Structures 37 (2022) 576–587

Contents lists available at ScienceDirect

Structures

journal homepage: www.elsevier.com/locate/structures

Mitigation of wind induced accelerations in tall modular buildings

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ARTICLE INFO

Keywords:
Modular construction
Wind induced vibration
Dynamic control methods
Tall buildings

ABSTRACT

This paper compares two methods of mitigating wind induced vibrations in tall modular buildings. Tall buildings experience wind induced motion which can result in serviceability and habitability issues associated with occupant discomfort. Modular construction, in which volumetric modules are assembled around a reinforced concrete core, can be particularly susceptible to wind induced accelerations due to the tall slender form of the core and the small and uncertain contribution of the modules to global lateral stiffness and damping. This study analyses the acceleration response of modular buildings. Broadly speaking, for this form of construction, two approaches exist to mitigate excessive vibrations; increasing core dimensions or the addition of auxiliary damping. This study evaluates these two approaches for a large number of archetype modular structures in order to investigate which method is more effective. Of more than 6000 archetypes studied, it was found that over 40% required vibration control measures to meet ISO acceleration limits. Installing a Tuned Liquid Damper (TLD) proved significantly more efficient than increasing RC core dimensions; a 1% increase in damping achieves a similar level of acceleration reduction to approximately a 2100 mm increase in core breadth and depth. The estimated level of auxiliary damping available from a TLD is sufficient to control accelerations for the majority of archetypes considered. A method for developing curves defining the maximum feasible height of a modular building based on its dimensions and the provision of an optimum TLD is also developed. The results show that modular buildings can be used as a viable form of construction for high-rise buildings and quantify the extent to which the maximum heights of modular tower buildings can be increased using existing vibration control technologies.

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 [1–4]. 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 labour is required and there is more accuracy, less injuries and less waste in the construction process [5]. 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 [1,5,6]. Modular construction continues to increase in height due to economic drivers, with building heights of over 130 m now realised [7,8]. 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 its further development that the inherent properties and limits of this form of construction are better understood and characterised.

All buildings are subject to environmental loads such as wind and experience a resulting dynamic response. This dynamic response can cause issues with the serviceability of the structure and give rise to habitability issues associated with motion sickness in occupants (for example [9–11]). Kwok et al. [12] provides a detailed summary of various studies that have attempted to quantify human perception of vibration and tolerance thresholds (for example [13–15]), and various

Abbreviations: RC, Reinforced Concrete; NBCC, National Building Code of Canada; ISO, International Standards Organisation; AIJ, Architectural Institute of Japan; TMD, Tuned Mass Damper; TLD, Tuned Liquid Damper.

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https://doi.org/10.1016/j.istruc.2022.01.037
Received 4 August 2021; Received in revised form 23 December 2021; Accepted 12 January 2022
Available online 19 January 2022

codes and standards offer guidance on acceptable levels of wind induced acceleration [16]. The first code to do so was the 1975 National Building Code of Canada (NBCC) [17], while The International Standards Organization (ISO) (1984) [18] and the Architectural Institute of Japan (AIJ) (1991) [19] also produced guidelines. ISO 10137 (2007) [20] is an updated version of the ISO 6897 (1984); the suggested acceleration limits are shown in Fig. 1. This standard imposes limits on the peak acceleration for a 1 in 1 year wind event for both residential and commercial buildings. These criteria are based on the accelerations which are disruptive to approximately 2% of the people occupying the upper third of the building.

Modular construction is often employed in residential construction, as the natural division between modules make it more suitable for deployment in buildings with repetitive divisions between rooms, rather than open-plan offices [3,5,6]. From a dynamic response viewpoint, this means that modular developments are often required to meet the more stringent criteria imposed for residential buildings, typically limiting response acceleration to less than 5 milli-g (50 mm/s^2).

The main source of lateral load resistance in tall modular construction is typically a reinforced concrete core that is connected directly or indirectly to all of the modules [1–3,5,8,21]. A typical modular scheme is illustrated in Fig. 2, where modules are stacked around and connected to the core, which provides lateral wind resistance. There are many different schemes and materials used for the individual modules, typically incorporating some form of lateral bracing within module walls to satisfy transportation, erection and stability requirements. However, the contribution of the modules to global lateral stiffness also depends on intra- and inter-module connection stiffness which is challenging to estimate at a preliminary design stage [4,6,22]. There is no one connection type which is used predominantly within construction of modular buildings. This suggests that a connection type is yet to be developed which sufficiently supplies continuity to the structure and meets structural needs along with manufacturing and construction requirements [5,23]. Furthermore, the potential increase in global lateral stiffness offered by greater in situ continuity between individual modules and between the modules and the core is difficult to achieve within existing modular construction practice [22,23]. There is insufficient conclusive research on optimal module connections and the modelling and estimating the behaviour and stiffness contribution of module connections [4–6,22,23]. Therefore in this study, it is conservatively assumed that the entire lateral load resisting capacity of the structure is provided by the RC core.

Other properties that influence dynamic response can also be significantly different for modular construction, including the total building mass due to the combined core and modules, and the inherent damping of a hybrid structural form that employs both reinforced concrete and steel elements. As economic drivers and advancements in manufacturing and material technology [8] lead to modular construction being employed in increasingly tall structures, the ability of the RC core to control the wind induced accelerations within acceptable limits is unproven [24], implying that, it may be necessary to employ some form of acceleration control methods. Typical methods employed to control building accelerations include:

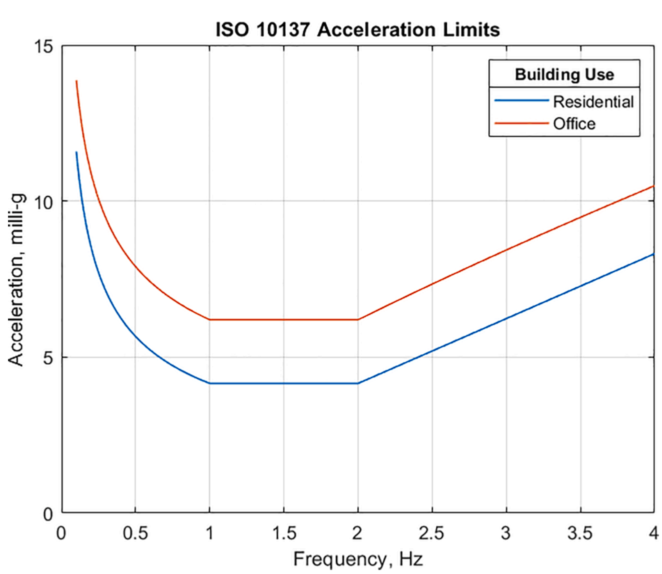


Fig. 1. ISO peak acceleration limits [20].

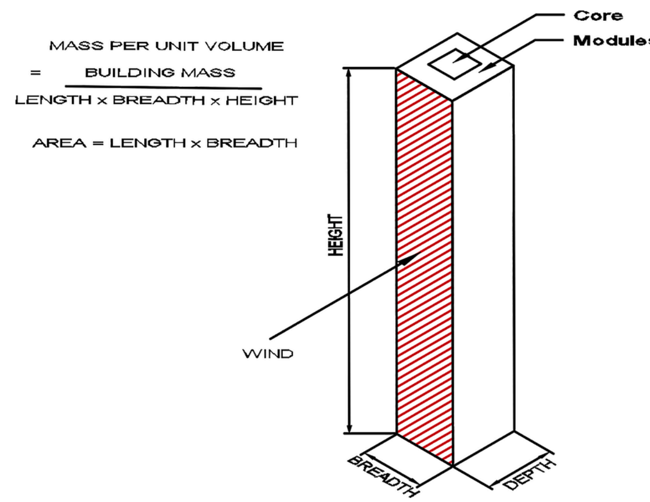


Fig. 2. Example of an archetype building.

- Changing physical properties of the structure such as building shape, materials and dimensions to favourably alter aerodynamics. Varying the shape to improve aerodynamics is generally not possible in modular construction due to its cellular nature. However, it is often possible to adapt the lateral load resisting system, i.e. the RC core, to improve performance.
- Employing passive control systems such as the inclusion of visco-elastic materials, Tuned Mass Dampers (TMDs), Tuned Liquid Dampers (TLDs) or variations of TMDs and TLDs such as Tuned Mass Damper Inerter or a Tuned Liquid Column Damper, to name a few [25,26]. Often where the decision is made to include an additional damping mechanism, a TLD is favoured due to its low maintenance, easy tuning and low cost compared to other forms of damper [26–28]. Another advantage of TLDs is their ability to be integrated into the water system of the building or be used in fire resistance systems [9,27].
- Employing active control systems incorporating a continuous monitoring strategy, a signal processing system and a damping mechanism such as a TMD which can be actively modified or induced to control accelerations based on the results of continuous monitoring.

This paper examines two practical methods for controlling wind-induced accelerations in high rise modular buildings: modifying the breadth and depth of the building's core, and installing a TLD. The goal of the work is to characterise the influence of key structural and control parameters on acceleration response of modular high rise buildings, providing novel insights into the behaviour of this form of construction which is only recently expanding into high-rise buildings. Given the limited amount of data on existing structures of this kind, assessing the potential applicability of a modular scheme at a preliminary design stage can be challenging. In particular, outstanding questions exist about the maximum feasible height for single-core modular construction; at what height acceleration serviceability becomes a limiting factor in design and at what height interventions to control this become unfeasible or uneconomical. This work provides new information insight that can be used early in the design of a wide range of modular towers; including what the maximum feasible height may be and what structural properties can be altered to better control wind-induced motion. This informs the preliminary design process and helps mitigate the need to carry out detailed, building specific dynamic analyses at an early design stage.

In order to assess the practicality and efficiency of increasing core dimensions or adding a TLD, a sample set of 6125 archetype buildings is generated. The archetypes cover a range of building heights, masses and dimensions representative of the feasible parametric range encountered in modular construction. Peak acceleration response during a 1 year wind event is estimated for each archetype, and in cases where this exceeds the ISO residential limit, the effectiveness of the two mitigation techniques is assessed and compared.

2. Generation of Structural Dataset

A sample set of 6125 archetype buildings was generated in order to assess the effects of modifying building core dimensions and of adding auxiliary damping by means of a TLD. The sample buildings are all of single tower, single core form, generated for each possible combination of the parameters listed in Table 1. These parameters were chosen as they cover a range of values which are representative of a series of the design possibilities for high-rise modular buildings.

Total building mass is calculated separately for the core and surrounding structure. This approach allows the effects of changing the core dimensions on the building mass, and ultimately acceleration, to be fully assessed. The mass of the core is calculated using the core dimensions and density, which is taken as 2500 kg/m^3 which is typical of highly reinforced concrete. The mass of the surrounding structure is one of the parameters varied in the generation of the archetypes. Typical timber or steel framed modules have an approximate self-weight, W_m, of $4-6kN/m^2$, with modules that include thicker concrete floor slabs having self weights up to $9kN/m^2$ [3,5]. Therefore, acceleration response is investigated for archetype buildings with mass per unit volume of 0.14 Mg/m^3 to 0.32 Mg/m^3, which corresponds to module self-weights from $4kN/m^2$ to $9kN/m^2$ for a 3 m storey height, as in Eq. 1:

$$W_m = M_{exC} \times g \times H \tag{1}$$

The core breadth and depth, i.e. the outer core dimensions, labelled b_{core} and d_{core} respectively, are calculated as building height/13, but these are limited to a maximum of building breadth or building depth/3 in the relevant direction, meaning that:

$$d_{core} = min\left(\frac{H}{13}, \frac{D}{3}\right) \tag{2}$$

$$b_{core} = min\left(\frac{H}{13}, \frac{B}{3}\right) \tag{3}$$

The final values were then rounded to the nearest 250 mm.

The core wall thickness was limited to between 400 mm and 600 mm, which are typical values for buildings up to 200 m tall [29].

The fundamental natural frequency of each archetype was calculated using Eq. 4, which is the standard expression for the natural frequency of a cantilever with a uniform mass per unit length.

$$f_s = \frac{3.5161}{2\pi} \times \sqrt{\frac{EI}{MH^3}} \tag{4}$$

Where E is the Young's Modulus of the reinforced concrete in the core, I is the second moment of area of the core and M is the total mass of the archetype, considering both the core and surrounding modules. Therefore, the lateral stiffness of each archetype was assumed to come entirely from the building core with no contribution from the modules, which as discussed previously is a common design assumption. However, modules are considered in the calculation of the building mass, and consequently the natural frequency. The loaded area also takes into account the presence of modules. Therefore, while modules do not contribute to lateral stiffness, their presence does strongly influence wind load and structural response.

Table 1
Archetype building properties.

	Symbol	Archetype Property Values
Building Height (m)	H	100, 110, 120, 130, 140, 150, 160
Mass Excluding Core (Mg/m^3)	M_{exC}	0.14,0.17, 0.20, 0.23, 0.26, 0.29, 0.32
Building Breadth (m)	B	20, 25, 30, 35, 40
Building Depth (m)	D	20, 25, 30, 35, 40
Core Wall Thickness (mm)	t_{core}	400, 450, 500, 550, 600

3. Calculation of Building Acceleration

3.1. Calculation Process

The peak acceleration response of each archetype building is estimated using the method proposed in Eurocode 1991–1-4 Annex C. The calculations are performed assuming a wind climate typical of the London region. The fundamental basic wind velocity, $v_{b,0}$, is 21 m/s, corresponding to the value specified by the UK National Annex to the Eurocode 1991–1-4 [30]. The surrounding terrain is assumed to be urban, or Class IV in Eurocode; the 10-min mean wind velocity, $v_m(z)$, and the turbulence intensity, $I_v(z)$ are calculated using these assumptions.

Return period, which as mentioned is 1 year for the ISO limits, is considered in Eurocode 1991–1-4 through the c_{prob} factor, which is calculated using Eq. 5:

$$c_{prob} = \left(\frac{1-0.2ln(-ln(1-p))}{1-0.2ln(-ln(0.98))}\right)^{0.5} \tag{5}$$

where p is the annual probability of exceedance. This is often assumed to be the inverse of the return period, however this assumption is not valid for short return periods like 1 year. In this study a Poisson distribution is assumed [31] and annual probability of exceedance is calculated using Eq. 6:

$$p = 1 - e^{-1/T} \tag{6}$$

For a return period, T, of 1 year this leads to a c_{prob} value of 0.749.

Once the natural frequency and wind excitation have been established, the peak acceleration of each archetype structure can be calculated using the Eurocode 1991–1-4 Annex C approach. This is based on the spectral approach initially proposed by Davenport [32], with Steenbergen et al. [33] providing a detailed derivation. In this approach the standard deviation of the along-wind acceleration, $\sigma_{\ddot{x}}$, is given by:

$$\sigma_{\ddot{x}} = c_f \times \rho \times I_v(z_s) \times V_m(z_s)^2 \times \frac{K_y \times K_z \times R}{\mu_{ref}} \tag{7}$$

where c_f is the force coefficient (dependent on building shape), ρ is the density of air, $I_v(z_s)$ is the turbulence intensity at height z_s ($z_s = 0.6$ h for regular buildings), $V_m(z_s)$ is the mean wind speed at height z_s, K_y and K_z are non-dimensional coefficients representing aerodynamic admittance and are functions of mode shape and μ_{ref} is the mass per unit area. R is a factor which represents resonance between the buildings natural frequency and wind gusts given by:

$$R^2 = \frac{\pi^2}{2\delta} \times S_L(z,f) \times K_s(f) \tag{8}$$

where $K_s(f)$ is a size reduction function accounting for variation in the wind load over the height of a tall structure, δ is the logarithmic decrement of damping and $S_L(z,f)$ is the value of the non-dimensional wind speed spectrum given by the Equation:

$$S_L = \frac{6.8f_L(z,f)}{(1+10.2f_L(z,f))^{\frac{5}{3}}} \tag{9}$$

where f_l is a non-dimensional frequency defined in the code as a function of the local wind climate and building natural frequency.

In this study, the inherent damping of each archetype was assumed to be 1.27%. This figure was obtained from the value for the logarithmic decrement of damping, 0.08, recommended by EC 1991–1-4 Annex F for mixed concrete and steel composite structures. There is a large level of uncertainty associated with this damping value, however it is not examined in this study.

The peak response is calculated from the standard deviation using a standard gust factor approach [32], in which the peak acceleration is given by:

$$\ddot{x}_{max} = k_p \times \sigma_{\ddot{x}} \tag{10}$$

where the gust factor, k_p is calculated as:

$$k_p = \sqrt{2 \times ln(600 \times f_s)} + \frac{0.6}{\sqrt{2 \times ln(600 \times f_s)}} \tag{11}$$

There are a number of simplifications in this approach, primarily that only along-wind acceleration response is considered. To account for this any archetypes deemed susceptible to vortex shedding, which can induce significant across-wind accelerations, were removed from the dataset. This was done by calculating the critical wind velocity for vortex shedding for each archetype using the method given in Eurocode 1991–1-4 Annex E and removing any any case in which this was less than 1.25 times the one-year wind velocity, which is the criteria specified in the code for a structure to be deemed susceptible to vortex shedding. This resulted in the removal of 66 archetypes (or approximately 1% of those considered) from the data set.

3.2. Calculation Results

The method described above was employed to calculate the peak acceleration responses of 6059 archetype buildings employing all combinations of the parameters listed in Table 1, excluding those where vortex shedding was identified as a potential issue. A number of trends emerge from the results of these calculations. Fig. 3 shows the relationship between peak acceleration and building mass and Fig. 4 shows the relationship between peak acceleration and core wall thickness for the 120 m archetypes. It is evident that most buildings of this height experience peak accelerations in excess of 4 milli-g and so, by comparison with Fig. 1, potentially exceed the ISO habitability limit for residential buildings. It may also be seen that the peak acceleration is generally reduced for archetypes with larger mass per unit volume. This is as expected; accelerations are known to reduce with building mass (to the extent that some forms of construction deliberately include additional mass to control wind induced accelerations [34]).

In Fig. 3 it can be seen that for low mass the range of accelerations is quite large, with maximum values of up to 15 milli-g, but as mass increases this range becomes smaller. It is generally the aim of the module fabricator to reduce the module self weight as this reduces material costs and waste, and improves construction time, ease of transport and lifting. However, this can negatively impact the dynamic performance of modular high-rise buildings with respect to wind induced accelerations.

Peak acceleration may be expected to reduce with increased wall thickness due to greater flexural stiffness and mass, however, Fig. 4 indicates that this effect may not be highly significant. Average peak accelerations are seen to decrease slightly as core wall thickness increases, but this trend is weak. Similarly, the range of peak accelerations observed is similar for each core wall thickness value examined. Increasing core wall thickness leads to both increased mass and

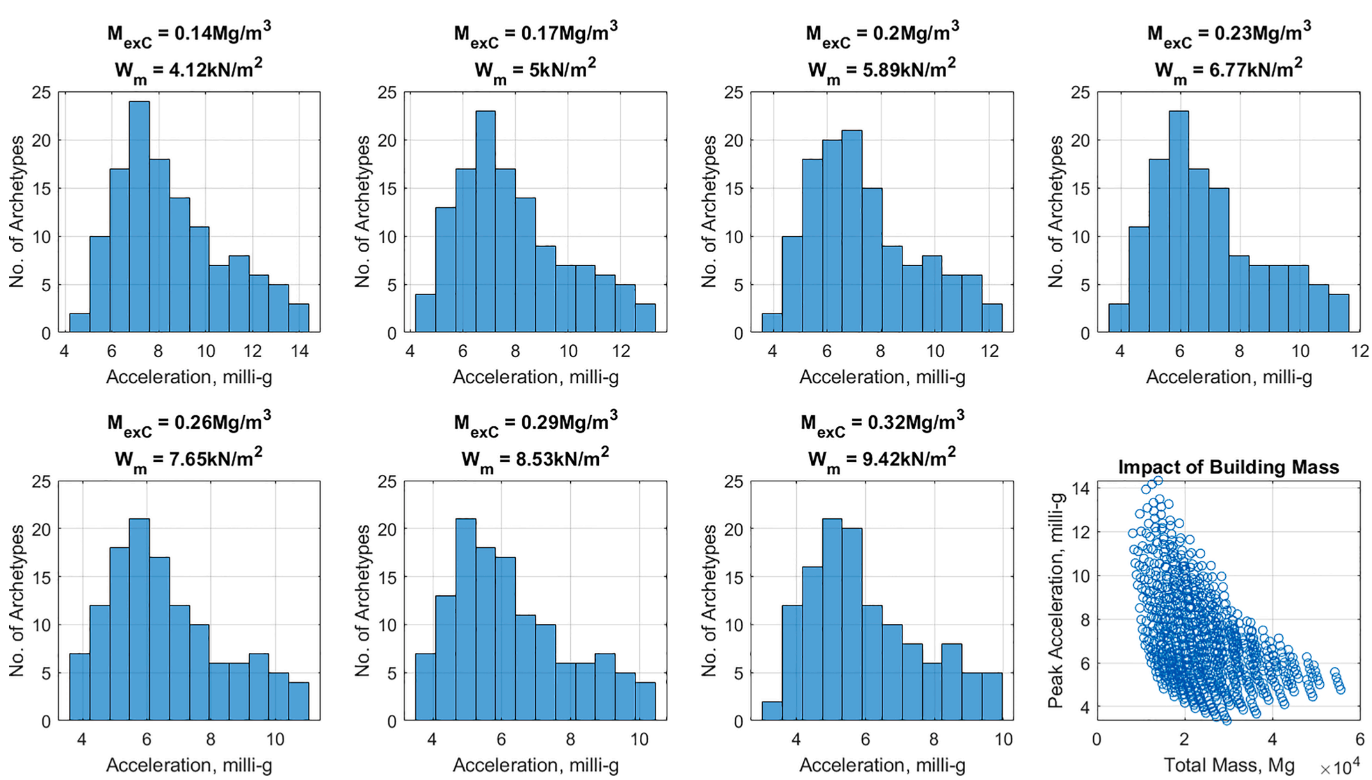


Fig. 3. Relationship between peak acceleration and building mass (shown in both mass per unit volume (Mg/m^3) and module self weight (kN/m^2) for a 3 m storey height) for the 120 m Archetypes.

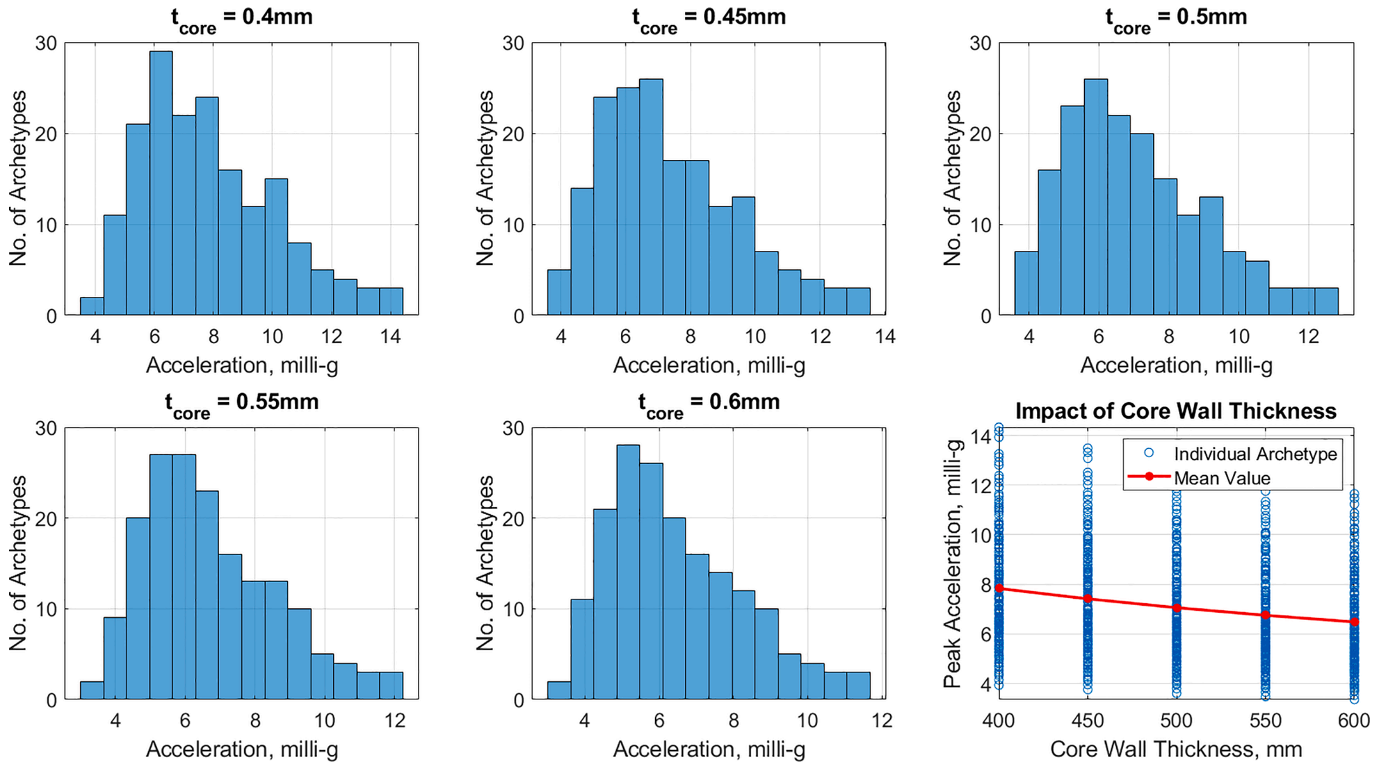


Fig. 4. Relationship between peak acceleration and core call thickness for 120 m Archetypes.

increased stiffness, both of which should reduce peak acceleration. However, the results obtained here suggest that the extent of these increases across the range of core wall thicknesses examined is not sufficient to substantially reduce peak accelerations.

The effects of building shape on the peak acceleration are also quite clear. Fig. 5 shows the predicted accelerations for the 120 m archetypes; it is observed that as the breadth of the building (dimension perpendicular to the acting wind force) decreases, so too does peak acceleration. The reverse is true for the depth of the building (dimension parallel to the acting wind force), with the peak acceleration decreasing as this increases. This is due to the extra mass and greater core depth, and therefore stiffness, of a deeper building. The variation of peak acceleration with breadth is not as stark, as the increase in mass and subsequent acceleration reduction is offset by the increase in area loaded by the wind.

3.3. Required Acceleration Reduction

For each archetype, the difference between the predicted peak acceleration from Eq. 10 and the frequency dependent ISO peak acceleration limit for a residential building was calculated. Of the 6125 archetypes considered, 3615 fall within ISO acceleration limits for residential buildings and require no additional measures to meet serviceability criteria. Modification of the remaining 2510 archetypes was required to reduce the wind induced response and eliminate acceleration serviceability issues. Fig. 6 shows the extent of the acceleration reduction required for each archetype.

In most cases an acceleration reduction of less than 5 milli-g is required to satisfy ISO acceleration limits. However, there are some extreme cases in which archetypes require as much as a 15 milli-g reduction in accelerations. In percentage terms, for the majority of cases where the acceleration limit is exceeded, less than a 40% reduction

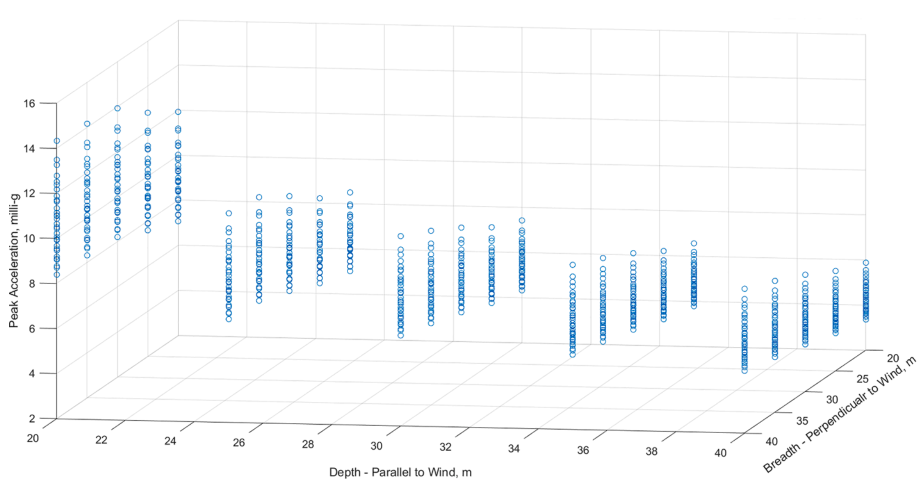


Fig. 5. Impact of building plan dimensions on peak accelerations for 120 m archetypes.

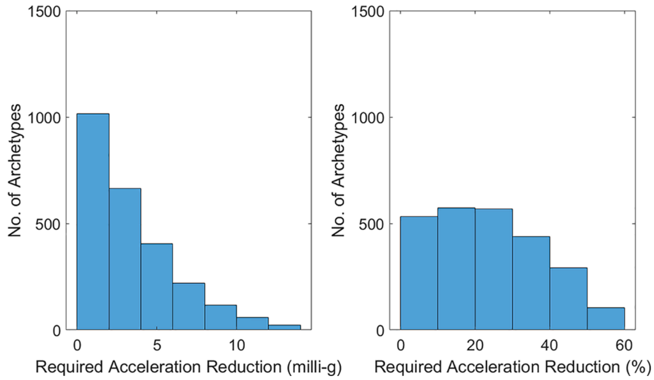


Fig. 6. Required acceleration reduction for archetypes exceeding ISO acceleration limits.

in acceleration is required. However in a small number of scenarios, reductions of up to 60% are required. The extreme cases which require reductions in the order of 15 milli-g or 60% are often somewhat unrealistic archetypes where the combination of parameters from Table 1 leads to a structure unlikely to be constructed in reality; for example the archetype in the dataset with the highest acceleration value is a 160 m tall structure with a 400 mm thick, 6.75 m deep core. If only cases with core slenderness less than 16 are considered, 1033 archetypes out of 4550 require remedial action, with this ratio reducing to 280 out of 2625 for structures with core slenderness less than 13.

Fig. 7 shows the percentage of archetypes requiring acceleration reduction interventions for the different module self weights considered. It can be seen that for lower module self weights, which correspond to the values typically seen in timber modules, the majority of accelerations exceed the ISO limit irrespective of building height. However, as module self weight is increased, the number of archetypes exceeding the limits decreases and building height begins to have a greater influence on whether or not the limit will be exceeded.

4. Modifications to Mitigate Excessive Accelerations

In the cases where the peak acceleration of an archetype exceeds the ISO limits, design modifications are required to meet serviceability criteria. As mentioned, in this study the modification methods considered to reduce accelerations are an increase in core dimensions and the installation of a TLD.

4.1. Required Increase in Core Dimensions

For each archetype, the increase in core dimensions, if any, required to reduce response accelerations sufficiently to satisfy the ISO residential limit was determined. It is important to note that the core wall outer dimensions, rather than core wall thickness, was altered to reduce accelerations. Core wall thickness is one of the parameters varied in the generation of the archetypes, but not adapted in the optimization, as Fig. 4 showed that this is not an efficient response mitigation approach. Instead the breadth and depth of the core are increased. This concept is illustrated in Fig. 8.

An incremental approach was employed to find the increase in core size required to bring the peak acceleration within ISO limits. The outer and inner core dimensions were increased in 100 mm steps and the peak acceleration recalculated for the modified archetype using the methods described in Section 3.1. This process of core resizing and acceleration recalculation was repeated until the acceleration of the archetype with increased dimensions was found to be less than the ISO residential

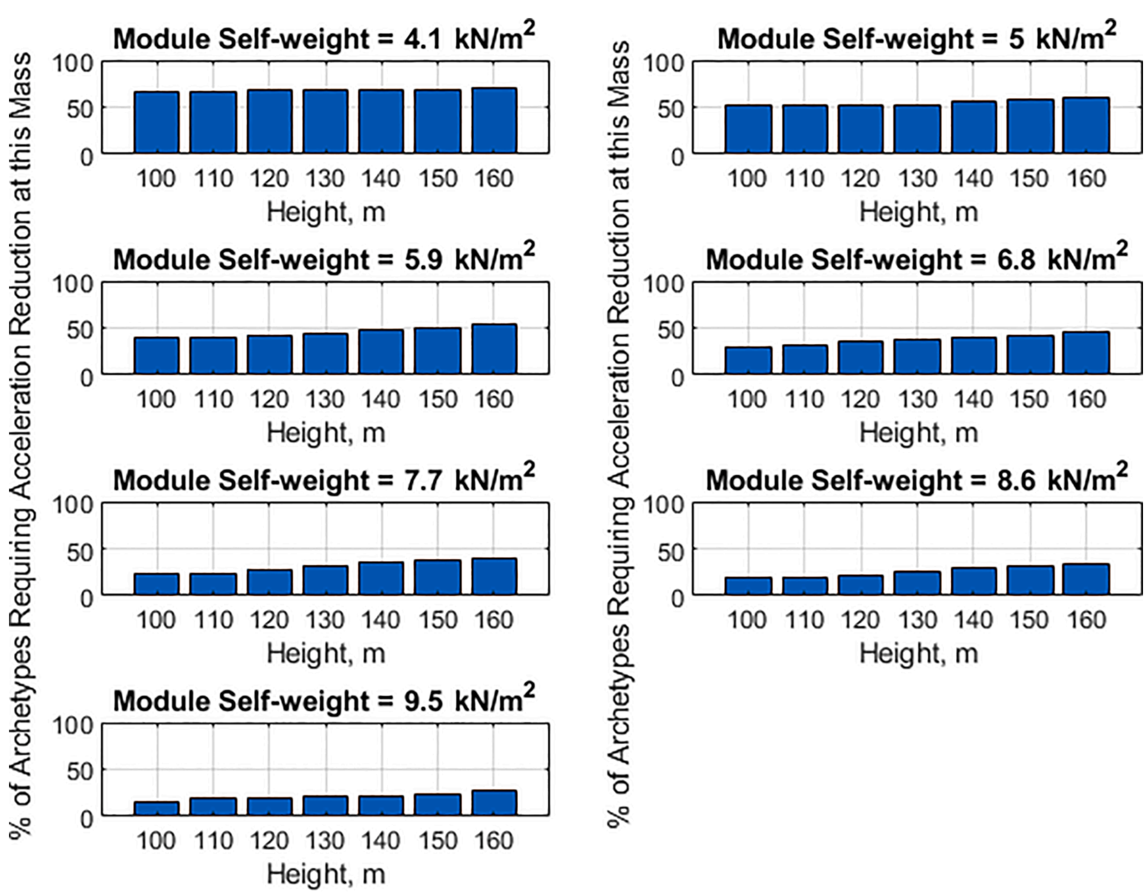


Fig. 7. Number of archetypes requiring acceleration reduction by mass.

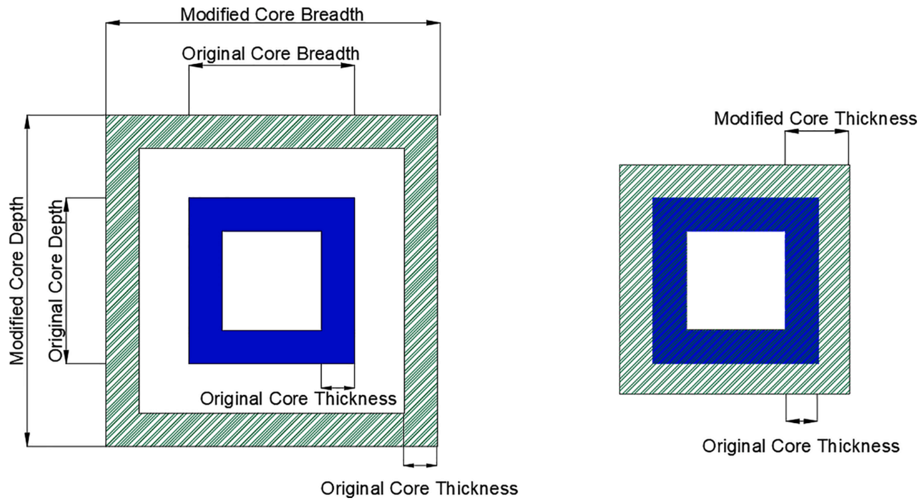


Fig. 8. Illustration of increasing core dimensions (left) and increasing core thickness (right). The method on the left was employed in this study.

limits, as illustrated in Fig. 9 for two archetypes.

The results of this process are shown in Fig. 10. The rate of acceleration reduction with increased core outer dimensions appears relatively low. Therefore, quite large increases in core dimensions are required to bring accelerations within the serviceability limits. In many cases these increases are in the order of some meters; in some instances it is above 5 meters. Fig. 10 (b) shows that the majority of cases do not require increases in core dimensions as extreme as 5 meters but that these instances tend to dominate Fig. 10 (a) visually.

From examining Fig. 10 (a) it can be seen that an increase in core dimensions of 1000 mm leads to an average 10% decrease in peak accelerations. This relationship becomes clearer when only the archetypes which required less than a 2000 mm increase in core dimensions are considered as in Fig. 11.

The feasibility of any increase in outer core dimensions is likely to be dictated by the financial loss associated with the loss of saleable floor area.Assuming the overall building footprint cannot be increased, increasing the core dimensions reduces the floor area outside the core. For tall modular construction, where typically the saleable area is limited to the area outside the core, even relatively minor increases in core dimensions can result in a substantial reduction in the value of a development. A square metre of residential property in London was valued at £8091 in 2019 [35]. Assuming only floor area outside the core can be sold, the reduction of income for the developer can be estimated as the product of the lost floor area per story, the number of storeys and the value of sell-able floor area. For example, for a 100 m tall archetype with an 8x8m core and 3 m storey height, a 100 mm increase in core outer dimensions translates to a loss in income of approximately £430,000 (1.61$m^2/storey$ x 33$storeys$ x 8091 £/m^2), a 1000 mm increase equates to £4,500,000 while a 5000 m increase leads to a loss of just under £30,000,000. While these relations are approximate, it can be reasonably concluded that for tall buildings any increase in core dimensions greater than several hundred millimetres is likely to be prohibitively expensive, particularly as a response measure to address a serviceability limit state issue. Therefore, comparing the cost and the extent of the required increases, it can be concluded that resizing core dimensions does not offer a feasible mitigation approach for the vast majority of potential modular buildings for which response acceleration is a design issue.

4.2. Required Additional Damping

A similar procedure was employed to calculate the additional damping required to bring the acceleration of each archetype below the ISO residential limit. As before this was done incrementally, this time by increasing the damping of the archetype in steps of 0.1%, and recalculating the peak acceleration as described in Section 3.1 until it reduces below ISO limits.

Fig. 12 shows the additional damping above the assumed inherent value of 1.27% required to bring accelerations below the ISO limits. Also shown is the analytical relationship between additional damping, ζ', and percentage reduction in acceleration $\Delta\ddot{x}$, which can be derived from manipulation of Eq. 7 and the standard expression for the relationship between logarithmic decrement and damping ratio to give:

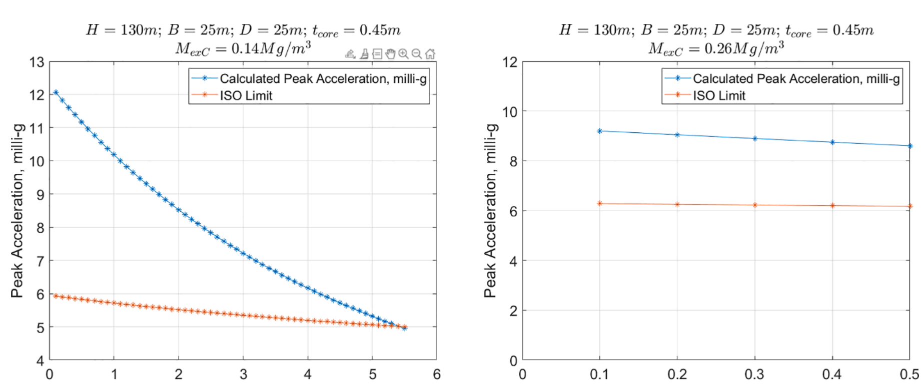
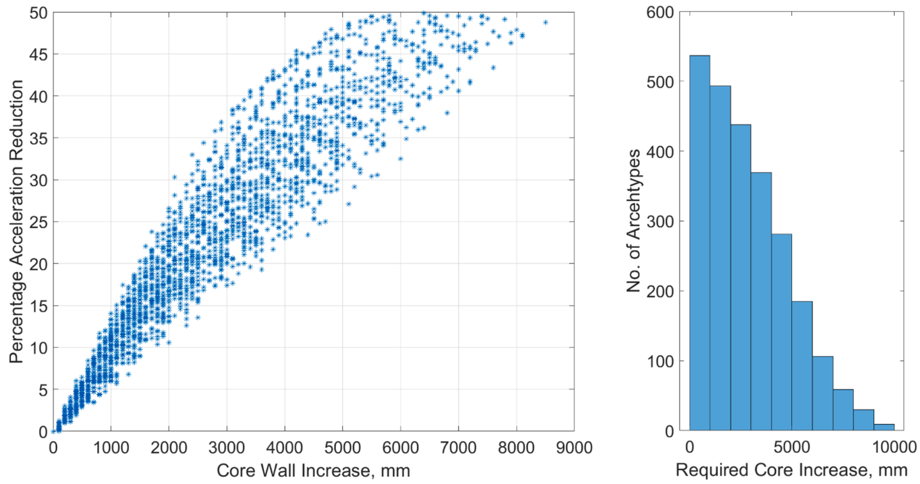


Fig. 9. Incremental increase in core dimensions to reduce response acceleration below ISO limits for two example archetypes.



(a) (b)

Fig. 10. Required increase in core dimensions.

$$\Delta_{\ddot{x}} = 100(1 - (\frac{1}{1 + \frac{2\zeta\zeta' + \zeta'^2}{\zeta^2 - \zeta^4 - \zeta^2\zeta'^2 + 2\zeta^3\zeta'^2}})) \tag{12}$$

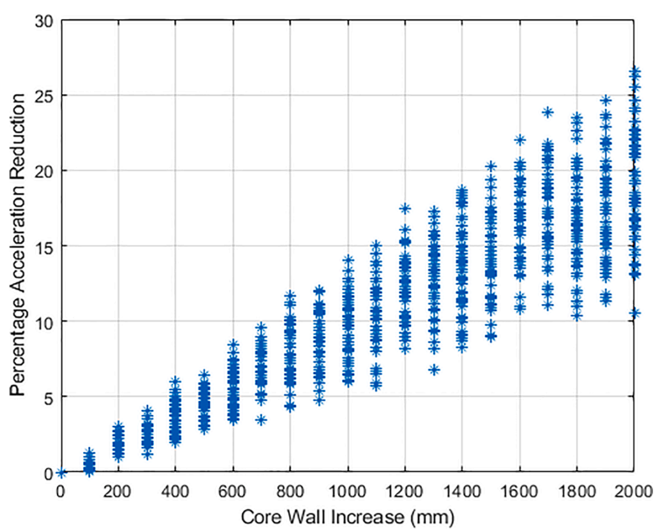


Fig. 11. Percentage reduction provided by increasing core dimensions for cases where less than a 2000 mm increase is required.

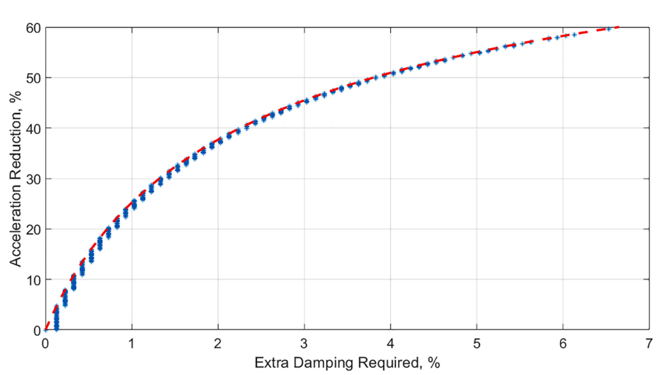


Fig. 12. Percentage reduction in acceleration provided by increase in damping.

As mentioned previously, the majority of archetypes examined require a reduction in peak acceleration of 40% or less. Fig. 12 shows that a 40% reduction can be achieved by providing additional damping of 2.4%.

In Fig. 13, the additional damping and additional core depth required to satisfy the ISO limit are compared for each archetype. While the trend displayed is not linear, an approximate relationship which can be inferred from this data is that roughly a 2100 mm increase in core wall depth and breadth corresponds to a 1% increase in inherent damping.

4.3. Additional Damping Provided by TLD

In order to assess whether the required additional damping is feasible, the maximum damping which can be provided by a TLD given the characteristics of each archetype was estimated using the method proposed by Tait [36] for the preliminary design of a TLD. In this process it is assumed that, as in Annex F of Eurocode 1991–1-4, the additional damping from a TLD can simply be added to the inherent structural damping to give an overall value for damping. As proposed in Tait [36], the method begins with the additional damping required, and a series of steps are followed to calculate the damper dimensions required to provide this level of damping. The process is reversed in this study; damper plan dimensions, i.e. damper breadth, b_d, and depth, L_d (see Fig. 14), are established based on the archetype properties and then the level of damping provided by a damper with these dimensions is calculated. The

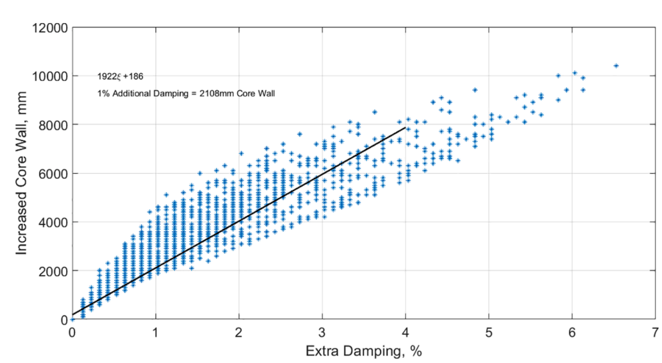


Fig. 13. Comparison of required increase in core depth and required additional damping for each archetype.

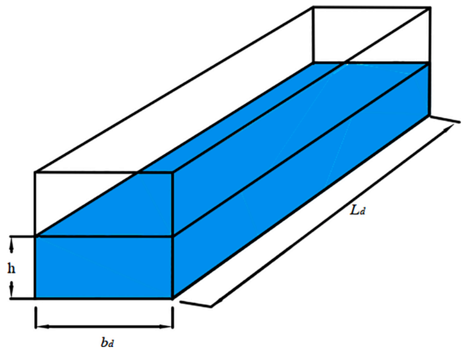


Fig. 14. Schematic of TLD.

steps undertaken to do this are as follows:

- Using the plan dimensions of the TLD and the natural frequency of the archetype, f_s, the height of water in the TLD, h, for which the sloshing frequency matches the building natural frequency is calculated as:

$$h = \frac{L_d}{\pi} tanh^{-1}\left(\frac{4\pi f_s^2 L_d}{g}\right) \quad (13)$$

- Once the height of water and the damper plan dimensions are established, the mass of water sloshing in the fundamental mode can be calculated as:

$$m_{eq} = \frac{8\rho b_d L_d^2}{\pi^3} tanh\left(\frac{\pi h}{L_d}\right) \quad (14)$$

- Using this value of m_{eq}, the mass ratio of the damper, μ, can then be calculated using Eq. 15:

$$\mu = \frac{m_{eq}}{M_s} \quad (15)$$

where M_s is the first modal mass of the building, which in this study is estimated as the first modal mass of a cantilever with a uniformly distributed mass, plus the mass of the non-participating component of the liquid associated with the fundamental sloshing mode.
- Once μ is calculated, an estimate of the additional damping provided by a TLD can be made using Eq. 16 [36,37]:

$$\zeta_{eff} = \frac{1}{4}\sqrt{\frac{\mu + \mu^2}{1 + \frac{3\mu}{4}}} \quad (16)$$

In the first step of this process it is assumed that the TLD is tuned to the building natural frequency. However the optimal sloshing frequency of a TLD, f_d, is given by:

$$f_d = f_s\frac{\sqrt{1 + \frac{\mu}{2}}}{1 + \mu} \quad (17)$$

Examination of Eq. 17 shows that for the low values of mass ratio (less than 0.1) typically encountered in TLD design, f_d will be very close to f_s. Therefore, at least at the preliminary design stage, it is reasonable to assume that the TLD can be tuned to the natural frequency of the archetype. This assumption avoids the need for any iteration in the calculation process.

The process outlined above is initialized by establishing the damper dimensions. For this study, this is done by constraining the maximum plan dimensions of the TLD, L_d and b_d, to match the core dimensions. This means that the TLD can be positioned on top of the archetype core without extending over the surrounding modules. This constraint ensures that no additional vertical load is imposed on the modules. Given that tall modular construction is often executed using a crane supported on top of the core, the critical vertical design load is often the temporary case during construction. Therefore, post-construction when the crane is dismantled, there is often unutilised vertical load carrying capacity in the core that can be exploited to support the mass of a damper. Consequently, by limiting the dimensions to match the core, it can be assumed that the damper can be installed without requiring consideration of the additional dead load. The alternative approach of strengthening the modules so that they can support some of the damper is not considered in this paper.

Fig. 15 shows the results of this calculation process for the 120 m and 150 m archetypes. It can be seen that for these archetypes, the estimated feasible additional damping ranges between approximately 0.5% and 3%. This value is a function of the mass ratio (Eq. 15), which depends on the first modal mass of the building and the size of the damper that can be installed, which in this study is controlled by the core size. Therefore, the amount of damping that can be provided generally increases with core size, as a larger core leads to a larger damper and a greater mass ratio, but decreases with building mass, as a larger building mass increases the modal mass and therefore reduces the mass ratio. Fig. 15 demonstrates this for the 120 m and 150 m archetypes.

In Fig. 16, the maximum additional damping available for each archetype is shown in blue, while the damping required to bring the accelerations within ISO acceleration limits are shown in red. In order for the TLD to provide sufficient auxiliary damping, the maximum damping available from the TLD (blue) must exceed the required damping (red). Whilst this is true for the majority of archetypes, as building height increases fewer archetypes can be made to satisfy the ISO serviceability limit by the addition of a TLD. This is because the required level of damping is generally greater for taller archetypes, as taller buildings tend to experience greater accelerations, but the available additional damping tends to decrease with building height, as taller buildings have greater mass leading to TLDs with smaller mass ratios. Furthermore, in Fig. 16 the mass of the building increases as the archetype number increases, i.e. the mass increases moving from left to right in each plot. Hence, it can also be seen that it is more difficult to control accelerations with a TLD in buildings with lighter modules.

5. Maximum Feasible Height of Modular Buildings

The methods presented above for calculating the response acceleration of a high-rise modular tower with a TLD tuned to the fundamental frequency of the building are used to evaluate the maximum feasible height that a modular building can attain without violating the serviceability limit state. It is shown that there is no unique value for the maximum height of a single core building which can be controlled by a TLD due to the dependence of acceleration response on both building mass and plan dimensions. However, it is possible to develop a series of curves which show the maximum feasible height as a function of building plan dimensions for a given mass per unit volume (excluding core) and core wall thickness. For a series of archetypes of incrementally increasing height, with a specific module self weight and core wall thickness, the peak acceleration was predicted using the method described in Section 3.1. The additional damping required and the maximum damping a TLD can provide were calculated using methods described in Sections 4.2 and 4.3. It is then possible to determine the maximum feasible height as the greatest height at which (1) maximum damping provided by the TLD is sufficient to maintain accelerations within the ISO limits for a residential building and (2) vortex shedding is not deemed problematic. Performing this process for a number of different building dimensions enables a curve to be generated which shows the approximate maximum feasible height of a building with a particular module self weight (excluding core) and core wall thickness

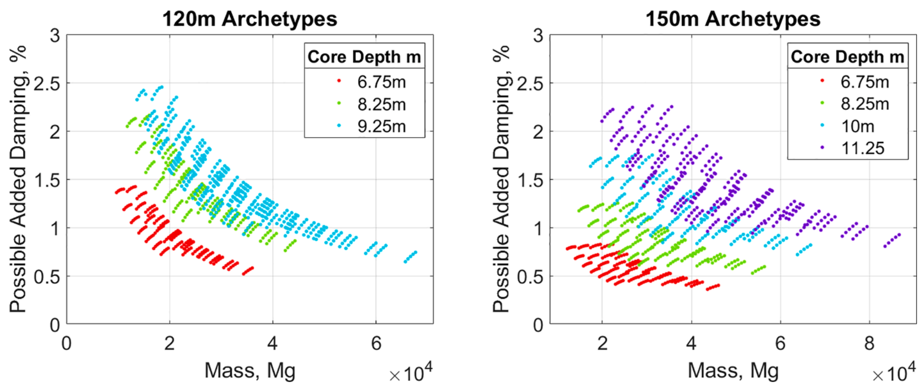


Fig. 15. Variation in added damping with building core size and total mass for the 120 m and 150 m archetypes.

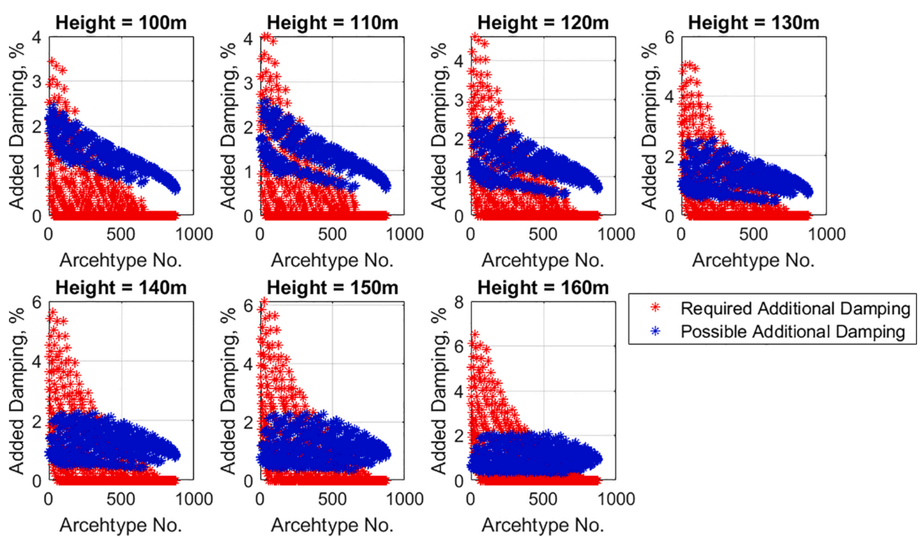


Fig. 16. Comparison of required damping and predicted available damping from an optimal TLD for archetypes.

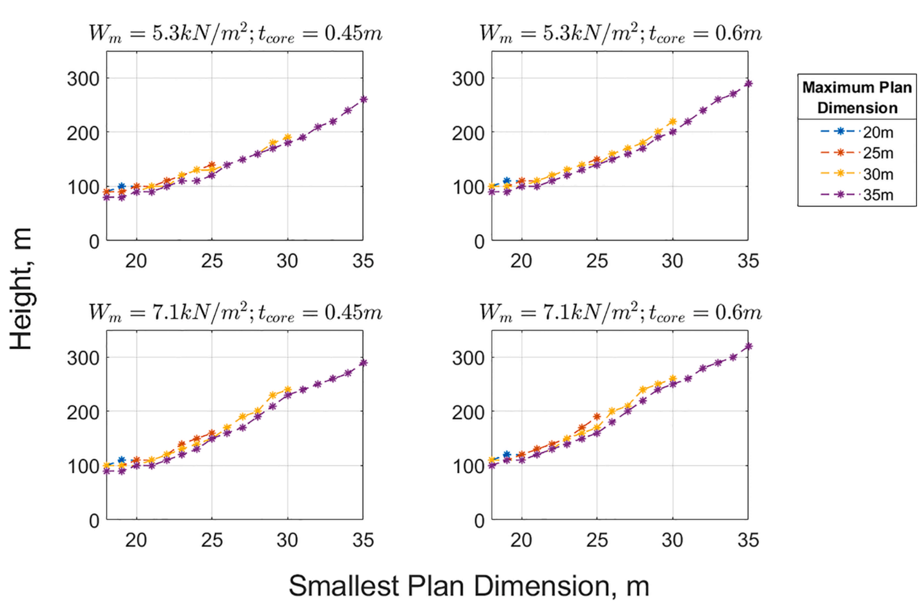


Fig. 17. Approximated maximum feasible height of single core modular buildings with varying combinations of self weight (approximated for a 3 m storey height) and core wall thickness.

given its smallest dimension on plan.

Fig. 17 shows examples of such curves for two combinations of module self-weights and core wall thicknesses. In the calculation framework adopted here, the critical case that dictates maximum height is that with the larger plan dimension perpendicular to the wind and the smaller plan dimension parallel to the wind, as this orientation results in the greatest loaded area and least stiffness. Examining the curves in Fig. 17 the most obvious trend is that the predicted maximum achievable height rises as the smaller plan dimension, i.e. building depth, is increased. For example in all four plots the estimated maximum height increases from about 100 m to 200 m as the building depth increases from 20 m to 30 m. This increase is attributable to the increased mass and stiffness of a deeper building. The larger plan dimension, i.e. the breadth perpendicular to the wind, has much less influence on estimated maximum height. This is because the increase in loaded area associated with a wider building is offset by an increase in mass and stiffness. By comparing the four different combinations of building mass and core wall thickness in Fig. 17 it can be seen that estimated maximum feasible height increases with mass, and to a lesser extent, core wall thickness. However, the influence of these parameters is minor compared to that of the smaller plan dimension. Finally, it is noted that the curves are not smooth and trends are not uniform with building depth. There are a number of reasons for this; firstly the ISO limit is a piecewise function meaning the required acceleration reduction is not smooth, and secondly the assumptions that govern core dimensions in this study result in abrupt changes in building frequency, and therefore response, with height.

6. Conclusions

The wind-induced acceleration responses of 6125 archetype high-rise modular buildings have been evaluated and their dependency on the key parameters of core and module mass, core stiffness and inherent and added damping assessed. The effectiveness of two methods for controlling response accelerations have been compared: increasing the breadth and depth of the core and installing a TLD. Both methods were assessed for their ability to control peak accelerations of the archetypes within ISO 10137 acceleration limits for residential buildings.

Whilst both methods can achieve the desired acceleration reduction for the majority of the archetypes studied, the practicality of increasing core dimensions to control accelerations is limited. On average, to reduce accelerations by 10% the core breadth and depth must be increased by approximately 1000 mm; this is inefficient, uneconomical and in many cases not feasible. The additional damping that could be provided by a TLD was estimated for each archetype and in the majority of cases this additional damping is sufficient to bring accelerations within the ISO limits. However, providing the required damping becomes more challenging as building height and mass increase.

A method for predicting the maximum feasible height of a single core modular tower has also been presented. The method produces curves that, for any combination of building mass per unit volume and core wall thickness, show the approximate maximum feasible height of a single core tower which can be controlled by a TLD given the building's plan dimensions. These curves provide a useful tool for preliminary design of modular high-rise buildings, and demonstrate that wind-induced dynamic response issues should not prevent modular buildings achieving greater heights in the future.

Declaration of Competing Interest

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.

Acknowledgements

The Funding for this research was provided by the Irish Government through the Irish Research Council [Grant No. EBPPG/2020/244]; and by Barrett Mahony Consulting Engineers. The authors are thankful for the support received through the financial funding.

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