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The significance of removing shear walls in existing low-rise RC frame buildings – Sustainable approach

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Abstract. According to The Concrete Centre, in the UK shear walls have become an inseparable part of almost every reinforced concrete frame building. Recently, the construction industry has questioned the need for shear walls in low to mid-rise RC frame buildings. This study tried to address the issue in two stages: The first stage, the feasibility of removing shear walls in an existing design for a residential building where ETABS and CONCEPT software were used to investigate the structural performance and cost-effectiveness respectively. The second stage, the same structure was examined in various locations in the UK to investigate regional effects. This study demonstrated that the building without shear wall could provide adequate serviceability and strength within the safe range defined by Eurocodes. As a result, construction time, overall cost and required concrete volume are reduced which in turn enhance the sustainability of concrete construction.

Keywords: low-rise RC buildings; wind actions; sustainability; non-linear static analysis; cost-effectiveness

1. Introduction

In the UK, shear walls have been widely used as elements to resist lateral forces in almost every concrete frame buildings' construction. However, as indicated by the Concrete Centre, the construction industry has questioned the significance of utilising such components. Removing shear walls can affect the performance of a structure accompanied by its cost-effectiveness and sustainability of construction in the economy and environment.

In this research the feasibility of removing shear walls in an existing residential concrete frame building is investigated by considering the following criteria:

- Serviceability of the design is an essential criterion in the performance of a building subjected to wind-induced forces for the occupants. In general, human response to building motion which is a complex psychological and physiological phenomenon is accounted to be more effectively measured by acceleration than other factors (Banks *et al*., 2014).
- Also in ULS criteria, the design checks for slab sections (deflection and punching shear) should be calculated.
- Interstorey drift as a damage limitation requirement is another factor that should be considered in the performance of a building. According to Eurocode 8, high values of interstorey drift may lead to severe damages and eventually collapse of a structure. Furthermore, for buildings having non-structural elements the limitation of interstorey drift is vital to control the serviceability cracks, connections between blocks or stud partition walls and slabs.
- The cost and duration in the construction industry are essential criteria to be taken into account. Providing approaches to reduce the overall construction expenses and the programme times will lead to more sustainable construction as such approach has a direct effect on two pillars of sustainability, namely environment (less environmental impact by reducing the required volume of concrete) and economy (reducing the construction cost).
- Since the wind force is directly dependant on the regional location, the response of a building to the subjected loads could be different in various areas in the UK. Furthermore, it is evident that climate change will lead to higher wind speed in the coming years.
- 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) and Global Wind Atlas (2018) confirm that Northern European countries including the UK, have high wind forces which can produce severe lateral loads on buildings.

Furthermore, the evaluation of the structural criteria will be performed using global analyses by ETABS v16.2.1 and the other criteria including the cost and time estimation will be performed using Concept v3.

2. Review of the previous work

2.1 Wind action and its mechanism

Wind is generated by the fluctuation of surface temperature and the rotation of the planet which creates

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imbalances in the atmosphere. In buildings as the height increases the wind speed increase which after a certain level called gradient height, mean wind velocity is almost the same (Hughes 2014).

The influence of wind on a structure is dependent on the movement of the air around and above the structure. The magnitude of the air that passes through and above the structure is affected by the dimensions of the building. In windward face the pressure is positive, and as the flow is continuous, it will be demanded that the wind flow accelerates around the structure causing a negative pressure on the other side of the building as well as the roof (Hughes 2014).

The primary objective in designing buildings is to calculate the lateral loads in each direction of the building. The forces are obtained from the combination of positive pressure on the windward side and the negative pressure (suction) on the leeward side (Hughes 2014).

2.2 Previous studies

Over the past few decades, various studies have been conducted to investigate the performance of reinforced concrete frame buildings with and without shear walls subjected to the lateral forces. In this field, there are many types of research regarding the effect of seismic actions including: Several studies conducted by Chandurkar and Pajgade (2013), Thakur and Singh (2014) and Aainawala and Pajgade (2014) who performed comparative analyses for multi-storey residential buildings with and without shear walls using STAAD and ETABS software. In their researches four types of structures were analysed, three with different locations of shear walls and one without any shear walls. Based on the results, the building with shear walls placed at the corner has the least lateral displacement comparing to other buildings. In the braced frames (with the shear wall) the shear forces and moments in members reduced in comparison with the bare moment resisting frame. However, their study did not examine the global performance of the buildings to investigate the overall lateral displacements and the lateral allowed deflections based on the design codes. Moreover, Jayalekshmi and Chinmayi (2015) studied the behaviour of RC frames with and without shear walls in different design codes (IS 1893 and IBC) to identify their differences without mentioning the lateral limitations in those codes.

In flat slab structures, some studies were conducted, including research from Tovi, Goodchild and B-Jahromi (2017) which investigated the deflection of flat slabs in an experimental work using Hydrostatic Cell Levelling system. The results show that the slab deflection develops quite slowly up to 2 mm during 142 days.

Ghorpade and Swamy (2018) in another research, investigated the performance of flat slab structures with and without shear walls using Pushover Analysis by SAP 2000 software. This research aimed to find a suitable structural system for flat slab buildings using Pushover Analysis. Results of this study showed that as the shear walls improve the stiffness of the structures, the period of the structures reduces and the frequency of the structures increases accordingly. Moreover, by adding shear walls the storey drift and displacement of the buildings reduce considerably and the flat slab structures with shear walls are preferable than RC frames with shear walls. It is worth mentioning that the frames without shear walls, despite their higher drifts and lateral displacements, are still in the safe range according to serviceability limit state criteria. However, they missed performing further investigation regarding the economic aspect of the study.

Furthermore, another study was conducted by Cismasiu *et al*. (2017) who used pushover analysis to investigate the applicability of this method in simulating the failure modes in RC shear walls. It was concluded that the applied element method could produce the results with reasonable accuracy.

On the other hand, there are not comprehensive studies regarding the evaluation of RC moment resisting frames with and without shear walls subjected to wind actions. Furthermore, according to Smith (2011), most of the international design codes have no guidance regarding the top deflection limit for wind actions, and even Eurocode 2 has not provided any inter-storey drift ratio limit within their design code for concrete frame buildings.

There are few studies to evaluate RC frame buildings' performance subjected to wind actions which investigate on arbitrary architectural plans, not real case scenarios neglecting the lateral displacement limitations in the design codes and their cost and time influence in the construction process. For example, in research conducted by Rasikan and Rajendran (2013), the performance of RC frame buildings with and without shear walls subjected to wind actions was investigated using STAAD software. In this study, two structures of different height with and without shear walls were analysed. The results obtained that the overall displacements of structures with shear walls, regardless of their height, were lower than the building without shear walls. As mentioned earlier, this study failed to consider the storey drifts limitations and the second order effect in their analysis.

Furthermore, Hosseini *et al*. (2014) investigated the effect of wind load on the behaviour of shear walls in concrete frame structures. It was evident that by utilising shear walls inside the frame, the torsional forces in the structure could be reduced.

In some other cases, the performance of shear wall systems was compared with other structural systems to assess the efficiency of such methods. Jayasundara *et al*. (2017) in their study investigated the application of utilising shear wall system to resist wind loads by different design methods. The case studies in this paper were two 60-storey buildings including one with diagrid system and the other with shear wall system. Their results demonstrated that the diagrid system could resist the same vertical and lateral forces while reducing the weight of the structure by 35 per cent. It can be concluded that shear walls despite their effectiveness, suffer from the extra weight that they are adding to the structures. However, this study did not cover the criteria above.

Perception of movement in buildings is a critical factor, and various studies have been conducted to assess this impact on the occupants' comfort. For example, Kwok, Hitchcock and Burton (2009) reviewed the perception of

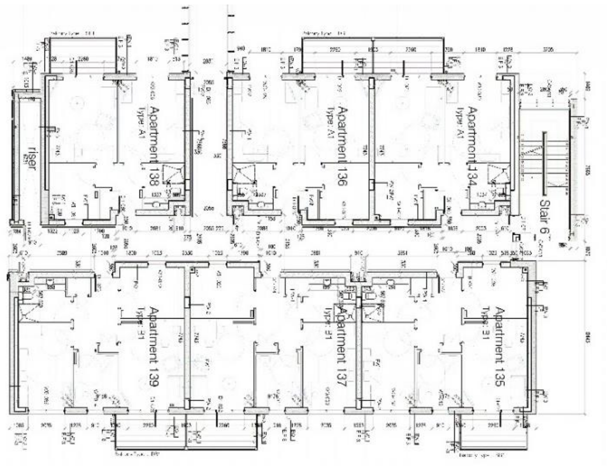
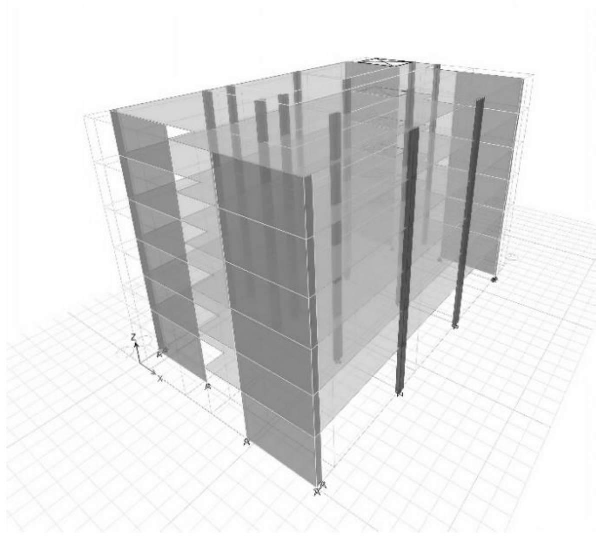
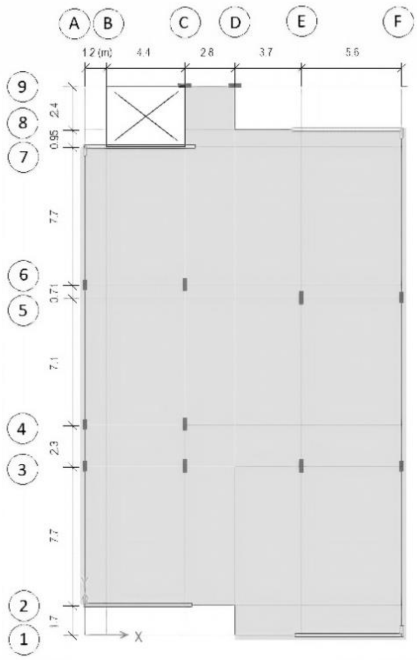


Fig. 1 Case study architectural plan

Fig. 2 Case study elevation view



(a) Three-dimensional view



(b) Plan view

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

vibration generated by wind actions considering acceptability and its influence on occupants comfort in tall buildings. At the time, it was concluded that there were no internationally accepted serviceability criteria for occupant comfort. However, in recent studies, the sensitivity and perception of humans to building vibrations and movement have been investigated and various values for comfort criteria, have been produced. Banks *et al*. (2014) pointed out a couple of values used in North America for a 10-year return period. Also, Johann, Carlos and Ricardo (2015) evaluated the comfort criteria in various design codes and mentioned that in the future, residents should be aware of the building motions and educated to cope with the situation.

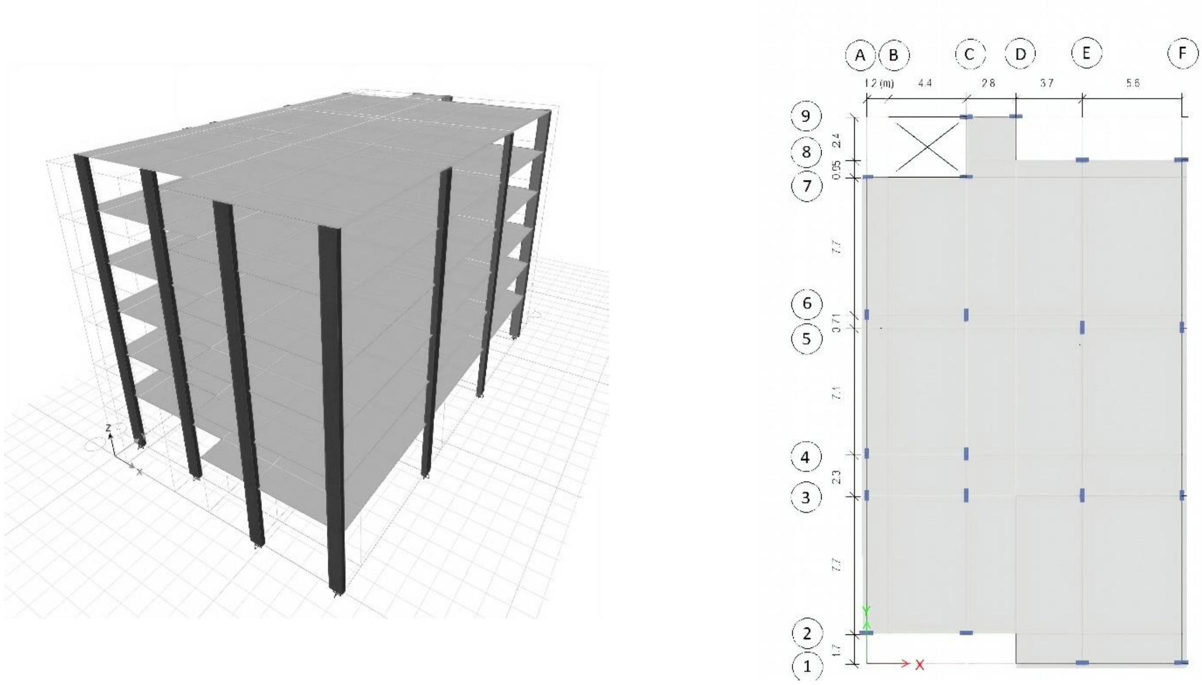
3. Case Study

3.1 First stage

In the first part of the study, global comparative analysis is performed to assess the influence of removing shear walls in the performance of the buildings, expenses and sustainability of the construction. This study is based on an architectural plan of an existing retirement village located in Home Counties (Fig 1 and 2), in the UK which was provided by Couch Consulting Engineers. This building was chosen because it was a real case scenario, not an arbitrary architectural plan. The building is a six storey reinforced concrete frame with flat slab floors.

To perform the analysis, two different types of buildings based on the proposed architectural plan were chosen. These include a building with shear walls which was the existing structure (Fig 3) and another building without shear walls (Fig 4). In the second building, shear walls were replaced by columns (with the same orientation as shear walls in the X direction). Other orientations will lead to having a much stiffer frame in the Y direction and less stiff frame in the X direction which will lead to failure in interstorey drift. These replaced columns were designed with the same size as the existing columns within each floor.

The building's specifications, including its dimensions, concrete and steel material properties and the applied vertical loads are presented in table 1.



(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 X 27.3 m	-	-	-
Floor	Flat Slab 275&325 mm	-	-	-
Column	600 X 275 mm	-	-	-
	750 X 250 mm	-	-	-
Shear wall	Shell Thin 250 mm	-	-	-
Grade	-	C30/37	C 40/50	-
f_c'	-	30 MPa	40 MPa	-
Weight per unit volume	-	25 kN/m^3	25 kN/m^3	- -
E (Modulus of Elasticity)	-	33000 MPa	35000 MPa	-
Poisson's Ratio	-	0.2	0.2	-
G (Shear Modulus)	-	13750 MPa	14580 MPa	-
Grade	-	-	-	B500B
R_e	-	-	-	500 MPa
R_m/R_e	-	-	-	1.08
A_{gt}	-	-	-	5
Roof loads: Permanent ($\frac{KN}{m^2}$)	8.125	-	-	-
Roof loads: Imposed ($\frac{KN}{m^2}$)	1.5	-	-	-
Floor loads: Permanent ($\frac{KN}{m^2}$)	6.875	-	-	-
Floor loads: Imposed ($\frac{KN}{m^2}$)	2.5	-	-	-
Stairs loads: Permanent ($\frac{KN}{m^2}$)	4.3	-	-	-
Stairs loads: Imposed ($\frac{KN}{m^2}$)	4	-	-	-

Table 2 Static structural design 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-Eqn (4.1)
Terrain factor	0.21	Cl 4.3-Eqn (4.5)
Roughness Factor	0.77	Cl 4.3-Eqn (4.4)
Terrain Orography Factor	1 (Recommended)	Cl 4.3
Mean Wind Velocity	16.5 m/s	Cl 4.3-Eqn (4.3)
Turbulence Intensity	0.27	Cl 4.4-Eqn (4.7)
Basic Velocity Pressure	0.17 kN/m^2	Cl 4.5-Eqn (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-Eqn (4.1)
External Pressure Coefficient *	1.3	Cl 5.2-Eqn (5.1)
Wind Force (X)	346 kN	Cl 5.3
Wind Force (Y)	201 kN	Cl 5.3

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

Also, for the exterior walls, the edge load is $5.4\ \frac{kN}{m}$ in all directions.

The wind velocity can be seen as a mean plus a fluctuating component. According to Eurocode classification, wind actions are classified as variable, fixed, direct actions. In this study, the procedure to calculate the static structural design load (for 3-sco2econd load once in 50 years) is presented in Table 2 for Home Counties.

The simulation part for the buildings was done by ETABS which is an engineering software to analyse and design multi-storey buildings (Wiki.csiamerica.com, 2018). Over the past two decades, ETABS was utilised in various large-scale projects and has become the standard in the industry (Ceanet.com.au, 2018). This software is capable of conducting linear, non-linear, static and dynamic analyses.

In this study a Non-linear Dynamic Analysis was performed, since the P-delta effect was included in the simulations which comparing to a linear static analysis could produce more accurate results. Furthermore, due to the buildings' low-height, the wind-induced forces were a constant value which was distributed uniformly across the buildings' height.

3.2 Second stage

In the second part of the research, the possibility of constructing the same structure without shear walls is investigated by using the same structural properties in various UK locations. Since the wind pressure activities increase toward the north (Table 3), several big cities in England, Scotland and Northern Ireland (Not Wales, since their latitude and wind pressure value were not significantly different from England's location) with different latitude and wind pressure values were selected within the UK. The locations were Birmingham, Belfast, Edinburgh and Shetland (as the worst possible scenario due to the highest wind pressure in the UK region). This selection could evaluate the influence of different climate in the UK on the building's structural performance.

Table 3 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

Table 4 Maximum Storey Displacement Case 1 & 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
Storey 5	3.15	0.507	3.34	9.71	5.27
Storey 4	3.08	0.386	2.99	8.65	4.72
Storey 3	3.08	0.269	2.48	7.02	3.88
Storey 2	3.08	0.159	1.79	4.89	2.75
Storey 1	4.13	0.0682	0.977	2.51	1.43

4. Results and discussion

4.1 First stage

4.1.1 Structural Results

<u>*Displacements*</u>

The following procedure is presented to check the acquired results from the first stage (the comparative analysis of the two buildings with and without shear walls):

The comparison of the maximum storey displacement in both structures with (Case 1) and without shear walls (Case 2) is given in table 4 and illustrated in Fig 5.

From maximum storey displacement in table 4, it can be observed that due to the location and orientations of the shear walls in the X direction (stiffer axis), the lateral displacement in the case 1 shows more rigid behaviour (lower movements) in the X direction. However, in Y direction the values of lateral displacements for case 2 show almost the same performance in both directions.

Moreover, according to Fig 5, the displacement values in case 2 are higher than case 1 which demonstrate the

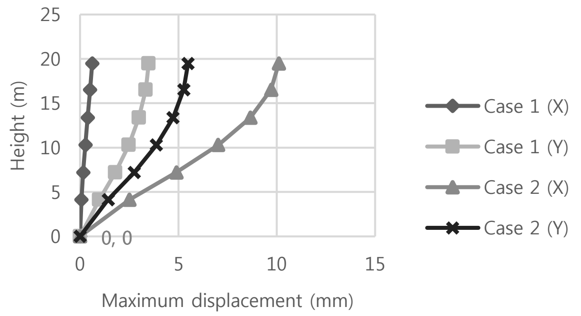


Fig. 5 Comparison of maximum displacement in Case 1&2

Table 5 Maximum Storey Drift

Storey	Height (m)	Case 1 Drift X d_r (mm)	Case 1 Drift Y d_r (mm)	Case 2 Drift X d_r (mm)	Case 2 Drift Y d_r (mm)	Limitation (mm)
Roof	2.96	0.111	0.152	0.444	0.221	5.92
Storey 5	3.15	0.122	0.345	1.06	0.548	6.31
Storey 4	3.08	0.119	0.521	1.63	0.846	6.15
Storey 3	3.08	0.111	0.688	2.13	1.13	6.15
Storey 2	3.08	0.092	0.814	2.38	1.32	6.15
Storey 1	4.13	0.068	0.977	2.51	1.43	8.25

influence of shear walls as the lateral resisting elements to withstand the horizontal displacements.

Interstorey drift

According to the acquired results from ETABS, maximum interstorey drift for wind actions in X and Y directions are presented in table 5.

According to BS 8110: Part 2 Cl 3.2.2.2, the limitation for interstorey drift in a concrete frame building subjected to wind actions is H/500.

The values of interstorey drift for case 1 and 2 are illustrated in table 5 which imply that neither of the buildings exceeds the limits defined by the BS 8110.

Figs 6a and 6b demonstrate a comparison between the interstorey drifts in each direction.

The interstorey drifts in case 2 appear to be higher than case 1, but as it can be observed from Fig 6a and 6b, they are within the safe range.

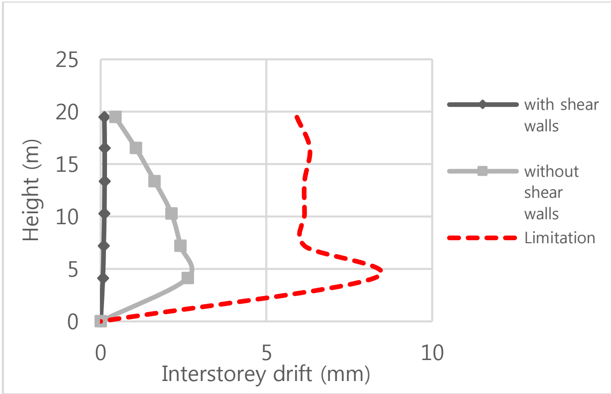
Accelerations

There are several guidelines regarding the occupants comfort criteria in buildings including BS 6472-1 and NBCC: Part 4. According to Banks *et al*. (2014), the typical values for a 10-year return period of wind-induced motion in North America (NBCC: Part 4) are:

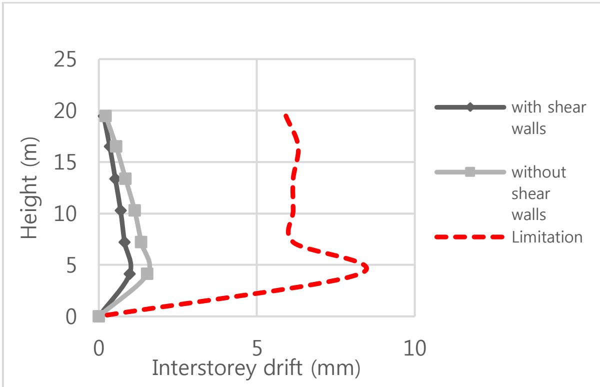
- 10 to 15 milli-g for residential occupancy
- 20 to 30 milli-g for office occupancy

Results of human perception and the acceleration of the buildings are presented in table 6.

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



(a) X-axis



(b) Y-axis

Fig. 6 Storey drifts ratio case 2

Table 6 Acceleration values for case 1 & 2

Mode of vibration	Frequency (cyc/sec) Case 1	Frequency (cyc/sec) Case 2	Max displacement (mm) Case 1	Max displacement (mm) Case 2	Acceleration (m/s^2) Case 1	Acceleration (m/s^2) Case 2	Acceleration (milli-g) Case 1	Acceleration (milli-g) 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

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

Which a, f and x represent acceleration, frequency and maximum displacement respectively.

Table 6 shows that the acquired acceleration from ETABS for case 1 and 2 fulfil the criteria defined by the NBCC design code.

Shear force

The storey shear represents the total lateral loads applied to the base of a structure which if the vertical elements are not strong enough it might result in shear failure.

In this study the storey shear applied by the wind actions are presented here and compared to the calculated design wind loads:

Based on the acquired wind shear forces in Fig 7, it can be observed that the obtained shear forces from the hand calculations are in the same range as ETABS results. The result is summarised in table 7.

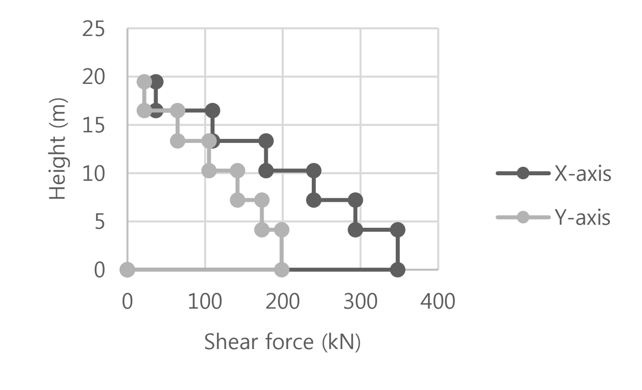


Fig. 7 Shear force comparison case 2 (X & Y axis)

Table 7 Comparison of Calculations

Method	Base Shear Force The x-axis (kN)	Base Shear Force Y-axis (kN)
Hand Calculations	346.1	201.1
ETABS	344.5	198.4
Percent Error (%)	0.46	1.3

Overturning moment

In a structure, the applied lateral loads will be multiplied by the height of the structure and create a moment at the base of the structure which in high magnitudes can result in overturning failure mechanism. The following calculations check the overturning moment for the building:

Based on the cumulative mass in both X & Y directions (49.15 MN) obtained from ETABS, the resisting moment could be calculated

Mass (calculated by ETABS) = 49.15 MN

$$M_ox = M_y + V_x h_f = 4.487 + 0.345 \text{ X } 1 = \mathbf{4.832 \text{ MN. m}} \text{ (Critical scenario)} \quad (2)$$

$$M_oy = M_x + V_y h_f = 2.545 + 0.198 \text{ X } 1 = 2.743 \text{ MN.m} \quad (3)$$

$$M_R = \text{Mass X Minimum lever arm (9.01 m or 8.69m)}$$

$$M_Rx = 49.15 \text{ X } 8.69 = \mathbf{427.1 \text{ MN.m}} \text{ (Critical scenario)}$$
$$M_Ry = 49.15 \text{ X } 9.01 = 442.8 \text{ MN.m} \quad (4)$$

The critical resisting moment is 427.1 ≥ 4.791 MN.m

These calculations demonstrated that the overall overturning moment was much lower than the resisting moment of the building.

Flat slab deflection

Eurocode 2 deals with a design for deflection in flat slabs by several approaches in which limiting span to depth ratio was used in this study.

In Table 8 the results for flat slab deflection are illustrated and the procedure to calculate the span to depth ratio is provided by Goodchild (2009) which is presented in Appendix A.

From Table 8, it can be observed that in both cases flat slab defection ratios are within the allowable L/d.

Table 8 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

Table 9 Punching shear reinforcement ratio (worst scenario)

Location	Ratio	Status
Case 1 (Building with shear walls)	0.75 (Storey5- Column F1)	Passed
Case 2 (Building without shear walls)	1.76 (Storey5- Column F1)	Passed

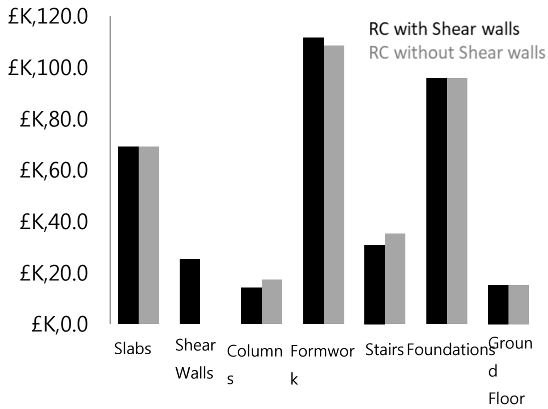


Fig. 8 Comparison of Structural Components cost

Punching shear (Case 1)

The results for punching shear ratio are presented in Table 9 and the detailed procedure to calculate the punching shear is presented in Appendix B.

The ratios in Table 9 demonstrate that the flat slabs in case 1 can provide adequate resistance to prevent shear failure, but in case 2 shear reinforcement is required.

4.1.2 Cost estimation

One of the objectives of this study was to estimate the cost difference between case 1 & 2. The cost estimates for construction in both cases is calculated by CONCEPT software which uses the rates obtained from Goodchild, Webster and Elliott (2009) publication to estimate the construction cost.

* It is required from the UK's building regulations that stair cores must be fire resistant. By eliminating shear walls around the staircase, fire resisting plasterboards were added to the cost estimation.

Based on table 10 and 11, since the shear walls were excluded from the building, the quantity of concrete and rebar were decreased, but the number of columns increased in the construction process

Moreover, the overall saving in case 2 is 0.67% of the estimated total cost (£2.89 million) which validates the cost-effectiveness of removing shear walls (even by replacing them with extra columns).

Concrete is an essential construction material but has negative impacts on the environment, e.g. CO_2 emissions,

Table 10 Cost Estimation Case 1

Component	Quantity	Rate	Quantity		Rate		Subtotal £K
Slabs	609 m³ @	£95 plus	15	T @	£750		69.1
Shear Walls	136 m³ @	£110 plus	13.6	T @	£750		25.2
Columns	42 m³@	£110 plus	12.6	T @	£750		14.1
Formwork (Vertical)			730	m² @	£32		23.4
Formwork (Horizontal - plain)			3043	m² @	£29		88.2
Formwork (Horizontal - ribbed)			0	m² @	£52.5		0.0
Hollow core units			0	m² @	See "Rates"		0.0
						Total "superstructure"	**220.0**
					72.3	***£/m²***	
Stairs as %age of superstructure cost						14%	30.8
Foundations			50772	kN @	£1.89		95.7
Ground floor slab			507	m² @	£30		15.2
Cladding			1816	m² @	£330		599.2
						Structure & cladding total	**960.9**
Prelims & external works						10%	293.1
Finishes & walls						21%	615.5
Mechanical & Electrical						35%	1025.9
						Total construction	2895.5
					951.6	*£/m²*	
						TOTAL	**2895.5**

Table 11 Cost Estimation Case 2

Component	**Quantity**	**Rate**	**Quantity**		**Rate**		**Subtotal £K**
Slabs	609 m³ @	£95 plus	15	T @	£750		69.1
Columns	71 m³ @	£110 plus	12.6	T @	£750		17.3
Formwork (Vertical)			630	m²@	£32		20.2
Formwork (Horizontal - plain)			3043	m²@	£29		88.2
Formwork (Horizontal - ribbed)			0	m²@	£52.5		0.0
Hollow core units			0	m²@	See "Rates"		0.0
						Total "superstructure"	194.8
					64.0	*£/m²*	
Stairs as %age of superstructure cost						14%	35.2
Sprayed mineral fibre coating (two hour) fire protection*	86 m² @	£11.79					1.0
Foundations			50772	kN @	£1.89		95.7
Ground floor slab			507	m² @	£30		15.2
Cladding			1816	m² @	£330		599.2
						Structure & cladding total	**941.1**
Prelims & external works						10%	293.1
Finishes & walls						21%	615.5
Mechanical & Electrical						35%	1025.9
						Total construction	2875.7
					945	*£/m²*	
						TOTAL	**2875.7**

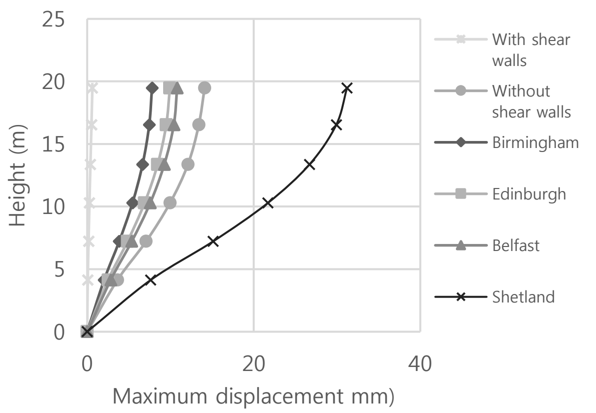
and in this study, it was illustrated the quantity of concrete in case 2 reduced. This has a positive impact on the sustainability of concrete construction. Rreducing the volume of concrete reduces the negative environmental impact.

Table 12 Maximum Displacement Case 3 to 6

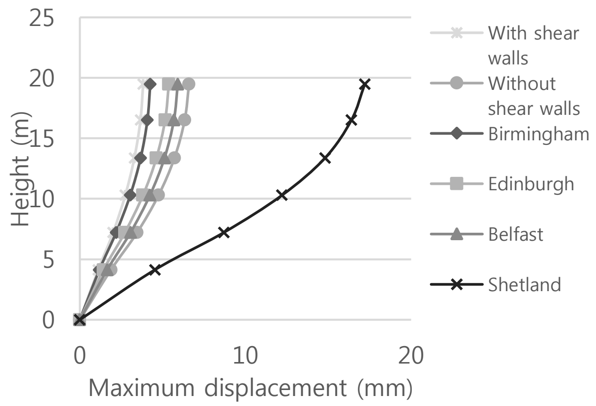
Storey	Height (m)	Case 3 (Birmingham)		Case 4 (Edinburgh)		Case 5 (Belfast)		Case 6 (Shetland)	
		X-Axis (mm)	Y-Axis (mm)	X-Axis (mm)	Y-Axis (mm)	X-Axis (mm)	Y-Axis (mm)	X-Axis (mm)	Y-Axis (mm)
Roof	2.96	7.78	4.25	9.93	5.37	10.8	5.91	31.2	17.2
Storey 5	3.15	7.45	4.08	9.49	5.15	10.4	5.68	29.9	16.4
Storey 4	3.08	6.65	3.67	8.46	4.62	9.26	5.11	26.7	14.8
Storey 3	3.08	5.44	3.04	6.89	3.81	7.57	4.23	21.7	12.2
Storey 2	3.08	3.84	2.19	4.84	2.72	5.35	3.06	15.1	8.69
Storey 1	4.13	2.03	1.18	2.51	1.43	2.82	1.65	7.62	4.53

Table 13 Maximum Storey Drift in Cases3-6

Storey	Case 3 (Birmingham)		Case 4 (Edinburgh)		Case 5 (Belfast)		Case 6 (Shetland)		Limitation (mm)
	Drift X dr (mm)	Drift Y dr (mm)	Drift X dr (mm)	Drift Y dr (mm)	Drift X dr (mm)	Drift Y dr (mm)	Drift X dr (mm)	Drift Y dr (mm)	
Roof	0.336	0.166	0.434	0.215	0.467	0.231	1.34	0.661	5.92
Storey5	0.799	0.412	1.03	0.533	1.11	0.573	3.23	1.66	6.31
Storey4	1.22	0.631	1.57	0.817	1.69	0.879	4.99	2.59	6.15
Storey3	1.59	0.847	2.06	1.09	2.22	1.18	6.63	3.53	6.15
Storey2	1.82	1.01	2.33	1.29	2.53	1.41	7.46	4.17	6.15
Storey1	2.03	1.18	2.51	1.43	2.82	1.65	7.62	4.53	8.25



(a) X-axis



(b) Y-axis

Fig. 9 Comparison of Maximum Displacements in all locations

4.1.3 Time estimation

It is evident that the existing RC frame building had been designed as a cast in-situ concrete frame. It has been suggested that cutting out shear walls might cut out a day in a 14-day cycle and this amount of time could be saved in the construction process.

4.2 Second Stage

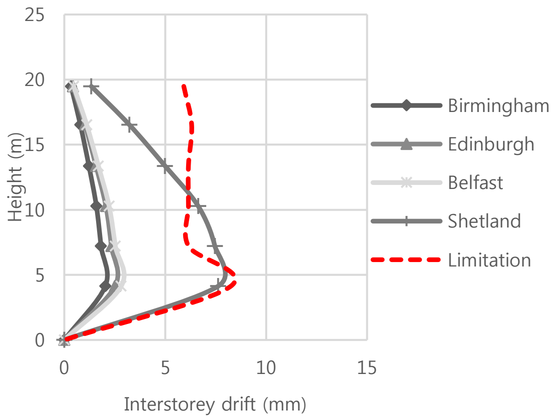
Maximum displacement

In this section, the structural analyses results obtained from the various UK locations are presented and discussed. These include the comparison of the maximum storey displacement for Birmingham (Case 3), Edinburgh (Case 4), Belfast (Case 5) and Shetland (Case 6) as demonstrated in table 12 and Figs 9a and 9b.

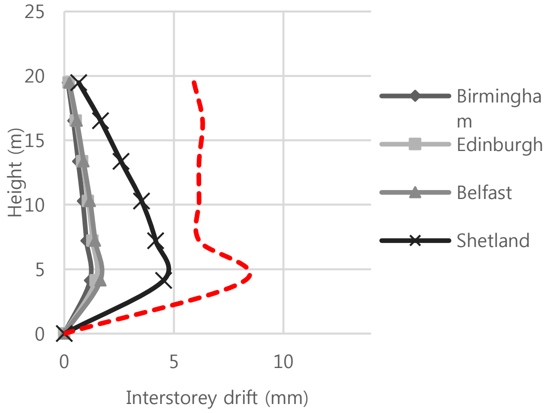
The maximum displacements in cases 3 to 6 are illustrated in table 12. Based on the acquired results in Fig 9a for all of the six cases, it was observed that the lateral displacement in the X direction for case 1 had the lowest value which is the result of utilising the shear walls as expected. The highest lateral displacement was for case 6 (Shetland) which is corresponding to the value of wind pressure calculated in table 3. Furthermore, as illustrated in Fig 9b, the lowest lateral displacement in Y direction was for case 1 (building with shear walls) which seems reasonable considering the presence of shear walls and the highest value was for case 6 (Shetland) as expected.

Interstorey drift

The comparison of interstorey drift in Case3-6 is presented in table 13.

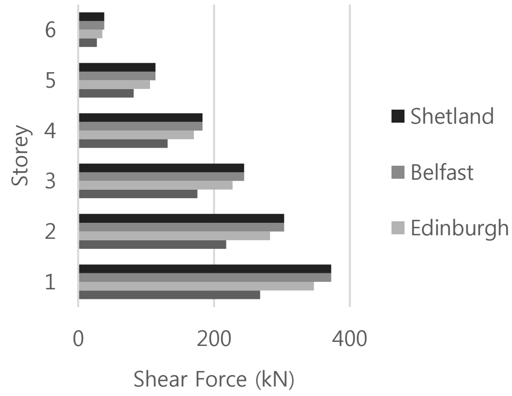


(a) X-axis

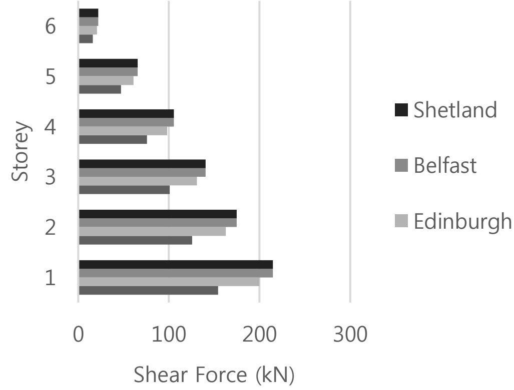


(b) Y-axis

Fig. 10 Comparison of interstorey Drift



(a) X-axis



(b) Y-axis

Fig. 11 Shear force comparison in case 3 to 6 (Birmingham, Edinburgh, Belfast and Shetland)

Table 14 Acceleration in case 3 to 6

Mode	Frequency (Hz)				Acceleration (milli-g)			
	Case3	Case4	Case5	Case6	Case3	Case4	Case5	Case6
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

The values of interstorey drift for cases 3 to 6, as it is shown in table 13, concludes 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. Additionally, Figs 10a and 10b demonstrate the results based on the height of the buildings which highlights that in all the cases the interstorey drift values decreased as the height of the structure increased. This is expected as usually the value of interstorey drifts in structures drop with the increment of the height.

Acceleration

The assessment of building response regarding human response for cases 3, 4, 5 and 6 are illustrated in table 14.

It is evident that the calculate accelerations for other locations (Birmingham, Edinburgh, Belfast) are within the defined safe range. However, this is not the case for Shetland because the acceleration has exceeded the safe range and it failed to provide occupants comfort.

Since the beginning of this study, it was pointed out that the Shetland case was chosen only as the worst possible case.

Shear force

Figs 11a and 11b demonstrate that the shear forces in the X direction in cases 3, 4, 5 and 6 tend to be higher than shear forces in the Y direction. It could be because of smaller dimensions and subsequently lower resistance in the X direction comparing to the Y direction.

It is also important to mention that based on the results, as the shear forces in a building increase the stability of the structure in global analysis increases accordingly. However, this needs further investigation since from a certain level the individual elements (especially columns) cannot resist the applied shear forces and will fail.

Flat slab deflection

The calculations to check deflection in flat slab were provided earlier. In table 15 only the values are presented in table 15:

These calculations are based on eqn. 6.11b in Eurocode 2 in which wind actions take place in the design calculations.

Table 15 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 16 Punching shear reinforcement ratio (worst scenario)

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

It was evident that deflection values in flat slabs for Birmingham, Edinburgh and Belfast were within the safe range defined by Eurocode 2; however, Shetland failed to fulfil the criteria.

Punching Shear

The calculation to design the punching shear reinforcement was provided earlier. In table 16 the results are presented.

In table 16 the shear reinforcement ratio is calculated. Except for Shetland which failed to provide enough resistance, all the other cases passed the criteria with shear reinforcement to prevent punching shear failure.

5. Conclusion

In this research, the significance of utilising shear walls in RC frame buildings are assessed in two stages. In the first stage, the feasibility of removing shear walls in an existing UK residential building is investigated when the building is subjected to wind-induced actions, and building performance, cost-effectiveness and sustainability of construction are reviewed by using ETABS and CONCEPT software. In the second stage, after validating the effectiveness of removing shear walls, the application of constructing the same building in various locations in the UK subjected to different wind loads are investigated, and global performance of the buildings are discussed.

Based on the acquired results from the first and second stage of this research the following conclusions can be drawn:

- The comparison of maximum lateral displacements in case 1 and 2 demonstrates that although case 1 (with shear walls) performed a stiffer behaviour, case 2 still can provide adequate serviceability and strength within the safe range defined by Eurocode 2.
- The results from interstorey drift in table 5 and 12 show that in all cases except for case 6 (Shetland) the values were within the safe range defined by BS 8110.
- The calculations section (Table 6 and 14) show the accelerations generated by the buildings with and without shear walls (in cases 1 and 2) and locations 3, 4 and 5 (Birmingham, Edinburgh and Belfast) are within the allowable range, and the comfort of the occupants will not be compromised. However, Shetland failed to provide occupants comfort by its high acceleration values.
- According to the acquired values from the overturning moment, it can be concluded that wind actions in low-rise buildings in the UK do not have a substantial impact to cause overturning failure mechanism.
- Deflection in flat slabs, as an essential check in Eurocode2, was checked in this study and the results obtained that, except for Shetland, all the other 5 cases (Buildings with and without shear walls, Birmingham, Edinburgh, and Belfast) provided enough safety in accordance to Eurocode 2.
- It was evident that flat slabs are vulnerable to punching shear and require shear reinforcement to prevent failure.
- By eliminating shear walls in the building, around 0.67% of the overall construction cost was saved, even when it was substituted by extra columns.
- Eliminating the shear walls also resulted in a reduction of construction time by one day in the 14-day cycle.
- This study illustrated that low-rise buildings in the UK could be constructed safely without shear walls and provide adequate serviceability and strength within the safe range defined by Eurocode 2.
- Constructing RC frame buildings in the UK without shear walls reduces the construction cost and time without affecting the buildings' structural performance.

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Appendix A

The calculations for flat slab deflection are presented here:

In case **1** (building with shear walls) the slab between EF-3 to EF-5 due to its wide span was chosen.

The combination of actions was taken from EN 1990-2002 eqn. (6.10b)

$$\sum_{j\geq 1} \xi_j \gamma_{G,j} G_{k,j} + \gamma_P P + \gamma_{Q,1} Q_{k,1} + \sum_{i>1} \gamma_{Q,i} \psi_{0,1} Q_{k,i} \tag{5}$$

$$n = 1.25 \times 8.125 + 1.5 \times 4 + 0.7 \times 0.73 = 16.7\ KN/m^2$$

Design moment in the span

$$M_{Ed} = (1.25 \times 8.125 \times 0.09 + 1.5 \times 4 \times 0.1 + 0.7 \times 1 \times 0.73) \times 9.4^2 \times 5.65 = 1010\ kN.m$$

Design moment in the support

$$M_{Ed} = 16.7 \times 0.106 \times 9.4^2 \times 5.65 = 883\ kN.m$$

Table 17 Apportionment of moments between column strips and middle strips

	M_{Ed}	
	Column strip	Middle strip
-ve (hogging)	0.7 x 883/2.35 = 263 kN.m/m	0.3 x 883/2.35 = 113 kN.m/m
+ve (sagging)	0.5 x 1010/2.35 = 215 kN.m/m	0.5 x 1010/2.35 = 215 kN.m/m

$$K = {M_{Ed}}/{bd^2 . f_{ck}} = 215 \times \frac{10^6}{1000 \times 287^2 \times \frac{30}{1.5}} = 0.13 \tag{6}$$

Z/d = 0.856 (Obtained from table 5 in Bond et al. (2006))
Z= 0.856 x 287 = 249 mm

$$A_s = M_{Ed}/f_{yd}Z = 215 \times 10^6/(500/1.15 \times 249) = 1985\ mm^2 \tag{7}$$

Provided 7 B500B @ 142 mm = 2198 mm^2

$$\rho = \frac{A_s}{bd} = 1985 \times \frac{100}{1000} \times 285 = 0.67\% \tag{8}$$

Deflection middle strip

$$\text{Allowable l/d} = N \times K \times F_1 \times F_2 \times F_3 \tag{9}$$

N = 21.2 (taken from table C10 in Goodchild (2009))

K = 1.2 (for flat slab)

$F_1 = F_2 = 1$

$$F_3 = 310/\sigma_s \text{ (Eqn. 7.17 Eurocode 2)} \tag{10}$$

$$\sigma_s = \sigma_{su}(A_{s,req}/A_{s,prov})/\delta \tag{11}$$

$\sigma_{su} =$ 250 MPa (taken from Fig C3 in Goodchild (2009))
δ = 1.03 (taken from table C14 in Goodchild (2009))

$$\sigma_s = 250 \times (1985/2198)/1.03 = 219$$

F_3 = 310/219 = 1.41

Allowable l/d = 21.2 x 1.2 x 1 x 1 x 1.41 =35.9

Actual l/d = 9400/287 = 32.7

Allowable l/d = 35.9 ≥ Actual l/d = 32.7 → OK

Based on the calculations, it was confirmed that deflection in the flat slabs is not an issue and the values are within the safe range defined by Eurocode 2.

In case 2 the deflection check values (building without shear walls) are:

Actual l/d = 9400/287 = 32.7

Allowable l/d = 33.4 ≥ Actual l/d = 32.7 → OK

Appendix B

Column F3 on roof (the worst scenario) was chosen. The procedure was given by Goodchild (2009):

The following values were taken from ETABS design results:

Effective punching perimeter = 3095 mm

Shear force = 439.4 kN

Design shear stress = 0.75 MPa

Concrete shear stress capacity = 0.48 MPa

Punching shear ratio = 1.57

According to Eurocode 2 CL 6.4.3 (2) the following checks should be carried out:

1. $V_{Ed} \leq V_{Rd,max} \rightarrow$ 0.75 MPa ≤ 3.6 MPa **Pass**

$$V_{Rd,max} = 0.4\ \upsilon f_{cd} = 0.4 \times 0.53 \times 17 = 3.6 \text{ MPa}$$
$$\upsilon = 0.6(1-f_{ck}/250) = 0.6 \times (1-30/250) = 0.53 \text{ MPa} \tag{12}$$

2. $V_{Ed} \leq V_{Rd,c} \rightarrow$ 0.75 MPa ≥ 0.48

Punching shear reinforcement is required

Also the UK NA recommends $V_{Ed} \leq 2V_{Rd,c}$ which in this case 0.75 MPa ≤ 0.96 MPa

Perimeter at which punching shear links are no longer required:

$$U_{out} = V_{Ed} \times \beta/(d \times v_{Rd,c}) = 621.8 \times 10^3 \times 1.4/287 \times 0.56 = 5416\ mm \tag{13}$$

Length attribute to column face = 600 + 2 x 275 = 1150 mm

Radius to U_{out} from face of column: $(5608 - 1150)/\pi = 1358\ mm$

Perimeter of shear reinforcement may stop at: 1414-1.5x275 =

1002 mm from face of column

S_r = 275 x 0.75 = 207 say = 200 mm (According to cl. 9.4.3(1) Eurocode 2)

S_t = 275 x 1.5 = 415 say = 400 mm (According to cl. 9.4.3(1) Eurocode 2)

$F_{ywd.ef}$ = (250 + 0.25d) = 250 + 0.25 x 287 = 322 MPa ≤ f_{yd} (14)

f_{yd} = 500/1.15 = 434 MPa → OK

$$Asw \geq \frac{\left(V_{Ed} - 0.75V_{Rd,c}\right)S_r U_1}{1.5 f_{ywd,ef}} = (1.46 - 0.75 \times 0.54) \times 200 \times 2677.5/(1.5 \times 322) = 1023\ mm^2 \quad (15)$$

Per perimeter

$$A_{sw,min} \geq 0.08 \times \frac{f_{ck}^{0.5}(S_r S_t)}{1.5 f_{yk}} = 0.08 \times 30^{0.5}(200 \times 400)/(1.5 \times 500) = 46 mm^2 \quad (16)$$

$$A_{sw}/U_i = 1023/2677.5 = 0.38 mm^2 \quad (17)$$

Using H8 (50 mm^2) maximum spacing = 50/0.38 = 132 mm

H8 shear reinforcement at 132 mm is provided to prevent punching shear failure.

Punching shear values for case 2 (building without shear walls) are:

Punching shear ratio = 1.76 and the ratio is less than 2 according to $V_{Ed} \leq 2V_{Rd,c}$

To prevent punching shear failure H8 at 124 mm shear reinforcement will be provided.