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Validation of Acceleration Response Modelling for Modular High Rise Structures through Full Scale Monitoring

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Abstract. For many tall building forms, habitability requirements associated with excessive acceleration response become a governing design criterion as building heights increase. The application of modular construction methods to high-rise construction is a relatively new concept with limited previous research being conducted on the dynamic properties of tall modular buildings. Further to this, the real contribution of individual modular elements to overall lateral stiffness is largely unknown leading to significant uncertainty in acceleration response predictions. As modular construction continues to be employed in structures of ever-increasing height, the susceptibility of this form of construction to wind induced accelerations requires further investigation. This research considers the comparison and validation of computational models of a tall volumetric corner post modular structure with an RC core. Both Finite Element Models (FEMs) and mathematically-equivalent mechanical models adapting an analytical stepped beam approach are developed and the inherent properties such as the natural frequencies and mode shapes are calculated. The inherent properties predicted by the models are compared to those obtained from the actual measured response as captured through a full-scale monitoring campaign.

A full-scale monitoring campaign employing two triaxial accelerometers, a data acquisition system and a data storage system recorded the white noise ambient acceleration response of two tall, slender modular structures with overall heights of 135m and 150m. Wind speed and direction were also recorded throughout the monitoring campaigns. Structural identification techniques were used to process the measured acceleration responses and obtain estimates of the actual natural frequencies and damping ratios of the partially- and fully complete structures. The acceleration response of the structure was captured at varying stages throughout the construction programme as more storeys of modules were added to the building and the contribution of the modules to the modal properties evolved.

The comparison between the measured inherent properties at the different stages of construction and the model results at the equivalent stage provides vital insight into the overall stiffness contribution of modules in high-rise modular structures. This can lead to more efficient modelling and design procedures for a novel form of building. Furthermore, comparison of the modelled properties and the results from the full-scale monitoring campaign helps to provide a better understanding of model accuracy and identifies opportunities for further refinement of the modelling of tall modular buildings to reduce model size, run time and computational expense, without loss of accuracy in wind-induced response prediction. The validation of the model and identification of stiffness contributions of the modules supports structural optimisation analyses and the numerical investigations required to include vibration response mitigation measures in future designs

1. Introduction

In recent years modular buildings have experienced increased interest due to their reduced environmental impact, improved quality and accuracy, and speed of construction [1]. 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 results in a significant saving in on site construction time, less labour being required, and more accuracy, less injuries and less waste in the construction process [2]. Whilst 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 [1, 2]. Modular construction continues to increase in height due to economic drivers, with building heights of over 130m now realised [3]. However, as with other structural forms, habitability requirements associated with excessive acceleration response become the governing design criterion as building heights increase. Hence, it is crucial for the further development of modular construction that modelling techniques used to identify inherent properties are validated and that the limits of this form of construction are better understood and characterised.

As the construction of modular buildings to new heights continues, issues with applying traditional modelling techniques to these structures are becoming apparent. It has been found that current modelling techniques do not cater for the variety of elements and connections seen in modular buildings and they do not capture the dynamic behaviour of modular buildings sufficiently [4, 5]. There is a significant lack of standards for modular buildings; both in the case of design and in applications of BIM [6, 7]. Design of modular buildings is currently completed using traditional, non-specific design codes with lateral stability elements such as concrete cores and modules designed independently rather than as a cohesive structure. Additionally, there are no standards for the modelling of modular buildings resulting in traditional BIM techniques being applied [2]. Using practices which were not created for this type of construction leads to inefficiency in design and modelling processes, impeding the progression of this form of construction [2].

Furthermore, the stiffness contribution of modules to the overall lateral stability of the structure is unknown [8]. At present, tall modular structures are designed considering only the lateral stability of the concrete core and ignoring any stiffness modules may add. This is a result of the stiffness contribution of modules not yet being quantified. In reality, the modules must add some stiffness and it is imperative to the future design of modular structures, and assessment of habitability requirements, that this value is identified.

This paper outlines research conducted by the authors to quantify the stiffness contributions of the modules in the world's tallest completed modular structure located in Croydon, London, United Kingdom. Attempts at quantifying this value include extensive finite element models and in situ acceleration monitoring schemes in order to identify the natural frequency of the structure and better understand the dynamic response. A comparison of the natural frequencies identified by each endeavour are provided. A mathematically equivalent mechanical model has been developed in order to efficiently identify the stiffness contribution of the modules in this case study structure, hence providing the first reporting of the stiffness of a volumetric modular building.

2. Description of Case Study Structure

The full-scale structure considered in this paper is a 44 storey, 135m tall modular building. The structure consists of a slip formed concrete core, transfer slab at level 4 and two adjoined towers of 37 and 44 stories that consist of volumetric corner post modules stacked around and connected to two concrete cores. The concrete cores are approximately 8 x 8 m in plan and have walls which vary in thickness between 300mm to 450mm. The landing slabs within the core are 300mm thick. For design purposes, the concrete core is assumed to act as the primary element for lateral load resistance.

The modules are typically 2.875m tall, are limited in length and width to 13m and 6m respectively due to transportation, and have a typical self-weight of 7 kN/m. Each module has its own set of structural members including beams, columns, bracing and a 150mm thick concrete slab.

The structure has an overall slenderness ratio (Height/Breadth) of 8. The contribution of the modules to the overall stiffness of the structure, and hence, the lateral load resistance of the modules is so far unknown and is a topic of current research by the authors.

Figure 1. Partially completed case study building showing tied cores (left) and completed case study building (right)

3. Finite Element Models of Case Study Structure

During the design process finite element models created in ETABS 2020 and Axis VM Software were relied upon to estimate the dynamic behaviour and modal properties of the structure. Both the free-standing cores and the full structure were modelled using traditional elements to represent the modules and bespoke connections unique to modular construction. The representation of the member stiffness of the steel elements was straightforward, as the steel elements were assumed to remain elastic at service loading. The concrete in the concrete cores and transfer slab was assumed to have cracked under service loading and so the Young's Modulus of the concrete was modified to represent that of cracked concrete. The individual slabs of the modules were assumed to act as one rigid diaphragm. Both the mass associated with the self-weight of the structure and the mass of the cladding were included in the modal analysis. The model of the free-standing cores included ties between the cores which were installed to ensure stability. These ties remained in place as the first number of floors of modules were installed and can be seen in Figure 1. Eigenanalysis was performed on both the finite element models of the tied cores and the completed structure in order to identify the modal properties, namely the natural frequency and mode shape in both the x and y direction The results are shown in Table 1.

Table 1. Results of modal analysis performed on finite element models of on free-standing cores and the completed structure.

Stories Installed Tower A	Stories Installed Tower B	Identified Natural Frequency x Direction (EW) (Hz)	Identified Natural Frequency y Direction (NS) (Hz)
0	0	0.31	0.33
44	37	0.248	0.253

The finite element model of the completed structure was found to be very computationally expensive. A significant issue with using current modelling practices for this novel form of construction arises from the nature of volumetric modular buildings. A typical modular building, when all modules are installed, will have double or quadruple of each member that a traditional structure would have due to the 'stacking' nature of the modules. This is due to each module being a separate structural system with a full set of members of its own. There is also a significant increase in the number and complexity of the connections throughout a modular structure and replicating the connections with current finite element software can prove difficult.

Whilst using current techniques to model a modular building, in which every element is modelled individually, may be possible for smaller modular structures, modelling of each element in tall modular buildings with over 40 storeys becomes problematic. The number of elements needed to be modelled becomes considerably large and requires significant computational power and run time to analyse the model. For reference, A PC with 32 GB of RAM and an 8 core Intel i9 processor takes approximately 180 minutes to run eigen modal analysis on such a model, and up to ten hours to produce the tables of results.

The accuracy of modal analysis performed using this type of model is not noteworthy with regards to global dynamic behaviour especially when the significant time producing and running the model are considered. Whilst a global model of a modular structure with every element modelled individually is useful for checking capacities, stresses and the localised behaviour of members, it is worth exploring whether such an extensive model is necessary to represent the properties of a modular structure for the specific purpose of ensuring that habitability requirements are satisfied. This is important because the computational demands associated with the dynamic analysis of the response of buildings to wind loading can be much greater than those required for static analysis of gravity load response. On the other hand, the low amplitude vibrations associated with habitability requirements imply small displacement, elastic response that can be well represented by modal properties, suggesting considerable scope for computational efficiency.

4. Full Scale Monitoring of Case Study Structure

Two separate in-situ monitoring campaigns were performed on the case study structure in different states of completion. The first in-situ campaign took place between February and March 2019 on the tied free-standing concrete cores prior to the installation of any modules. Ambient in-situ acceleration response recordings were taken at levels 30 and 43 of Core A with a sampling frequency of 1Hz. This monitoring campaign was preliminary and yielded only useful acceleration responses of the tied cores.

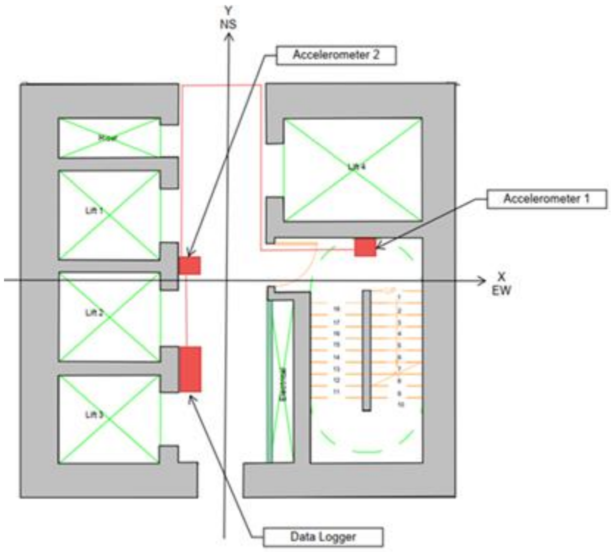

Figure 2. Location of accelerometers and data logger in the core

More detailed monitoring of the structure was undertaken over a two-month period, beginning in late August 2019 and ending in late October 2019 whilst the structure was still under construction. Two three-axis accelerometers and tilt sensors were directly mounted on a rigid support at approximately 1.84m height from the floor slab at level 43 as shown in Figure 2. One accelerometer was located at the centre of the concrete core and the other at the edge of the core to capture torsional modes. The sampling rate for the acceleration data was 5 Hz; given the height and slenderness of the structure and initial modal analysis, low natural frequencies below 0.5 Hz were expected for the first three modes of vibration. In total acceleration time history data were recorded for ten independent periods of 12 hours

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in which the structure's ambient acceleration due to wind excitation exceeded a threshold value. Of the ten sets of measurement data obtained, the final three were measured when the structure was fully complete. On the roof of one of the RC cores, a weather station was installed to record 10-minute average values for wind speed and direction and maximum/minimum wind speed and direction values within each 10-minute window, along with measurements of temperature, humidity, atmospheric pressure and rain. A 3G router was also installed to allow for remote access to all data. Data from the weather station was continuously monitored.

4.1. Modal Identification of Case Study Structure

Modal identification was performed on the ambient in-situ acceleration responses captured by both monitoring campaigns of the world's tallest modular building. The acceleration responses were analysed in order to identify the structures inherent properties such as the natural frequency. For the first preliminary campaign, nine recordings of ambient excitation of the tied concrete cores of three minute length were used for modal analysis. Results for this are shown in Table 2 and it can be seen that these values are in good agreement with those shown in Table 1 for the finite element model.

Table 2. Identified natural frequencies of the tied concrete cores from the first field monitoring campaign

Stories Installed Tower A	Stories Installed Tower B	Identified Natural Frequency x Direction (EW) (Hz)	Identified Natural Frequency y Direction (NS) (Hz)
0	0	0.31	0.33

From the second, more extensive campaign, acceleration time history data was recorded for a total of ten periods of 12 hours in which the structure's acceleration due to ambient white noise excitation from the wind exceeded a threshold value. Two methods were used: the Bayesian Fast Fourier Transform (BFFT) and a composite method of Analytical Mode Decomposition and the Random Decrement Technique (AMD-RDT). The BFFT was applied as described by Au, 2011 [9]. The AMD-RDT was applied as described by Wang et al., 2014 [10]. Hickey et al. provides further description of the application of these methods [8].

As mentioned in Section 4, the structure was still under construction with floors of modules being added as measurements were taken. Hence, the modal properties of the structure were identified using both methods at varying degrees of completion which are shown in Figure 3.

Table 3 shows the date each acceleration measurement was taken, the number of stories of modules installed in each of the two towers and the identified natural frequencies of the structure in the x (North-South) and y (East West) directions. The structure was fully complete for measurements 7-10. It can be seen that the natural frequency of the structure generally decreased as more stories of modules were added. This is due to the increase in mass at the top of the structure resulting from the additional modules.

Considering Table 1 and Table 3, it can be seen that the estimates of the natural frequency from performing modal analysis on the finite element model of the struture are far lower that those obtained from applying modal identification methods to the in-situ ambient acceleration response of the completed structure. However, comparitively, the finite element models provide accurate estimates of the natural frequency of the tied cores. For the completed structure in the x-direction the understimate is >20% and it is >30% in the y-direction, suggesting that the lateral stiffness contributions of the modules is underestimated in the finite model. This is likely due to the way in which connections between the modules and the core are modelled; there is no provision for the bespoke type of connections used in modular construction in traditional finite element softwares and these connections had to be simplified and represented using euler-bernoulli beam elements.

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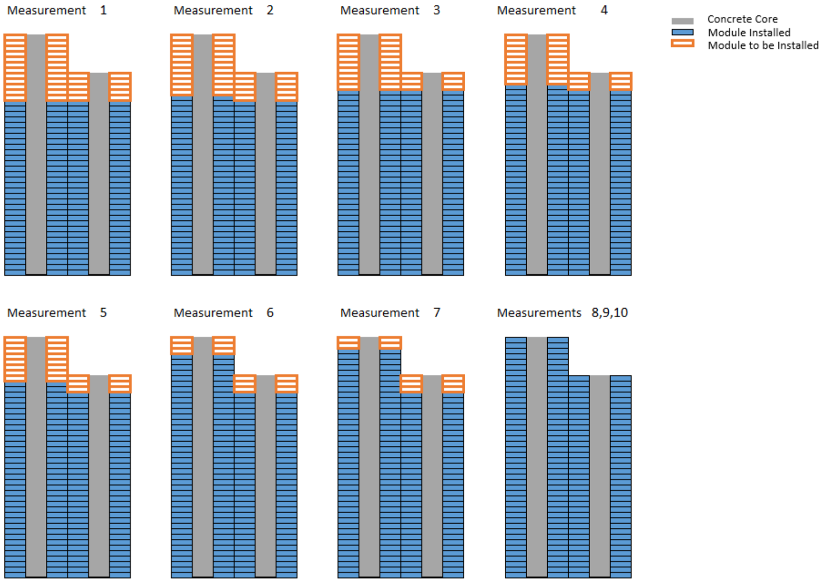

Figure 3. Graphical representation of the number of levels of modules installed for each set of measurement data

Table 3. Date, number of stories installed and identified natural frequency for each acceleration measurement.

Measurement	Date	Stories Installed Tower A	Stories Installed Tower B	Identified Natural Frequency x Direction (EW) (Hz)	Identified Natural Frequency y Direction (NS) (Hz)
1	31/08/2019	32	32	0.370	0.417
2	02/09/2019	33	32	0.368	0.415
3	06/09/2019	34	34	0.36	0.410
4	09/09/2019	35	34	0.359	0.407
5	11/09/2019	36	34	0.350	0.406
6	27/09/2019	41	34	0.321	0.375
7	29/09/2019	42	34	0.323	0.377
8	11/10/2019	44	37	0.316	0.368
9	13/10/2019	44	37	0.317	0.374
10	26/10/2019	44	37	0.308	0.365
Average (Measurements 7-10)				0.316	0.371

Finite element models are built during the design stage of a structure and used to inform decisions on member sizes, structural behaviour and mitigation measures. The underestimate of natural frequency of modular buildings, and hence dynamic behaviour, of a structure by modal analysis of finite element models to this extent could lead to issues being raised in relation to habitability during the design stage of a structure which in reality may not be problematic. There is an evident necessity for more efficient models of modular structures which accurately capture the stiffness contribution of modules in order to better inform design decisions. This paper sets out a numerical stepped-beam model which has been developed and verified using the in-situ monitoring data of the case study structure. The numerical stepped-beam model accurately captures the stiffness contribution of modules and the overall stiffness of a modular building.

5. Stepped Beam Numerical Model

An alternative numerical model was developed to provide a less computationally expensive model than the finite element models and one which suits the nature of modular construction better, more accurately representing the stiffness contribution of modules to the finished structure.

The numerical model is based on the free vibration analysis of a stepped beam using Adomian Decomposition as proposed by Mao et al., 2010 [11]. This stepped beam model lends itself well to the in-situ acceleration measurements obtained at different levels of completion. The model is verified using the frequencies estimated from the in-situ acceleration measurements. It is then used to replicate the in-situ structure at the varying levels of completion and compare the estimated natural frequencies.

The problem is set up by considering the ratio of mass per unit length (m) and stiffness (EI) of the sections of the building above and below the module installation level:

$$E_{cm}I_{cm} = \alpha E_c I_c \quad (1)$$

$$m_{cm} = \beta m_c \quad (2)$$

Where the subscripts c and cm refer to the core and core-plus-modules (i.e. completed structure) respectively.

The free vibration of an Euler-Bernoulli beam consisting of two uniform sections and elastically restrained at both ends as shown in Figure is considered. The stepped beam consists of two sections each with their own system of reference, x_1 and x_2. The partial differential equation which describes the free vibration of each section is shown in Equations 3 and 4.

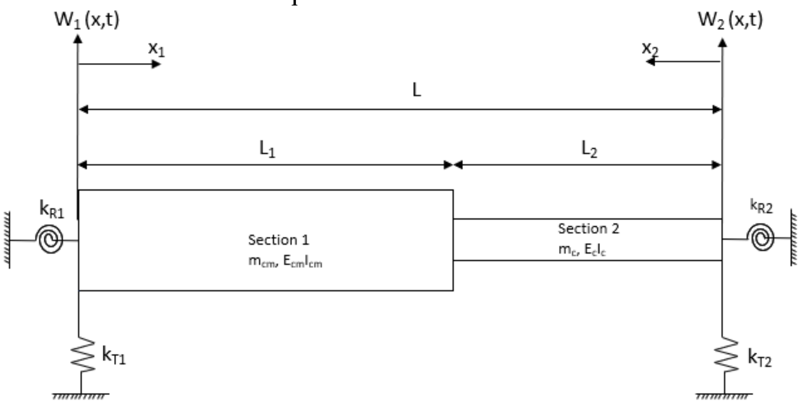

Figure 4. Coordinate system for the Euler Bernoulli stepped beam elastically restrained at both ends

$$\frac{\partial^4 \omega_1(x_j,t)}{\partial x_j{}^4} + \frac{\beta}{\alpha}\,\frac{m_c}{E_c I_c}\,\frac{\partial^2 \omega_1(x_j,t)}{\partial t^2} = 0,\ x_j \in \left[0\ L_j\right] \quad (3)$$

$$\frac{\partial^4 \omega_2(x_j,t)}{\partial x_j{}^4} + \frac{m_c}{E_c I_c}\,\frac{\partial^2 \omega_2(x_j,t)}{\partial t^2} = 0,\ x_j \in \left[L_j\ H\right] \quad (4)$$

Where ω_1 and ω_2 are the natural frequencies of the respective sections of the stepped beam and t is time. Considering a modal analysis approach for harmonic free vibration and Adomian decomposition, the dimensionless natural frequency, Ω_1, of the stepped beam defined in Equation 2, can be found by obtaining the determinant of the matrix given in Equation 6.

$$\Omega_1{}^4 = \frac{m_1\,\omega^2\,L^4}{E_1 I_1} \quad (5)$$

Where $m_1$is the mass of section 1 per unit length, ω is the natural frequency of the structure, L is the combined length of both sections and $E_1$and I_1 are the Young's Modulus and Second Moment of Area of section 1 respectively.

$$\det\begin{bmatrix} D_1(R_1) & D_2(R_1) & -D_3(R_2) & -D_4(R_2) \\ \frac{dD_1(R_1)}{dR_1} & \frac{dD_2(R_1)}{dR_1} & \frac{dD_3(R_2)}{dR_2} & \frac{dD_4(R_2)}{dR_2} \\ \frac{d^2D_1(R_1)}{dR_1{}^2} & \frac{d^2D_2(R_1)}{dR_2{}^2} & -\frac{1}{\alpha}\frac{d^2D_3(R_2)}{dR_2{}^2} & -\frac{1}{\alpha}\frac{d^2D_4(R_2)}{dR_2{}^2} \\ \frac{d^3D_1(R_1)}{dR_1{}^3} & \frac{d^3D_2(R_1)}{dR_2{}^3} & \frac{1}{\alpha}\frac{d^3D_3(R_2)}{dR_2{}^3} & \frac{1}{\alpha}\frac{d^3D_4(R_2)}{dR_2{}^3} \end{bmatrix} = 0 \tag{6}$$

Where

$$D_1(R_1) = \sum_{m=0}^{M-1}\left[\Omega_1{}^{4m}\ \frac{R_1{}^{4m}}{(4m)!}\right] - K_{T,1}\sum_{m=0}^{M-1}\left[\Omega_1{}^{4m}\ \frac{R_1{}^{4m+3}}{(4m+3)!}\right], \tag{7}$$

$$D_2(R_1) = \sum_{m=0}^{M-1}\left[\Omega_1{}^{4m}\ \frac{R_1{}^{4m+1}}{(4m+1)!}\right] - K_{R,1}\sum_{m=0}^{M-1}\left[\Omega_1{}^{4m}\ \frac{R_1{}^{4m+2}}{(4m+2)!}\right], \tag{8}$$

$$D_3(R_2) = \sum_{m=0}^{M-1}\left[(\mu\Omega_1{}^4)^m\ \frac{R_2{}^{4m}}{(4m)!}\right] - K_{T,2}\sum_{m=0}^{M-1}\left[(\mu\Omega_1{}^4)^m\ \frac{R_2{}^{4m+3}}{(4m+3)!}\right], \tag{9}$$

$$D_4(R_2) = \sum_{m=0}^{M-1}\left[(\mu\Omega_1{}^4)^m\ \frac{R_1{}^{4m+1}}{(4m+1)!}\right] - K_{R,2}\sum_{m=0}^{M-1}\left[(\mu\Omega_1{}^4)^m\ \frac{R_2{}^{4m+2}}{(4m+2)!}\right], \tag{10}$$

Where M is determined by the convergence requirement in practice and is in this case set as 10. $R_1 = \frac{L_1}{L}$ and $R_2 = \frac{L_2}{L}$. $K_{R,j}$, $K_{T,j}$ and μ are defined in Equations 11, 12 and 13 respectively.

$$K_{R,j} = \frac{k_{R,j}\,L^3}{E_jI_j} \tag{11}$$

$$K_{T,j} = \frac{k_{T,j}\,L^3}{E_jI_j} \tag{12}$$

$$\mu = \frac{m_c}{m_{cm}}\frac{E_{cm}I_{cm}}{E_cI_c} = \frac{\alpha}{\beta} \tag{13}$$

For the purposes of this paper, the modular structure is considered a cantilever stepped beam as shown in Figure 5, with fixed-free boundary conditions such that $k_{R,1}$ and $k_{T,1}$ are sufficiently large ($1\text{x}10^9$) and $k_{R,2}$ and $k_{T,2}$ are 0.

Thus Eqns 7-10 simplify to Eqns 16-19:

$$K_{R,1} = K_{T,1} = 1\times10^9\ \frac{L^3}{\alpha EI} \tag{14}$$

$$K_{R,2} = K_{T,2} = 0 \tag{15}$$

$$D_1(R_1) = \sum_{m=0}^{M-1}\left[\Omega_1{}^{4m}\ \frac{R_1{}^{4m}}{(4m)!}\right] - 1\times10^9\ \frac{L^3}{\alpha EI}\sum_{m=0}^{M-1}\left[\Omega_1{}^{4m}\ \frac{R_1{}^{4m+3}}{(4m+3)!}\right], \tag{16}$$

$$D_2(R_1) = \sum_{m=0}^{M-1}\left[\Omega_1{}^{4m}\ \frac{R_1{}^{4m+1}}{(4m+1)!}\right] - 1\times10^9\ \frac{L^3}{\alpha EI}\sum_{m=0}^{M-1}\left[\Omega_1{}^{4m}\ \frac{R_1{}^{4m+2}}{(4m+2)!}\right], \tag{17}$$

$$D_3(R_2) = \sum_{m=0}^{M-1}\left[(\frac{\alpha}{\beta}\Omega_1{}^4)^m\ \frac{R_2{}^{4m}}{(4m)!}\right], \tag{18}$$

$$D_4(R_2) = \sum_{m=0}^{M-1}\left[(\frac{\alpha}{\beta}\Omega_1{}^4)^m\ \frac{R_1{}^{4m+1}}{(4m+1)!}\right], \tag{19}$$

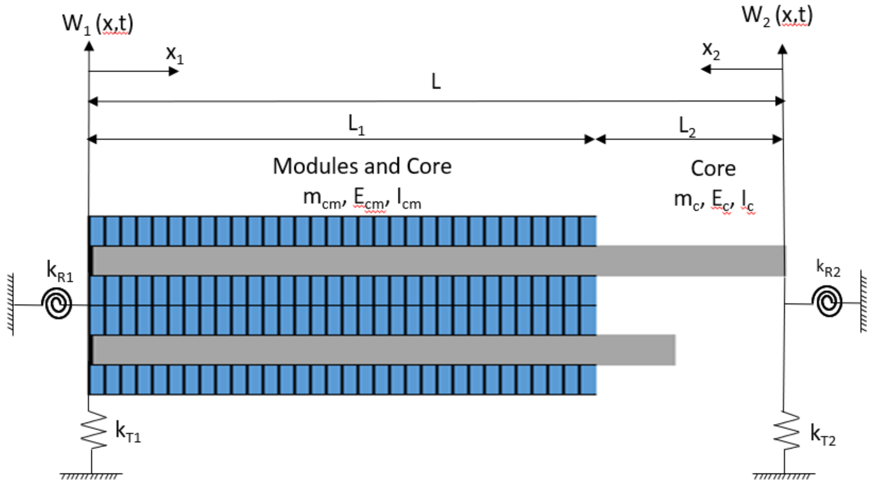

Figure 5. Partially completed modular structure represented as a stepped beam

The stepped beam numerical model is first run for the completed modular structure with $L = L_1 = 135$m and the corresponding natural frequencies of 0.316 Hz in the x-direction and 0.371 Hz in the y-direction in order to obtain an equation for $E_{cm}I_{cm}$. The stepped beam model was then run using the partially completed structure with 32 stories of modules installed, with $L = 135$m, $L_1 = 100.5$m and $L_2 = 34.50$m, and the corresponding natural frequencies of 0.37 Hz in the x and 0.417 Hz in order to obtain an equation for $E_{cm}I_{cm}$ and E_cI_c. These two equations were then solved simultaneously for both x and y directions to identify values for $E_{cm}I_{cm}$ and E_cI_c which could be used in the numerical stepped beam model for predicting the natural frequency of a modular structure. The results are shown in Table 4.

Table 4. Identified EI values found using the numerical model and natural frequencies from modal identification

Direction	$E_{cm}I_{cm}$	E_cI_c	α
x	4.06E+13	1.40211E+13	3.4
y	2.13E+13	7.36165E+12	2.9

6. Frequency Prediction

The model was used alongside the stiffness values shown in Table 4 to predict the natural frequency of the structure with each storey of modules installed. Figure shows a comparison of these predicted natural frequencies to those obtained from performing modal analysis on the finite element model and the values obtained from performing modal identification on the in-situ acceleration measurements. It can be seen that the in-situ measurements and the numerical stepped beam values are in close agreement, with the numerical stepped beam model providing a slight overestimate in some cases, particularly in the y-direction.

The natural frequency values from the finite element models are significantly lower than those from the in-situ measurements and the numerical stepped beam model as discussed in Section 4.1. The lower estimates of natural frequency and hence stiffness of the structure given by modal analysis of the finite element model show the difficulties and inaccuracy of applying traditional modelling methods to modular structures. The numerical stepped beam model provides a more efficient and accurate method for identifying the natural frequency of a modular structure than traditional methods. This will allow for better informed design and analysis of modular structures to ensure compliance with habitability requirements.

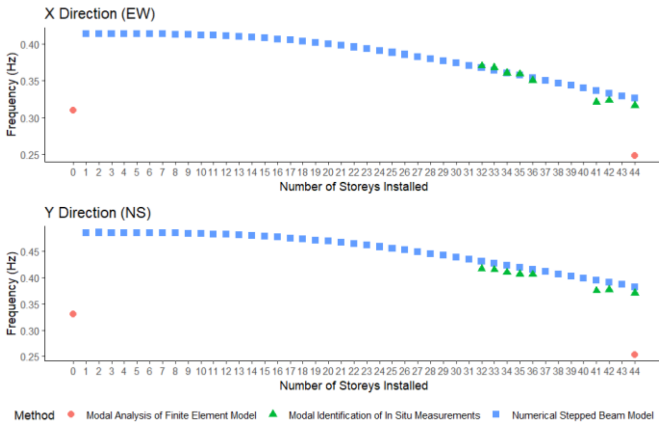

Figure 6. Comparison of predicted frequency using the different methods

7. Conclusion

Modular construction is a rapidly growing industry offering many advantages over traditional structural forms such as lower environmental impact, reduced construction times and increased efficiency of design. As modular construction moves away from more traditional applications in low and medium rise buildings to structures over 135m tall it is essential that the dynamic behaviour of this form of construction is better understood, in particular the lateral stiffness contribution of modules to the overall lateral resistance of a structure.

This paper has considered a 135m tall modular structure located in Croydon, London, United Kingdom. Modal analysis performed on finite element models of the structure found the models to be computationally expensive and inaccurate in predicting the natural frequency of the completed structure. In situ monitoring Campaigns provided ambient acceleration responses of the structure at varying degrees of completion. These responses have been analysed using modal identification methods to identify the actual natural frequency of the structure. Using these natural frequencies, a numerical stepped beam model was built which identified the stiffness contribution of the modules. It was found that the stiffness of the competed sections of the structure (cores and modules) is 4.05801E+13 in the x-direction and 2.13432E+13 in the y-direction. To the best of the authors knowledge, this is the first time that the stiffness contribution of modules to a completed structure has been reported.

The identified stiffness values can in turn be used in the numerical stepped beam model to predict the natural frequency of a modular structure for a range of parameters including height, breadth, width, mass and number of stories installed. This numerical model provides an efficient and fit for purpose method for identifying the natural frequency of a modular structure, enabling a more accurate dynamic analysis of modular structures to be performed during design stages in order to ensure habitability requirements are met.

References

[1] Hough M J and Lawson R M 2019 *P I CIVIL ENG-CIV EN.* **176** p 37-44
[2] Ferdous W, Bai Y, Ngo T D, Manalo A and Mendis P 2019 *Eng Struct.* **183** p. 883-93
[3] Lane T 2019 *The sky's the limit: See the world's tallest modular tower in Croydon.* ***Building.co.uk***
[4] Ramaji I J and Memari A M 2018 *Archit. Eng. Des*. **141** 2 p. 158-76
[5] Chua Y S, Liew J Y R and Pang S D 2020 *J. Constr. Steel Res*. **166** p. 105901
[6] Sun Y, Wang J, Wu J, Shi W, Ji D, Wang X and Zhao X 2020 *Appl.* **10** 20
[7] Wang Z and Tsavdaridis K D 2022 *Eng. Struct.* **251** A
[8] Hickey J, Moore H, Broderick B and Fitzgerald B 2023 *Struct. Des. Tall Build.* submitted
[9] Au S 2011 . *J. Eng. Mech.* **1373** p. 214-26.
[10] Wang Zand Chen G 2014 *Eng. Struct*. **59** p. 173-84
[11] Mao Q and Pietrzko S 2010 *Appl. Math. Comput*. **2177** p. 3429-41