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# Enhancing Sustainability of Low to Medium-Rise Reinforced Concrete Frame Buildings in the UK

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*Received 7 January 2023, Accepted 20 February 2023*

**Abstract.** In low-to-medium-rise RC frame buildings, to sustain gravity and lateral loads, mostly, two main load-bearing systems are used; moment-resisting (sway) and moment-resisting with shear walls. In the UK, the use of shear walls has now become an expansive technique in these types of RC frame building, as indicated by The Concrete Centre. Furthermore, considering the structural behaviour and performance of shear walls in low-to-medium-rise RC frame buildings, in the areas with low risk of seismic load they may negatively affect the economic and environmental efficiency of the construction without any substantial improvement to the structural performance of the building. In this study, several steps will be undertaken to enhance the sustainability of construction in RC frame buildings, including the possibility of removing shear walls in low-to-medium-rise buildings which can directly influence the consumption of aggregates (as natural resources) and cement in concrete and speed up the construction process. Reducing the amount of concrete in the construction of RC frames may be a feasible option that would have a significant effect on the reduction of $CO_2$. emissions from the construction industry. The results indicate that, considering the practical side of the design, it is advisable to design the buildings up to 12 storeys using moment-resisting frame with flat slab. The results, also, shows 12% reduction of consumed concrete and 4.6% of $CO_2$. emission in moment-resisting frames compared to the frame with shear walls. According to the outcome of this research and in collaboration with the Concrete Centre a design guide is developed for the use and implementation of moment-resisting frames in the UK construction industry.

**Keywords:** Moment-resisting, shear wall, sustainability, $CO_2$. emission, displacement, punching shear,



## 1. Introduction

In the construction industry, moment-resisting (unbraced) frames and moment-resisting frame with shear wall are the two most common load-bearing systems in reinforced concrete structures where both systems can sustain gravity and lateral loads with required lateral stiffness and structural strength. Both systems have several advantages and drawbacks that require detailed technical considerations. It is obvious that each system could provide an economical solution up to a specific height. To investigate the most suitable system for RC structures various factor including sustainability, structural performance, and construction method must be considered.

It is well established that shear walls are one of the most effective resistance components to use against lateral forces in reinforced concrete structures. The characteristics of the shear walls control the performance of the building against the lateral loads (Chandurkar and Pajgade, 2013). Typically, shear walls are necessary for the mid to high-rise buildings to withstand wind or earthquake actions. Shear walls are used to withstand lateral forces acting on buildings in active seismic regions due to their substantial lateral stiffness and strength.

Musson and Sargeant (2007) reported that the Peak Ground Acceleration (PGA) value is less than 0.02g, which indicates that there is very low seismic activity in the UK; hence there is no need to consider the effect of earthquake load on the structures (BS EN 1998-1, 2013). On the other hand, Northern European countries, including the UK experiencing large wind load which can generate high lateral load on medium and high-rise structures (Archer and Jacobson 2005, Global Wind Atlas 2018). In addition, the intensity of wind-induced loads on the structures can be

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influenced by the height of the buildings. Raskin and Rejendram (2013) reported that the use of concrete shear walls in high-rise buildings could considerably improve required strength and stiffness. In the UK, despite those facts, shear walls have been commonly used, particularly in buildings with three up to six storeys that are known as low-rise building (Emporis Codes, 2008 and 2009; Banks et al., 2014 and NFPA, 2016). The use of shear walls in low-to-medium rise structures has been criticized by the construction industry, as reported by the Concrete Centre. It worth mentioning that, previous experiments have demonstrated that the height of the building will directly influence the applied load, which makes this aspect important to the design (Ghorpade, A. and Swamy, B. 2018). As discussed already, the focus of this research is on low to mid-rise buildings that face smaller lateral forces compared to high-rise buildings. As a result, it might be possible to develop low to mid-rise RC frame buildings without shear walls. From the other side, as discussed, regardless of design criteria, shear walls may have a negative impact on the sustainability of construction, environment and economy; hence it is very desirable to remove them from some types of RC structures up to a certain height without risking the safety of the inhabitants.

To study the efficiency of RC frame structures either with or without shear walls subjected to lateral forces, numerous experimental and numerical studies have been conducted over last few decades. A comparative analysis on multi-storey RC structures with and without shear walls have been conducted by Chandurkar and Pajgade (2013), Thakur and Singh (2014) and Aainawala and Pajgade (2014) using STAAD and ETABS software. In these studies, four types of structures were modelled; three with shear walls in various locations and one without shear walls. As expected, due to high stiffness of shear walls, in all cases frame with shear walls experienced considerably less lateral displacement and internal forces in the columns and beams compared to the frame without shear walls. Main deficiency of these studies is that the overall lateral displacement and the permissible lateral displacement based on the design codes were not considered. Furthermore, to identify their distinctions without considering the lateral limitations of those codes, a numerical study has been conducted by Jayalekshmi and Chinmayi (2015) on the behaviour of RC frames with and without shear walls in multiple design codes i.e., IS 1893 and IBC. To define an optimum system for flat slab structures and the efficiency of this system with and without shear walls subjected to gravity and lateral loads, a numerical study was performed by Ghorpade and Swamy (2018) using Pushover analysis. The results indicate that, lateral displacement of the flat slab with shear walls is significantly less than flat slab without shear walls. They concluded that flat slab systems without shear walls are superior to RC frames with shear walls.

There are few studies on the efficiency of RC frame buildings subject to wind load considering arbitrary architectural plans, not real-world situations that ignore the constraints of lateral displacement in the design codes. Rasikan and Rajendran (2013) studied the performance of RC frame buildings with and without shear walls subjected to wind load using STAAD software. In this research two RC structures with different heights were considered. The results indicated that, regardless of their height, overall lateral displacement of structures with shear walls was much lower than that of a structure without shear walls and the permissible lateral displacement required by the code which shows the structures with shear walls fail to provide an economical solution.

One of the key requirements for constructing a cost-effective building is the ability of the structural system to tolerate a high lateral displacement before failure to be able to dissipate the absorbed energy using the maximum potential of the system. As indicated in the literature, due to significant in-plan lateral stiffness of shear walls, the lateral displacement of frame system with shear walls is significantly less than moment-resisting frame, hence prohibits the structure from dissipating the absorbed energies. This behaviour restricts the frame from taking part in lateral stiffness, which potentially affects the design and leads it to be conservative. However, the greater the stiffness of the structure, the higher the level of absorbed lateral forces with less ductility, which may potentially lead to the building's failure (Cao, Xue and Zhang, 2020) unless with a complex reinforcements' detailing at the end of shear walls. On the other hand, only small part of concrete at the ends of shear walls e.g., 15% of wall length are under compression and contributing to the second moment area and finally lateral stiffness of the shear walls. It can be concluded that, for low-medium rise buildings moment-resisting frame with shear wall will provide more expensive solution compared to the moment-resisting frame systems.

Due to high ductility, moment-resisting frame systems can dissipate more energy by large and periodic lateral displacement, but at the same time the perception of movement would be a vital aspect due to their ductile behaviour when subjected to lateral loading. To determine this effect on the safety of the inhabitants, numerous studies have been conducted in last decade. Hitchcock and Burton (2009) examined the perception of the vibration produced by wind action and concluded that there were no generally agreed requirements for occupant comfort, while the Concrete Centre (Banks et al., 2014) pointed out a variety of values used in North America for a 10-year return cycle.

Height of buildings can be considered as one of the key parameters effecting structural performance. For the low and medium-rise buildings, seismic loads dominate design of structures while wind action provides more critical internal forces for the high-rise buildings. The distinction between low and high-rise buildings is important in structural analyses since the structural performance of a building can change with the overall height. There is no universally agreed academic definition for low and high-rise buildings, but the ratio of overall height to the lowest dimension of the plan can be used to classify the buildings. The Concrete Centre (Banks et al., 2014) classifies buildings as high-rise if the ratio of height to the lowest lateral dimension is greater than 5:1. This research follows the concept of The Concrete Centre criteria, where the classification of buildings height is based on the dimensions of individual buildings rather than a predefined value for height limitation.

doi: 10.36828/efs.216 ISSN: 2753-4693

Due to considerable ductility of rigid frame compared to frame with shear wall systems, the second order (P- Δ) effect will be one of the main factors that effects structural behaviour in these types of structures and limits the maximum overall height. Concrete grade, column size and shape, also, have significant effect on structural performance of building (Murty et al. 2012, Singh et al. 2016). According to the literature, as the dynamic reaction might be deemed negligible, low-rise structures are designed based on strength requirements while in high-rise structures, stiffness dominates the structural design process.

It is obvious that, the capacity of planet to support life has hit an alarming limit, resulting in irreversible disruption to the planet, resources, inhabitants and ecosystem (Uher and Lawson, 1998; Our Common Future, 2008; Building a lowcarbon economy, 2008; Ortiz, Castells and Sonnemann, 2009; Yılmaz and Bakış, 2015). Accordingly, sustainability is becoming the most important issue worldwide; hence radical changes have been proposed to address global problems such as natural resource consumption, air pollution, climate change, waste production and environmental degradation in major cities. To this end, planet must now decrease emissions up to about 50 % by 2050 since significant environmental problems such as global warming and climate change have been caused by carbon dioxide ($CO_2$.) emissions and other greenhouse gases that are already affecting human lives (Building a low-carbon economy, 2008).
As construction industry provides infrastructure and buildings needed for the society and economy, it is responsible for a considerable amount of $CO_2$. emissions resulting from production of cement, other greenhouse gases (GHGs) added to the atmosphere due to material production process, construction process, renovation and demolition waste (González and García Navarro, 2006; Malhotra, 2010). According a comprehensive parametric FE analysis, Jayasinghe et al. (2020) showed that by relaxing the deflection limit in flat slabs the embodied carbon can be reduced by 20%. Furthermore, they suggested that optimizing column spacing, using lower grades of concrete, and minimizing slab depth will lead to further reduction in embodied carbon.

As stated, his situation must be changed before the exhaustible natural resources of the planet run out. Enhancing construction practises to minimise these harmful environmental effects has also drawn the interest of building experts around the world (Sev, 2009). In line with this global strategy, the UK Building Leadership Council and the UK Government released the Construction Industry deal in July 2018 and invested £420 m to fund the transformation of the industry (Davies, 2018). Although various fields in construction industry can be taken into account, but to reduce cement and aggregate in construction process, sustainability in the design of different structural elements in RC structures need to be considered as a priority.

## 2. Research Significances

After the Second World War, due to rapid economic developments in construction and huge demand in urban areas, natural resources such as fossil fuels, minerals, forests and lands, have been harmfully over-excavated. To address main global problems, such as natural resource consumption, air pollution, climate change, waste production and environmental degradation in major cities, radical changes have been proposed in last decade. In line with this strategic `plan, sustainability in construction is becoming a top priority issue in the design of all the projects.

This study covers significant gaps in the application of reinforced concrete moment-resisting frames and addressing of the identified issues in the construction industry with regard to development in the design of moment-resisting frames in high-wind regions. These advances include the prospect of eliminating shear walls in UK typical buildings, examining the effect of various factors on structural performance and the overall height constraint of reinforced concrete (RC) moment-resisting frames and producing a guideline for the design and construction of moment-resisting frames in the UK.

Furthermore, the significance of this research is to reduce cement and aggregate consumption to minimize $CO_2$. emission in RC structures by proposing a more sustainable load bearing system in the UK.

## 3. Research Strategies

To improve sustainability in the construction industry and reducing the consumption of concrete in RC structures, in this research, three different methods have been considered; the feasibility of eliminating shear walls and their effect on the behaviour of low-to-medium-rise residential buildings in the UK, using an innovative beam-to-column connection, and replacing fine aggregates with Polypropylene. In this paper, only the results of possibility of using moment-resisting system without shear walls for low-to-medium rise structures are presented and the results of two other scenarios will be presented in a separate paper.

To identify the key factors affecting the structural performance of shear walls in the moment-resisting frames, studying the influence of shear walls on the behaviour of RC structures is crucial. The results will help the potential for the removal of shear walls in low-to medium-rise RC moment-resisting frames and assessing the maximum overall height for RC frame buildings considering the various factors such as concrete grade, column size, column shape and slab thickness. The outcome this study led to produce a design guide for RC moment-resisting frames entitled 'How To Design Moment-Resisting Frames' as one of the Concrete Centre 'How-To' publications (MPA The Concrete Centre, 2018).

The significance of removing shear walls is investigated in three stages. (1) A broad comparative analysis is conducted to determine the feasibility of eliminating shear

doi: 10.36828/efs.216 ISSN: 2753-4693

walls in an existing UK residential building and its effect on the sustainability of construction when the building is subjected to wind-induced actions. Furthermore, building performance, cost-effectiveness and sustainability of construction are taken into consideration by using ETABS and Concept software. (2) Following the validation and effectiveness of removing shear walls, the global performance and application of the moment-resisting frame is investigated in various locations of the UK subjected to different wind loads. (3) A parametric study is conducted to identify the influence of various variables such as concrete grade, column size, column shape, column orientation, and slab thickness.

## 4. The Significance of Removing Shear Walls in Existing Low-Rise RC Frame Buildings

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 & 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. 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 will have 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, whilst maintaining and not compromising the occupants' safety.

For the first stage, to reflect a common multi-storey structure, an architectural plan for an established retirement community-based in Home Counties (Fig. 1 and 2) in the UK, provided by Couch Consulting Engineers was chosen for the comparative analyses. The building is a six-storey reinforced concrete structure with flat slab floors.

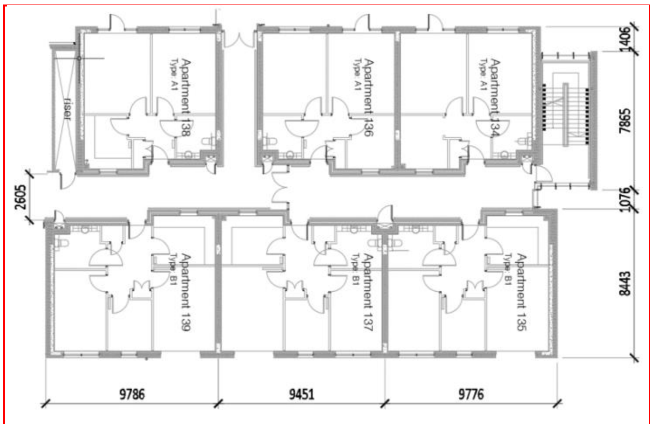



Fig. 1 Case study architectural plan

The comparative analyses are carried out using moment-resisting frame with and without shear walls just in X direction (Fig 3 and 4). The shear walls are replaced by the same column size as the current columns on each storey. The characteristics of the structures, materials properties, and gravity loads are described in Table 1 and measuring the static structural design of wind load (3-second load once in 50 years) could be found in EN 1991.1.4, and it has been summarized in Table 2 for Home Counties. It is to be noted that, lateral forces generated by seismicity in many countries dominate the resisting design of buildings, but in the UK wind actions are the critical lateral loads (Archer and Jacobson, 2005; Global Wind Atlas, 2018). For both load bearing systems, a linear analysis is used considering various criteria such as human response, serviceability limit state, interstorey drift, cost and construction duration, and various locations of the structures in the UK.

doi: 10.36828/efs.216
ISSN: 2753-4693

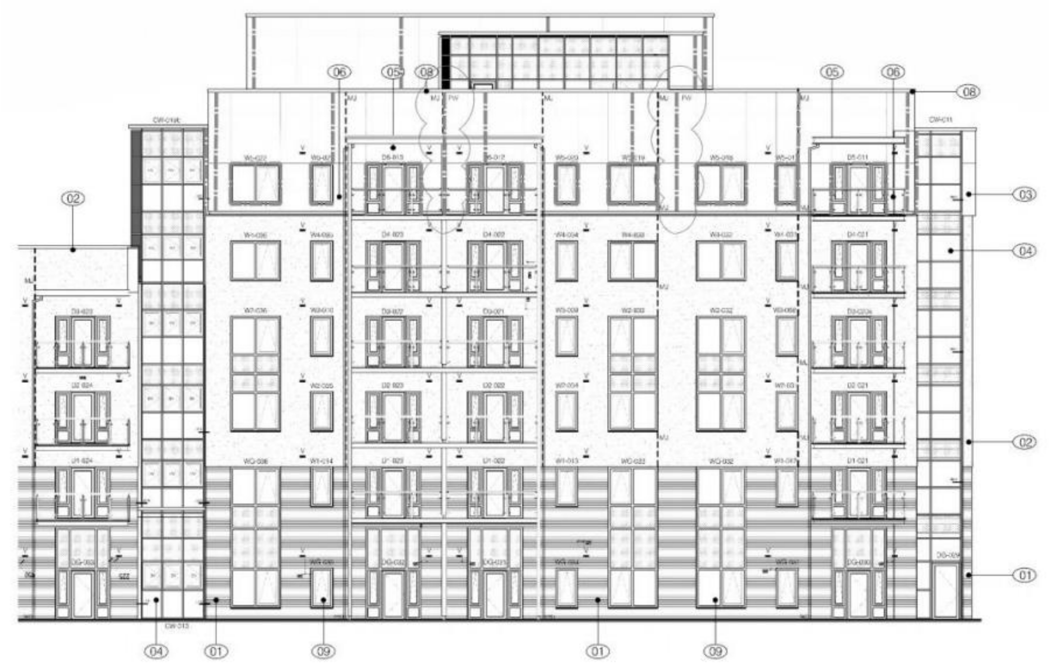
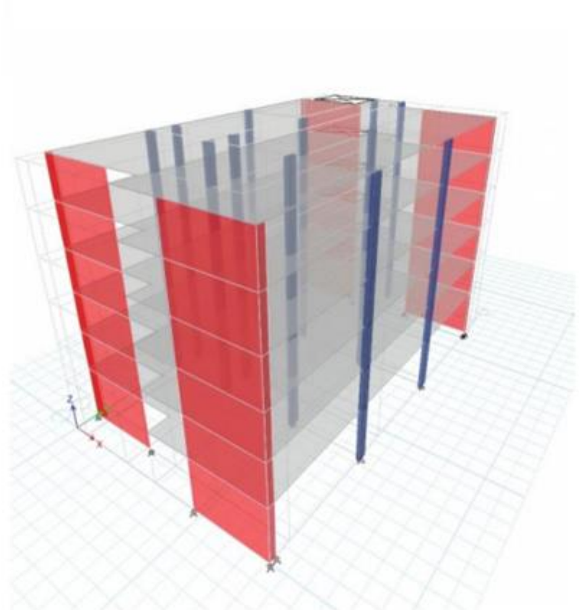
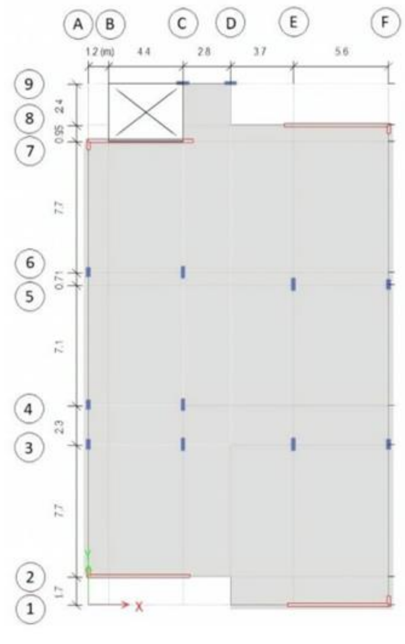



Fig. 2 Case study elevation view



(a) Three-dimensional view

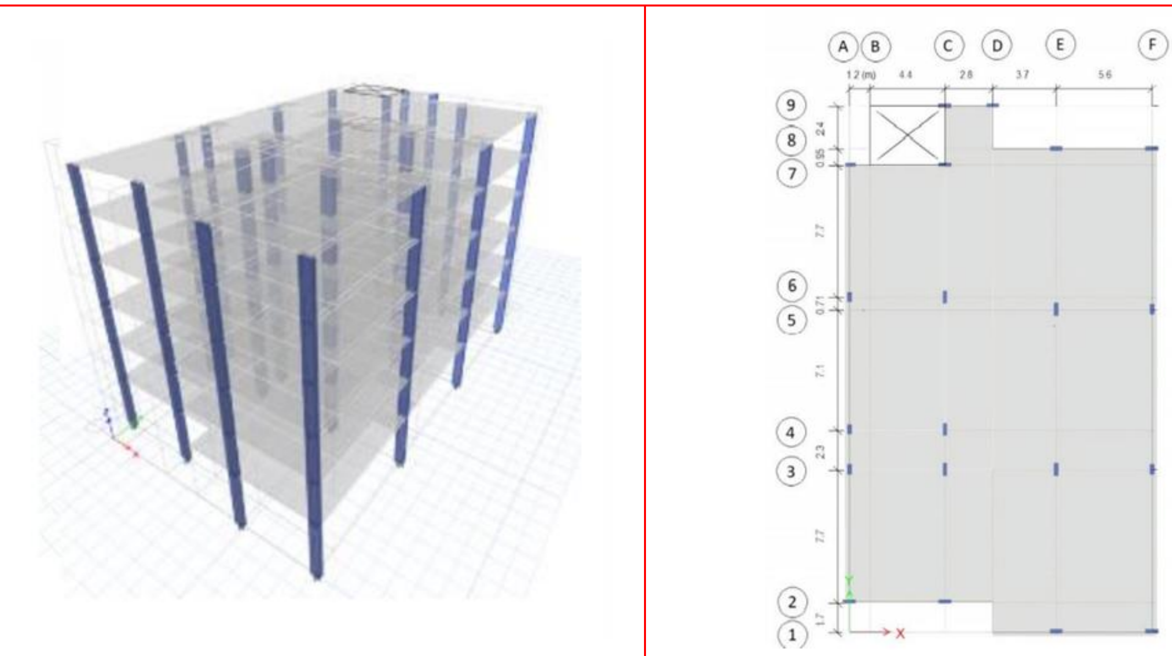



(b) Plan view

Fig. 3 Moment-resisting frame with shear walls (Case 1)

doi: 10.36828/efs.216

ISSN: 2753-4693




(a) Three-dimensional view | (b) Plan view

Fig. 4 Moment-resisting frame without shear walls (Case 2)

**Table 1.** Building specifications

| **Specification** | | **Value** | **Concrete** | | **Steel rebar** |
|---|---|---|---|---|---|
| Height | | 19.46 m | - | - | - |
| Number of Storeys | | 6 | - | - | - |
| Typical Floor Height | | 3.075 m | - | - | - |
| Ground Floor Height | | 4.125 m | - | - | - |
| Overall dimensions | | 17.7 × 27.3 m | - | - | - |
| Floor | | Flat Slab 275 an | - | - | - |
| Column | | 600 × 275 mm | - | - | - |
| | | 750 × 250 mm | - | - | - |
| Shear wall | | Shell Thin 250 m | - | - | - |
| Grade | | - | C30/37 | C 40/50 | - |
| $f_c$' | | - | 30 N/mm$^2$ | 40 N/mm$^2$ | - |
| Weight per unit volume | | - | 25 kN/m$^3$ | 25 kN/m$^3$ | - |
| E (Modulus of Elasticity) | | - | 33000 N/mm$^2$ | 35000 N/mm$^2$ | - |
| Poisson's Ratio | | - | 0.2 | 0.2 | - |
| G (Shear Modulus) | | - | 13750 N/mm$^2$ | 14580 N/mm$^2$ | - |
| Grade | | - | - | - | B500B |
| $R_e$ | | - | - | - | 500 N/mm$^2$ |
| $R_m/R_e$ | | - | - | - | 1.08 |
| $A_{gt}$ | | - | - | - | 5 |
| Roof loads | **Permanent (kN/m$^2$)** | 8.125 | - | - | - |
| | **Imposed (kN/m$^2$)** | 1.5 | - | - | - |
| Floor loads | **Permanent (kN/m$^2$)** | 6.875 | - | - | - |
| | **Imposed (kN/m$^2$)** | 2.5 | - | - | - |
| Stairs loads | **Permanent (kN/m$^2$)** | 7.9 | - | - | - |
| | **Imposed (kN/m$^2$)** | 4 | - | - | - |

Note: for the outer walls, the edge load is 5.4 kN/m in all directions.

**Table 2.** Static structural design wind load (Home Counties)

| Specification | Value | Reference (EN 1991-1-4:2005) |
|---|---|---|
| Terrain Category | III (Town) | Cl 4.3.2 |
| Reference Height | 11.67 m | Cl 6.3 |
| Directional Factor | 1 (Recommended) | Cl 4.2 |
| Season Factor | 1 (Recommended) | Cl 4.2 |
| Fundamental Wind Velocity | 21.5 m/s | Fig NA.1 |
| Basic Wind Velocity | 21.5 m/s | Cl 4.2-Exp (4.1) |
| Terrain factor | 0.21 | Cl 4.3-Exp (4.5) |
| Roughness Factor | 0.77 | Cl 4.3-Exp (4.4) |
| Terrain Orography Factor | 1 (Recommended) | Cl 4.3 |
| Mean Wind Velocity | 16.5 m/s | Cl 4.3-Exp (4.3) |
| Turbulence Intensity | 0.27 | Cl 4.4-Exp (4.7) |
| Basic Velocity Pressure | 0.17 kN/m$^2$ | Cl 4.5-Exp (4.10) |
| Peak Velocity Pressure | 0.49 kN/m$^2$ | Fig NA.1 |
| Structural Factor | 1 (Recommended) | Cl 6.2 |
| Wind Pressure | 0.64 kN/m$^2$ | Cl 4.2-Exp (4.1) |
| External Pressure Coefficient * | 1.3 | Cl 5.2-Exp (5.1) |
| Wind Force (X) | 346 kN | Cl 5.3 |
| Wind Force (Y) | 201 kN | Cl 5.3 |

*The external pressure coefficient for the wider face (X direction) is selected

## 5. Results

### 5.1 Displacements

The results of maximum lateral displacement of systems with shear walls (case 1) and without shear walls (case 2) are shown in Table 3. As expect, due to high lateral stiffness, frame with shear wall system (case 1) results in significantly lower than permissible displacements in both directions (especially in X axis) compared to moment-resisting frame system. Furthermore, the results for case 2 indicate that, due to the orientation of the columns, the lateral displacement in X-axis is approximately twice of Y-axis. It can be concluded that, to improve the structural performance, the minor axis of the cross section of columns needs to be align with the width of the building plan.

**Table 3** Maximum storey displacement case 1 and 2

| Storey | Height (m) | Case 1 | | Case 2 | |
|---|---|---|---|---|---|
| | | X-Axis (mm) | Y-Axis (mm) | X-Axis (mm) | Y-Axis (mm) |
| Roof | 2.96 | 0.616 | 3.49 | 10.1 | 5.49 |
| Storey5 | 3.15 | 0.507 | 3.34 | 9.71 | 5.27 |
| Storey4 | 3.08 | 0.386 | 2.99 | 8.65 | 4.72 |
| Storey3 | 3.08 | 0.269 | 2.48 | 7.02 | 3.88 |
| Storey2 | 3.08 | 0.159 | 1.79 | 4.89 | 2.75 |
| Storey1 | 4.13 | 0.0682 | 0.977 | 2.51 | 1.43 |

Table 4 Maximum storey drift

| Storey | Height (m) | Case 1 | | Case 2 | | Limitation (mm) |
|---|---|---|---|---|---|---|
| | | Drift X $d_r$ (mm) | Drift Y $d_r$ (mm) | Drift X $d_r$ (mm) | Drift Y $d_r$ (mm) | |
| Roof | 2.96 | 0.111 | 0.152 | 0.444 | 0.221 | 5.92 |
| Storey5 | 3.15 | 0.122 | 0.345 | 1.06 | 0.548 | 6.31 |
| Storey4 | 3.08 | 0.119 | 0.521 | 1.63 | 0.846 | 6.15 |
| Storey3 | 3.08 | 0.111 | 0.688 | 2.13 | 1.13 | 6.15 |
| Storey2 | 3.08 | 0.092 | 0.814 | 2.38 | 1.32 | 6.15 |
| Storey1 | 4.13 | 0.068 | 0.977 | 2.51 | 1.43 | 8.25 |

### 5.2 Drift

Interstorey drift is one of the key parameters in a practical analysis and design of structures. For an economical design the story drift, in both directions, needs to be slightly less than code requirement. Table 4 shows the result of interstorey drift for both structures. The results indicate that, although, the interstorey drift in both cases did not exceed the limits defined by the BS 8110 for each storey but due to drift in case 1 is extremely less than code limit, it can be clearly observed that frame with shear walls cannot provide an economical solution for this these types of structural systems.

doi: 10.36828/efs.216 ISSN: 2753-4693

**Table 5** Acceleration values for case 1 and 2

| Mode of vibration | Frequency (cyc/sec) | | Maximum displacement (mm) | | Acceleration ($m/s^2$) | | Acceleration (milli-g) | |
|---|---|---|---|---|---|---|---|---|
| | Case 1 | Case 2 | Case 1 | Case 2 | Case 1 | Case 2 | Case 1 | Case 2 |
| **1** | 0.731 | 0.586 | 3.49 | 10.3 | 0.036 | 0.071 | 3.67 | 7.24 |
| **2** | 2.39 | 0.617 | 0.616 | 5.61 | 0.069 | 0.042 | 7.03 | 4.28 |

## 5.3 Occupants' Comfort

There are ranges of standards for occupant comfort requirements in buildings (NBCC: Section 4; Bank et. Al., 2014). According to Concrete Centre, the standard values for a 10-year wind-generated return period are as follows:

- 10 to 15 milli-g for residential occupancy
- 20 to 30 milli-g for office occupancy

To calculate the acceleration based on the frequency and the maximum displacement Eq. (1) from SpaceAge Control (2001) can be used

$$a = \frac{2\pi^2 f^2 d}{g} \quad (1)$$

Where *a*, *f*, *d* and *g* represent acceleration (m/s$^2$ ), frequency (Hz), maximum displacement (m) and ground acceleration (m/s2 ), respectively. The findings of the acceleration of buildings and human perception are shown in Table 5. It was noted that for the first two principal modes of vibration, the acceleration values in both cases were lower than the threshold, and both buildings were within the safe range for the occupants' comfort criteria.

## 5.4 Slab Deflection

Eurocode 2 Part 1-1 (BS EN 1992-1-1, 2014) deals with a design for deflection in flat slabs by several approaches, in this study limiting span to depth ratio and the procedure provided by Goodchild (2009) is used. It can be observed that in the worst scenarios for both cases, flat slab defection ratios were within the allowable span length to effective depth ratio; L/d (Table 6).

**Table 6** Flat slab deflection check (worst scenario)

| Location | Allowable L/d | Actual L/d | Status |
|---|---|---|---|
| Case 1 Building with shear walls | 35.9 (Storey 5- EF-1 to EF-3) | 32.7 | Passed |
| Case 2 Building without shear walls) | 33.4 (Storey 5- EF-1 to EF-3) | 32.7 | Passed |

## 5.5 Punching Shear Failure

Flat slabs are susceptible to punching shear failure, where slab is penetrated around column and leads to an immediate local failure that may results to a progressive collapse of structure. To study the punching shear behaviour of flat slab, for both cases, column F1 on the first roof, as the worst scenario, is chosen and procedure presented by Goodchild (2009) is used to calculate punching shear ratios. The results for punching shear ratio are presented in Table 7. The ratios in Table 7 demonstrate that the flat slabs in case 1 can provide adequate resistance to prevent shear failure, but in case 2, shear reinforcement is required.

**Table 7** Punching shear reinforcement ratio (worst scenario)

| Location | Ratio | Status |
|---|---|---|
| Case 1 Building with shear walls | 0.75 (Storey1- Column F1) | Passed |
| Case 2 Building without shear walls | 1.76 (Storey1- Column F1) | Passed |

doi: 10.36828/efs.216 ISSN: 2753-4693

**Table 8** Cost estimation case 1

| Component | Quantity | Rate | Quantity | | Rate | | Subtotal £K |
|---|---|---|---|---|---|---|---|
| Slabs | 609 m³ @ | £95.00 + | 15 | T @ | £750.00 | | 69.1 |
| Shear Walls | 136 m³ @ | £110.00 + | 13.6 | T @ | £750.00 | | 25.2 |
| Columns | 42 m³ @ | £110.00 + | 12.6 | T @ | £750.00 | | 14.1 |
| Formwork (Vertical) | | | 730 | m² @ | £32.00 | | 23.4 |
| Formwork (Horizonta | | | 3043 | m² @ | £29.00 | | 88.2 |
| Formwork (Horizonta | | | 0 | m² @ | £52.50 | | 0.0 |
| Hollow-core units | | | 0 | m² @ | see "Rates" | | 0.0 |
| | | | | | | *Total "superstruc* | 220.0 |
| | | | | | *72.3* | *£/m²* | |
| Stairs as %age of su | | | | | | 14% | 30.8 |
| Foundations | | | 50772 | kN @ | £1.89 | | 95.7 |
| Ground floor slab | | | 507 | m² @ | £30.00 | | 15.2 |
| Cladding | | | 1816 | m² @ | £330.00 | | 599.2 |
| | | | | | | *Structure and cl* | 960.9 |
| Prelims and external | | | | | | 10% | 293.1 |
| Finishes and walls | | | | | | 21% | 615.5 |
| Mechanical and Elect | | | | | | 35% | 1025.9 |
| | | | | | | *Total constructio* | 2895.5 |
| | | | | | *951.6* | *£/m²* | |
| | | | | | | TOTAL | 2895.5 |

**Table 9** Cost estimation case 2

| Component | Quantity | Rate | Quantity | | Rate | | Subtotal £K |
|---|---|---|---|---|---|---|---|
| Slabs | 609 m³ @ | £95.00 + | 17 | T @ | £750.00 | | 70.61 |
| Columns | 71 m³ @ | £110.00+ | 12.6 | T @ | £750.00 | | 17.3 |
| Formwork (Vertical) | | | 630 | m² @ | £32.00 | | 20.2 |
| Formwork (Horizontal | | | 3043 | m² @ | £29.00 | | 88.2 |
| Formwork (Horizontal | | | 0 | m² @ | £52.50 | | 0.0 |
| Hollow-core units | | | 0 | m² @ | see "Rat | | 0.0 |
| | | | | | | *Total "superstruc* | 194.8 |
| | | | | | *64.0* | *£/m²* | |
| Stairs as %age of sup | | | | | | 14% | 35.2 |
| Sprayed mineral fibre | 86 m² @ | £11.79 | | | | | 1.0 |
| Foundations | | | 50772 | kN @ | £1.89 | | 95.7 |
| Ground floor slab | | | 507 | m² @ | £30.00 | | 15.2 |
| Cladding | | | 1816 | m² @ | £330.00 | | 599.2 |
| | | | | | | *Structure and cl* | 941.1 |
| Prelims and external | | | | | | 10% | 293.1 |
| Finishes and walls | | | | | | 21% | 615.5 |
| Mechanical and Electr | | | | | | 35% | 1025.9 |
| | | | | | | *Total constructio* | 2875.7 |
| | | | | | *945.0* | *£/m²* | |
| | | | | | | TOTAL | 2877.2 |

## 5.6 Cost Estimation

It is obvious that, in any proposed load bearing system cost of projects is crucial, hence a cost effective between case

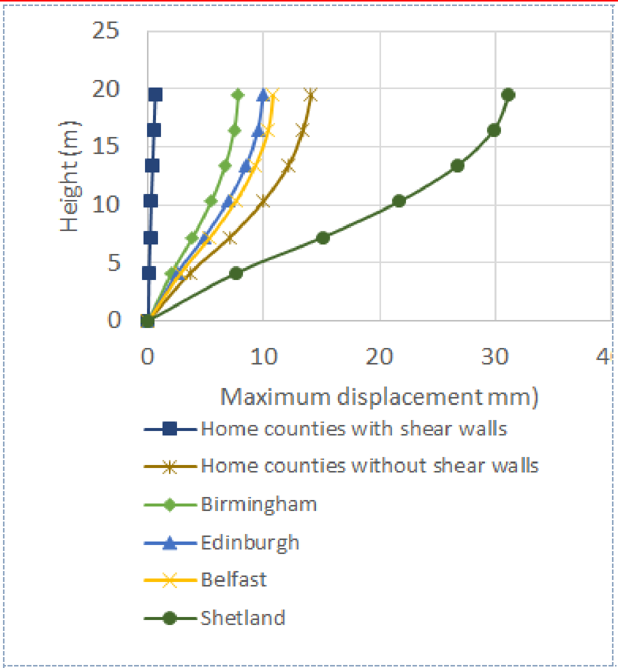
1 and 2 is performed. The cost estimates for construction in both cases are calculated by Concept software using the rates obtained from Goodchild, Webster and Elliott (2009) publication. It is to be noted that, only the cost of superstructure structures is taken into account, as the cost of other parts for both cases, more or less, are the same.

To show the efficiency of the shear walls removal scenario, only the quantity of concrete and reinforcements bars in the vertical elements (shear walls and columns) in both cases is compared. The results indicate that, the quantity of concrete of columns in moment-resisting system (case1) is around 40% that of columns and shear walls in case 2. Furthermore, the quantity of reinforcement bars in vertical elements in moment-resisting system is around 48% that of frame with shear walls system (Table 8 and 9). It is due to using minimum flexural and shear reinforcement bars in the most part of shear walls. The results of shear wall's design indicated that, except for the first two storeys, both flexural and shear reinforcements bars was in the minimum range required by the code. It is worth mentioning that, the construction time of moment-resisting frame is, also, decreased by 7% compared to frame with shear walls.

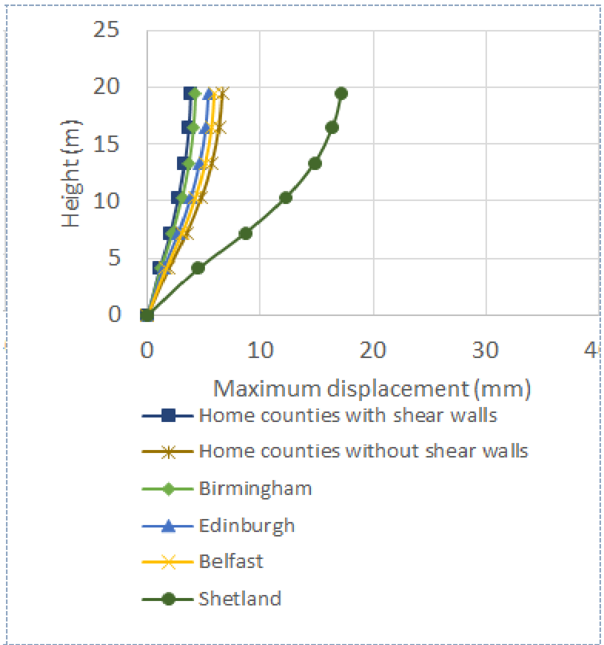
Concrete is an essential construction material but has negative impacts on the environment, e.g. $CO_2$. emissions, and in these analyses, it was illustrated that the quantity of concrete in case 2 is significantly reduced. This has a positive impact on the sustainability of concrete construction; reducing the volume of concrete reduces the negative environmental impact which is strongly in line with sustainability in construction industry.

## 6. The Global Performance and Application of the Moment-Resisting Frame

To investigate the probability of designing the RC structures without shear walls some structures in different locations of the UK have been studied using the same properties. As wind pressure activities rise towards the north (Table 10), many major cities in England, Scotland and Northern Ireland (not Wales, as their latitude and wind pressure were not substantially different from the area of England) with differing latitudes and wind pressure values were chosen within the UK from Birmingham, Belfast, Edinburgh and Shetland (as the worst possible). This selection could determine the effect of various climates in the UK on the structural performance of buildings. The maximum storey displacement for Birmingham (Case 3), Edinburgh (Case 4), Belfast (Case 5) and Shetland (Case 6) is demonstrated in Figure 5.




(a) X-axis



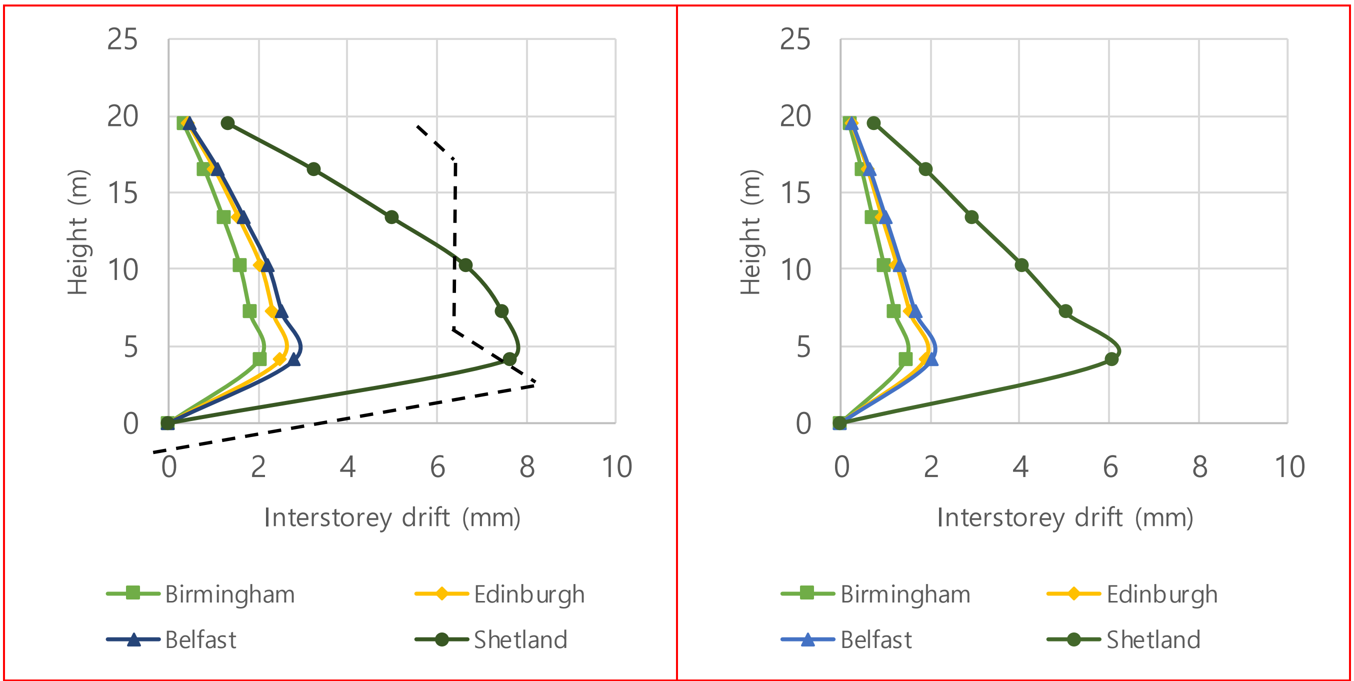

(b) Y-axis

**Fig. 5** Maximum displacements in all cases

**Table 10** Static structural design load (3-second load once in 50 years)

| Specification | Birmingham (Case 3) | Edinburgh (Case 4) | Belfast (Case 5) | Shetland (Case 6) |
|---|---|---|---|---|
| Terrain Category | IV (Town) | IV (Town) | IV (Town) | I (Country) |
| Wind Pressure ($W_e$) | 0.45 kN/m$^2$ | 0.59 kN/m$^2$ | 0.63 kN/m$^2$ | 2.10 kN/m$^2$ |
| External Pressure Coefficient ($C_{pe}$) | 1.3 | 1.3 | 1.3 | 1.3 |

doi: 10.36828/efs.216 ISSN: 2753-4693




(a) X-axis

(b) Y-axis

**Fig. 6** Interstorey drift for cases 3-6

**Table 11** Acceleration in case 3 to 6 years)

| Mode | Frequency (Hz) | | | | Acceleration (milli-g) | | | |
|---|---|---|---|---|---|---|---|---|
| | Case 3 | Case 4 | Case 5 | Case 6 | Case 3 | Case 4 | Case 5 | Case 6 |
| 1 | 0.586 | 0.591 | 0.586 | 0.595 | 5.37 | 6.97 | 7.48 | 22.2 |
| 2 | 0.617 | 0.623 | 0.617 | 0.626 | 3.25 | 4.18 | 4.54 | 13.5 |

As expected, Figure 5 shows that lateral displacement for case 1 (with shear walls) in X and Y directions had the lowest value (significantly less than code limit) and maximum displacement is related to case 6 which is corresponding to higher wind load. Figure 6 displays the interstorey drift in cases 3-6 with the relevant limitations. The results indicate that, except for case 6 (Shetland) the other cases were within the safe range defined by BS 8110: Part 2 Cl 3.2.2.2. Furthermore, Figure 6 shows that interstorey drift decreases with the increment of the height. It is to be noted that, in practice economical design will be achieved with interstorey slightly less than the permissible drift in both directions.

The assessment of building response regarding to the human response for cases 3, 4, 5 and 6 are illustrated in Table 11. It was evident that the calculated accelerations for three locations (Birmingham, Edinburgh, Belfast) were within the defined safe range but this is not the case for Shetland because the acceleration has exceeded the safe range, which means that it failed to provide occupants comfort. It can be concluded that the Shetland case can be chosen as of the worst possible case. It is to be noted that, the displacements are calculated based on Eq. 6.11b in Eurocode 2 Part 1-1 (BS EN 1992-1-1, 2014) in which wind actions take place in the design calculations.

**Table 12** Flat slab deflection check (worst scenario)

| Location | Allowable L/d | Actual L/d | Status |
|---|---|---|---|
| Case 3 Birmingham | 36.6 (Storey 5- EF-1 to EF-3) | 32.7 | Passed |
| Case 4 Edinburgh | 34.9 Storey 5- EF-1 to EF-3 | 32.7 | Passed |
| Case 5 Belfast | 34.5 (Storey 5- EF-1 to EF-3) | 32.7 | Passed |
| Case 6 Shetland | 24.1 (Storey 5- EF-1 to EF-3) | 32.7 | Failed |

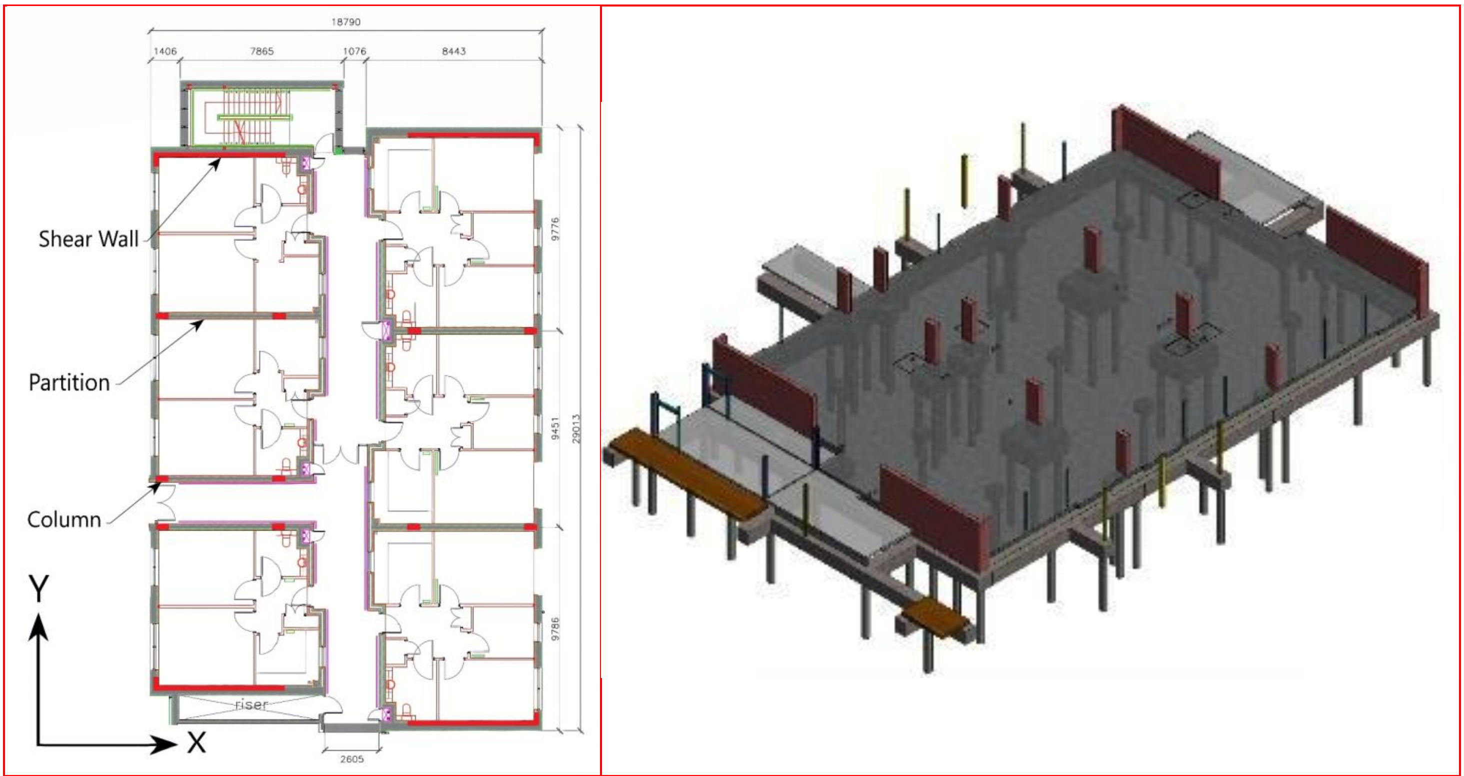
**Table 13** Punching shear ratio (worst load combination)

| Location | Ratio | Status |
|---|---|---|
| Case 3 Birmingham | 1.84 Storey5- Column F1 | Passed |
| Case 4 Edinburgh | 1.84 Storey5- Column F1 | Passed |
| Case 5 Belfast | 1.87 Storey5- Column F1 | Passed |
| Case 6 Shetland | 2.34 Storey2- Column F1 | Failed |

doi: 10.36828/efs.216

ISSN: 2753-4693

**Fig. 7** The village overview (the reference building is highlighted in yellow)




(a) Architectural plan (values in mm)

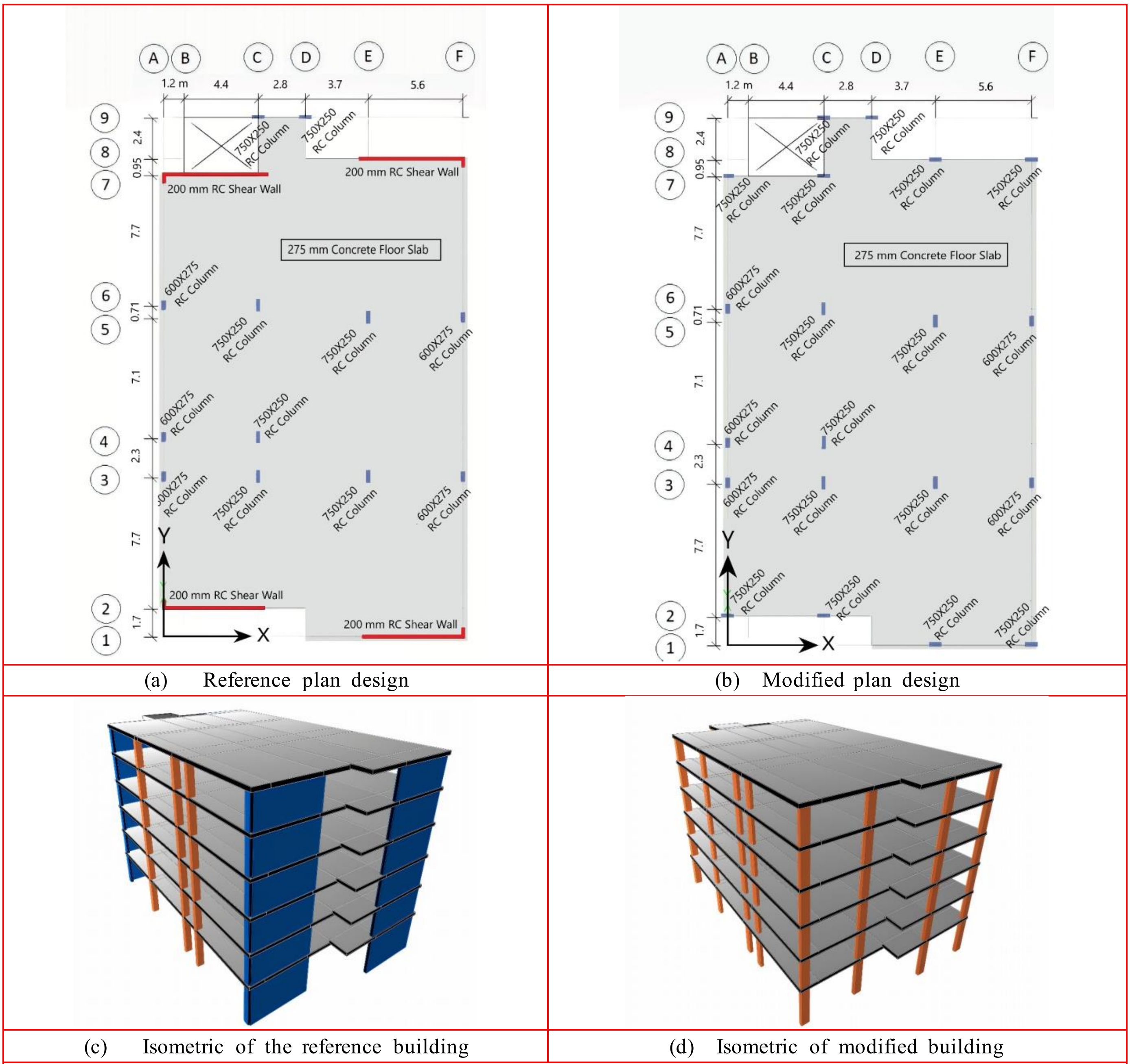
(b) Columns and shear walls location

Fig. 8 Reference building with shear walls

It was evident that deflection values in flat slabs for Birmingham, Edinburgh and Belfast were within the safe range defined by Eurocode 2 Part 1-1 (BS EN 1992-1-1, 2014); however, Shetland failed to fulfil the criteria (Table 12). Table 13 shows that, except for Shetland, all other cases passed the punching shear criteria using shear reinforcement to prevent punching shear failure.

## 7. Parametric Study

The above analysis, design and discussions clearly indicated that, moment-resisting frame system is able to provide safe and more economical and sustainable solution for low-to-medium rise RC structures compared to the frame with shear wall systems. To study the structural performance of moment-resisting frame without shear walls various factors such as concrete grade, column size, column shape, slab thickness, occupants' comfort, building performance, cost-effectiveness and sustainability of construction are taken into consideration. The influence of different factors on buildings' height in the absence of shear walls in low seismic regions is, also, studied.




(a) Reference plan design

(b) Modified plan design

(c) Isometric of the reference building

(d) Isometric of modified building

**Fig.9** Structural arrangement of reference and modified structures (values in m)

## 7.1 Geometry of the Structures

To assess the influence of various factors on maximum overall height of in RC rigid frames, architectural plans taken from a six-storey residential RC building in the UK provided by COUCH Consulting Engineers are used (Fig. 8 and 9). Belfast was chosen for the current analyses as it would be the most onerous of the different locations in the UK (excluding Shetland Island) in terms of wind loading. In all models, shear walls are replaced by two additional columns from the same section size of current columns at the ends of the walls (Fig. 9).

## 7.2 Loading

The characterstics of permenet and imposed load is shown in Table 14. As stated in the literature, in the high-rise buildings wind load is the dominant load while for low and medium-rise structurers seismic load will provide critical internal forces. In this study only wind laod requierd by Eurocode 1 Part 1-1 (BS EN 1991-1-1, 2009) is considered. The input values for the simulations and the wind flow are shown in Table 15 for the wind loads in Belfast.

doi: 10.36828/efs.216 ISSN: 2753-4693

**Table 14** Permanent and imposed actions

| | | | |
|---|---|---|---|
| Roof | **Permanent** | 6.875-7.5 | kN/m$^2$ |
| | **Imposed** | 1.5 | kN/m$^2$ |
| Floors | **Permanent** | 6.875-7.5 | kN/m$^2$ |
| | **Imposed** | 2.5 | kN/m$^2$ |
| Stairs | **Permanent** | 7.6 | kN/m$^2$ |
| | **Imposed** | 4 | kN/m$^2$ |
| Exterior walls | **Permanent** | 5.4 | kN/m |

**Table 15** Wind load (Belfast)

| 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 | Figure NA.1 |
| Basic Wind Velocity | 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$^2$ | Figure 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).

## 7.3 Material Properties

The design of RC buildings in Eurocode 2 Part 1-1 (BS EN 1992-1-1, 2014) is according to the characteristic cylinder strength rather than the cube strength. Eurocode 2 Part 1-1 (BS EN 1992-1-1, 2014) is used for the design of concrete classes up to C90/105, steel with the characteristic strength of 400-600 MPa while additional modifications and rules may be added for concrete classes beyond C50/60. In this study concrete strength classes ranging between C40/50 to C80/95, using Mander stress-strain curve (Mander, Priestly Park, 1988) and steel ***class S500***, due to its variety of applications in the UK, was selected in compliance with BS EN 1992-1-1, 2014.

The characteristics of the structure, including its dimensions, the properties of the concrete and steel components, are described in Table 16.

**Table 16** Reference building (with shear walls) specifications

| **Parameter** | **Value** | | **Units** |
|---|---|---|---|
| Height | 19.46 | | m |
| Number of Storeys | 5 | | - |
| Typical Floor Height | 3.075 | | m |
| Roof Height | 2.96 | | m |
| Ground Floor Height | 4.125 | | m |
| Overall dimensions | 18.8 × 29 | | m |
| Floor | Flat Slab 275* | Flat Slab 300 * | mm |
| Column | 600 × 275 | 750 × 250 | mm |
| Shear wall | 250 | | mm |
| Concrete | | | |
| Grade | C 30/37 | C 40/50 | - |
| $f'_c$ | 30 | 40 | N/mm$^2$ |
| Weight per unit volume | 25 | 25 | kN/m$^3$ |
| E (Modulus of Elasticity) | 33000 | 35000 | N/mm$^2$ |
| Poisson's Ratio | 0.2 | 0.2 | - |
| G (Shear Modulus) | 13750 | 13750 | N/mm$^2$ |
| Steel (Rebar) | | | |
| Grade | B500B | | - |
| $f_y$ | 500 | | N/mm$^2$ |
| $f_{yd}$ | 435 | | N/mm$^2$ |
| $R_m/R_e$ | 1.08 | | - |

## 7.4 Simulation Procedure

The overall design process for modelling, analysis and design in this study is shown in Fig. 10. To investigate the safety and to monitor the ductile behaviour of the moment-resisting frames with flat slabs, in each simulation the overall displacement, interstorey drift, horizontal acceleration and punching shear ratio ($V_{Ed}$ /$V_{Rd,c}$) were campared with the design limitations according to Eurocode 2 Part 1-1 (BS EN 1992-1-1. If the structure's design values were smaller than the threshold, the number of storeys was raised, and if the design values were nearly equal to the threshold, the simulation was terminated. This process was repeated until the maximum number of storeys with punching shear ratios of 2 and 2.5 were obtained.

doi: 10.36828/efs.216

ISSN: 2753-4693

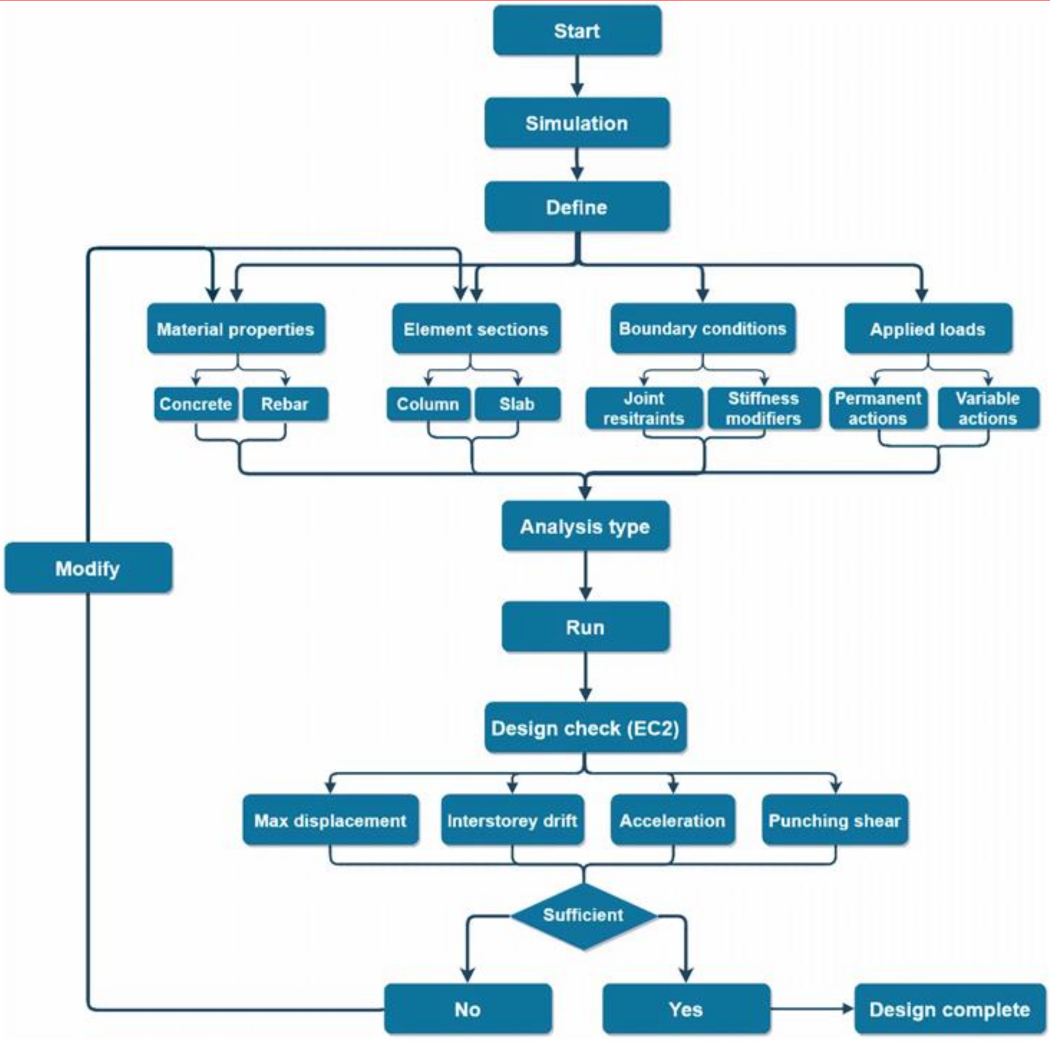



**Fig. 10** Overall design procedure

## 7.5 Structural Modeling

Columns with rectangular, square and circle shape accompanied by choice of column size, concrete grade and reinforcement details were selected. To account for the cracking behaviour of the concrete and to modify the elastic stiffness of the bilinear force-deformation relationship in RC structures as per the Eurocodes a modification factor of 0.5 is applied on the gross second moment of area of the columns.

The concrete strength class of the flat slab is set to be C30/37. In order to accurately represent the behaviour of the flat slab a shell-thin model available in the software is used. Also, to compensate for the crack behaviour of the slab, the property modifiers in both axes were set to 0.5 for the moment of inertia. In this analysis, the flat slab was perceived to be in accordance with the UK construction practice.

In RC moment-resisting frames, connections between columns and other components (beams and slabs) and base columns to the foundation were assumed to be rigid and fixed connection, respectively.

In all analyses, the applied permanent and imposed loads were determined based on Eurocode 0 (BS EN 1990, 2010) and shown in Table 14. Furthermore, wind load requierd by Eurocode 1 Part 1-1 (BS EN 1991-1-1, 2009) is applied (Table 15). In addition, the load combinations for the finite element simulations were specified according to the ULS and SLS load combinations.

To simulate actual behaviour of structures, analysis type could play a significant role especially in the buildings subjected to lateral forces as it can involve consideration of second-order (P-Δ) effect. Based on the the material and geometric nonlinearity a non-linear analysis offers more realistic results which could be used for both the ultimate limit state (ULS) and the serviceability limit state (SLS) criteria. In this study various numerical analyses were conducted using ETABS software v16.2.1, to analysis and design of all the concrete elements and punching shear ratio based on Eurocode 2 Part 1-1 BS EN 1992-1-1, 2014 (Saisaran, Prasad and Venkat Das, 2016; Jolly and Vijayan, 2016; Tsay, 2019).

**Table 17** 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 | - | - | - | - | - | - |

* The optimised concrete was an improved mix of the grades of concrete, starting with a higher strength of concrete on the lower floors and a reduction in strength over the height

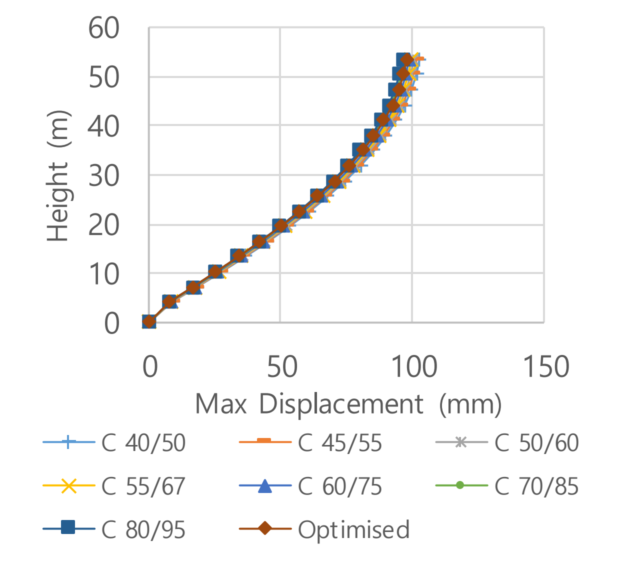
The first model (Fig. 9b) is develped with column size of 750 × 250 mm, flat slab thickness of 275 mm, garvity and lateral load according to Eurocode 1 Part 1-1 (BS EN 1991-1-1, 2009) (Table 14 and 15). To investigate structural performance of moment-resisting frames, four variables are considered; concrete grade, column size, column shape, slab tickness (Table 17).

Table 17 indicates various adopted factors for the structural analysis, comprising eight concrete grades for columns, one concrete grade for flat slabs, six different column dimensions, three column shapes and two slab thicknesses. The practicality and economic considerations of the building have been taken into account throughout this selection. For instance, eight different grades of concrete were chosen for columns, while only one grade of concrete was chosen for flat slabs, as the volume of concrete used for flat slabs was almost ten times more than those used for columns. As a consequence, raising the range of concrete grade in flat slabs could dramatically increase the cost of construction; thus, only one grade of concrete was preferred for flat slabs.

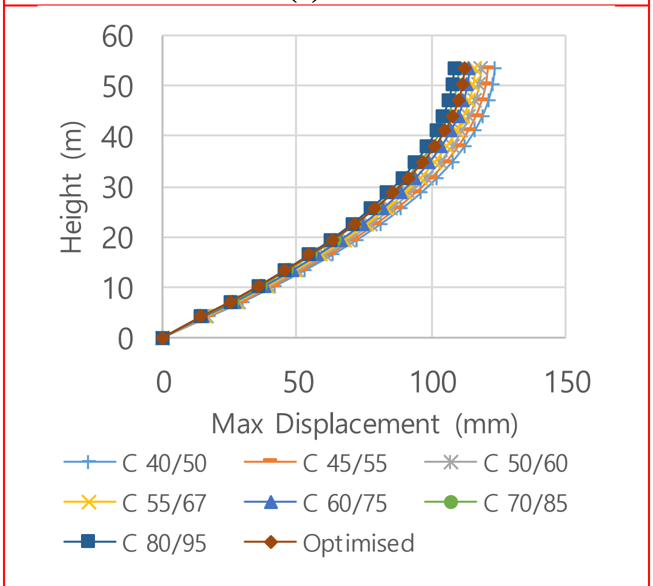
## 7.6 Results

### 7.6.1 Concrete Grades

According to the advise of the Concrete Centre experts, to determine effect of concrete grades on the structural performance of the building. various grades of concrete ranging from C40/50 to C80/95, were used in the columns and optimised with a higher strength in the lower storeys and a lower strength in the upper storeys while the column dimention and slab thikness asumed to be constant with column size of 750 × 250 mm and slab thickness of 275 mm. It can be observed that the structural stiffness is gradually enhanced with increasing the concrete grade (Fig. 11). Due to providing a high-strength concrete is considerably more expensive than concrete with normal grade, it will be more favourable to utilize a variation of concrete strength classes rather than using high-strength concrete such as C80/90 for the entire structure.




(a) X-axis

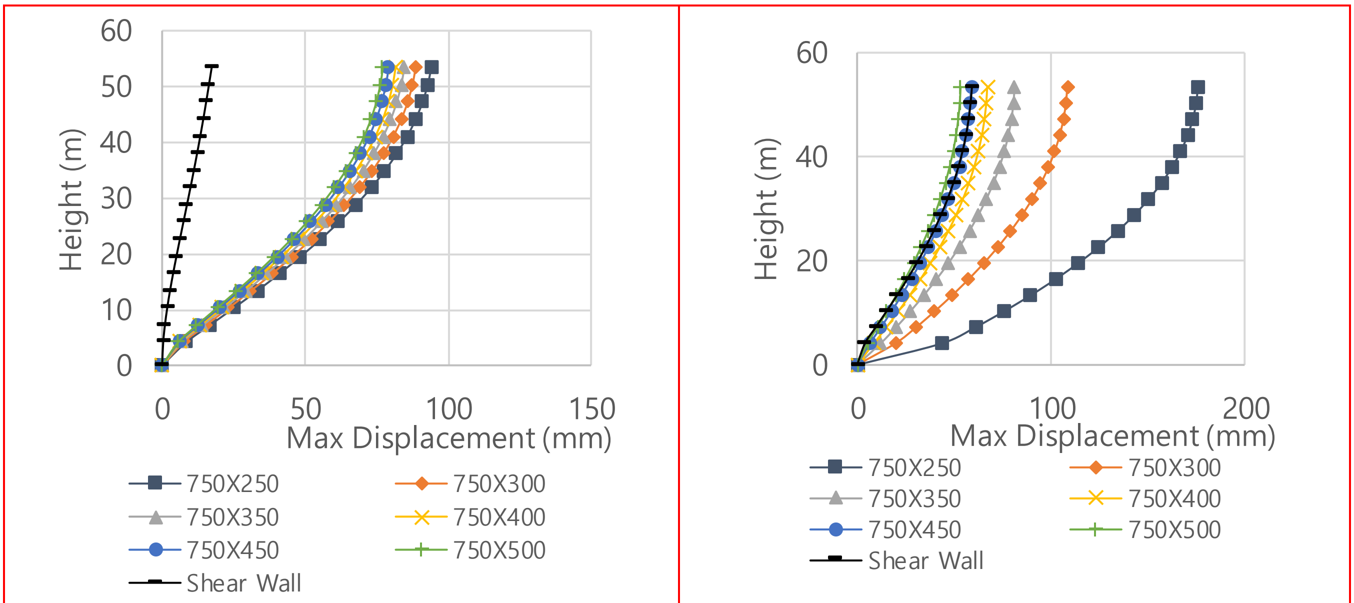



(b) Y-axis

Figure 11 Influence of concrete strength on the maximum displacement of RC framed buildings

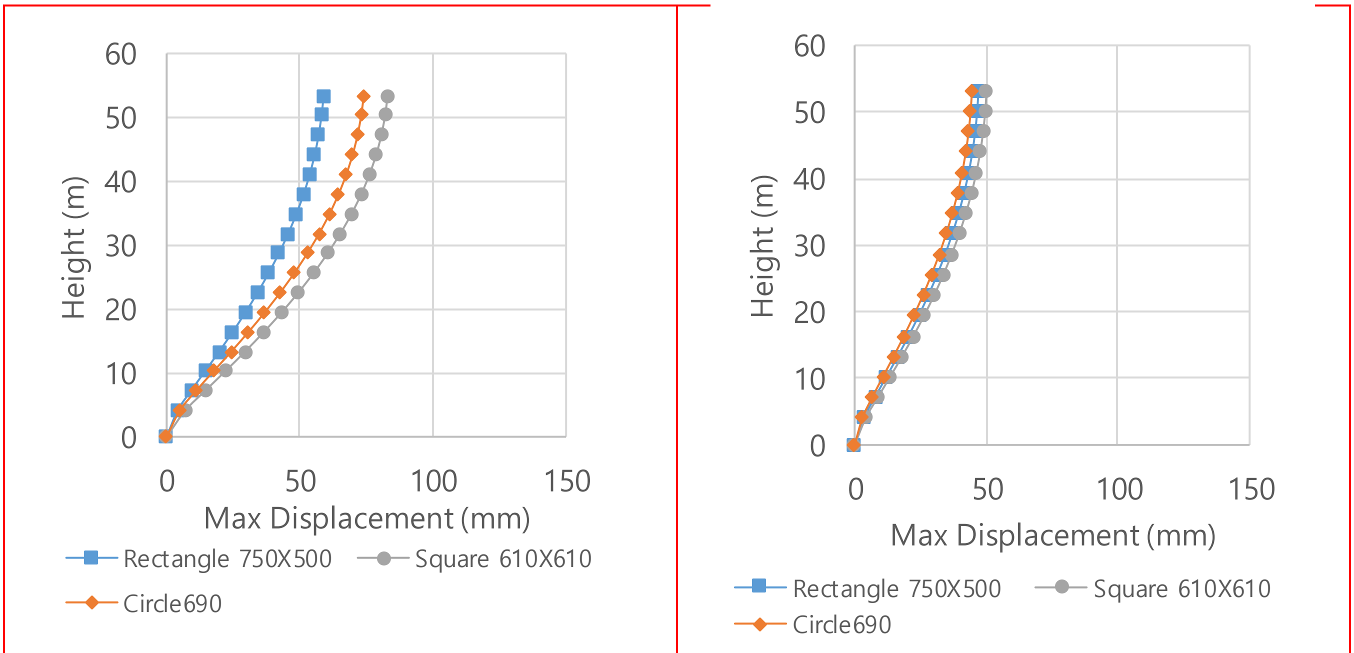
doi: 10.36828/efs.216

ISSN: 2753-4693




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

**Fig. 12** Influence of column size on the maximum displacement of RC framed buildings




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

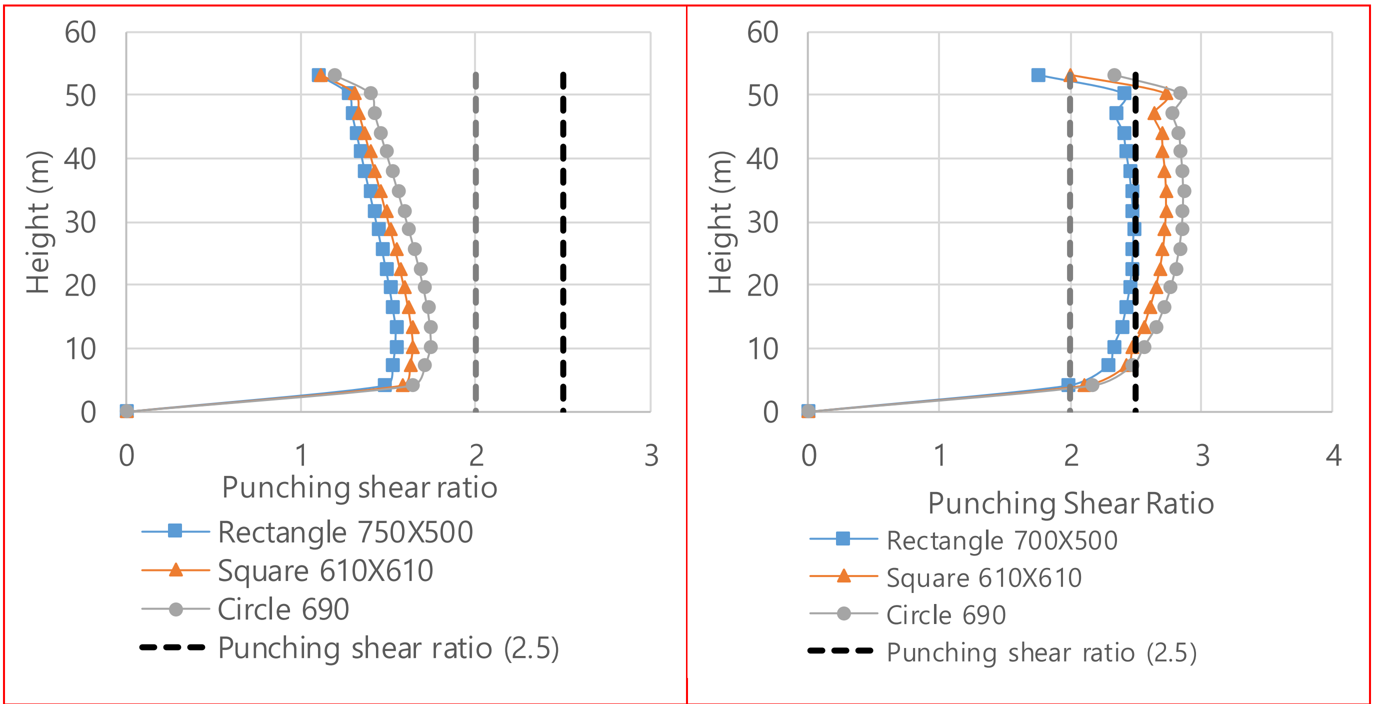
**Fig. 13** Influence of column shape on the maximum displacement of RC framed buildings

### 7.6.2 Column Shape and Size

To study the impact of column size on the structural performance, columns with the depth of 750 mm with various width ranging from 250 mm to 500 mm in increment of 50 mm using grade of C40/50 were considered.

Slab thickness in all models was set to be 275 mm. The results indicate that column size has a significant effect on the leteral deflection (Fig. 12b) but only up to a certain dimension (Fig. 12b), which shows that for each project the optimum colmn size and orintation need to be optomized.

To study the effcet of column shape on the structural behaviour, specilly on the lateral displacemnet and punching shear, columns with shapes of circle, square, and rectangular with the same cross section area, approximatetly 0.37 $m^2$,




(a) Internal column (b) Corner column

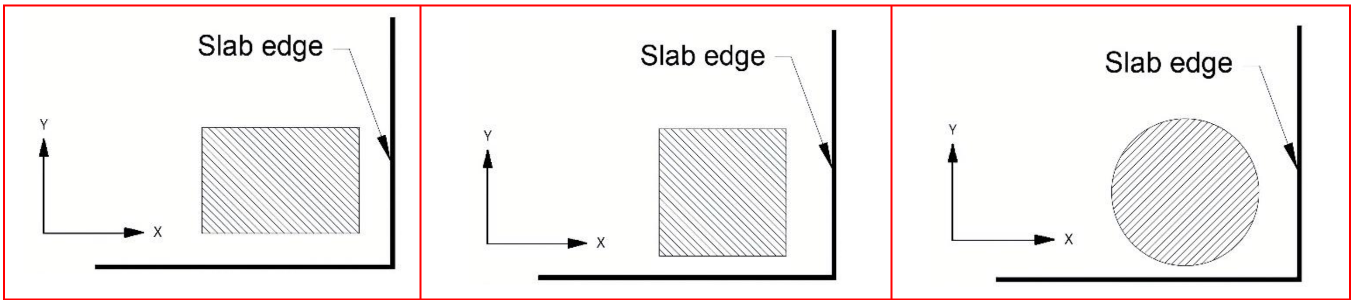
**Fig. 14** Influence of column shape on the punching shear ratio of RC framed buildings

were considered in the modelings using concrete grade of C40/50 and slab thikness of 275 mm. Figure 13 shows that, in the X-direction, the rectangular shape had the lowest while the square cross-section resulted highest displacement. It is due to this fact that, for the sections with same cross section area, rectangular sections provide more second moment of area and stiffness around major axis compared to other sections. The lateral displacements in the Y-direction are approximately the same as X axis with the rectangular section (Fig.13), which indicates the importance of section's orientation on the performance of the structures. For the economic purpose, it is recommended that major axis of cross section need to be aligned with the length of structural plan.

### 7.6.3 Punching Shear Failure

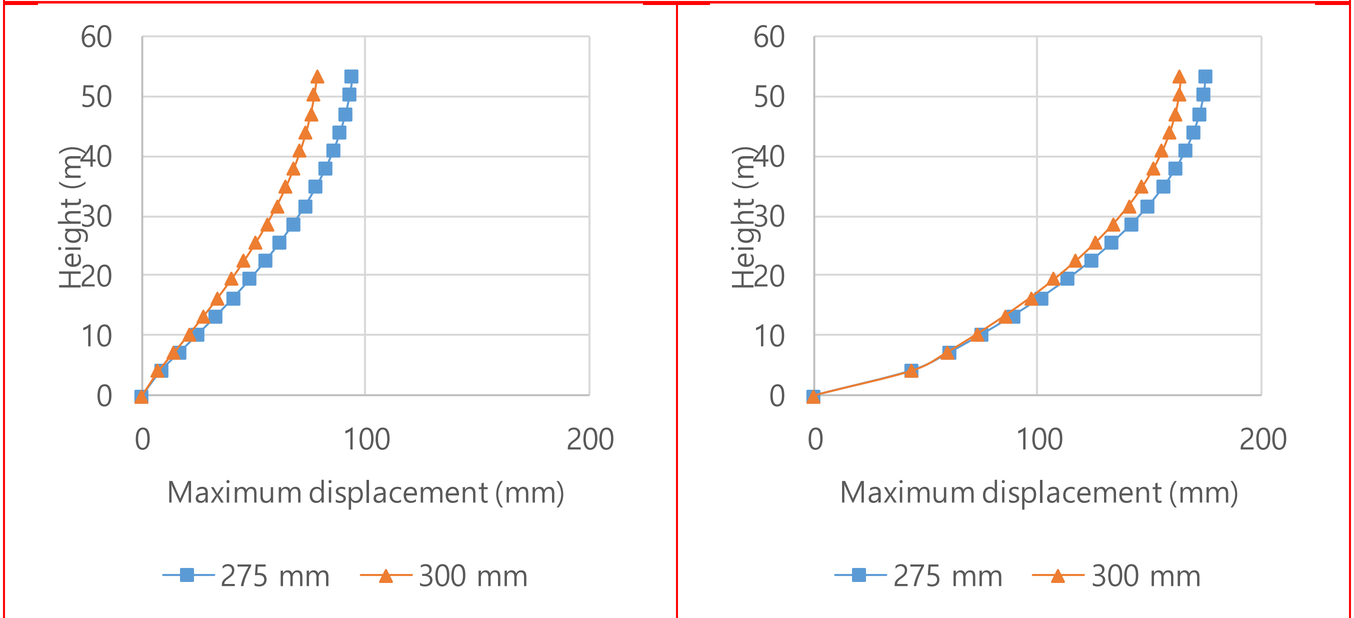
There are several thresholds for punching shear ratio ($V_{Ed}/V_{Rd,c}$) in design guides, by defining limiting ratio for shear force over allowable shear without reinforcement. Two of them are utilised in these analyses. The UK National Annex suggests limiting the punching shear ratio to 2.5, while this value for Eurocodes is 2. The influence of column shape on the punching shear in internal and corner columns are presented in Figure 14. It is obvious that punching shear failure, as a significant issue in flat slabs, is more likely to happen in corner columns than edge or internal columns, as shown in Figure 15 (Sacramento et al., 2012; Aalto and Neuman, 2017). The results indicate that in the internal columns the punching shear ratios were within the safe rang, while for the corner columns only rectangular shape was lower than threshold punching shear ratio of 2.5. It is due to providing more control perimeter around the loaded area in the rectangular shape compared to the others shapes.

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 the implementation of the Shear rails leads to increase the overall construction cost (Max Frank, 2020).




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

**Fig. 15** Corner columns with different shapes




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

Fig. 16 Influence of slab thickness on the maximum displacement of RC framed buildings5.5

### 7.6.4 Slab Thickness

The influence of Slab thickness was, also, explored to show the effcet of slab tickness on the strenght and letaral stiffnees of the structures using slab thicknesses of 275 mm and 300 mm with the concrete grade C40/50 and column size of 750 × 250 mm. By increasing the slab,s thickness the stifness of structures are incresaed henec the leteral deflection in both direction is reduced (Fig. 16). Due to only column strip in the flat slabs contribute in the lateral stiffeness, and in a commen flat slab nearly 85% of cross section of the slabs are umder tension, which do not contributing in strenght and lateral stiffness, slabs with less thickness results in more sustainable construction. On the other hand, although punching shear resitance is enhansed

by increasing of the slab tickness but as it effcets only the areas around columns, it is not recommended. Punching shear capacity can be achavide using drop panle or column head with considerably less concrete than enhanced slab thickness. Furthermore, drop panel can increase the lateral stiffness and, also, descreases the negetive reinforcement bars at the column-slab connection.

### 7.6.5 Maximum Overal Height

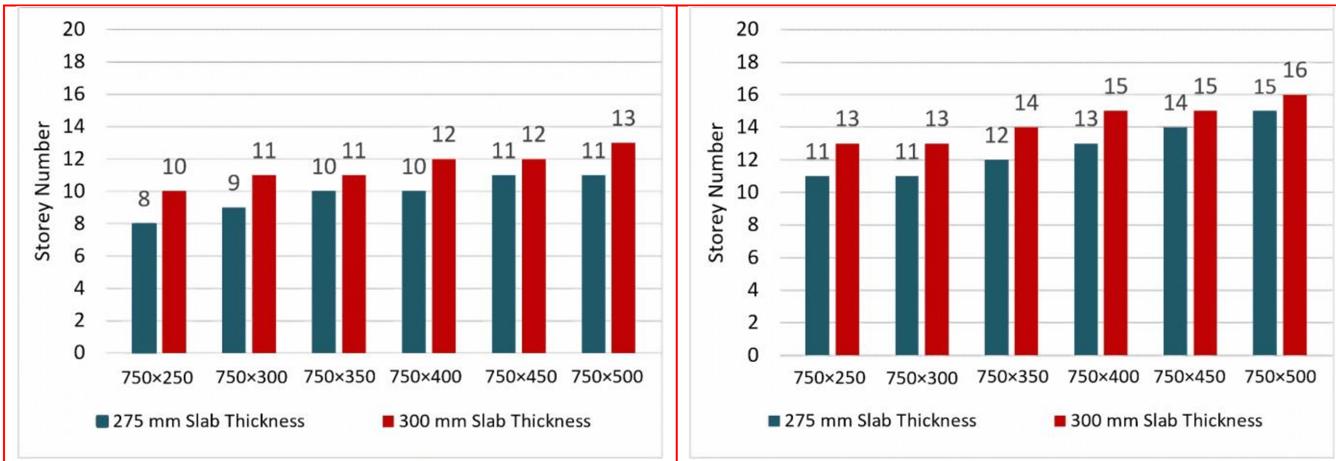
The optimised concrete grade with different column sizes, slab thicknesses and punching shear ratios ($V_{Ed}/V_{Rd,c}$) was investigated to achieve the maximum overall height (Table 17). Figure 17 presents the results for the maximum overall height in an RC moment-resisting frame with flat slab. The results demonstrated that the overall height could be increased up to 2 more storeys, for each column section, by increasing the slab thickness. Furthermore, the results showed that increasing the punching shear ratio limit, between 2 and 2.5, directly increase the maximum overall height up to 3 storeys. Therefore, for the proposed building with the optimised concrete grade, 750 × 500 mm column section size and flat slab with a thickness of 300 mm could reach up to 16 and 13 storeys with punching shear ratio of 2.5 and 2, respectively.

doi: 10.36828/efs.216 ISSN: 2753-4693

**Table 17** Concrete strength grade variation for each column size

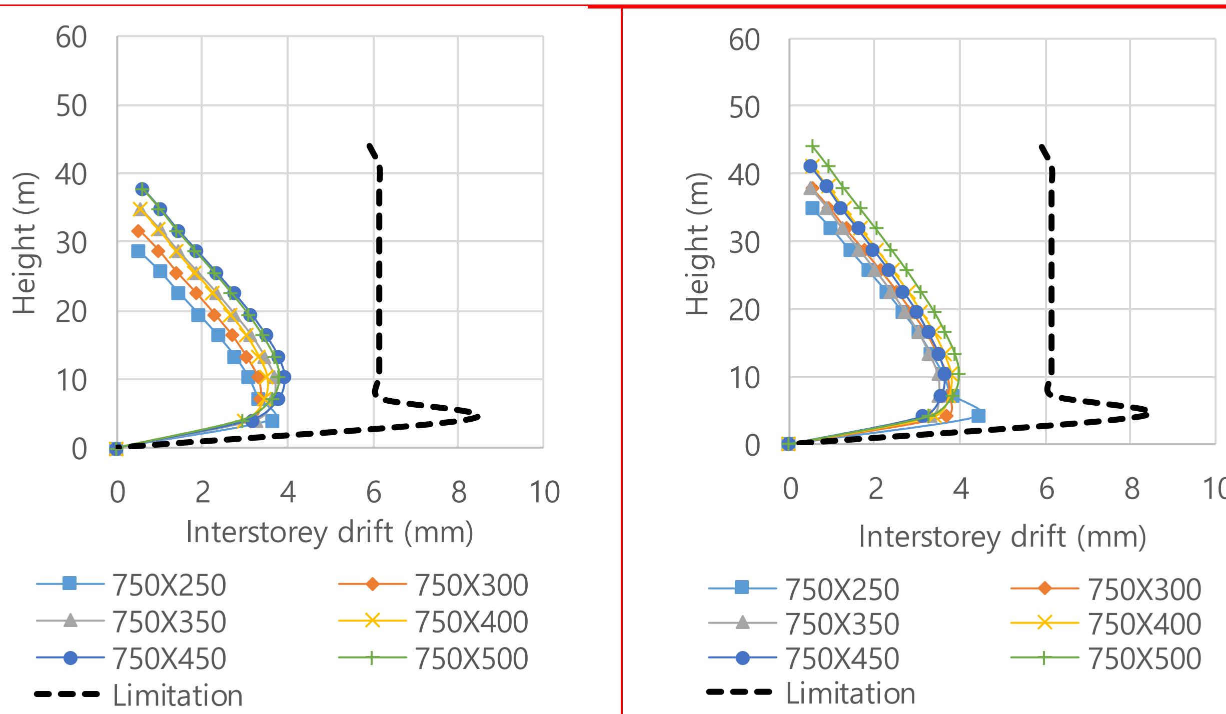
| **Concrete grade** | **750 × 250** | **750 × 300** | **750 × 350** | **750 × 400** | **750 × 450** | **750 × 500** |
|---|---|---|---|---|---|---|
| C80/95 | Storey 1-3 | Storey 1-3 | Storey 1-3 | Storey 1-3 | Storey 1-3 | Storey 1-3 |
| C70/85 | Storey 4-6 | Storey 4-6 | Storey 4-6 | Storey 4-6 | 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 | - | - | - | - | - | - |

**W**



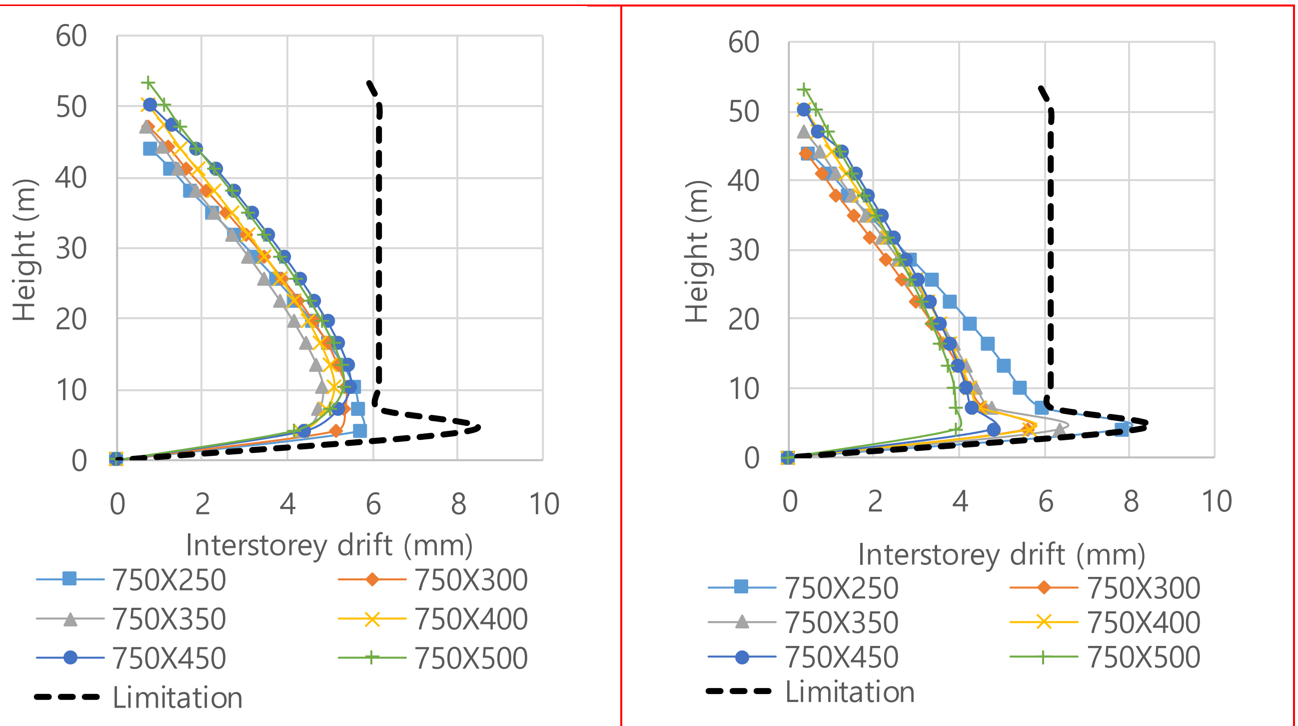

(a) $V_{Ed}/V_{Rd,c} = 2$ (b) $V_{Ed}/V_{Rd,c} = 2.5$

**Fig.17** Maximum overall height with various column sections and two slab thicknesses




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

**Fig. 18** Influence of column size on the interstorey drift of RC framed buildings (punching shear ratio 2)




(a) X-direction

(b) Y-direction

**Fig. 19** Influence of column size on the interstorey drift of RC framed buildings (punching shear ratio 2.5)

## 8. Performance of Moment-Resisting Frame Systems

Figure 18 and 19 shows that interstory drift in the all designed structures are within the safe range defined by Eurocode 2 Part 1-1 (BS EN 1992-1-1, 2014) in both X and Y directions. The fluctuation in interstorey drift's limit is due to the change in storey height between the ground storey and other storeys (from 4.125 m to 3.075 m).

Furthermore, the results of analyses, using slab thickness of 275 and 300 mm with various column sizes, indicate that the punching shear ratios were, also, within the safe range (2 and 2.5 punching shear ratio limits) for 13 (39 m)- and 16 (48 m) -storey buildings (Fig. 20 and 21).

Since the maximum overall 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:*

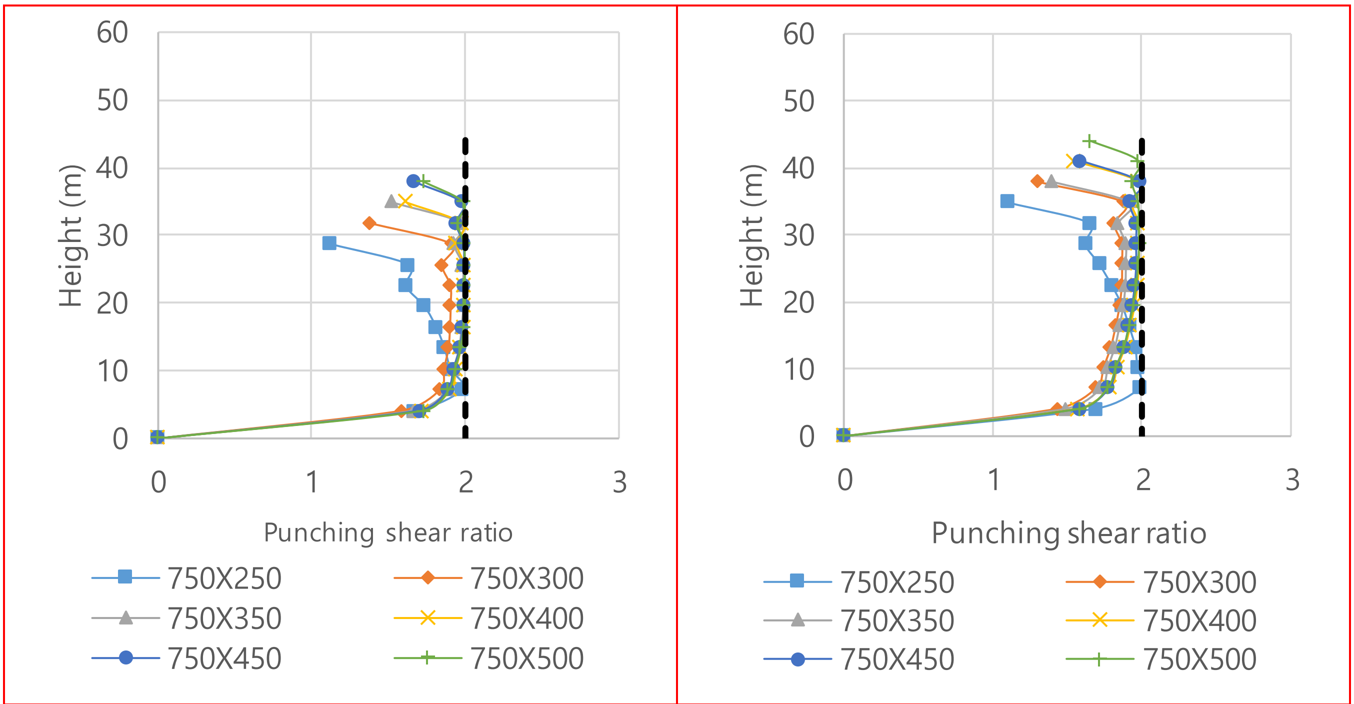
The horizontal acceleration threshold for residential occupancy with a 10-year return period is 15 milli-g, which is shown in Figures 22. In this part, only the buildings with 300 mm flat slab thicknesses were chosen.

In Figures 22, 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 punching shear ration of 2. Furthermore, for punching shear ratio of 2.5, the horizontal accelerations were 8.76 to 9.70 milli-g in X-direction and 5.56 to 5.70 milli-g in Y-direction. 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.

It can be concluded that 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, and the residents' comfort was not compromised.

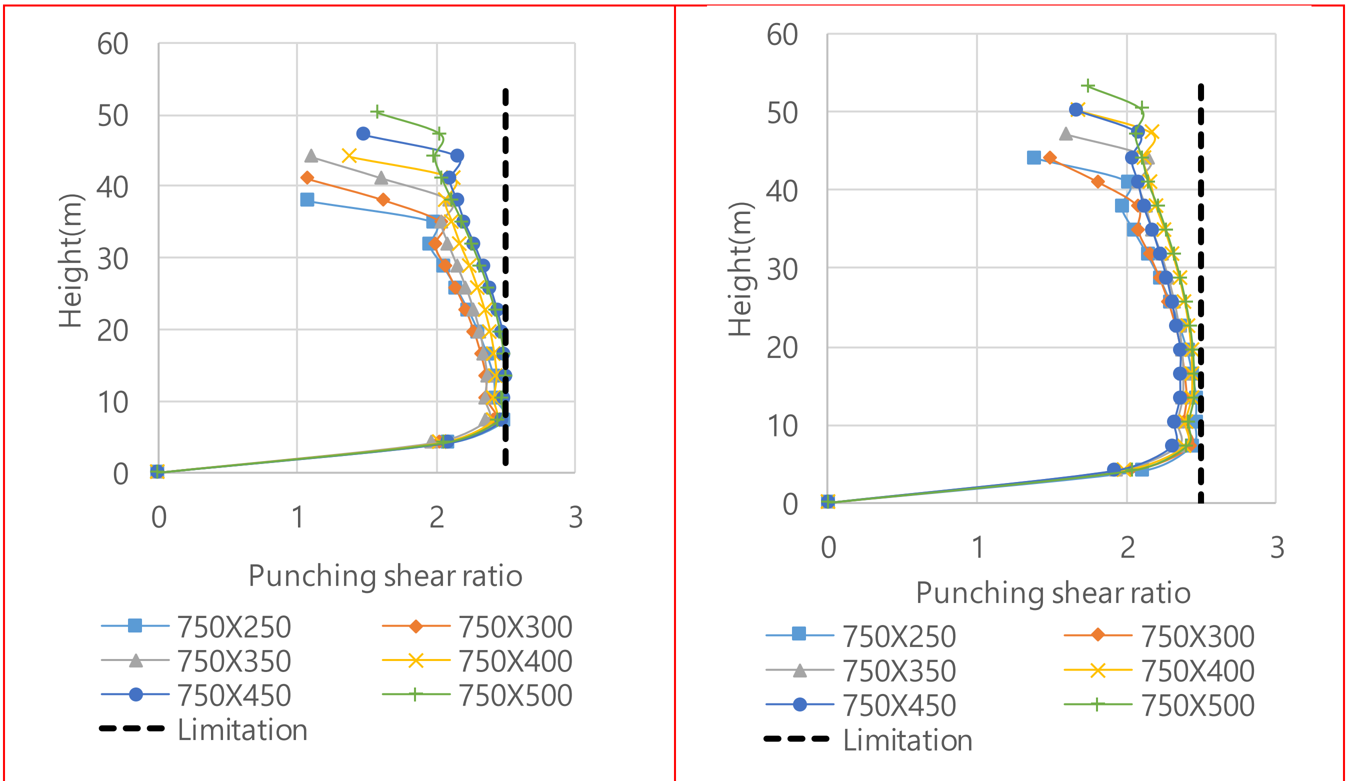
doi: 10.36828/efs.216

ISSN: 2753-4693




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

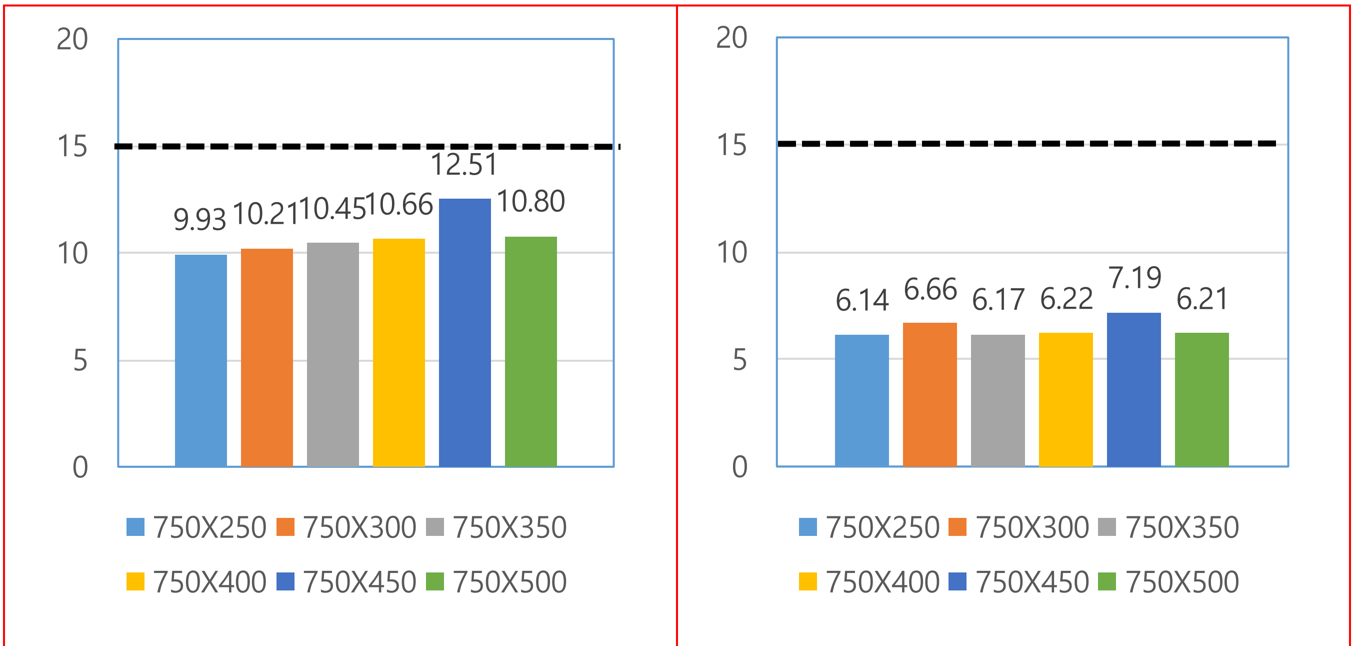
**Fig. 20** Influence of column size on the punching shear ratio of RC framed buildings (punching shear ratio 2)




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

**Fig. 21** Influence of column size on the punching shear ratio of RC framed buildings (punching shear ratio 2.5)

doi: 10.36828/efs.216
ISSN: 2753-4693




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

**Fig. 22** Influence of column size on horizontal acceleration (NBCC) of RC framed buildings (punching shear ratio 2)

## 9. Enhance the Practical Design of Reinforced Concrete Structures

### 9.1 Sustainable Load Bearing System

In the UK, shear walls have been extensively used as vertical resisting elements to withstand the wind forces in almost every reinforced concrete frame building above three storeys. Recently the construction industry has questioned the need for shear walls in low-to-medium-rise RC frame buildings and the possibility of replacing them with moment-resisting frame. The main benefits of moment-resisting frames are reduced cost and increased environmental sustainability due to the reduction in the consumed volume of concrete during the construction process.

It was observed that, in all models, except for the first two storeys, due to significant cross section of the shear walls, flexural and transvers reinforcement bars are less than minimum reinforcement bars required by BS EN 1992.1.1. Further optimization of frame with shear wall systems is out of question while more optimization can be applied on frame without shear walls using various concrete grade, various column size, using rectangular section with suitable orientation (major axis of cross sections align with length of plan), and optimum slab thickness.

Furthermore, for all the cases, depth of compression area under lateral load is around 20% of shear wall's length and 80% of cross section of shear walls is under tension which leads to a considerable consumption of concrete without any structural benefit. It is to be noted that, concrete in tension area do not contribute in providing lateral stiffness and strength.

According to Table 18 removing shear walls saves 50% of the consumed concrete on vertical elements (columns and shear walls) and 11% on the total superstructure's cost. Besides, in this case study, 165 m$^3$ of concrete was saved by removing the shear walls during the construction process which leads to more reduction in $CO_2$. emission.

**Table 18** Total superstructure

| Component | With shear walls (Case 1) £K | Without shear walls (case 2) £K |
|---|---|---|
| Slabs | 69.1 | **70.61** |
| Shear Walls | 25.2 | 0 |
| Columns | 14.1 | 17.3 |
| Formwork (Vertical) | 23.4 | 20.2 |
| Formwork (Horizontal – plain) | 88.2 | 88.2 |
| **Total "superstructure."** | **220** | **194.8** |

The performed structural analyses and above discussions clearly indicate that, removing shear wall in low-to-medium rise buildings will provide an economical and sustainable solution in construction industry, hence for low-to-medium rise RC structures up to 12 storeys, moment resisting frame is recommended. Considering transport and works on the site, $CO_2$. emission reduction will be more than 11%.

Furthermore, in terms of the construction time, it has been suggested that replacing shear walls might cut out a day in a 14-day cycle, and this amount of time could be saved in the construction process. The saved time can also indirectly reduce the construction cost and $CO_2$. emission.

doi: 10.36828/efs.216
ISSN: 2753-4693

## 9.2 Concrete grade

Concrete grade do not have significant effect on the lateral stiffness of the moment-resisting frames compared to the column dimensions, due the relationship between lateral stiffness and concrete grade is linear. On the other hand due to concrete with higher grade are very expensive, for the slabs normal concrete and for the columns concrete with high compressive strength in lower storeys e.g. 30% of storeys and normal concrete for the rest of the storeys is recommended.

## 9.3 Slab Thickness

According to the results, in the moment-resisting frames lateral loads only effect reinforcement bars in areas with negative bending moment of slabs, with the length around 15-20 % of span length from the columns, while the area of reinforcement bars in the middle of slabs (around 60-70% of span length) are the same as bars required for the gravity loads. On the other hand, in those areas only singly reinforced bars are able to provide adequate bending moment resistance for critical load combinations (in the all cases). The slab thickness can be reduced, approx. 10%-15%, by taking into account the compression reinforcement bars and using results of non-linear finite elements analyses to calculate vertical deflection of slabs. These arrangements will have significant impact on reduction of concrete consumption in the slabs compared to column optimisations as consumed concrete in the slabs is around 10 times of the columns in each floor. Furthermore, according to the results of non-linear analyses, for all cases the span to depth ratio (L/d) required by the codes are conservative..

For the slabs with heavy gravity loads, instead of using slabs with higher thickness, increasing the number of columns, drop panel or column head will provide more economical solution. Furthermore, in the project with architectural constraints, for long span lengths, using drop panel leads to a more economical solution compared to the enhanced slab thickness. The drop panels enhance the lateral stiffness, reduce vertical deflection of slabs, improve punching shear resistance, and reduce cross section area of the negative reinforcement bars at the column-slab connections. For the common span lengths, to improve punching shear resistance, instead of using drop panel shear reinforcement bars is recommended.

According to performed structural analyses and above discussions, for the flat slabs minimum thickness which is able to sustain the loads and provide permissible deflection is highly recommended.

## 9.4 Height Limit

A vital aspect of column shape is its impact on punching shear and different column stiffness around major and minor axis. For the same cross section, rectangular section provides more perimeter than square and circle section, hence the punching shear ratio ($V_{Ed}/V_{Rd,c}$) will be less than other two shapes. Furthermore, lateral stiffness of rectangular shapes around major axis is significantly more than square and circle shape. The influence of column size and slab thickness on the maximum overall height of a building using the optimised concrete grade is shown in Figure 23.


**Fig. 23** Maximum overall height in a different column and slab sections

The results demonstrate that increasing width to depth ratio of columns' cross-sections can increase the building's overall height. According to the results, to provide relatively the same stiffness and storey drift in both directions, for structures with rectangular plan only rectangular columns with minor axis of section align with the width of the plan is recommended. An optimisation analyses are needed to finalise the column's dimensions to maximise the numbers of storeys and minimise the cost of structures.

It was also evident that increasing the slab thickness by 25mm results increase of building height by two storeys. Although, the building with 750×500 column section and 300mm slab thickness provides 13 storey RC moment-resisting frame buildings constructed in the UK (Fig. 19), but due to considerable consumption of concrete and increased $CO_2$. emission, slabs with higher slab thickness is not highly recommended.

## 9.5 Optimum Dimensions

An approach to decide between the options is to conduct a cost-benefit analysis (CBA). This approach is a systematic process where the decisions for applications are analysed to decide whether or not benefits outweigh the costs and by what margin. Here, the difference in the construction cost is presented in Figure 24 for three types of 13 storey buildings with various element sections. The results clearly indicate that, the cost of structures with slab thickness of 300 mm is considerably more than structures with slab thickness of 275 mm for the same numbers of storeys, which again shows that slab with minimum thickness provides more sustainable solution.

Considering the practical side of the design, it is advisable to design the buildings up to 12 storeys, since by increasing the height to 13 storeys the structural performance of the buildings is pushed to the defined limitations in the Eurocodes. In this situation, another factor such as human errors could lead the structural performance of the building to fail.

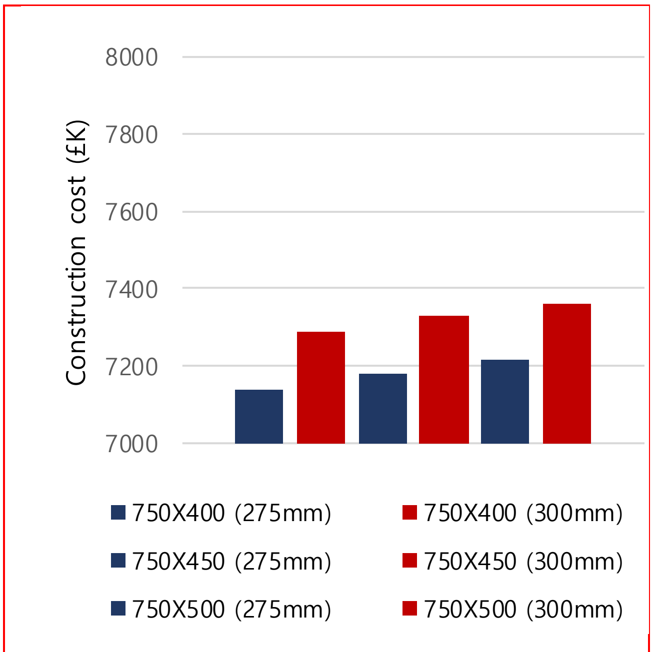



**Fig. 24** Cost estimation for various element sections

## 9.6 Slab-Column Connections

Due to the concentration of bending moments and shear stresses at the perimeter of columns and the weakness of flat slabs to transfer such loads without failure, it is essential to limit the transferrable loads at the column-slab connections, particularly at the edge and corner columns. Furthermore, as gravity and lateral loads is directly transferred from slabs to the columns and inversely, column-slab connection play significant role in structural resistance and stability under lateral loads. To improve the capacity of connections to transfer the applied loads only limited options are available: Using column head, increasing slab thickness (not recommended), using higher concrete grade, using less span length and more columns, and relocate the columns relative to the edge of the slab.

## 9.7 Robustness

According to EN 1991-1-7:2006, a building should be designed in a way to provide robustness (withstand events like accidental explosions, fire, impact or the consequences of human error) without being damaged to an extent disproportionate to the original cause. Moment-resisting frame with flat slab are highly susceptible to progressive collapse. To enhance the robustness of flat slabs, the following detailing for bottom rebars are suggested:

1. Extend the bottom rebars within the clear cover distance of slabs' free edge.
2. Increase the area of the bottom rebars anchored to the edge and corner columns. Standard hooks need to be located into inside of the slab-column connections.
3. Every second rebar has to be lap spliced with the adjacent panel's bottom rebar.
4. To provide more safe alternate load paths, minimum two reinforcement bars at the bottom of slab's cross section, within the column's dimensions, need to be continued in both directions.

## 9.8 Buckling

As effective length of columns highly depends on the stiffness of slabs at the top and bottom of columns, hence they are more susceptible to the buckling. Providing lateral support align with major axis of the cross section, design columns with maximum area in the middle and least area at the ends like bulged columns, use of BRBF (Buckling-restrained braced frames) to prevent buckling, using column head or drop panel can increase the capacity of columns against the buckling

To improve the structural performance of buildings, in horizontal accelerations, higher column size, increased slab thickness (not recommended), and passive, active and semi-active vibration control methods can be used.

## 10. Conclusions

In this research, the possibility of eliminating shear walls in the typical UK reinforced concrete (RC) frame buildings was investigated using various finite element simulations with ETABS software. In the first part, a UK case study was used and analysed with and without shear walls in the various locations of the UK to determine the climate influences on the structural performance of buildings. The same architectural design was then built with a moment-resisting system, and the influence of several factors including the concrete grade, column shape, column size and slab thickness on the overall height of the building was explored through various simulations. It was found that, moment-resistance frame can be used for the RC structures up to 12 storeys. Furthermore, the results indicated that in the moment-resisting frames the concrete and CO2. emission is

doi: 10.36828/efs.216 ISSN: 2753-4693

reduced by 11% and 4.6% respectively, on superstructure compared to the moment-resisting frame with shear walls.

To reduce further concrete and embodied carbon in RC structures, an optimization on column spacing, column shape, column orientation, concrete grade,beam shape, thickness of slabs, and using voided slab are crucial. Furthermore, as concrete in slabs is considerably more than column and beams, an innovative floor system to change flexural behavior to compressive membrane action seems to be an effective alternative.

## Acknowledgments

We would like to express our gratitude to Concrete Centre (London) for their generous support by funding this research project.

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ISSN: 2753-4693