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Investigations of elastic vibration periods of tall reinforced concrete office buildings

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Abstract. The assessment of wind-induced vibration for tall reinforced concrete (RC) buildings requires the accurate estimation of their dynamic properties, e.g., the fundamental vibration periods and damping ratios. In this study, RC frame-shear wall systems designed under gravity and wind loadings have been evaluated by utilising 3D FE modelling incorporating eigen-analysis to obtain the elastic periods of vibration. The conducted parameters consist of the number of storeys, the plan aspect ratio (AR) of buildings, the core dimensions, the space efficiency (SE), and the leasing depth (LD) between the internal central core and outer frames. This analysis provides a reliable basis for further investigating the effects of these parameters and establishing new formulas for predicting the fundamental vibration periods by using regression analyses on the obtained results. The proposed constrained numerically based formula for vibration periods of tall RC frame-shear wall office buildings in terms of the height of buildings reasonably agrees with some cited formulas for vibration period from design codes and standards. However, the same proposed formula has a high discrepancy with other cited formulas from the rest of design codes and standards. Also, the proposed formula agrees well with some cited experimentally based formulas.

Keywords: reinforced concrete; shear walls; office buildings; vibration period; wind load; space efficiency; leasing depth

1. Introduction

The assessment of wind-induced vibration for tall RC buildings requires the accurate estimation of their dynamic properties, e.g., fundamental vibration periods and damping ratios. These dynamic properties are required not only for assessing the effect of dynamic loading, e.g., wind load for design purposes, but also for ascertaining the comfort of the occupants, in particular for tall buildings (Stafford Smith and Coull 1991). Thus, the dynamic properties of various structural systems for buildings should be investigated to identify their dynamic behaviour under lateral loading, i.e., earthquake and wind. These structural systems perform differently due to the inclusion of structural and non-structural elements, e.g., columns, beams, shear walls, floor slabs, infills and the interactions between them in the whole structure to resist the applied actions. Therefore, some systems are used for limited heights, e.g., RC moment-resisting frame systems can be used for up to 20-25 storeys. With the increase the height of buildings, however, other alternative structural systems, e.g., RC shear walls, RC frame-shear walls, RC tube structures, etc., should be used due to the dominant effect of the lateral loadings, e.g., wind and earthquake loads rather than gravity loadings, on tall RC buildings (Stafford Smith and Coull 1991, Taranath 2009).

To simplify the modelling in practice, the bare frame is used to simulate structural elements for a typical structural analysis. On the other hand, the effects of non-structural elements, e.g., infill walls and floor slabs (assumed as rigid diaphragms), are simply ignored. Actually, the lateral stiffness of infill walls significantly contributes towards the lateral stiffness of the bare frame structure as investigated in the previous study (Al-Balhawi and Zhang 2017). In particular, RC floor slabs, e.g., semi-rigid or flexible diaphragms with the flexural stiffness contributed by floor slabs in RC shear wall structures, may affect their lateral stiffness and then alter their dynamic properties (Ju and Lin 1999, Lee *et al.* 2002). Also, the ignorance of the effects of these non-structural elements in RC shear wall structures can result in high discrepancy on the results obtained from full-scale tests on RC structures for obtaining dynamic properties (Su *et al.* 2005, Kim *et al.* 2009).

Various methods have been so far applied to obtain the dynamic properties, including experimental approaches (i.e., ambient vibration tests) and analytical and numerical approaches (i.e., continuum mechanics and finite element methods (FEM)). The former methods are used to calibrate the latter ones and verify the obtained dynamic properties of a structure (Brownjohn *et al.* 2000, Balendra *et al.* 2003, Kim *et al.* 2009, Panzera *et al.* 2013, Yoshida and Tamura 2015, Li and Yi 2016). However, the higher cost and other barriers of performing experimental investigations and the continuous development of powerful computer software provide the opportunity of adopting other methods, e.g.,

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FEM, for obtaining the dynamic characteristics of various structural systems. Also, the latter methods provide the opportunity to investigate different parameters that may affect the dynamic properties of RC systems.

In this study, RC frame-shear wall systems for tall RC office buildings were investigated in terms of the elastic vibration period. These systems were designed by considering the effects of dead, imposed and wind loads according to Eurocodes, i.e., BS EN 1990 (BSI 2005a), BS EN 1991-1-1 (BSI 2002), BS EN 1991-1-4 (BSI 2005b), BS EN 1992-1-1 (BSI 2004a) and BS EN 1992-1-2 (BSI 2004b), and then a parametric study was conducted to investigate the effects of several design parameters on the dynamic response of the models in terms of the elastic vibration period. The parameters studied were the number of storeys, the plan aspect ratio (AR) of buildings, the core dimensions, the space efficiency (SE), and the leasing depth (LD) between the internal central core and outer frames. Due to the plan configurations of these systems, these parameters largely influence the dynamic response of tall RC office buildings.

However, there is a lack of investigations in particular on the effect of the last three parameters for tall RC office and tall RC residential shear wall buildings though they have attracted more attention than their counterparts. Thus, with the availability of powerful computers, numerical analysis using FEM can be used to evaluate the dynamic behaviour of these systems, i.e., elastic vibration periods. Also, the obtained numerical results of the elastic vibration periods can be used as a reliable basis for establishing the formulas of the vibration period for these systems. Accordingly, these proposed formulas will be compared with the corresponding experimentally and numerically based formulas cited from literature.

2. Design codes and standards for buildings

For evaluating the vibration period of RC shear wall structures, many design codes and standards have adopted formulas to estimate this dynamic property. The American Standard ASCE7 (2010) provides various expressions for predicting the fundamental vibration period of RC shear wall buildings. For wind design, the standard adopts the upper bound expression which was originally proposed by Goel and Chopra (1998). The standard states that the adoption of the upper bound expression for wind design instead of the recommended lower bound expression in the seismic design is related to providing conservative wind design in terms of the gust effect factor and the design wind pressure. Some limitations need to be verified with respect to the height and effective width of the building with a regular plan to apply the recommended expression. Hence, the cited design formulas are presented in terms of the period of vibration, T, rather than the fundamental frequency (the inverse of T), and the International System of Units (SI) is used as

$$T = 0.0085 \frac{H}{\sqrt{C_w}} \tag{1}$$

with

$$C_w = \frac{100}{A_B} \sum_{i=1}^{n} \left(\frac{h}{h_i} \right)^2 \frac{A_i}{\left[1 + 0.83 \left(h_i / D_i \right)^2 \right]} \tag{2}$$

where H or h is the building height, C_w represents the ratio of the effective area of shear walls to the total floor area of the building in the evaluated direction, n is the number of shear walls in the evaluated direction, A_i is the area of the single shear wall in the considered direction, A_B is the plan area of the building, h_i is the height of the single shear wall in the evaluated direction, and D_i is the length of the single shear walls in the evaluated direction.

The standard also provides other expressions obtained from analytical studies on the wind tunnel tests, which can be applied to all buildings with a height less than 122 m, regardless of their material types. These expressions are in terms of the building height H based on the regression analyses on the obtained analytical results as

$$T = \alpha H^{\beta} \tag{3}$$

where α and β are empirical coefficients which are listed in Table 1. BS EN 1991-1-4 (BSI 2005b) and the Australian and New Zealand Standard AS/NZS 1170.2 (2011) recommend a similar formula as Eq. (3) for obtaining the fundamental period of vibration for all types of buildings, which was first proposed by Ellis (1980), with the corresponding empirical coefficients cited in Table 1. In Japan, many studies, such as those done by Suda *et al.* (1996), Sasaki *et al.* (1997), Tamura *et al.* (2000) and Satake *et al.* (2003), have used the form of Eq. (3) to predict the natural vibration period of various structures. These studies used the obtained data from ambient vibration tests on buildings in Japan and have been reflected in the specified document for damping in buildings proposed by the Architectural Institution of Japan (AIJ 2000), which recommends a similar formula for obtaining the fundamental vibration period with the corresponding empirical coefficients cited in Table 1.

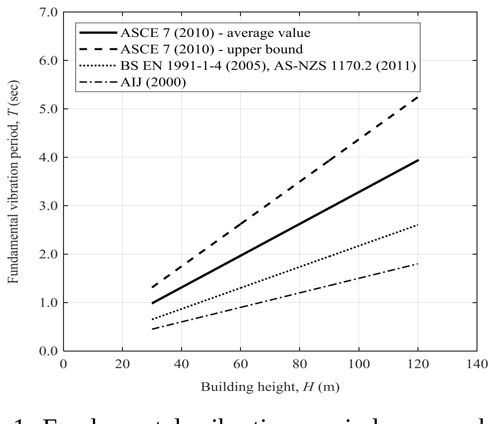


Fig. 1 Fundamental vibration period versus building height in the design codes and standards

Table 1 Empirical coefficients in the formulas for vibration period in the design codes and standards

Codes/Standards	Empirical coefficients		Remarks
	α	β	
ASCE 7 (2010)	0.0328	1	Average-value expression
	0.0437	1	Upper-bound expression
BS EN1991-1-4 (2005b) / AS/NZS 1170.2 (2011)	0.0217	1	-
AIJ (2000)	0.0150	1	For habitability level

Table 2 Empirical coefficients in the formulas for vibration period from the recent experimental studies on RC framed buildings with shear walls

References	Empirical coefficients		Remarks
	α	β	
Lagomarsino (1993)	0.0182	1	For 52 RC buildings
Goel and Chopra (1998)	0.0268	0.9650	For all data
Su *et al.* (2003)	0.0130	1	For $H > 50$ m
Poovarodom *et al.* (2004)	0.0190	1	For 50 RC buildings
Yoon and Ju (2004)	0.0190	1	For 17 RC wall buildings
Jalali and Milani (2005)	0.0260	0.8500	For RC dual-systems with infills
Kwon and Kim (2010)	0.0366	0.7500	For the lower-bound expression
Michel *et al.* (2010)	0.0130	1	For 127 RC buildings
Gilles and McClure (2012)	0.0190	1	For best fit constrained expression
Velani and Kumar (2016)	0.0150	1	For best fit constrained expression

The variations in the recommended formulas for vibration periods among the design codes and standards indicate the various dynamic properties of RC buildings around the world due to various influencing factors. These factors include the structural system of buildings, the plan of buildings, the height of buildings, the construction practice, the history response of buildings under different amplitudes of motion, etc. (Shan *et al.* 2013). Hence, the cited formulas of the design codes and standards are shown in Fig. 1.

3. Experimental studies

So far, most data for the vibration periods of shear wall structures are only available for residential buildings where the box form is the most popular system. However, there is a lack of such information for RC frame-wall office buildings. In this research, this type of buildings is investigated by conducting a parametric study with performing numerical analyses to obtain the fundamental vibration periods. More experimental results are needed to verify the behaviour of these buildings and to provide the basis for predicting this dynamic property when designing new RC frame-wall office buildings. Here, the available experimental studies about RC frame and shear wall buildings are included in this study where the results for low amplitude motions, e.g., ambient vibration tests, are cited for wind design assessment and the expected response will be linearly elastic.

Lagomarsino (1993) investigated the vibration periods and damping ratios of various structural systems and construction materials in Italy. He performed regression analyses in correlation with the analytical cantilever beam model on the testing data collected from 185 buildings to establish formulas for predicting the vibration periods for both lowest and higher modes. Due to the scope of the current study in relation to RC shear wall buildings, only the corresponding proposed formula for evaluating the fundamental period is quoted. For 52 RC buildings, the proposed formula for the fundamental vibration period in terms of the building height is in the same form as Eq. (3) with the empirical coefficients listed in Table 2.

Goel and Chopra (1998) investigated the fundamental vibration periods of RC shear wall buildings based on the data obtained from the recorded earthquake motions. They proposed the best-fit formulas with two bounds for estimating the vibration periods of shear wall buildings in terms of the building height and the effective area of shear walls in the evaluated direction. ASCE 7 (2010) adopts the upper-bound expression instead of the lower-bound one in the earthquake design for predicting the vibration period of shear wall buildings designed under wind load as indicated in Eq. (1). They stated that the proposed formulas can only be applied to uncoupled shear wall buildings. However, for coupled shear walls with other systems, i.e. moment-resisting frames, other formulas should be used. By performing an unconstrained regression analysis on their experimental data, a formula similar to Eq. (3) was also obtained with the empirical coefficients presented in Table 2. However, the corresponding linear correlation coefficient R^2 = 0.5560 is very low, and hence they adopted another formula in terms of the height and effective area ratio of shear walls to the building area in the considered direction.

Lee *et al.* (2000) performed ambient vibration tests on 50 RC apartment buildings with shear walls in Korea to evaluate their fundamental vibration periods. The number of storeys ranged between 10 and 25. They indicated that the vibration period obtained from the earthquake excitation is greater than those obtained from the ambient vibration tests in relation to the stiffness degradation due to cracking. They proposed a formula for evaluating the vibration period of RC shear wall apartment buildings in terms of the building height and the ratio of the wall length to the floor area through the constrained regression analysis for the measured periods as follows

$$T = 0.4\ \frac{H^{0.2}}{\sqrt{L_{\mathrm{w}}}} - 0.5 \tag{4}$$

where L_{w} is the ratio of the shear wall length to the floor area in the evaluated direction. Su *et al.* (2003) performed the ambient vibration tests for six RC residential buildings in Hong Kong to evaluate their fundamental vibration periods and damping ratios. The height of buildings ranged between 53 m and 126 m. They also used the 3-D finite element modelling to calibrate the numerically simulated results against the full-scale measurements. They proposed the equations for predicting the vibration periods based on different modelling assumptions, i.e., bare frames, frames with non-structural elements, etc. They found that the numerical results for bare frames without non-structural components and with the accurate stiffness of concrete modelling can significantly overestimate the behaviour of the existing buildings. They proposed the formula for the fundamental vibration period of tall buildings with $H > 50$ m, in terms of the building height as Eq. (3). The corresponding empirical coefficients are listed in Table 2.

Poovarodom *et al.* (2004) examined the natural periods and mode shapes by performing the ambient vibration tests on 50 RC buildings in Bangkok. The number of storey varied between 5 and 54 with the height ranging between 20 m and 210 m. They stated that the buildings of 15 to 25 storeys were located on soft soil and most were sensitive to earthquakes. These buildings were not designed seismically. The constrained regression analyses were performed on the obtained data to establish formulas for evaluating the fundamental vibration period of the buildings in terms of the number of storeys and the height of buildings. The corresponding empirical coefficients for the proposed formula in terms of the building height similar to Eq. (3) are listed in Table 2.

Yoon and Ju (2004) investigated the natural periods and damping ratios of tall buildings in Korea, including 21 steel buildings and 17 RC wall buildings. The number of storeys varied between 11 and 25, and the building height ranged between 28.5 m and 67 m. They used the microtremor tests on these buildings to obtain the dynamic properties. For the tall RC wall buildings, the corresponding empirical coefficients for the proposed formula in terms of the building height similar to Eq. (3) are listed in Table 2.

Jalali and Milani (2005) performed the ambient vibration tests on 30 RC buildings and 30 steel buildings in Iran to evaluate their fundamental vibration periods. The tested RC buildings included dual system of shear walls and moment-resisting frames with infill walls. Also, the height of buildings ranged between 16 m and 75 m. Based on the obtained data, they performed regression analyses to establish similar expressions for evaluating the fundamental vibration periods in terms of the height of buildings to Eq. (3), with the corresponding empirical coefficients listed in Table 2. Also, they propose another expression for RC dual systems in terms of the height and plan dimension (depth) of buildings in the considered direction as follows

$$T = 0.07\ \frac{H}{\sqrt{D}} \tag{5}$$

Kwon and Kim (2010) investigated the vibration periods of different structural systems in situ. 141 buildings were selected from the California Geological Survey (CGS) stations. Also, they used other data in literature to combine with those obtained in their study. To minimise the effect of nonlinear responses from structures or soils, the periods from low-intensity seismic events were used in the study. They used a total of 91 RC buildings including 56 RC shear walls, 23 reinforced masonry (RM) and unreinforced masonry (URM) shear walls, and 12 precast concrete (PC) tilt-up shear walls to establish a lower bound expression for evaluating the vibration periods for seismic design similar to Eq. (3) and the corresponding empirical coefficients are listed in Table 2.

Michel *et al.* (2010) investigated the dynamic properties of 127 RC buildings in France by conducting the ambient vibration tests. They found that the non-structural elements in the shear wall buildings had minor influence due to the high stiffness of the shear wall buildings. They indicated that the ambient vibration data provided the opportunity for accepting or rejecting the relationships obtained using the analytical methods. In addition, they observed that the height or storey number of buildings contributed to 85-90% of the variances in the vibration period. However, the length of the building in the considered direction had a low partial correlation coefficient. The corresponding empirical coefficients for the proposed formula in terms of the building height similar to Eq. (3) are also listed in Table 2.

Gilles (2011) explored the vibration periods and damping ratios for the low and high modes by conducting the ambient vibration tests on 39 multi-storey buildings in Montreal, Canada. The height of buildings ranged between 12 m and 195 m. 27 RC shear wall buildings were used for establishing the formulas for predicting the vibration period by utilising unconstrained and constrained regression analyses similar to Eq. (3). The corresponding empirical coefficients of the constrained regression analysis are listed in Table 2. As the current study deals with the design of RC frame-shear wall buildings under wind load, the best fit constrained expression is applied instead of the lower bound expression used for conservative earthquake design.

Velani and Kumar (2016) performed ambient vibration tests on 32 tall RC buildings in India to evaluate the corresponding fundamental vibration periods. The number of storeys ranged between 16 and 42. Based on the obtained data, they used both unconstrained and constrained regression analyses to establish the formulas for evaluating the fundamental vibration period in terms of the height of buildings, the dimension of buildings in the considered direction, and the area of buildings. Similarly, only the best fit constrained expression similar to Eq. (3) is applied with the corresponding empirical coefficients listed in Table 2.

4. Numerical studies

Balkaya and Kalkan (2003) performed 3-D finite element analyses on 80 RC shear wall (tunnel form) buildings with various configurations to evaluate the fundamental vibration period and propose new expressions to evaluate this dynamic property. The number of storeys ranged between 2 and 15. Hence, the simulated models were shear walls and flat plate slabs without beams and columns. This type of buildings is commonly used for public and residential usages. Based on the nonlinear regression analyses on the numerical results, an expression was the proposed as Eq. (6) by including the polar moment of inertia to consider the fundamental torsional behaviour of many models relevant to the plan dimensions and shear wall configurations

$$T = C\, h^{b_1} \beta^{b_2} \rho_{as}^{b_3} \rho_{al}^{b_4} \rho_{min}^{b_5} J^{b_6} \tag{6}$$

where h is the total height of the building in m, β is the dimension ratio of the long-side to the short-side, ρ_{as} is the ratio of the short-side shear wall area to the total floor area, ρ_{al} is the ratio of the long-side shear wall area to the total floor area, ρ_{min} is the ratio of the minimum shear wall area to the total floor area, J is the polar moment of inertia of the plan, and C and b_1 to b_6 are the constants obtained from the nonlinear regression analyses. Here, no formula for the vibration period in terms of the height of buildings was proposed only as the majority of the simulated models have fundamental torsion mode behaviour.

Vuran *et al.* (2008) examined the response parameters of dual-frame-wall systems in Turkey by performing 3-D fibre-based finite element modelling and using the displacement-based adaptive pushover analyses instead of the force-based analyses to obtain the response parameters, e.g., yield periods, deformed shape, and effective heights of the studied buildings in order to define the single degree of freedom system (SDOF) characteristics of dual-structures. Due to the various behaviours of the studied buildings in the considered directions, the studied buildings were divided into three groups, i.e., frame behaviour, dual behaviour and wall behaviour. The proposed vibration period formula for dual behaviour buildings is given as follows

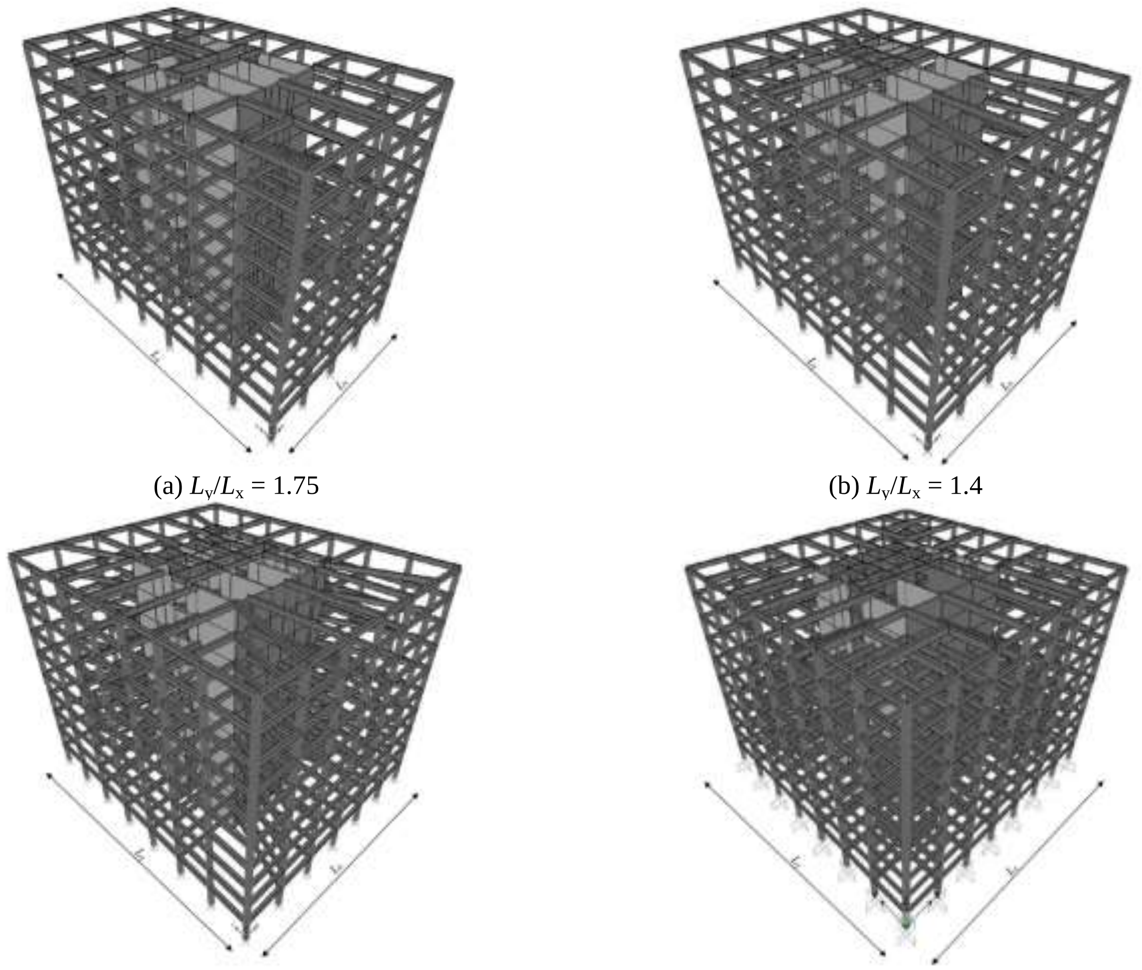
$$T = 0.075H \tag{7}$$

Hence, the above proposed formula is believed to overestimate the vibration periods which yield from the force-based analysis because Chopra and Goel (2000) suggested the use of the upper bound equation from the force-based analysis in their previous study (1998) to assess the seismic displacements. For RC frame-wall office buildings, there is lack of information on their dynamic properties in the prior experimental and numerical studies. Thus, the current study is to investigate this type of buildings through a parametric study by performing numerical analyses to obtain the fundamental vibration period and exploring the feasibility of establishing formulas for predicting this dynamic property with taking into account various influencing factors.

5. Parametric investigations

In this study, the dynamic response of RC office buildings was investigated analytically by taking into account a number of parameters including the height of buildings, the plan or side aspect ratios (AR) of buildings, the core dimensions, the space efficiency, and the leasing depth between the internal central core and outer frames. The simulated models included the frames with double central U-cores. The number of storeys ranged between 10 and 40 with the storey height as 3 m. Also, the plan aspect ratio L_y/L_x ranged between 1 and 2 depending on the bay length and the number of bays. The bay lengths were assumed to be 5 m and 6 m, while the number of bays ranged from 3 to 7 with varied building areas and bay lengths. As an example, the models were square and rectangular in plan due to the assumed number of spans as shown in Fig. 2. In addition, Fig. 3 illustrates typical configurations of the cores adopted in the models.

To design a tall building, a conceptual design should be conducted to ensure the stability and functionality of the building. The design criteria include strength, stability, serviceability and human comfort. The strength and stability states are satisfied by limited stresses and safety factors against P-Delta effects, respectively. However, the serviceability states are satisfied by limiting the drift limits to stabilise claddings and by limiting accelerations for human comfort (Jayachandran 2009). The models were designed under gravity and wind loads to relevant Eurocodes, i.e., BS EN 1990 (BSI 2005a), BS EN 1991-1-1 (BSI 2002) and BS EN 1991-1-4 (BSI 2005b). Hence, the gravity loads included the self-weight of structural elements and finishes as dead loads and the imposed loads as live loads. Together with wind loads, different combinations were taken according to the corresponding Eurocodes. For ultimate limit state design, the wind velocity was taken as 30 m/s for the 50-year return period, while for serviceability limit state design, the wind velocity was taken as 22.5 m/s for 1-year return period. Also, BS EN 1992-1-1 (BSI 2004a) and BS EN 1992-1-2 (BSI 2004b) were used to design reinforced concrete elements such as floor slabs, beams, columns and shear walls according to ultimate limit state and serviceability limit state criteria. These criteria verify the stresses in flexural bending and shear and the vertical



(a) $L_y/L_x = 1.75$ (b) $L_y/L_x = 1.4$

(c) $L_y/L_x = 1.167$ (d) $L_y/L_x = 1$

Fig. 2 3-D views of ten-storey RC frame-shear wall office building models with various plan aspect ratios

deflections to design different structural elements. For example, the floor slabs were designed mainly under flexural bending and governed by deflection. However, the design of RC beams was mainly referred to bending, shear and deflection, and the deflection gradually governed the design with increasing the span of the beam. The interactions between axial forces and bending moments were verified when designing RC columns and shear walls. Hence, the RC shear walls were designed to limit the lateral drifts of the models due to the high lateral resistance provided.

In addition to the ultimate limit state criteria, the models were also verified against serviceability limit state criteria to limit the maximum lateral drift to 1/500 of the total height of the building under wind load (PEER/ATC 2010). The compressive strength of concrete used ranged between 25 and 40 MPa. The floor slab was designed typically with a thickness of 0.2 m. The shear walls (cores) thickness was uniform through height, plan, space efficiency and loads for each building. The shear walls (cores) were checked also using the software CSiCOL V.9 (CSI 2003). The typical core areas were arranged based on the study by Sev and Özgen (2009) who largely explored the design parameters of tall RC office buildings around the world, particularly in Turkey. For the space efficiency of RC office buildings, Table 3 adopted by Sev and Ozgen (2009) is used as the primary criteria to determine the core areas. However, the simulated models had the space efficiency ratios ranging from 80% to 84% and the leasing depths between the internal central core and outer frames ranging between 5 m and 12 m based on different plan aspect ratios. Also, the areas of cores were taken as a ratio of the gross plan area. Hence, the considered parameters are listed in Table 4.

Table 3 Building efficiency (net-to-gross floor area) of multi-storey office buildings

Number of storeys	Efficiency (%)
2-4	83-86
5-9	79-83
10-19	72-80
20-29	70-78
30-39	69-75
40+	68-73

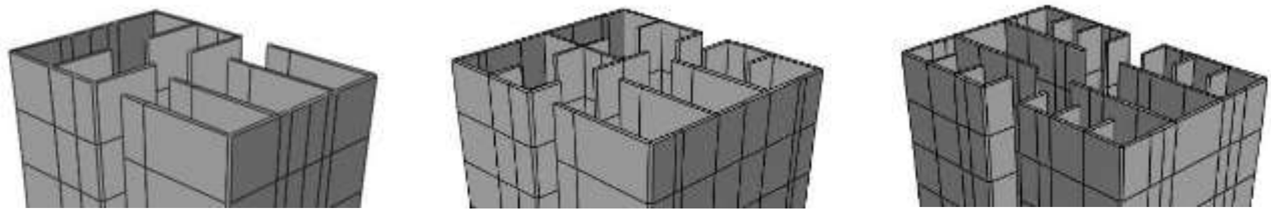


Fig. 3 Typical plan configurations for the cores of RC frame-shear wall office building models

Table 4 Investigated parameters of multi-storey office buildings

Parameters	Details
Number of storeys	10, 20, 30, 40
Height of storeys	3 m
Number of bays	3, 4, 5, 6, 7
Panel widths	5 m, 6 m
Plan of models	20 m × 15 m, 20 m × 20 m, 25 m × 15 m, 25 m × 20 m, 25 m × 25 m, 30 m × 15 m, 30 m × 20 m, 30 m × 25 m, 30 m × 30, 35 m × 20 m, 35 m × 25 m, 35 m × 30 m, 35 m × 35 m, 24 m × 18 m, 24 m × 24 m, 30 m × 18 m, 30 m × 24 m, 30 m × 30 m, 36 m × 18 m, 36 m × 24 m, 36 m × 30 m, 36 m × 36 m, 42 m × 24 m, 42 m × 30 m, 42 m × 36 m, 42 m × 42 m
Number of models	104
Plan aspect ratio (AR) (L_y/L_x)	1.0, 1.167, 1.2, 1.25, 1.33, 1.4, 1.5, 1.67, 1.75, 2.0
Space efficiency	80% - 84%
Leasing depth	5 m, 6 m, 6.5 m, 7 m, 7.5 m, 8 m, 8.5 m, 9 m, 10 m, 10.5 m, 12 m
Core areas	40 m^2, 50 m^2, 60 m^{2*}, 64 m^2, 65 m^2, 78 m^2, 91 m^2, 96 m^2, 100 m^{2*}, 104 m^2, 117 m^2, 120 m^2, 144 m^2, 162 m^2, 165 m^2, 169 m^{2*}, 195 m^2, 208 m^2, 216 m^2, 225 m^2, 256 m^2, 270 m^2, 324 m^2

*Adopted in different models

The commercial software SAP2000 (CSI 2016) was used for simulating the models where the RC beams and columns were modelled as two-node beam elements with six degrees of freedom for each node, while the slabs and shear walls were modelled as shell elements. Also, the core integrity with the outer frames was provided by the floor slabs and beams. The modelling of floor slabs was adopted as many studies (Ju and Lin 1999, Lee *et al.* 2002, Su *et al.* 2005, Kim *et al.* 2009) indicated that the flexural stiffness of floor slabs could affect the lateral stiffness of shear wall structures and then alter their dynamic properties. Hence, the majority plans of the simulated models were similar to those of the real buildings investigated in the previous studies (Kim *et al.* 2009, Sev and Özgen 2009, Zekioglu *et al.* 2007). Based on the conducted parameters in this study, a total of 104 models were simulated.

6. Analysis and discussion of the numerical results

In this section, the numerical results of the fundamental vibration periods with respect to the employed parameters in the 3D FE eigen analyses are presented and discussed. New formulas for the vibration period are then proposed based on the single or multiple unconstrained and constrained regressions performed on the obtained results. Here, the single regression analysis represents the relationship between a set of two variables in a database with one dependent variable and one independent variable only. However, the multiple regression analysis represents the relationships between more than two variables with one dependent variable and two or more independent variables. Also, the obtained constants or coefficients from the regression analyses (single or multiple) are considered as unconstrained coefficients if no restriction has been applied to any coefficient in the regression analysis, while the analysis is called a constrained regression when one of the coefficients is forced to some value to test the proposed formulas to provide the best-fit line for the tested numerical data. Moreover, there are two important statistical factors called the standard error of estimate (*RMSE*) and the coefficient of determination (R^2) that are used to adopt the best-fit line for the tested data. *RMSE* should be minimised to a small value because it represents the overall accuracy of fitting the regression line equation to the actual data. The quality of the regression best-fit line is assessed by R^2 which ranges between 0 and 1. For example, R^2 approaching 0 indicates a poor fitting for the test data, while R^2 approaching 1 indicates a best fitting for the actual data (Al-Balhawi 2018). As stated above, the contributions of the floor slabs were considered in the modelling due to the effect of flexural stiffness of floor slabs on the lateral stiffness of shear wall structures in addition to their connectivity roles to link the shear walls (cores) to the outer resisting-frames. Thus, the bare frame models were ignored and the results of full models including columns, beams, shear walls and floor slabs are discussed here. It is worthwhile to indicate that no infill walls were used in the models.

6.1 The effects of heights and plan dimensions of buildings

As suggested in the design codes and standards and the experimental studies, the height of buildings is the best predictor for evaluating the vibration periods of different structural systems of RC buildings. Thus, first the obtained numerical results of the fundamental vibration periods were explored by performing various unconstrained and constrained regression analyses in relation to the building height of the simulated models on the basis of natural log-log scales for the considered parameters to obtain the empirical coefficients for each proposed formula. Also, the regression analyses were performed and the figures obtained using Matlab software (MWI 2017). Fig. 4 illustrates the numerical results obtained from the simulated models with the fitted unconstrained and constrained regression formulas with respect to the height of buildings in the two horizontal and combined directions. The fitted regression formulas are expressed in the figure in red, green and black for the trends in x-direction, y-direction and combined directions, respectively. The used regression is given as follows

$$T = a H^{b} \tag{8}$$

where T is the fundamental vibration period, a and b are the empirical coefficients, and H is the building height. The corresponding empirical coefficients and the values of R^2 and the root mean square error (*RMSE*) or the standard error are listed in Table 5. It can be seen that the high correlation coefficients for the formula to predict the vibration period indicate the significant dependence of this dynamic property on the height of buildings and reflect the reason for including this high correlation parameter "height of building" in the recommended formulas in the design codes and standards and in the experimental studies. This evidence consistently agrees with the statement by Michel *et al.* (2010) that the height or number of storeys of buildings contributed to 85-90% of the period variance.

Other parameter added to the previous regression form Eq. (8) is the dimension of buildings (length or depth) in the evaluated direction and the corresponding expression is given as follows

$$T = a H^{b} D^{c} \tag{9}$$

where D is the dimension of the building corresponding to the evaluated direction and c is an empirical coefficient based on the regression analysis on the obtained results. The empirical coefficients and the values of R^2 and the root mean square error (*RMSE*) are listed in Table 6. Fig. 5 illustrates the obtained numerical results from the simulated models with the fitted unconstrained regression formulas in terms of the height and dimension of buildings in the considered directions. The fitted unconstrained regression formulas expressed in red, green and black are the trends in x-direction, y-direction and combined directions, respectively. Fig. 6 illustrates the obtained numerical results from the simulated models with the fitted constrained regression formula in terms of the height and dimension of buildings in the considered directions.

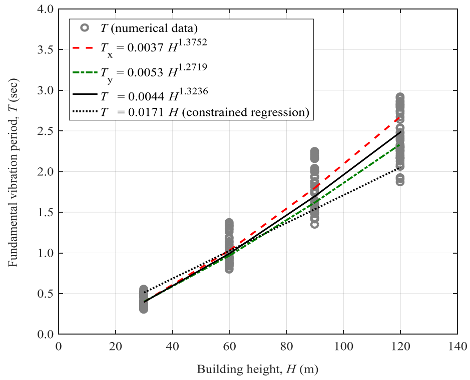


Fig. 4 Fundamental vibration periods versus building height for RC frame-shear wall buildings with AR = 1:1 to 1:2

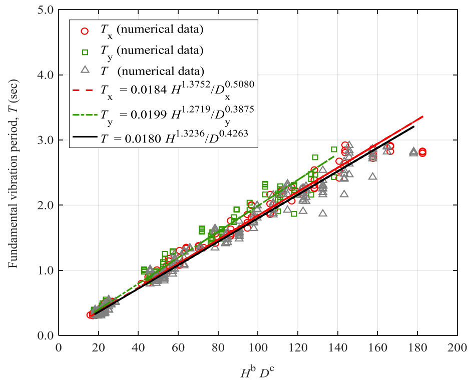


Fig. 5 Fundamental vibration period versus height and plan dimensions of RC frame-shear wall buildings with unconstrained regression formulas

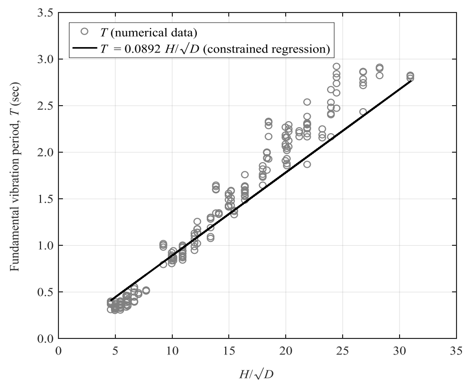


Fig. 6 Fundamental vibration period versus height and plan dimensions of RC frame-shear wall buildings with constrained regression formula

Table 5 Empirical coefficients in the proposed vibration period formulas for RC frame-shear wall buildings in terms of the building height only

Empirical coefficients		R^2	*RMSE*	Remarks
a	***b***			
0.0037	1.3752	0.9515	0.1633	Transverse period
0.0053	1.2719	0.9689	0.1198	Longitudinal period
0.0044	1.3236	0.9554	0.1496	Period for combined directions
0.0171	1.0000	0.8983	0.2253	Constrained regression for combined directions

Table 6 Empirical coefficients in the proposed vibration period formulas for RC frame-shear wall buildings in terms of the height and dimension of buildings

Empirical coefficients			R^2	*RMSE*	Remarks
a	***b***	***c***			
0.0184	1.3752	-0.5080	0.9898	0.0751	Transverse period
0.0199	1.2719	-0.3875	0.9839	0.0865	Longitudinal period
0.0180	1.3236	-0.4263	0.9844	0.0886	Period for combined directions
0.0892	1.0000	-0.5000	0.9265	0.1916	Constrained regression for combined directions

Table 7 Empirical coefficients in the proposed vibration period formulas for RC frame-shear wall buildings in terms of the height and dimension of buildings and shear walls

Empirical coefficients			R^2	*RMSE*	Remarks
a	***b***	***d***			
0.0040	1.3696	-0.3066	0.9722	0.1242	Transverse period
0.0052	1.2729	-0.0534	0.9693	0.1196	Longitudinal period
0.0045	1.3235	-0.1794	0.9617	0.1391	Period for combined directions
0.0180	1.0000	-0.5000	0.8847	0.2400	Constrained regression for combined directions

6.2 The effects of plan dimensions of buildings and cores

In the previous subsection, two parameters, the height and dimension of buildings in the evaluated directions, were investigated. The results demonstrated that the former was much more influential on the fundamental vibration period than the latter. In this subsection, the correlations of the vibration period with the height and dimensions of buildings and the sizes of shear walls in the evaluated direction are examined.

Unconstrained and constrained regression analyses were performed by utilising a similar expression to Eq. (9) except that the last term of the equation was replaced by another parameter to take into account the effect of the length of shear walls in the evaluated direction on the calculated fundamental vibration period

$$L = \frac{D_{\text{shear-wall}}}{D} \tag{10}$$

where L is the ratio of the length of shear walls to the dimension of buildings in the evaluated direction, $D_{\text{shear-wall}}$ is the length of shear walls in the evaluated direction, and D is the dimension of the buildings in the evaluated direction.

Thus, the fitted expression is given as follows

$$T = aH^{b}L^{d} \tag{11}$$

where d is an empirical coefficient. The corresponding empirical coefficients and the values of R^2 and the root mean square error (*RMSE*) are listed in Table 7. Fig. 7 illustrates the obtained numerical results from the simulated models with the fitted unconstrained regression formulas in terms of the height of buildings and the ratio of the length of shear walls to the plan dimension of buildings in the considered directions. The fitted unconstrained regression formulas expressed in red, green and black are the trends in x-direction, y-direction and combined directions, respectively. Fig. 8 illustrates the obtained numerical results from the simulated models with the fitted constrained regression formula in terms of the height of buildings and the ratio of the length of shear walls to the plan dimension of buildings in the considered directions.

Similarly, Table 7 demonstrates an enhancement in the prediction of the vibration periods when including the ratio of the length of shear walls to the dimension of buildings in the considered directions. However, the correlation between the fundamental vibration period and this ratio is very low as R = -0.0794 where the negative sign still means that the vibration period decreases when the ratio increases.

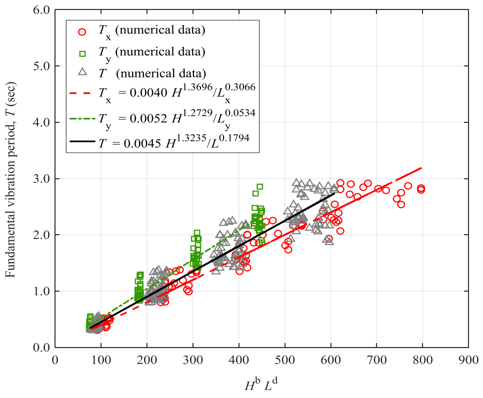


Fig. 7 Fundamental vibration period versus building height and ratio of shear wall length to building dimension for RC frame-shear wall buildings with unconstrained regression formulas

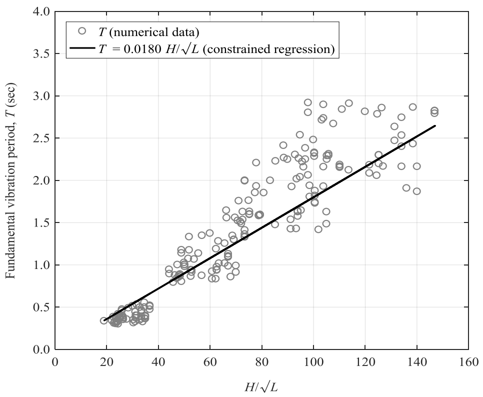


Fig. 8 Fundamental vibration period versus building height and ratio of shear wall length to building dimension for RC frame-shear wall buildings with constrained regression formula

This indicates again the dominant influence of the height of buildings on the evaluation of the vibration periods with high correlation coefficients in the regression analyses.

6.3 The effects of plan areas of buildings and cores

In this subsection, the effects of the area of buildings and the area of the shear walls (cores) of buildings are investigated. Goel and Chopra (1998) stated that the proposed formula including the height of building and the effective area of shear walls could only be applied to uncoupled shear wall (without coupling beams) buildings, while for coupled shear walls with other resisting structural systems, other formulas should be used. Even though, other regression analyses were performed by including the effective shear area ratio of the central shear walls (core) to the plan area of buildings in addition to the height of buildings and the following regression form is proposed as

$$T = a H^b A^e \tag{12}$$

where A is the effective shear area ratio of shear walls to the building plan area corresponding to the evaluated direction, and e is an empirical coefficient. Here, the effective shear area ratio A was evaluated based on Eq. (2). The corresponding empirical coefficients and the values of R^2 and the root mean square error (*RMSE*) are listed in Table 8. Also, Fig. 9 illustrates the obtained numerical results from the simulated models with the fitted unconstrained regression formulas in terms of the height of buildings (H) and the effective shear area ratio of shear walls to the building plan area (A) in the considered direction. Hence, the fitted unconstrained regression formulas expressed in red, green and black indicate the trends in x-direction, y-direction and combined directions, respectively. The diversions in the numerical results in Fig. 9 are related to the various areas of shear walls and plan areas of buildings in the two horizontal directions. Fig. 10 illustrates the obtained numerical results from the simulated models with the fitted constrained regression formulas with respect to the height of buildings and the effective shear area ratio of shear walls to the building plan area in the considered direction. It can be seen from Table 8 that the inclusion of the effective shear area ratio of shear walls to the building plan area in the considered direction enhances the correlation coefficients for the vibration period formulas more than the previous regression formulas presented in Tables 5-7.

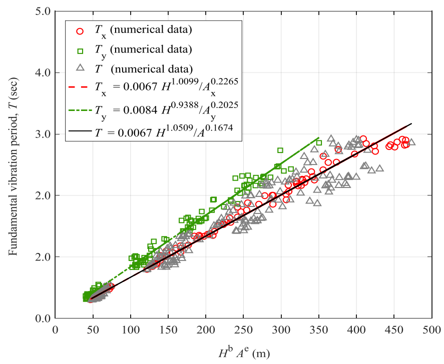


Fig. 9 Fundamental vibration period versus building height and effective shear area ratio for RC frame-shear wall buildings with unconstrained regression formulas

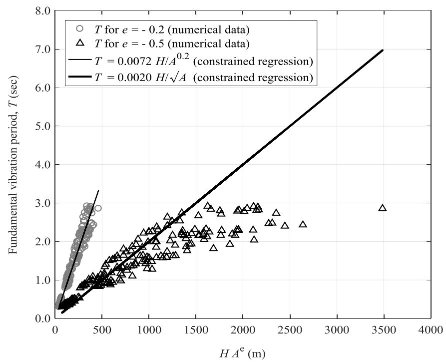
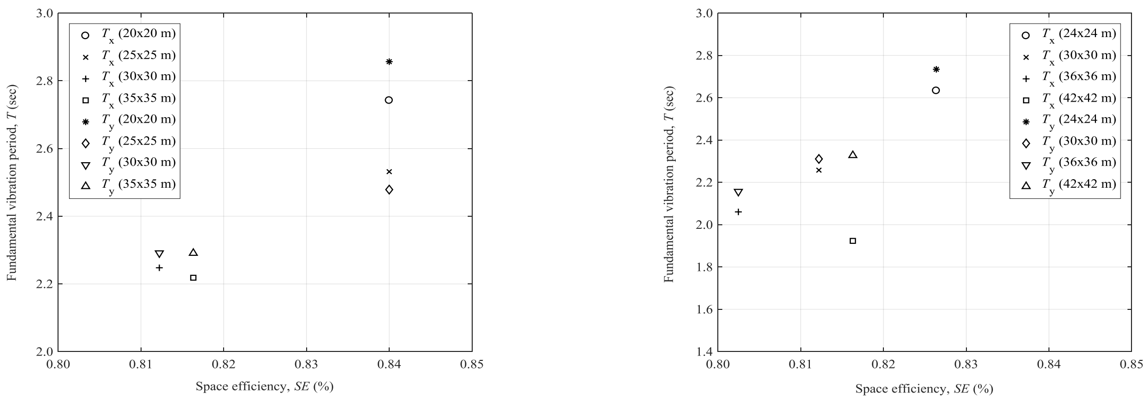


Fig. 10 Fundamental vibration period versus building height and effective shear area ratio for RC frame-shear wall buildings with constrained regression formulas

Table 8 Empirical coefficients in the proposed vibration period formulas for RC frame-shear wall buildings in terms of the height of buildings and the effective shear area ratio

Empirical coefficients			R^2	*RMSE*	Remarks
a	***b***	***e***			
0.0067	1.0099	-0.2265	0.9961	0.0464	Transverse period
0.0084	0.9388	-0.2025	0.9906	0.0662	Longitudinal period
0.0067	1.0509	-0.1674	0.9785	0.1041	Period for combined directions
0.0072	1.0000	-0.2000	0.9776	0.1057	Constrained regression for combined directions
0.0020	1.0000	-0.5000	0.7559	0.3491	Constrained regression for combined directions



(a) L_{span} = 5 m (b) L_{span} = 6 m

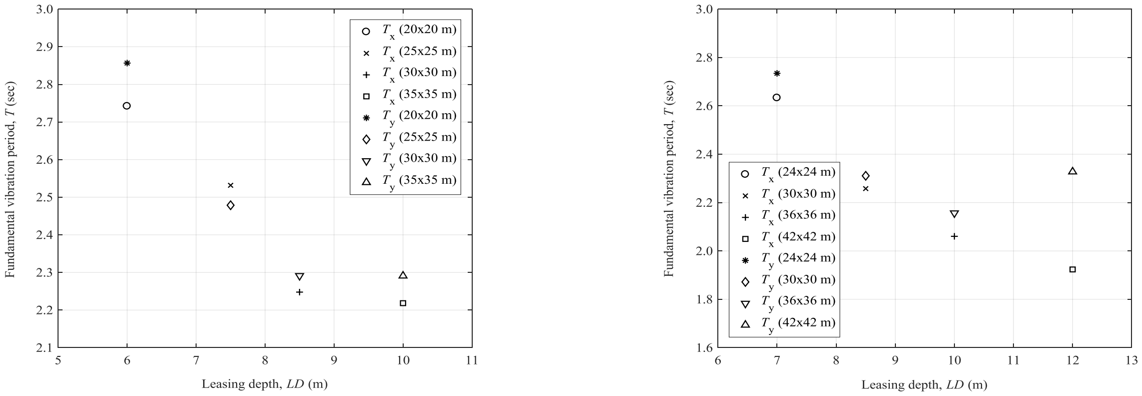
Fig. 11 Fundamental vibration period versus space efficiency for 40-storey buildings with AR = 1:1

The standard error of estimation significantly decreased as well in particular for the proposed unconstrained regression formulas. This effect is highly related to the inclusion of the effective shear area ratio in addition to the height of buildings for determining the fundamental vibration periods. Here, the correlation between the fundamental vibration period and the effective shear area ratio is R = -0.8718. This evidently indicates the significant effect of the effective shear area ratio of shear walls to the building plan area on the fundamental vibration period, which is similar to what stated by Goel and Chopra (1998).

6.4 The effects of space efficiency and leasing depth between cores and outer frames

The space efficiency (SE) and the leasing depth (LD) are crucial parameters for designing the tall office buildings because they are used to specify the available net floor area out of the gross floor area for renting and assessing the benefit of investing money. Here, these two parameters are assessed in relation to the fundamental vibration period. When the area of shear walls (core) increases the space efficiency and the leasing depth decreases if the plan area of a building is fixed. Rationally, this increase enhances the lateral stiffness of the frame-shear wall systems so as to decrease the fundamental vibration period or increase the fundamental frequency. Figs. 11 and 12 illustrate the effect of the space efficiency and the leasing depth on the fundamental vibration periods of some 40-storey buildings for floor span L = 5 m and 6 m, respectively. It can be seen from Fig. 11 that with the increase in the space efficiency of buildings the fundamental vibration period increases due to the reduced contribution of the shear wall area towards the whole lateral stiffness of the frame-shear wall buildings in the two horizontal directions, i.e., x and y directions. In fact this enhancement is not only related to the space efficiency but also to the decrease in the number of floor spans in the two directions, i.e., from 7 to 4. The difference between the fundamental vibration periods in the two horizontal directions is also related to the varied shear wall areas as stated in Fig. 3.

As indicated in Figs. 11 and 12, the values of the fundamental vibration period in x-direction are lower than those in y-direction due to the higher effective shear wall areas in x-direction. Fig. 12 illustrates the effect of the leasing depth on the fundamental vibration period. With the increase in the leasing depth, the vibration period decreases due to the increase in the number of floor spans, i.e., from 4 to 7, resulting in the increase in the lateral stiffness of frame-shear wall buildings. These trends are similar to those for other plan aspect ratios (AR), i.e. the fundamental vibration period for higher plan aspect ratios will be higher than that for lower plan aspect ratios for RC with symmetric plans due to the inequality in AR along the two horizontal directions which results in different lateral stiffnesses. In addition, the interactions between the outer frame-resisting systems for shear behaviour and the internal shear walls (core) for flexural behaviour play a significant role in



(a) L_{span} = 5 m (b) L_{span} = 6 m

Fig. 12 Fundamental vibration period versus leasing depth for 40-storey buildings with AR = 1:1

enhancing the lateral stiffness of the dual-systems and then reducing the fundamental vibration period or increasing the fundamental vibration frequency. With the increase in the number of floor spans for the same building height, the lateral stiffnesses of frames and shear walls increase due to the increase in the stiffnesses of the columns and beams of the frames and the effective shear areas from the shear walls in the evaluated directions.

7. Comparison between the proposed formulas and those in the literature

In this section, the proposed formulas for evaluating the vibration periods of tall RC frame-shear wall office buildings are compared with those cited in the prior studies. As most of the cited formulas are mainly dependent on the height of buildings, the proposed constrained formula in terms of the height of buildings is compared with those cited in the literature. Also, the proposed constrained formula for the models in combined directions (see Table 5), T = 0.0171 H, is compared with the recommended empirical equations in the design codes and standards, i.e., BS EN 1991-1-4 (BSI 2005b), ASCE 7 (2010), AS/NZS 1170.2 (2011) and AIJ (2000), and in the cited experimental studies done by Goel and Chopra (1998), Lagomarsino (1993), Su *et al.* (2003), Jalali and Milani (2005), Kwon and Kim (2010), Gilles (2011), and Velani and Kumar (2016), as illustrated in Figs. 13 and 14, respectively.

Fig. 13 illustrates the high discrepancy between the proposed constrained formula and those recommended in BS EN 1991-1-4 (BSI 2005b), ASCE 7 (2010), and AS/NZS 1170.2 (2011) (see Table 1). It is interesting to see that the proposed constrained formula agrees reasonably well with the formula recommended in AIJ (2000). The formulas in BS EN 1991-1-4 (BSI 2005b), ASCE 7 (2010) and AS/NZS 1170.2 (2011) overestimate the fundamental vibration periods of tall RC office buildings, while the formula for Japanese buildings in AIJ (2000) slightly underestimates the fundamental vibration periods of tall RC office buildings. Hence, the proposed constrained formula (T = 0.0171 H) can be used to predict the vibration periods for RC shear walls buildings subjected to earthquakes because this formula provides lower values for this dynamic property in correspondence to high base shear forces in comparison with those obtained from BS EN 1991-1-4 (BSI 2005b), ASCE 7 (2010), and AS/NZS 1170.2 (2011).

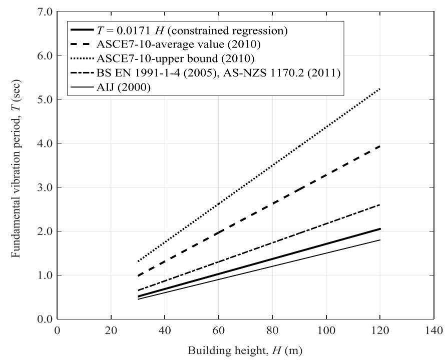


Fig. 13 Comparison of the proposed formula for fundamental vibration period versus building height for RC frame-shear wall buildings with those in the design codes and standards

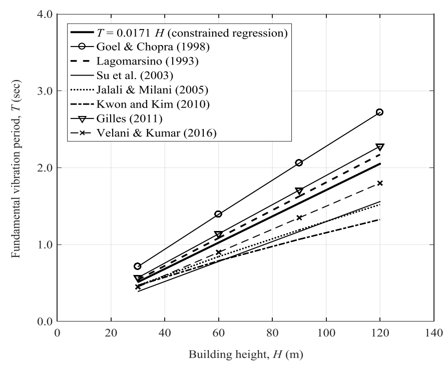


Fig. 14 Comparison of the proposed formula for fundamental vibration period versus building height for RC frame-shear wall buildings with those from the prior experimental studies

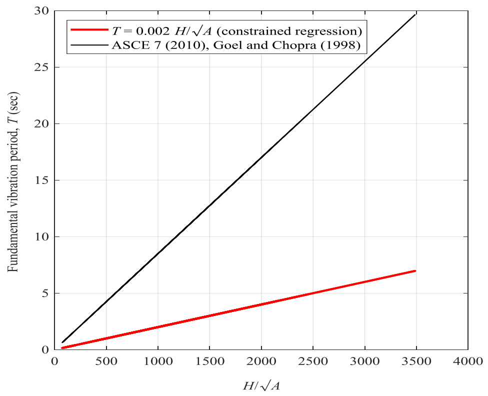


Fig. 15 Comparison of the proposed formula for fundamental vibration period versus building height and effective shear area ratio for RC frame-shear wall buildings with those in the prior studies

Fig. 14 illustrates the comparisons between the same constrained formula and those cited from the recent experimental studies done by Goel and Chopra (1998), Lagomarsino (1993), Su *et al.* (2003), Jalali and Milani (2005), Kwon and Kim (2010), Gilles (2011) and Velani and Kumar (2016). Some studies are not included in the previous figure as their formulas coincide with each other. Thus, only one study is included to represent the others. It can be seen that the proposed constrained formula underestimates the vibration period based on the formula derived by Goel and Chopra (1998) which has a low correlation coefficient R^2 = 0.5560 as stated in Section 3. On one hand, the proposed constrained formula for the vibration period has a good agreement with the formulas cited by Lagomarsino (1993) and Gilles (2011). Also, the constrained formula lies between the lower and upper bounds proposed by Gilles (2011). On other hand, the constrained formula for the vibration period overestimates the values obtained based on the formulas by Su *et al.* (2003), Jalali and Milani (2005), Kwon and Kim (2010), and Velani and Kumar (2016). These differences are due to the fact that those studies investigated RC residential buildings usually having more infills and shear walls than RC office buildings. Thus, the lateral stiffness of RC residential buildings will be higher than that of RC office buildings, leading to shorter fundamental vibration periods or higher fundamental frequencies. Hence, the study done by Kwon and Kim (2010) showed the shorter estimated vibration period values as they investigated different types of shear wall buildings.

The cited analytically based formulas by Balkaya and Kalkan (2003) and Vuran *et al.* (2008) are not used for comparison as the former researchers studied the shear walls with floor slabs only without columns and beams (tunnel form), and the latter researchers suggested a lower vibration period formula based on the displacement-based analysis instead of the force-based analysis which normally results in longer vibration period values. The lack of information on the dynamic properties of RC office buildings in literature by either experimental or numerical studies should be further explored to search for establishing such formulas to evaluate the fundamental vibration periods of these buildings in different regions and also to justify the proposed vibration periods obtained from the numerical analyses. Fig. 15 illustrates the comparison of the proposed constrained formula (red trend) indicated in Table 8 for the fundamental vibration period in terms of the height of buildings and the effective shear area ratio with the formula in ASCE7 (2010) as Eq. (1). High discrepancy between the proposed constrained formula and the formula given in the standard can be seen.

Thus, the formula in the design standard ASCE7 (2010) largely overestimates the fundamental vibration periods of tall RC frame-shear wall office buildings. Even though, Eq. (1) is an upper-bound value formula but it was derived from buildings in California under earthquake motion records where these buildings could have invisible cracking resulting in longer vibration periods. However, the simulated models were designed under wind loading and the corresponding lateral stiffness will be higher. This discrepancy between the upper-bound Eq. (1) and other formulas in literature was also reported by Gilles (2011) who specified that this equation should be used for experimental data of shear wall buildings with the ratio of the building height to the effective area, ($H/\sqrt{A}$), less than 300, otherwise a high discrepancy will be expected for higher ratios as illustrated in Fig. 15. In addition, the proposed constrained formula in Fig. 15 (mean value) can be used for primarily predicting the vibration periods of RC shear wall buildings subjected to earthquakes because these formulas provide lower values for the vibration periods in comparison with those obtained from ASCE7 (2010) and then conservative base shear forces.

8. Conclusions

In this study, the elastic vibration periods of tall RC frame-shear wall office buildings designed under gravity and wind loads were numerically evaluated by utilising FE modelling. A number of influencing parameters were investigated, including the height of the buildings, the plan aspect ratio (AR) of buildings, the core dimensions, the space efficiency, and the leasing depth between the internal central core and outer frames. Based on these numerical analyses and comparisons with the experimentally obtained formulas cited in the literature, the following conclusions can be drawn accordingly.

- The regression analyses indicated that the height of buildings is a significant parameter for evaluating the fundamental vibration periods of tall RC frame-shear wall office buildings. The proposed constrained formula in terms of the building height fairly well agrees with some cited formulas from the literature.
- In the regression analyses, the dimensions of buildings in the considered directions are less influential on the fundamental vibration period.
- The plan aspect ratio affects the fundamental

vibration period. With the increase in the plan aspect ratio the vibration period will be different in the two horizontal directions due to the variations in the lateral stiffness of the frames and shear walls in those directions.

- The ratio of the shear wall length to the plan dimension of the building in the considered direction is less influential on the fundamental vibration period.
- The effective shear area ratio of shear walls to the building plan area shows a clear impact on the fundamental vibration period due to the inclusion of the area of shear walls.
- The space efficiency and the leasing depth of buildings are crucial parameters affecting the fundamental vibration period and should be taken into account when assessing the lateral stiffness of tall RC office buildings.
- The proposed formulas can be used to assess the fundamental vibration period of tall RC office buildings designed under gravity and wind loads with the studied parameters.
- The constrained proposed formulas in terms of the building height only or combined with the effective shear area ratio can be used to predict the vibration periods of RC shear walls buildings subjected to earthquakes with lower values of vibration periods and conservative base shear forces.
- The lack of information on the evaluation of the fundamental vibration period of tall RC office buildings should be addressed and more experimental work should be conducted to establish such formula and verify the formulas proposed from the numerical analyses.

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