higher values of damping ratio may apply at the higher speeds required to reach the design habitability threshold.

The two BFFT methods investigated gave significantly different estimates of damping ratio, with the more computationally efficient “asymptotic” approach giving inconsistent and sometimes unrealistically high values. In contrast, the estimates obtained using two RDT methods were in close agreement, with the AMD-RDT method giving more consistent results across all 10 measurement events. These results emphasize the need for careful selection of modal identification techniques for application to real world monitoring data. The damping ratios reported here represent the first values obtained from ambient vibration monitoring of tall modular structures during wind loading events. They provide greater confidence in the modal properties used in dynamic response assessments of future modular structures, supporting further design innovation. However, further full-scale monitoring of a wider range of modular structures and loading conditions is required to fully characterize their dynamic behavior.

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CONFLICT OF INTEREST STATEMENT

The authors declare that they have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

DATA AVAILABILITY STATEMENT

The data that support the findings of this study are available from Barrett Mahony Consulting Engineers. Restrictions apply to the availability of these data, which were used under license for this study. Data are available from the author(s) with the permission of Barrett Mahony Consulting Engineers.

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