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# The influence of different factors on buildings' height in the absence of shear walls in low seismic regions

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**Abstract.** Shear walls are structural members in buildings that are used extensively in reinforced concrete frame buildings, and almost exclusively in the UK, regardless of whether or not they are actually required. In recent years, the UK construction industry, led by the Concrete Centre, has questioned the need for such structural elements in low to mid-rise reinforced concrete frame buildings. In this context, a typical modern, 5-storey residential building is studied, and its existing shear walls are replaced with columns as used elsewhere in the building. The aim is to investigate the impact of several design variables, including concrete grade, column size, column shape and slab thickness, on the building's structural performance, considering two punching shear limits ($V_{Ed}/V_{Rd,c}$), lateral drift and accelerations, to evaluate its maximum possible height under wind actions without the inclusion of shear walls. To facilitate this study, a numerical model has been developed using the ETABS software. The results demonstrate that the building examined does not require shear walls in the design and has no lateral displacement or acceleration issues. In fact, with further analysis, it is shown that a similar building could be constructed up to 13 and 16 storeys high for 2 and 2.5 punching shear ratios ($V_{Ed}/V_{Rd,c}$), respectively, with adequate serviceability and strength, without the need for shear walls, albeit with thicker columns.

**Keywords:** high-rise RC buildings; wind actions; concrete grade; concrete section size; column shape; slab thickness; shear wall

## 1. Introduction

Shear walls are components typically included in reinforced concrete framed structures to resist lateral actions (Taleb *et al.* 2012). They are employed almost exclusively in the UK, especially in so-called low-rise buildings, which are up to five storeys or more (Emporis Standards 2008, Emporis Standards 2009, Banks *et al.* 2014, NFPA 2016). In recent years, experts at the Concrete Centre in the UK have questioned the extensive usage of shear walls, which is very costly to the construction industry, and the current work has been conducted as a direct consequence. Such elements, if used based on the design necessities, can provide stiffness to a structure that enables it to resist the applied lateral loads. On the other hand, if shear walls are employed regardless of the design requirements, this has a negative effect on the sustainability credentials of the final design, as well as the economic and structural efficiency. Accordingly, there is significant interest amongst the reinforced concrete construction sector into an investigation of the requirement for shear walls, whist maintaining and not compromising the occupants' safety.

In earlier studies (Keihani, Bahadori-Jahromi and Goodchild 2019) the significance of removing shear walls in an existing five-storey reinforced concrete (RC) building near London, in the UK, was investigated. The results demonstrate that the frame itself, with rigid connections between the elements, can withstand the applied loads after the shear walls are removed and the structural performance remains within the safe range, as defined by Eurocode 2 Part 1-1 (2014). Furthermore, it was shown that the same building can be safely constructed in various locations in the UK with different latitude and wind pressure values. The current study aims to build on this work and investigate the possibilities and limitations of increasing the height of a five-storey RC frame residential building without shear walls, and to develop a deep understanding of the influential parameters and limits.

There are a number of different classifications of multi-storey buildings, with no globally-accepted definition for low-, medium- or high-rise structures. For some researchers (e.g. Höweler 2003), the classification of the building is defined by relationship between the height and width of the structure, whereas others use the overall height as the measure. For example, Emporis Standards (2008, 2009) categorise low-rise buildings as structures below 35 m and high-rise buildings as structures between 35 m and 100 m. Moreover, the National Fire Protection Association (NFPA 2016) defines a high-rise building as a structure greater than 23 m in height. Scott (1998) refers to a high-rise building as a structure with a very tall facade, a small roof area and a small footprint. Banks *et al.* (2014) consider buildings as

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Fig. 1 The village overview and the reference building highlighted in yellow (COUCH Consulting Engineers)

high-rise if the ratio between the height and the lowest lateral dimension is greater than 5:1. This study adopted the Emporis Standards definition, where a high-rise building is considered 35 m to 100 m high.

In order to investigate the potential and limitations for the maximum overall height of reinforced concrete buildings without shear walls, it is essential to identify the key parameters that influence how the structure responds to lateral loads, specifically. These are the variables that influence the building's structural performance and hence have an impact on the maximum height that can be achieved. These variables include:

- Concrete strength

Several studies have been conducted regarding the impact of concrete grade on the ultimate capacity of concrete elements, including the study done by Ibañez, Hernández-Figueirido and Piquer (2018), in which the influence of concrete grade C30 and C90 on CFST (concrete-filled steel tube) columns was investigated. The results illustrate that the concrete strength has a positive impact on the columns' ultimate capacity, which means, by the increment of concrete grade from C30 to C90, the column sections could resist higher loads.

- Column size

There are not many studies on the influence that the section size of different concrete columns can have on their load capacity or ultimate strength, however, Murty *et al.* (2012) mentioned that a column size has a direct influence on a building's stiffness and mass in which the increment of the column's size subsequently increases the mass and stiffness. Furthermore, Avşar, Bayhan and Yakut (2012) identified that the axial load level, amount of reinforcement for tension and compression, concrete strength and geometry all directly affect the rigidity of concrete beams and columns. It can be concluded that larger columns may result in higher rigidity in a building's structural performance.

- Column shape

There are no comprehensive studies for the effect of the concrete column's shape on a building's structural performance and its impact on the punching shear. However, an essential factor that can affect an element's section strength is its moment of inertia, which represents the mechanical characteristics of a material in response to the applied stress due to the load (Singh, Nagar and Agrawal, 2016). This value might vary for rectangle and square shapes, depending on the axis, while for circle shapes, it is the same in all directions. That is why a rectangular shape, compared to a circular one with the same area, can have a higher moment of inertia in one axis and lower value in the other, and the combination of X and Y axes is important.

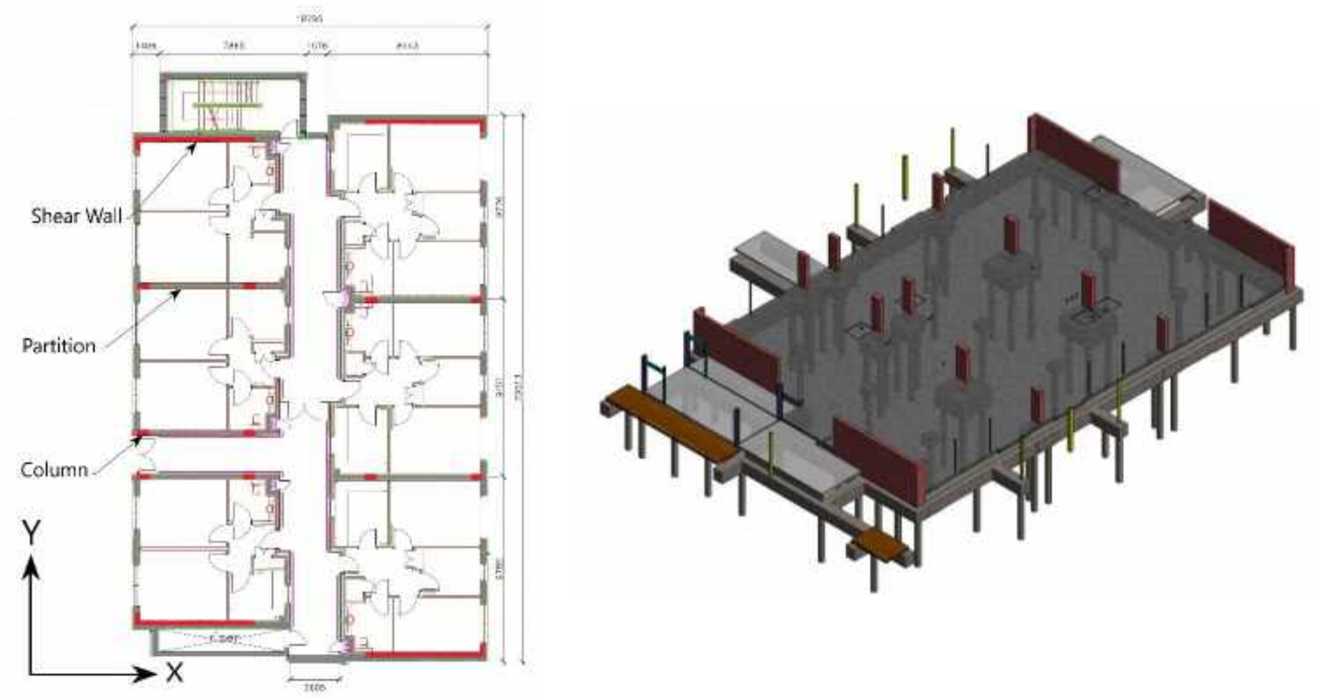
Moreover, Ibañez, Hernández-Figueirido and Piquer (2018) studied the shape effect on axially loading high-strength CFST stub columns. In this study, three different cross-sectional shapes with the same area were utilised: a circle, rectangle and square. The results obtained showed that the circular CFST columns could resist higher axial forces, shear forces and bending moments to a greater extent than rectangular or square columns.

- Slab thickness

The slab thickness is another factor that can influence a building's structural performance on the lateral stiffness and punching shear, especially in flat slabs. One major issue with such an element is since its flexural stiffness is relatively low, the concentration of bending and shear stresses in the surroundings of the columns could lead to punching failure (Moreno and Sarmento, 2011, Lapi, Ramos and Orlando, 2019, Hyeon-Jong, Gao and Chang-Soo, 2019). Besides, punching failure can happen in internal, edge or corner columns, and its ratio on the corner columns is more critical than is the case with the other two (Bond 2011, Alkarani and Ravindra 2013). On the other hand, changing the slab thickness can have a direct impact on the building's dynamic performance, as increasing the slab thickness escalates both the natural frequency and stiffness (Islam Khan *et al.* 2013). Since the slab thickness variation can greatly affect the punching shear ratio ($V_{Ed}/V_{Rd,c}$) on flat slabs (Goodchild 2009), two punching shear ratios, being 2 and 2.5, are recommended by the UK National Annex and are used in this analysis.

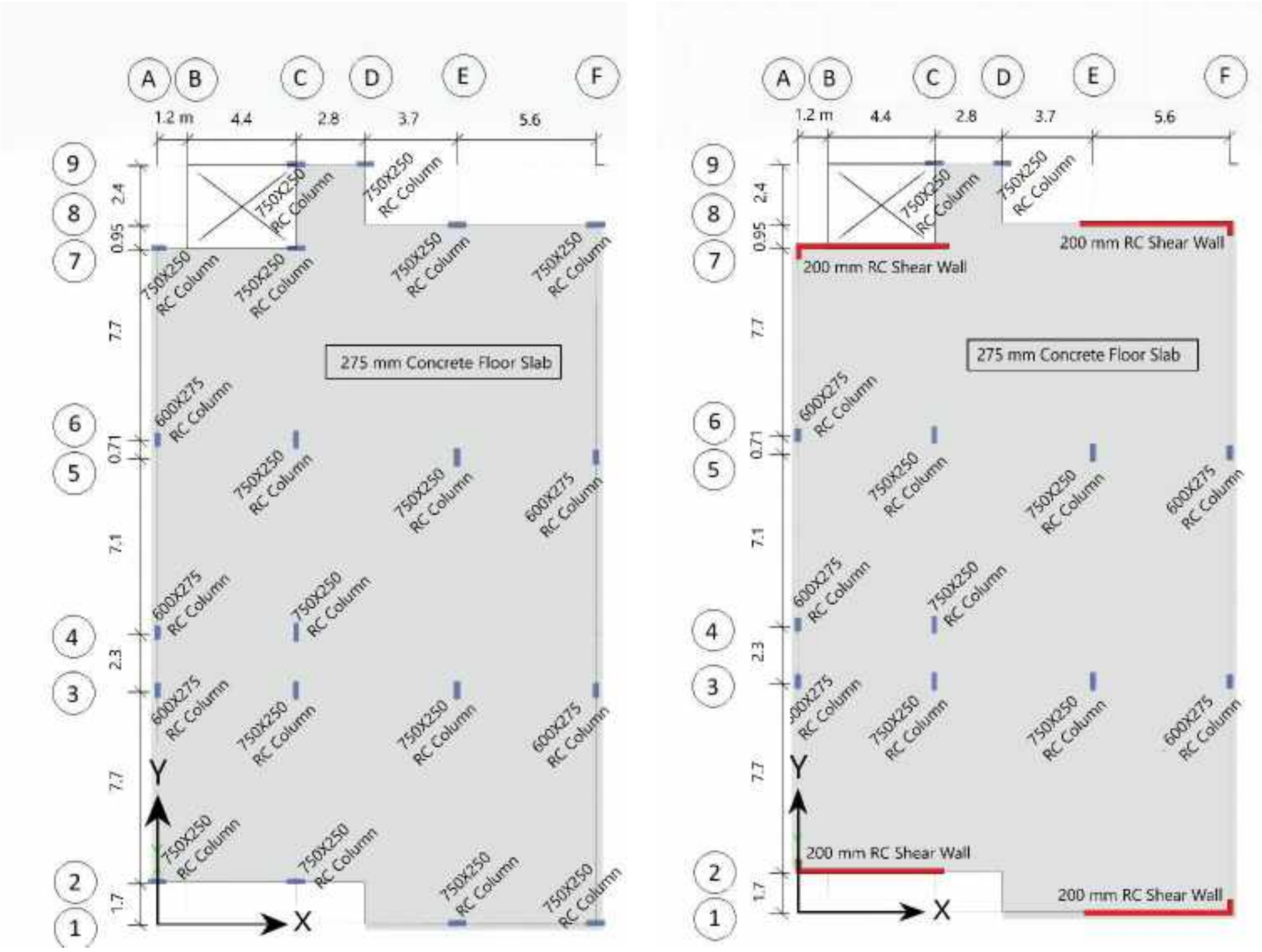
## 2. Numerical modelling

In the current study, a reference architectural plan is taken from a five-storey residential RC frame building in the UK, as is shown in Figs. 1 and 2. Belfast is selected for the current analysis since it has been shown previously by Keihani, Bahadori-Jahromi and Goodchild (2019) to be the most onerous of various UK locations (excluding Shetland Island) in terms of wind loading (Keihani, Bahadori-Jahromi and Goodchild 2019).

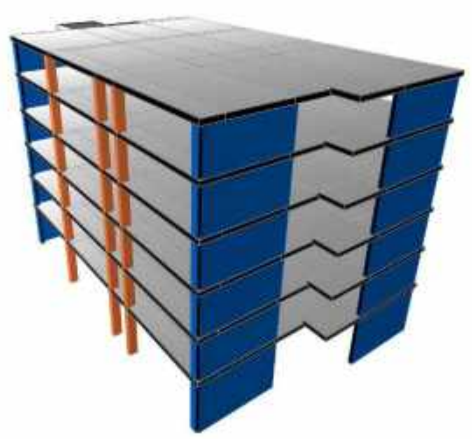



a) Architectural plan (values in mm) b) Columns and shear walls location

Fig. 2 Reference building with shear walls




(a) Reference plan design (b) Modified plan design



(c) Isometric of reference building

(d) Isometric of modified building

Fig. 3 Structural arrangement of reference and modified structures (values in m)

Table 1 Details from the reference building (with shear walls)

| Parameter | Value | |
|---|---|---|
| Height (m) | 19.46 | |
| Number of Storeys | 5 | |
| Typical Floor Height (m) | 3.08 | |
| Roof Height (m) | 2.96 | |
| Ground Floor Height (m) | 4.13 | |
| Overall dimensions (m) | 18.8 × 29.0 | |
| Floor (mm) | Flat Slab 275* | Flat Slab 325* |
| Column (mm) | 600 × 275 | 750 × 250 |
| Shear wall (mm) | 250 | |
| **Concrete** | | |
| Grade | C 30/37 | C 40/50 |
| $f'_c$ (Compressive strength) (N/mm$^2$) | 30 | 40 |
| Weight per unit volume (kN/m$^3$) | 25 | 25 |
| E (Modulus of Elasticity) (N/mm$^2$) | 33000 | 35000 |
| Poisson's Ratio | 0.2 | 0.2 |
| G (Shear Modulus) (kN/m$^2$) | 13750 | 13750 |
| **Steel (Rebar)** | | |
| Grade | B500B | |
| $f_y$ (Yield strength) (N/mm$^2$) | 500 | |
| $f_{yd}$ (Design yield strength) (N/mm$^2$) | 435 | |
| $R_m/R_e$ (Ratio of tensile strength/Yield strength) | 1.08 | |
| Roof loads Permanent (kN/m$^2$) | 6.875-7.5* | |
| Roof loads Imposed (kN/m$^2$) | 1.5 | |
| Floor loads Permanent (kN/m$^2$) | 6.875-7.5* | |
| Floor loads Imposed (kN/m$^2$) | 2.5 | |
| Stairs loads Permanent (kN/m$^2$) | 4.3 | |
| Stairs loads Imposed (kN/m$^2$) | 4 | |
| Exterior walls Permanent (kN/m$^2$) | 5.4 | |

*Depending on the slab thickness, the permanent load varies between 6.8 and 7.5 kN/m$^2$

Table 2 Parameters required for the design wind load for Belfast, UK

| Specification | Value | Reference (EN 1991-1-4, 2005) |
|---|---|---|
| Terrain Category | IV (Town) | Cl 4.3.2 |
| Reference Height | 31.8 m | Cl 6.3 |
| Directional Factor | 1 (Recommended) | Cl 4.2 |
| Season Factor | 1 (Recommended) | Cl 4.2 |
| Fundamental Wind Velocity | 25.6 m/s | Fig. NA.1 |
| Basic Wind Velocity (3-second gust) | 25.6 m/s | Cl 4.2-Exp (4.1) |
| Terrain Factor | 0.23 | Cl 4.3-Exp (4.5) |
| Roughness Factor | 0.79 | Cl 4.3-Exp (4.4) |
| Terrain Orography Factor | 1 (Recommended) | Cl 4.3 |
| Mean Wind Velocity | 20.48 m/s | Cl 4.3-Exp (4.3) |
| Turbulence Intensity | 0.29 | Cl 4.4-Exp (4.7) |
| Basic Velocity Pressure | 0.26 kN/m$^2$ | Cl 4.5-Exp (4.10) |
| Peak Velocity Pressure | 0.78 kN/m$^2$ | Fig. NA.1 |
| Structural Factor | 1 (Recommended) | Cl 6.2 |
| Wind Pressure | 1.01 kN/m$^2$ | Cl 4.2-Exp (4.1) |
| External Pressure Coefficient * | 1.3 | Cl 5.2-Exp (5.1) |
| Wind Force (X) | 540 kN | Cl 5.3 |
| Wind Force (Y) | 324 kN | Cl 5.3 |

*External pressure coefficient is selected for the wider face (X direction).

In the current study, the shear walls, which were included in the original design, as shown in Fig. 2(a), are removed and replaced with columns of similar section size to those already at other locations in the buildings, as it is illustrated in the Fig 3.

The specification of the building, including the dimensions, concrete and steel material properties and the applied vertical loads are presented in Table 1.

Once the reference building has been selected, the next steps in the analysis are to (i) determine the wind loading, (ii) develop the simulation procedure, and (iii) verify the

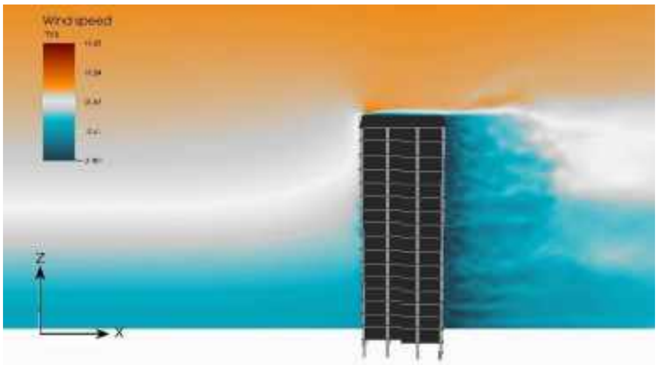



Fig. 4 Wind flow (velocity) on the building (Ingrid Cloud, 2018)

design. Each of these steps is described in more detail in the following sub-sections.

### *2.1 Design wind load*

Wind is the dominant lateral design load for high-rise buildings, and it consists of both a static and a dynamic component. In high-rise buildings, extreme localised varying loads and large aerodynamic forces may be applied to the façade and structural system. Under the influence of such loads, a building oscillates and the amplitude depends on the dynamic characteristics of the structure and the aerodynamic nature of the applied loads. If the vortex-shedding frequency and natural frequency of a building occur simultaneously, it can result in large-scale displacement of the building's response, called the critical velocity effect (Mendis *et al*. 2007, Li, Zhang and Li 2014, Zhi, Chen and Fang 2015).

A wind gust as a sudden rise in the wind's strength is dependent to the velocity at the time, representing the worst case scenario due to its force and high velocity, and it usually happens for only a few seconds (Ambrose and Vergun 1995, Schueller 1977). Due to the fluctuating components of the wind or gust, calculating the pressure is difficult. This is because pressure depends on various factors, including the nature of the wind, the local terrain and shape, size and dynamic characteristics of the structure.

In order to design the wind load, the European standards (Eurocode 1 Part 1-4, 2005) present a procedure for different locations. As Belfast is the location adopted in the current study, the input values for the simulations and the wind flow for this location are presented in Table 2 and Fig. 4. Fig. 4 illustrates wind load simulation and its impact on the building using an Ingrid Cloud Simulator (Ingrid Cloud 2018).

### *2.2 Simulation procedure*

The overall design procedure to perform the simulations is demonstrated in Fig. 5.

#### *2.2.1 Material properties*

The design of RC buildings in accordance with Eurocode 2 Part 1-1 (2014) is based on the characteristic cylinder strength rather than the cube strength, which is determined using the guidance in BS 8500-1 (2015).

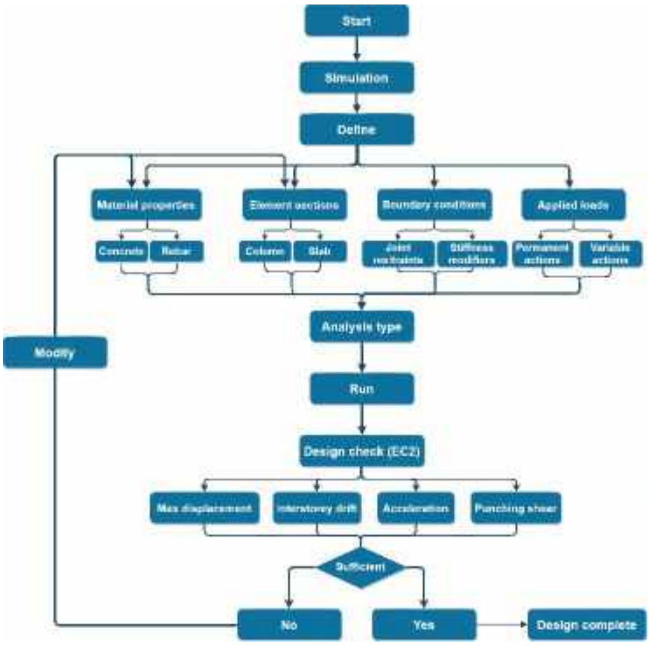



Fig. 5 Overall design procedure

Eurocode 2 Part 1-1 (2014) can be used to design concrete class up to C90/105, although for classes above C50/60, additional variations and rules could be applied. Furthermore, Eurocode 2 Part 1-1 (2014) can be utilised for reinforcement of characteristic strength ranging from 400 to 600 N/mm$^2$ and the related reinforcement properties for the UK could be found in BS 4449 (2005), in which 500 N/mm$^2$ characteristic strength is adopted in the UK construction industry.

In the ETABS software, the concrete material properties were defined according to Eurocode 2 Part 1-1 (2014) per EN 206-1 (2000), with different concrete strength classes ranging from C40/50 to C80/95 using the stress-strain curve proposed by Mander, Priestley and Park (1988). For the steel material properties, grade S355 was selected according to EN 1993-1-1 (2005) per EN 10025-2 (2019) due to its wide range of applications in the UK construction.

#### *2.2.2 Element section*

The column and slab section properties are defined according to the different variables mentioned in this study (Table 3). For columns, first the shape of column is defined (either rectangle, square, circle or special shapes). This was followed by defining the concrete grade, column dimensions and reinforcement details such as rebar material, clear cover for confinement bars, number of longitudinal bars along X and Y directions, longitudinal and corner bar sizes and size of confinement bars. Also, in order to account for cracked behaviour of concrete, the elastic stiffness of the bilinear force-deformation relation in reinforced concrete elements should be adjusted according to Eurocode 2 Part 1-1 (2014). In this case, the property modifiers for moment of inertia about X and Y-axes are adjusted to 0.5.

For slabs, first the material is defined as concrete (C30/37) and the modelling type is taken as shell-thin in order to properly simulate the behaviour of flat slab in the analysis. Moreover, to account for crack behaviour of slab, the property modifiers for moment of inertia about X and

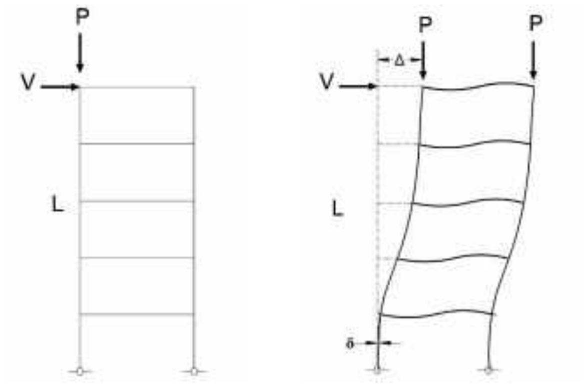



Fig. 6 Second order effect

Y-axes are adjusted to 0.5. In this study, in line with UK construction practice, flat slab is considered.

### 2.2.3 Boundary conditions

The boundary conditions depend on the design assumptions and can be different from one structure to another. In RC moment-resisting frames, the joints between columns and other elements (beams and slabs) and base columns to the foundation are considered to be fixed to transfer the stress distribution.

### 2.2.4 Applied loads

In a load case, the design value of an action ($F_d$) is:

$$F_d = \gamma_F \psi F_k$$

where
$\gamma_F$ = Partial factor for actions;
$\psi$ = Factor defining representative values of variable actions; and
$F_k$ = Characteristic value of an action

#### Ultimate limit state (ULS)

The designer can choose between expression 6.10, 6.10a or 6.10b that are defined by Eurocode 0 (2017) for the design value according to ultimate limit state, given as:

Exp. (6.10) $1.35\, G_k + 1.5 Q_{k,1} + \sum(\psi_{O,i}\, 1.5 Q_{k,i})$

Alternatively, the worst case of:

Exp. (6.10a) $1.35\, G_k + \psi_{O,1} 1.5 Q_{k,1} + \sum(\psi_{O,i}\, 1.5 Q_{k,i})$
Exp. (6.10b) $1.25\, G_k + 1.5 Q_{k,1} + \sum(\psi_{O,i}\, 1.5 Q_{k,i})$

where
$G_k$ = Characteristic value of a permanent action;
$Q_{k,1}$ = Characteristic value of a leading variable action;
$Q_{k,i}$ = Characteristic value of an accompanying variable action;
$\psi_{O,1}$ = Characteristic combination factor for 1st variable load; and
$\psi_{O,i}$ = Characteristic combination factor for ith variable load.

Expression (6.10) tends to utilise $\gamma_F = \gamma_G = 1.35$ for permanent actions and $\gamma_F = \gamma_Q = 1.5$ for variable actions and is always considered to be equal or more conservative than the less favourable of (6.10a) and (6.10b) expressions.

Except when the permanent actions are greater than 4.5 times the variable actions, or there are concrete structures supporting storage loads, expression (6.10b) will apply to most concrete structures.

#### Serviceability limit state (SLS)

For the SLS, there are three load combinations, which are given in Table A1.4 of Eurocode 0 (2017). Depending on the checked limit state, the combinations could be utilised. In the current study, the applied permanent and imposed loads are calculated according to Eurocode 0 (2017) and mentioned in Table 1. Moreover, the load combinations for the simulations were defined according to ULS and SLS load combinations.

### 2.2.5 Analysis type

Depending on the building's geometry, material properties, support conditions and structural loads, the type of analysis can be chosen. In case of low-rise buildings, the linear elastic analysis would suffice, however, for high-rise buildings, due to the complexity of the building and non-linearity of materials, a non-linear analysis provides more realistic results with less computing time compared to dynamic analyses which could be used for both ultimate limit state (ULS) and serviceability limit state (SLS) criteria and it assumes non-linear behaviours for the materials. Nowadays, in advanced structural analysis, in order to analyse and design buildings Finite Element Methods have been extensively used in the construction industry to capture more accurate structural performance of buildings.

A vital aspect of the analysis is to simulate the structural behaviour of an RC frame building with accuracy and reliability. When the building is subjected to lateral forces (V), it tends to deform, which requires consideration of the second-order (P-Δ) effects. Furthermore, the P-Δ shear (the force generated at the bottom and top of the columns due to P-Δ moments) produces an extra demand on the lateral shear resistance of the structural system (Fig. 6). This additional demand is added to the applied shear load, which may be critical.

Hence, in this study, due to the building's maximum height and the impact of lateral displacements on the building's structural performance, a non-linear static analysis is conducted. This type of analysis applies a nonlinear relation between forces and displacements that can originate from material nonlinearity, geometrical nonlinearity and constraint and contact nonlinearity. These factors result in a stiffness matrix that varies with the applied loads and can be used for both the ultimate (ULS) and serviceability (SLS) limit states. The results are obtained by conducting various numerical analyses using ETABS software (version v16.2.1), which is engineering software used to analyse the structural performance and design of multi-storey buildings (Saisaran *et al.* 2016, Tsay 2019, Jolly and Vijayan 2016). In this study, ETABS software is used due to its efficiency in performing reliable wind analysis, concrete elements design and deriving punching shear ratios based on Eurocode 2 Part 1-1 (2014).

To perform the analysis, first, the frame with 750 × 250 mm column section size and 275 mm flat slab thickness was modelled in ETABS (Fig. 3b), and the vertical and

Table 3 Investigated variables

| Specification | Variable 1 | Variable 2 | Variable 3 | Variable 4 | Variable 5 | Variable 6 | Variable7 | Variable 8 |
|---|---|---|---|---|---|---|---|---|
| Concrete grade (column) | C40/50 | C45/55 | C50/60 | C55/67 | C60/75 | C70/85 | C80/95 | Optimised |
| Concrete grade (flat slab) | C30/37 | - | - | - | - | - | - | - |
| Column size | 750 × 250 | 750 × 300 | 750 × 350 | 750 × 400 | 750 × 450 | 750 × 500 | - | - |
| Column shape | Square | Rectangle | Circle | - | - | - | - | - |
| Slab thickness | 275 mm | 300 mm | - | - | - | - | - | - |

lateral loads were applied according to Eurocode 1 Part 1-4 (2005) (as presented in Tables 1 and 2). Then, a combination of values for the selected factors was adopted (as shown in Table 3) and the number of storeys was increased. For each simulation, the design limitations for maximum displacement, interstorey drift and horizontal acceleration and punching shear ratio ($V_{Ed}/V_{Rd,c}$) according to Eurocode 2 Part 1-1 (2014) were checked to investigate the safety of the buildings and control the ductile behaviour of moment-resisting frames with flat slabs. If the building's design limitations were lower than the acceptable threshold, the number of storeys was increased, and if the design limitations were close to the threshold, the simulation was stopped. This procedure was repeated until the highest number of storeys with punching shear ratio 2 and 2.5 was achieved.

The influence of four factors with predefined ranges (Table 3) on the building's structural performance was investigated. The four factors investigated were:

- Concrete strength;
- Column size;
- Column shape; and
- Slab thickness

In the first stage, different concrete grades, ranging from C40/50 to C80/95, were utilised and an optimised concrete design created, with higher strength in the lower storeys and lower strength in the higher storeys for the columns, in order to assess their influence on the building's structural performance. These values were selected based on advice from the Concrete Centre, and represented the typical range available in the UK. At this stage, the minimum values for column size and slab thickness were adopted (750 × 250 mm and 275 mm, respectively) to observe the concrete grade's impact.

For the optimised concrete grade, Table 4 presents the variation of concrete grade over height for each column section. As shown in Table 4, higher strength concrete grades were used in the bottom storeys and the strength reduced over the height. To achieve the best results, each concrete grade was assigned to one or two storeys, however, considering the practical aspect of the study and the lower influence of this approach on the structural performance of the buildings, it was decided to assign at least two storeys and more to each concrete grade.

The effect of varying the column sizes was then investigated. As one axis was already relatively large, the column thicknesses were investigated in 50 mm increments, from 250 mm up to 500 mm. Grade C40/50 concrete and a slab thickness of 275 mm were assumed in the models.

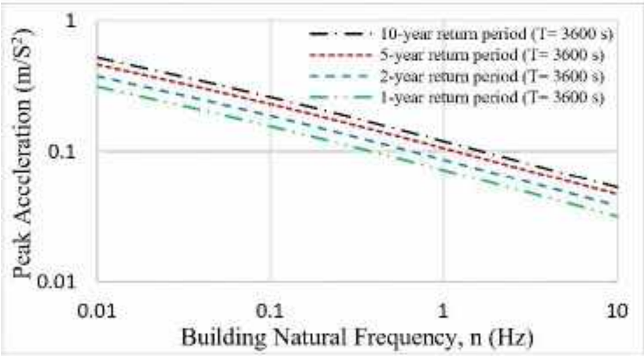



Fig. 7 Limits for horizontal peak acceleration based on Breeze (2011)

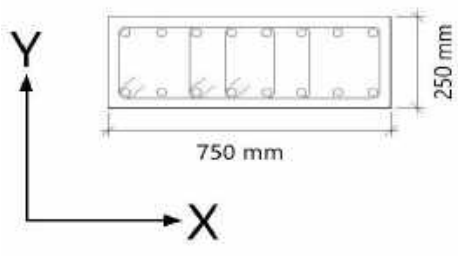
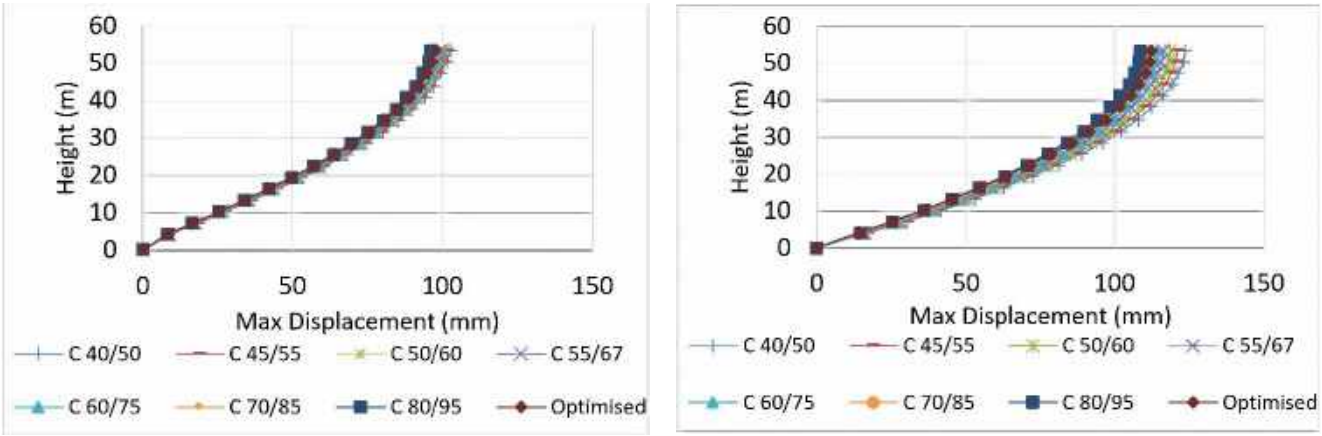



Fig. 8 Column cross-section

Different column shapes were also studied to investigate their influence on the building's structural performance, including punching shear. The shapes examined were circular, square and rectangular, each providing the same cross-sectional area (around 0.375 m$^2$). It was assumed that the concrete was grade C40/50 and the slab thickness was 275 mm. Finally, the slab thickness was investigated and both 275 mm and 300 mm thicknesses were simulated. In these analyses, grade C40/50 was again adopted and the columns were assumed to be rectangular with cross-sectional dimensions of 750 × 250 mm. Table 3 summarises the parameters studied in the structural analyses including eight concrete grades for columns, one concrete grade for flat slab, six different column sizes, three column shapes and two slab thicknesses.

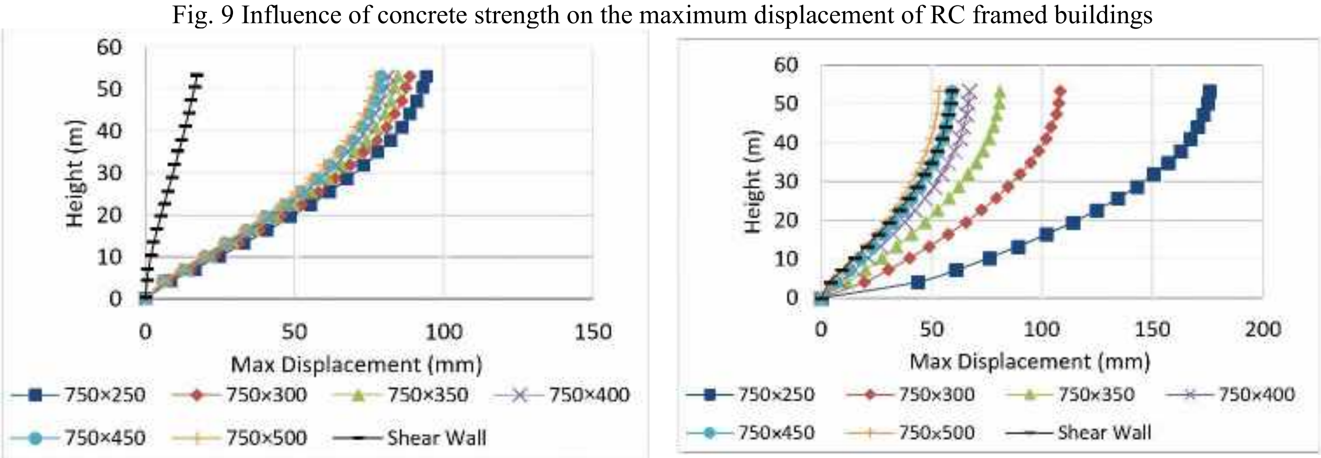
### 2.3 Design check

Increasing a building's height can give rise to various issues, including excessive lateral displacements, interstorey drift, acceleration and punching shear, which, if limitations are not taken into account, might result in the building's failure. Therefore, the numerical analyses in this study were conducted in accordance with the provisions in Eurocode 2 Part 1-1 (2014) and Eurocode 0 (2017), which are as follows:




(a) -direction (b) Y-direction

Fig. 9 Influence of concrete strength on the maximum displacement of RC framed buildings




(a) X-direction (b) Y-direction

Fig. 10 Influence of column size on the maximum displacement of RC framed buildings

- Horizontal vibrations can have significant effects on the comfort of occupants if they lead to excessive accelerations. Human response to a building's movements is a multiple psychological phenomenon, which is usually determined by acceleration (Banks *et al.* 2014). To assess a building's allowable acceleration in which the occupants' comfort is considered, a number of different guidelines are available (National Building Code of Canada, NBCC, Part 4, 2010, Melbourne and Palmer 1992) and these define limitations for residential and office occupancy.

Currently, there is no defined limitation within the Eurocodes for occupants' comfort. According to Banks *et al.* (2014), the standard values for a 10-year return period of motion subjected to wind actions are:

- 10 to 15 milli-g (an acceleration unit that is equal to 1 cm/s$^2$) for residential occupancy; and

- 20 to 30 milli-g for office occupancy.

The acceleration can be determined using Eq. 1:

$$a = \frac{2\pi^2 \times f^2 \times d}{g} \tag{1}$$

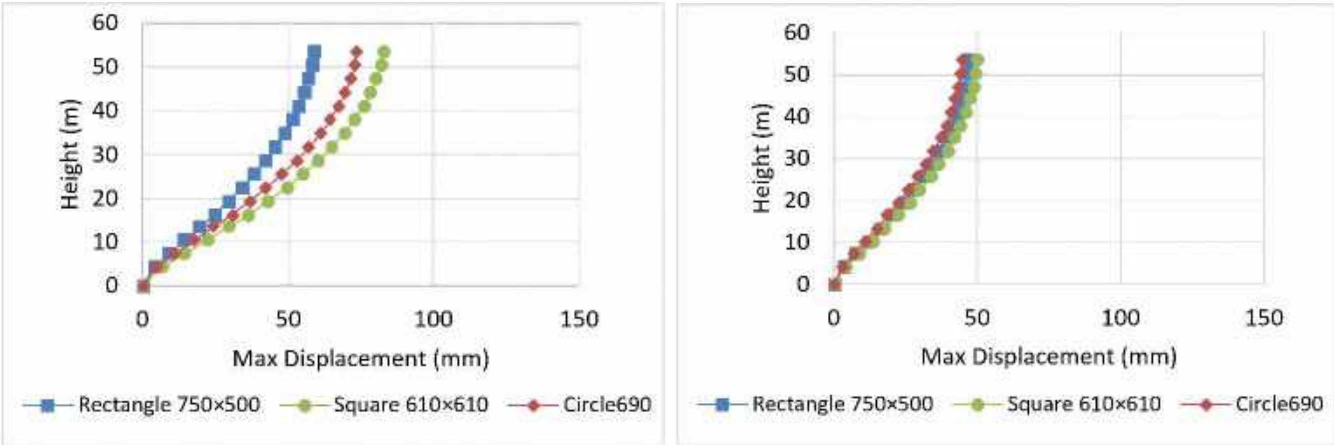
in which a, f, d and g represent acceleration, natural frequency (Hz), maximum displacement (m) and gravitational acceleration (m/s$^2$), respectively.

Currently, the Melbourne criteria is the most commonly used criteria for the design and evaluation of wind-generated horizontal acceleration in the UK buildings (Breeze 2011). Eq. 2 presents an equation for the determination of the threshold for (un-weighted) peak horizontal acceleration:

$$a = \sqrt{2 \ln nT}\left(0.68 + \frac{\ln R}{5}\right)\exp(-3.65 - 0.41 \ln n) \tag{2}$$

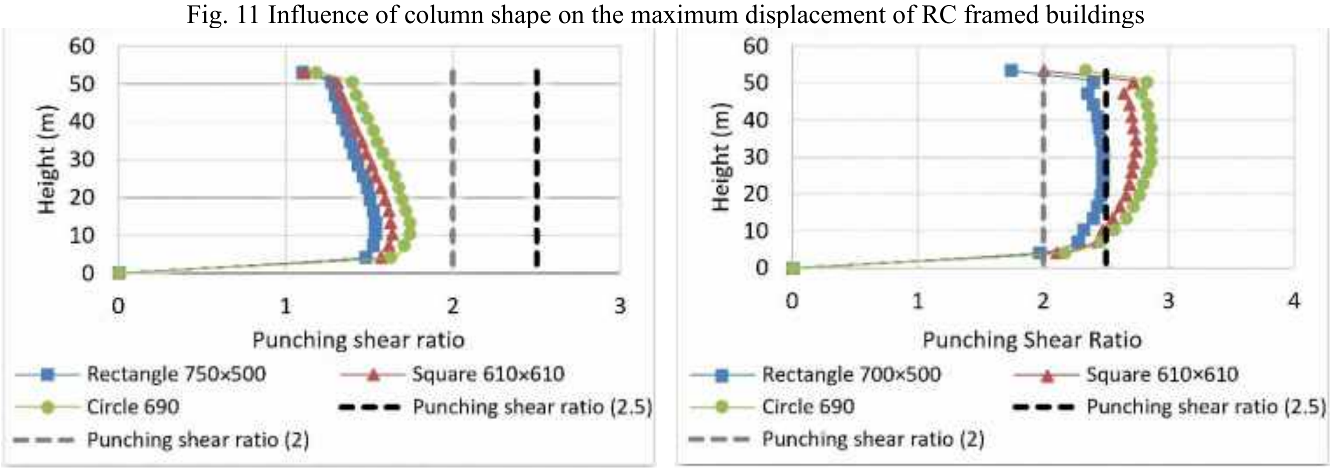
in which a, n, R and T represent acceleration (m/s$^2$), natural frequency (Hz), the return period (years) and time duration (seconds), which takes the nature of the wind action into account. In some countries such as the United States, T is assumed to be 10 minutes (600 seconds), due to the dominant storm activities. On the other hand, in the UK, since the storms typically occur over a longer time period, T is assumed to be 60 minutes (3600 seconds). The horizontal peak acceleration shown in Fig. 7 are obtained from Eq. 2.

- Lateral forces can cause horizontal displacements in a building, which, depending on the magnitude of the displacements, may result in severe damage to the building and its facade. It is also essential to control these displacements for non-structural elements, such as the connections between blocks and stud partition walls. To control the overall horizontal displacements and interstorey drifts, EN 1990 defines a limitation of H/500, where H is the overall storey height, to minimise the lateral movements (EN 1990, 2017).




(a) X-direction (b) Y-direction

Fig. 11 Influence of column shape on the maximum displacement of RC framed buildings




(a) Internal column (b) Corner column

Fig. 12 Influence of column shape on the punching shear ratio of RC framed buildings

- Flat slabs, despite their economic advantages, require design checks for deflections and punching shear. The calculations are provided in Appendix A.

## 3. Results and discussion

### 3.1 Concrete grade

The results for the influence of concrete grade were obtained using 750 × 250 mm column size (Fig. 8) and 275 mm slab thickness on a building's lateral displacements in each storey subjected to wind action in the X and Y directions. The results are presented in Fig. 9.

It can be observed that the building stiffness was gradually enhanced as the concrete grade increased, and the maximum displacement in both directions was reduced accordingly. It also was evident that the optimised concrete-grade performance was quite close to C80/95 in both directions, which was more favourable in terms of economy by utilising a variation of concrete strength classes instead of using a high-strength concrete such as C80/95 for the whole structure.

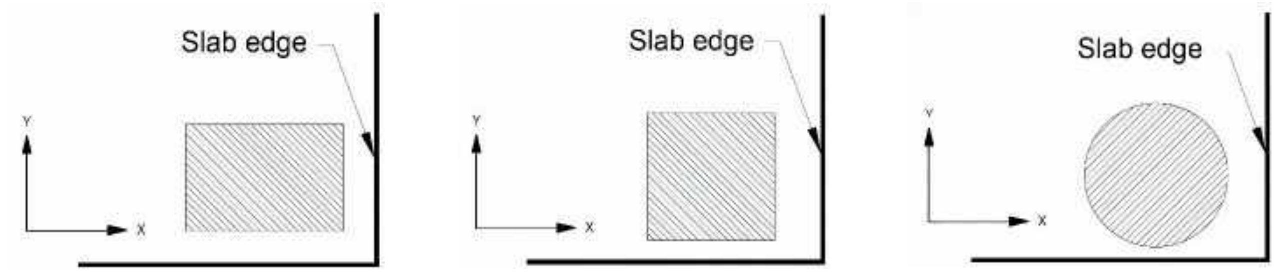
### 3.2 Column size

The results for the maximum displacements in buildings with different column sizes and another one with shear walls (200 mm thickness) were obtained using concrete-grade C40/50 and 275 mm slab thickness subjected to the wind load in X and Y directions, and are illustrated in Fig. 10. It was evident that by increasing the column size, the maximum displacement was reduced; however, the trend was not the same in both directions. Due to the similarities in the columns' dimensions in the X direction, the range of displacements was lower than that of the Y direction. In both directions, the maximum displacement decreased with the increment of column size from 750 × 250 mm to 750 × 500 mm. Furthermore, the shear walls showed quite stiff behaviour in the X direction with the least displacement, and in the Y direction, it had the second-least displacement. The difference between shear walls in the X and Y directions was due to their orientation in the building's design (Fig. 10).

### 3.3 Column shape

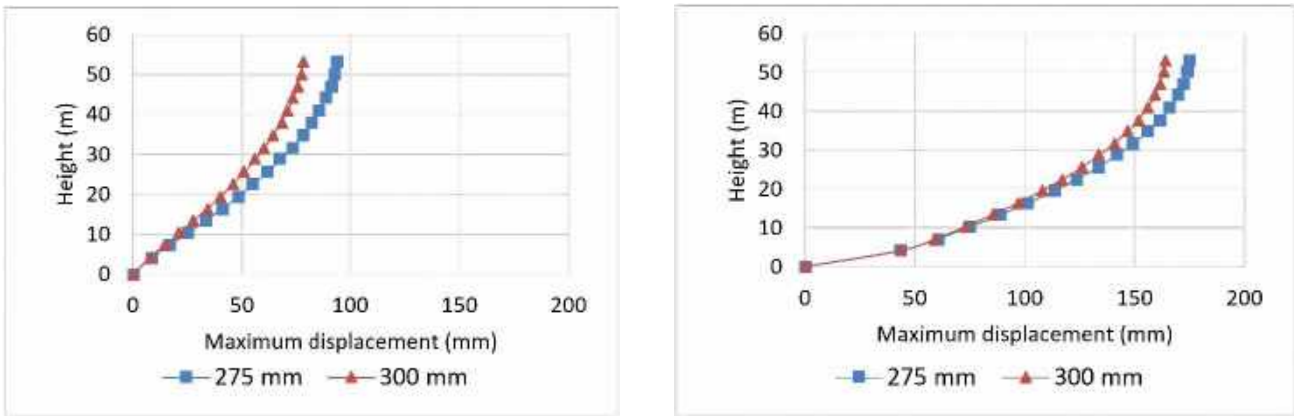
#### 3.3.1 Maximum displacement

Fig. 11 provides a summary of the column's shapes effect using concrete grade C40/50 and 275 mm slab thickness on the building's lateral movements subjected to the wind load in the X and Y directions. The selected column section sizes were the most-used shapes for



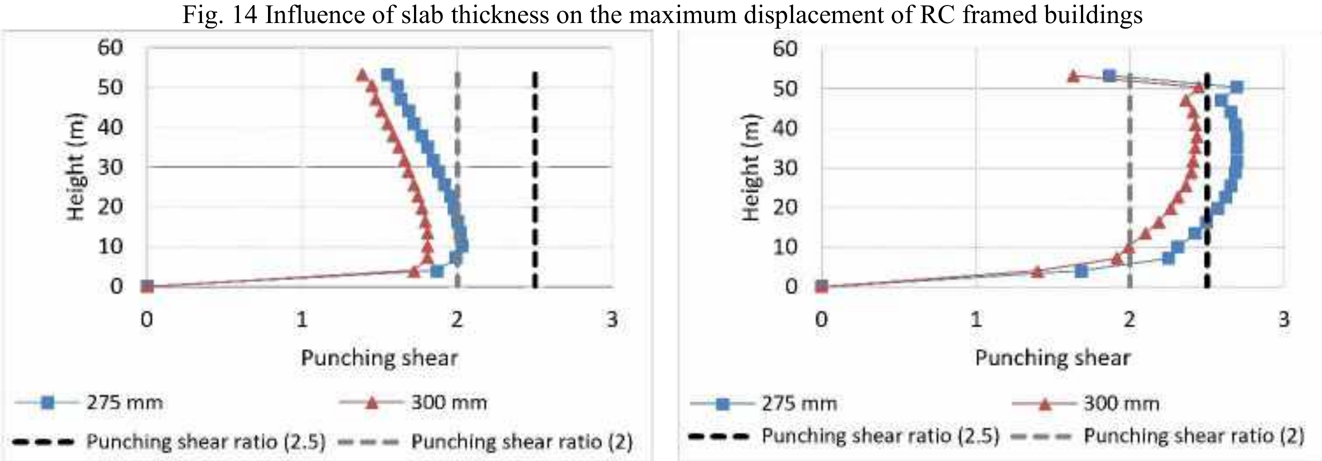

(a) Rectangle column (b) Square column (c) Circle column

Fig. 13 Corner columns with different shapes




(a) X-direction (b) Y-direction

Fig. 14 Influence of slab thickness on the maximum displacement of RC framed buildings




(a) Internal column (b) Corner column

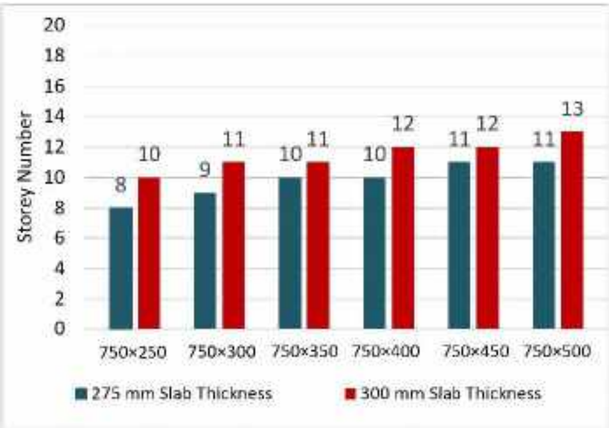
Fig. 15 Influence of slab thickness on the punching shear ratio of RC framed buildings

columns in the construction and all of the three shapes had the same cross-sectional area. It was evident that, in both directions, the shape with the larger dimension resulted in stiffer behaviour and lowered lateral movements, compared to the other shapes. In the X-direction, the rectangular shape had the lowest displacements, while the square cross-section had the highest one. On the other hand, the lateral displacements in the Y-direction were different, in which the circle had the lowest lateral movements, and the square had the highest movements.

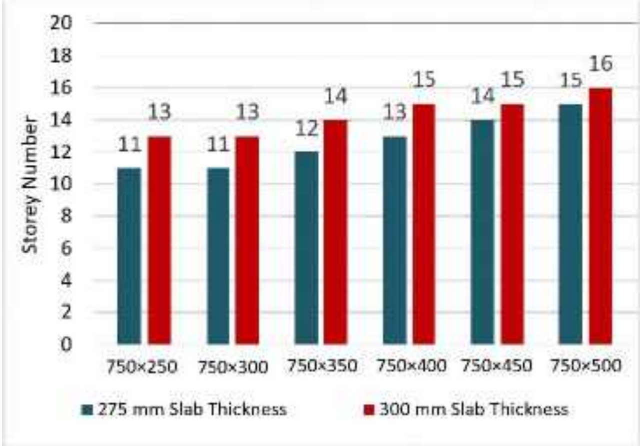
### *3.3.2 Punching shear*

There are several thresholds for punching shear ratio ($V_{Ed}$/VRd,c) in design guides, by defining limiting ratio for shear force over allowable shear without reinforcement, and two of them are utilised in this study. The UK National Annex suggests to limit the punching shear ratio to 2.5, while this value for Eurocode 2 Part 1-1 (2014) is 2. The results for the influence of column shape on the punching shear in internal and corner columns are presented in Fig. 12.

As it was demonstrated in Fig. 12, punching shear failure, as a significant issue in flat slabs, is more likely to happen in corner columns than edge or internal columns (Sacramento *et al.* 2012, Aalto and Neuman 2017). Moreover, Fig. 12a demonstrated in the internal columns the punching shear ratios were within the safe range, both with 2 and 2.5 punching shear ratio limits, while, for the




a) $V_{Ed}/V_{Rd,c} = 2$




b) $V_{Ed}/V_{Rd,c} = 2.5$

Fig. 16 Maximum height with various sections

Table 4 Concrete strength grade variation for each column size

| Concrete grade | **750 × 250** | **750 × 300** | **750 × 350** | **750 × 400** | **750 × 450** | **750 × 500** |
|---|---|---|---|---|---|---|
| C80/95 | Storey 1-3 | Storey 1-3 | Storey 1-3 | Storey 1-3 | Storey 1-3 | Storey 1-3 |
| C70/85 | Storey 4-6 | Storey 4-6 | Storey 4-6 | Storey 4-6 | Storey 4-6 | Storey 4-6 |
| C60/75 | Storey 7-10 | Storey 7-9 | Storey 7-9 | Storey 7-9 | Storey 7-9 | Storey 7-9 |
| C55/67 | - | Storey 10-11 | Storey 10-11 | Storey 10-12 | Storey 10-12 | Storey 10-13 |
| C50/60 | - | - | - | - | - | - |
| C45/55 | - | - | - | - | - | - |
| C40/50 | - | - | - | - | - | - |

corner columns (Fig. 12b), only the rectangular shape was lower than the threshold of 2.5 punching shear ratio, and none of them passed the 2 punching shear ratio. Besides, the shape of the columns' impact was more evident in the corner columns. Fig. 13 shows where the control perimeter around the loaded area in the rectangular shape was more than the others, providing more space to distribute the applied loads.

It is possible to overcome the punching shear failure for corner columns with circular or square cross-sections by introducing Shear rails (Punching shear reinforcement), but this option was not considered here, as implementation of the Shear rails lead to increase the overall construction cost (Max Frank 2020).

### *3.4 Slab thickness*

#### *3.4.1 Maximum displacement*

Fig. 14 illustrates the results for the impact of flat slabs' thickness using 750 × 250 mm column size and C40/50 concrete grade on the building's lateral movements subjected to the wind actions in the X and Y directions.

It can be observed that by increasing the slab's thickness by 25 mm, the building became stiffer, and the lateral displacements in the X and Y directions reduced accordingly. In both directions, the building with a 300 mm flat slab thickness resulted in lower lateral movements (around 14 mm displacement), compared to the building with a 275 mm flat slab thickness.

#### *3.4.2 Punching Shear and slab thickness*

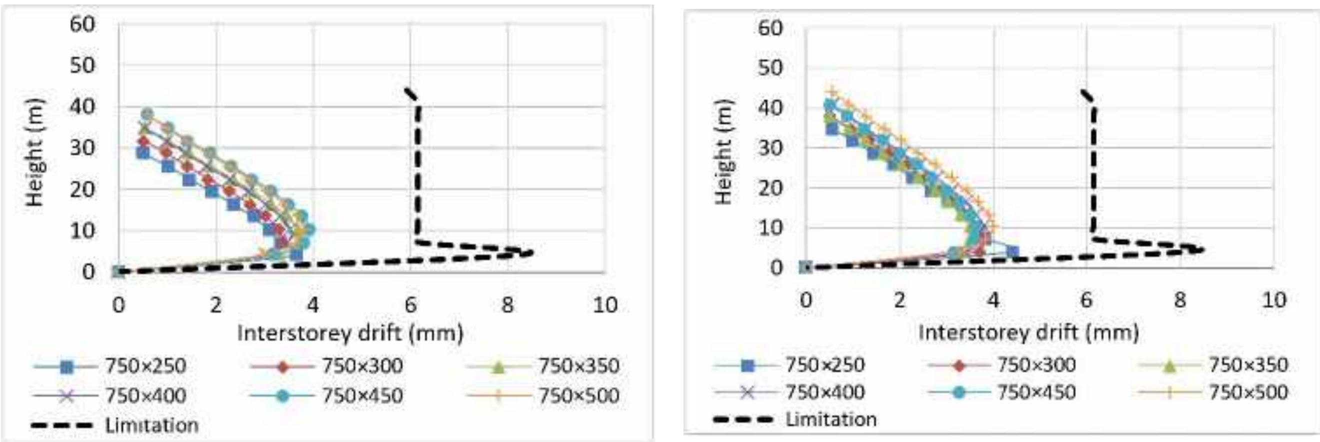
The results for the influence of increasing the slab thickness on punching shear are presented in Fig. 15.

As shown in Fig. 15, the punching shear ratios in the internal columns were lower than the corner columns. Furthermore, increasing the slab thickness by 25 mm could lead to lower punching shear ratios within the safe range defined by Eurocode 2 Part 1-1 (2014). However, compared to the internal columns, the corner columns failed to pass the 2 punching shear ratio limits, showing the vulnerability of flat slabs in punching shear failure.

### *3.5 Maximum height*

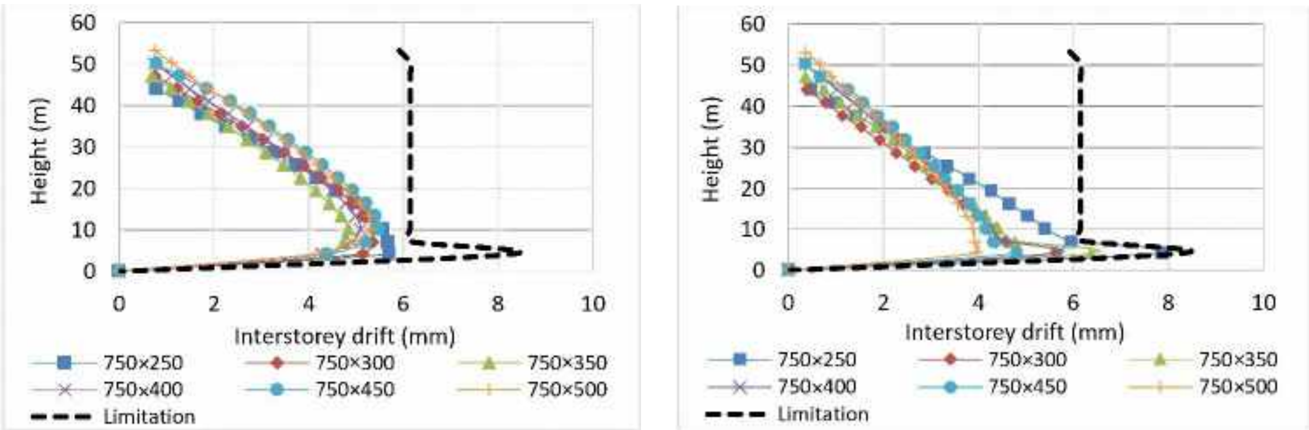
In the previous section, the impacts of individual factors were assessed in relation to maximum allowable height. In this section, the optimised concrete grade with different column sizes (the details are shown in Table 4), slab thicknesses and punching shear ratios ($V_{Ed}/V_{Rd,c}$) was investigated to achieve the maximum height.

Fig. 16 presents the results for the maximum height in an RC moment-resisting frame. The results demonstrated the maximum height for each column section; by increasing the slab thickness, the buildings could be built up to 2 more storeys. Furthermore, a comparison of the two graphs showed that increasing the punching shear ratio (between 2 and 2.5 punching shear ratios) directly increased the maximum height up to 3 storeys. Therefore, for the investigated building with the optimised concrete grade, 750 × 500 mm column section size and flat slab with a



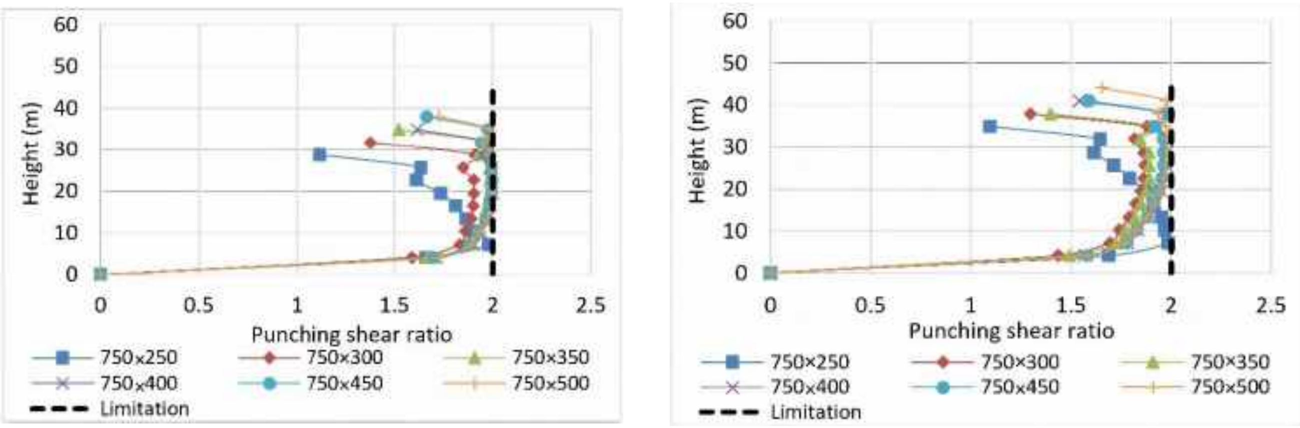

(a) X-direction (b) Y-direction

Fig. 17 Influence of column size on the interstorey drift of RC framed buildings (punching shear ratio 2)




(a) X-direction (b) Y-direction

Fig. 18 Influence of column size on the interstorey drift of RC framed buildings (punching shear ratio 2.5)




(a) 275 mm slab thickness (b) 300 mm slab thickness

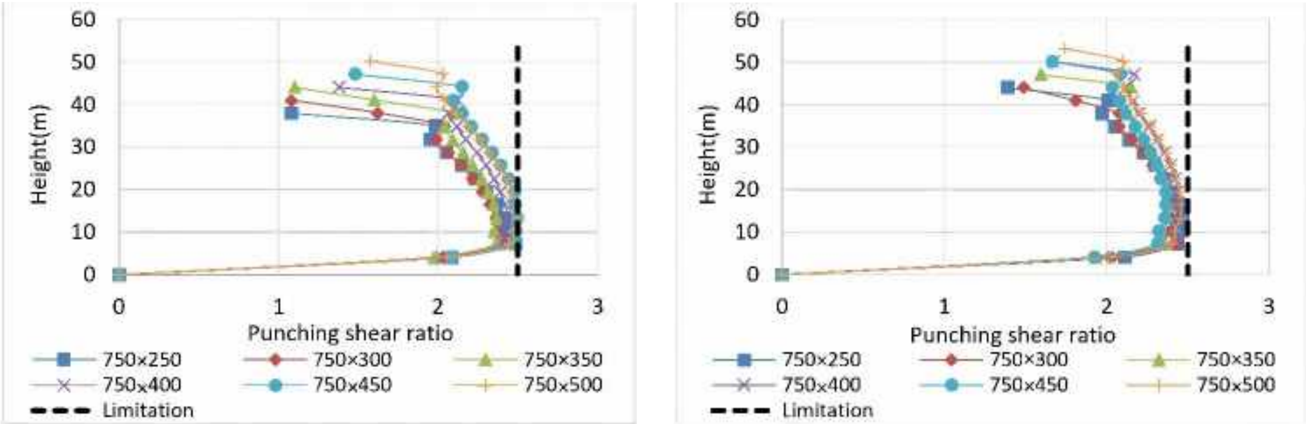
Fig. 19 Influence of column size on the punching shear ratio of RC framed buildings (punching shear ratio 2)

thickness of 300 mm could reach up to 16 storeys with 2.5 punching shear ratio, and up to 13 storeys with 2 punching shear ratio. These were the maximum heights for the proposed architectural plan.

#### *3.5.1 Design check*

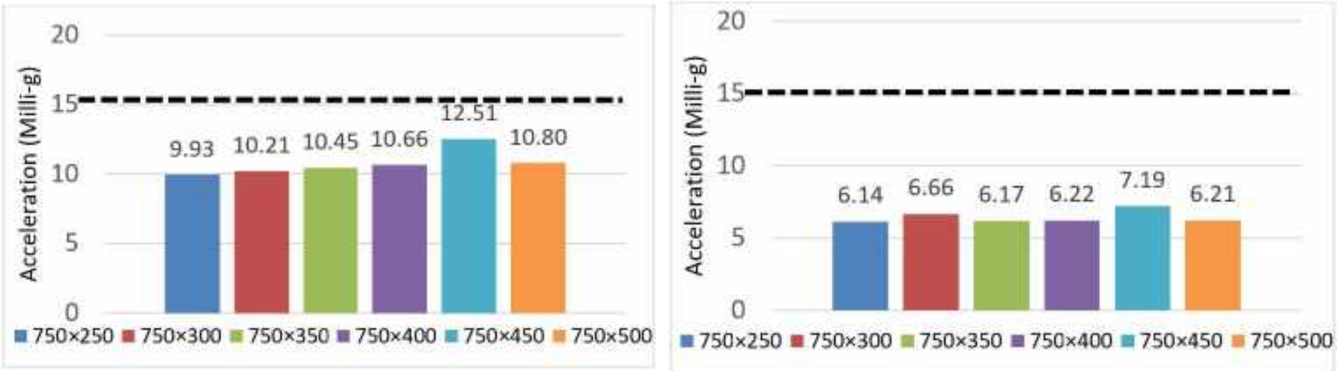
*Interstorey drift*

The comparison for the interstorey drift in the designed buildings with different column sizes and 300 mm slab thickness and punching shear ratio limits of 2 and 2.5 are illustrated in Figs. 17 and 18. It is shown that these designs were still within the safe range defined by Eurocode 2 Part 1-1 (2014) in both X and Y directions. The fluctuation in interstorey drift's limit was due to the change in storey height between the first storey and other storeys (from 4.125 m to 3.075 m).



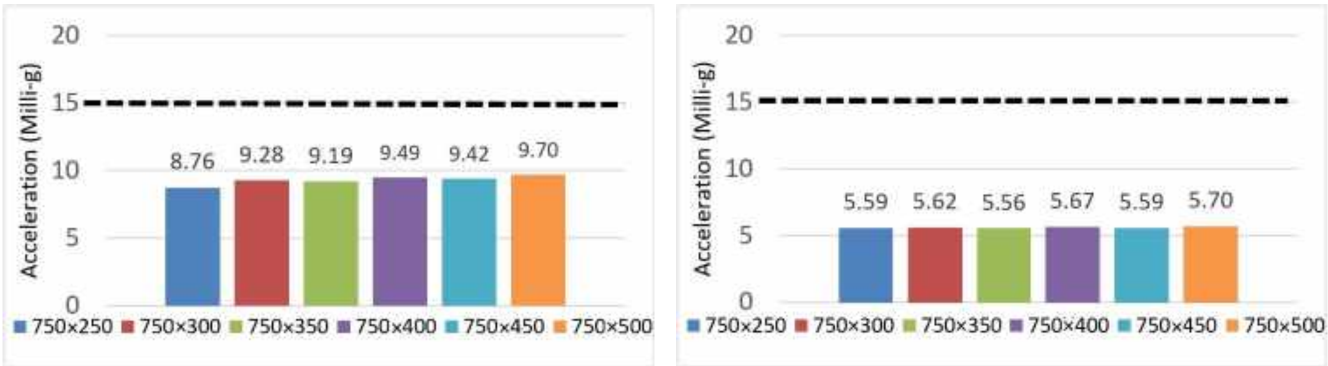

a) 275 mm slab thickness b) 300 mm slab thickness

Fig. 20 Influence of column size on the punching shear ratio of RC framed buildings (punching shear ratio 2.5)




(a) X-direction (b) Y-direction

Fig. 21 Influence of column size on the horizontal acceleration (NBCC) of RC framed buildings (punching shear ratio 2)




(a) X-direction (b) Y-direction

Fig. 22 Influence of column size on the horizon tal acceleration (NBCC) of RC framed buildings (punching shear ratio 2.5)

*Punching shear ratio*

The results for punching shear ratios in each case with 275 mm and 300 mm slab thickness are presented in Figs. 19 and 20. Changing the slab thickness can have an impact on the punching shear ratio. Punching shear ratios are given in Figs. 19 and 20 for the buildings with slab thicknesses of 275 mm and 300 mm, and it can be observed that the punching shear ratios were within the safe range (2 and 2.5 punching shear ratio limits) for 13 (39 m)- and 16 (48 m) - storey buildings.

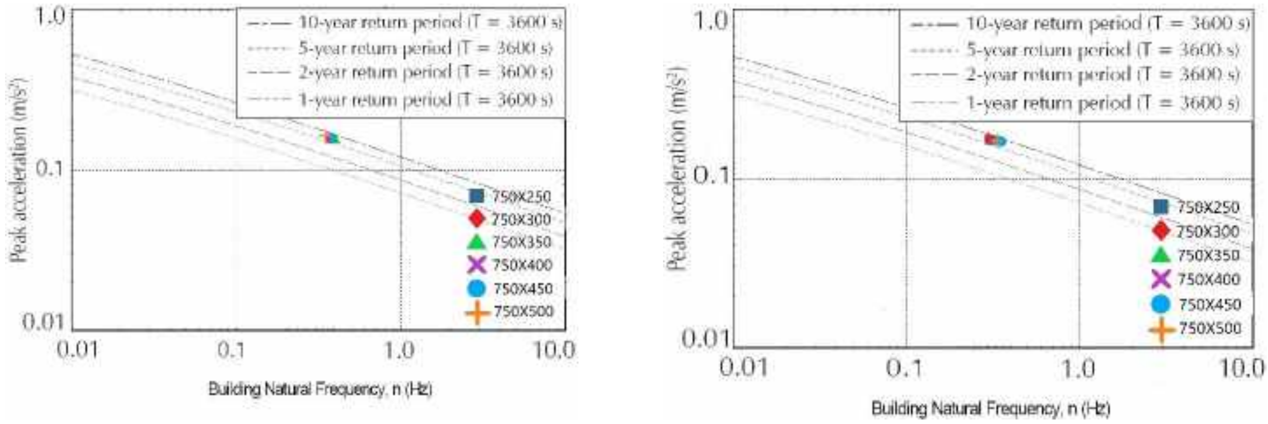
*Acceleration*

Since the maximum height for the building was achieved with 300 mm flat slab thickness, the results for the occupants' comfort measured in the top floors of each building are according to the following:

*NBCC Part 4 limitations:*

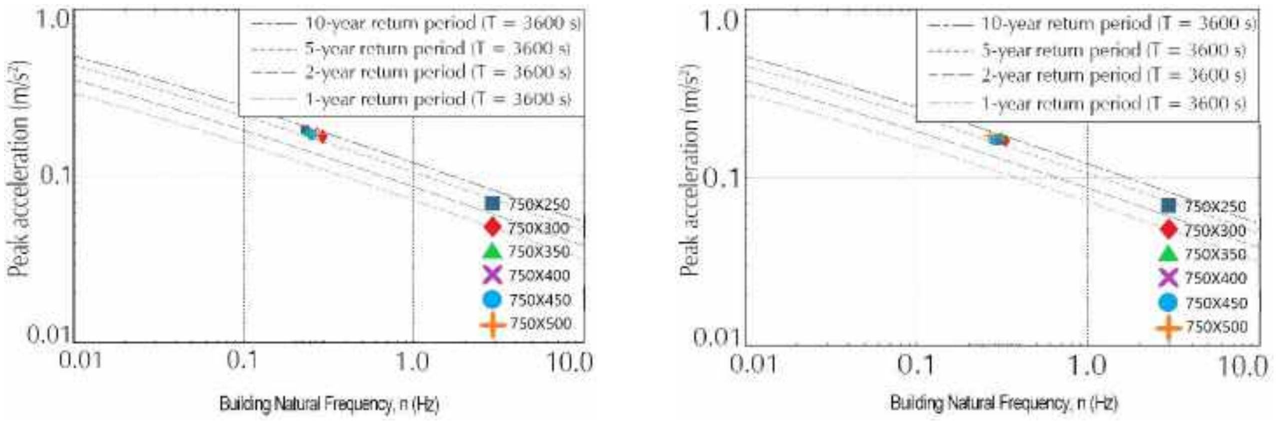
The horizontal acceleration threshold for residential occupancy with a 10-year return period is 15 milli-g, which is shown in Figs. 21 and 22. In this part, only the buildings with 300 mm flat slab thicknesses were chosen.

In Figs. 23 and 24, the horizontal accelerations in all buildings were within the acceptable limit, ranging from 9.93 to 12.51 milli-g in X-direction and 6.14 to 7.19 milli-g in Y-direction for 2 punching shear limit and 8.76 to




(a) X-direction (b) Y-direction

Fig. 23 Influence of column size on horizontal acceleration (Melbourne) in RC buildings (punching shear ratio 2)




(a) X-direction (b) Y-direction

Fig. 24 Influence of column size on horizontal acceleration (Melbourne) in RC buildings (punching shear ratio 2.5)

9.70 milli-g in X-direction and 5.56 to 5.70 milli-g in Y-direction for 2.5 punching shear limit. The difference between the accelerations in X and Y directions was due to the difference between the dimensions of columns in each direction being 750 mm in X-direction and ranging from 250 mm to 500 mm for Y-direction.

*Melbourne Criteria:*

The horizontal acceleration threshold for residential occupancy with a 10-year return period of exceedance in Melbourne criteria is represented in Figs. 23 and 24, and the results were based on the natural frequencies taken from the simulations and equation 2. In this part, only the buildings with 300 mm flat slab thicknesses were chosen.

It can be observed that in both directions for 2 and 2.5 punching shear ratio limits, the buildings with a maximum overall height ranging from 13 to 16 storeys were acceptable for the residential occupancy with a 10-year return period in both criteria, and the residents' comfort was not compromised.

## 4. Conclusions

In this study, the feasibility of the maximum height for an existing UK residential building, which is designed and constructed with shear walls, is investigated without shear walls when the building is subjected to wind-induced forces. To achieve the maximum height, contributory factors including concrete grade, column size, column shape and slab thickness are taken into account, and their impact on the building's structural performance is assessed according to Eurocode 2 Part 1-1 (2014), using ETABS software.

Based on the acquired results of this research, the following conclusions can be drawn:

- Increasing the concrete grade results in stiffer behaviour in a building and reduces the lateral displacements.
- Optimising the concrete grade in the building is a more practical approach and can result in acceptable structural behaviour.
- It was evident that there is a direct relationship between a column section size and the lateral displacements, in which the increase of column size reduces the lateral displacements.
- Different column shapes can change the buildings' lateral movements and influence the punching shear ratio, in which the rectangular shape achieved the lowest ratio.
- The change in flat slab thickness can directly affect the lateral stiffness of a building, which means by increasing the thickness, the lateral displacements and the punching shear ratio reduce.
- Increasing the slab thickness can add up to 2 more

storeys to the maximum height in the reinforced concrete frame building.

- In this study, the governing limitation was punching shear ratio, and if the simulations were based on $V_{Ed}/V_{Rd,c} = 2$, the maximum height would have been reduced to 13 storeys to comply with Eurocode 2 Part 1-1 (2014) limits.
- The reference building was designed and constructed as a five-storey RC frame building with shear walls, and the achievements of this study demonstrated a practical potential to increase buildings' height only by removing the shear walls and optimising the key factors.
- This study demonstrated that depending on the architectural plan and the influencing factors, it is feasible to achieve the buildings' full potential in structural performance.

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*CC*

## Appendix A

*Flat slab deflection*

Eurocode 2 Part 1-1 (2014) provides several approaches for checking the deflection in flat slabs. In this study, the span to depth ratio was used. The procedure to calculate the span to depth ratio was provided by Goodchild (2009). In this 16-storey building, the slab between EF-3 to EF-5 due to its wider span was chosen.

$$\text{Allowable l/d} = 49 \geq \text{Actual l/d} = 36 \rightarrow \text{Pass} \quad (1)$$

In which L and d are the length and depth of flat slab, respectively.

It can be seen that deflection in the flat slab was not an issue, and the values were within the safe range defined by Eurocode 2 Part 1-1 (2014).

*Punching shear*

In the 16-storey building, column F1 in the fifth storey (as the worst case with the highest punching shear value) was chosen based on the given procedure by Goodchild (2009). The following values were taken from ETABS design results.

Design shear stress ($V_{Ed}$) = 1.21 N/mm2

Concrete shear stress capacity ($V_{Rd,c}$) = 0.49 N/mm2

Punching shear ratio = 2.45

According to Eurocode 2 Part 1-1 (2014) CL 6.4.3 (2), the following checks should be carried out:

1. Design effect shear $V_{Ed} \leq$ Design resistance shear $V_{Rd,max} \rightarrow 1.21$ kN/m$^2 \leq 3.6$ kN/m$^2$ **Pass** (1)

$V_{Rd,max} = 0.4\,\upsilon f_{cd} = 0.4 \times 0.53 \times 17 = 3.6$ kN/

Strength reduction factor for concrete cracked in shear $\upsilon=0.6(1-f_{ck}/250) = 0.6 \times (1-30/250) = 0.53$ kN/m$^2$

2. $V_{Ed} \leq V_{Rd,c} \rightarrow 1.21$ kN/m$^2 \geq 0.6$ **Punching shear reinforcement is required** (2)

In addition, the United Kingdom NA recommends $V_{Ed} \leq 2.5V_{Rd,c}$ (without shear reinforcements), which in this case was 1.21 kN/m$^2 \leq 1.22$ kN/m$^2$