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On the fundamental period of infilled RC frame buildings

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Abstract. This paper investigates the fundamental period of vibration of RC buildings by means of finite element macro-modelling and modal eigenvalue analysis. As a base study, a number of 14-storey RC buildings have been considered "according to code designed" and "according to code non-designed". Several parameters have been studied including the number of spans; the span length in the direction of motion; the stiffness of the infills; the percentage openings of the infills and; the location of the soft storeys. The computed values of the fundamental period are compared against those obtained from seismic code and equations proposed by various researchers in the literature. From the analysis of the results it has been found that the span length, the stiffness of the infill wall panels and the location of the soft storeys are crucial parameters influencing the fundamental period of RC buildings.

Keywords: fundamental period; infilled frames; masonry; modal analysis; reinforced concrete buildings

1. Introduction

In the context of seismic risk assessment and mitigation, a robust method for the estimation of the fundamental period of vibration is essential both for the design of new buildings and the performance assessment of existing ones. The distribution of stiffness and mass along the height of a building impacts its fundamental period. Consequently, any element (structural or non-structural) with rigidity, mass or both has an effect on the fundamental period of a building. Some of the parameters influencing the vibration period of buildings are: the structural regularity, the height of the building, the provision of shear walls, the number of storeys, the number of spans, the dimensions of the member sections, the presence of openings in the infill panels, the position of load, the soil flexibility etc. The influence of the above parameters and their interactions for the estimation of the fundamental period of a building is a non-trivial task.

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Worldwide, several earthquake design codes provide formulas for estimating the fundamental period of buildings. Typically, such formulas are derived from regression analysis of numerical values, which have been obtained from measured periods of vibrations of real buildings during past earthquakes. Despite the fact that several other parameters affect the period of vibration, the formulas given by the design codes are, typically, a function of the building's height or the number of storeys. However, the fundamental periods calculated by these expressions (design codes' formulas) reveal large discrepancies. Today, with the use of sophisticated computational methods of analysis, it is possible to determine the fundamental period of buildings by means of exact eigenvalue analysis or by other robust quantitative methods (i.e., Rayleigh's method). Computational values of fundamental period have been found to be significantly higher than those measured from the motion of buildings during earthquakes. This is attributed to the fact that the effects of secondary components/non-structural components, like the infills, are not considered in the computational analysis. In fact, masonry infill walls affect the strength and stiffness of infilled frame structures and thus have a significant impact on the building's performance (El-Dakhakhni *et al.* 2003, Dolsek and Fajfar 2005, Kose 2009).

Despite the extensive experimental efforts (Smith 1966, Smith and Carter 1969, Page *et al.* 1985, Mehrabi *et al.* 1966, Buonopane and White 1999, Santhi *et al.* 2005a, b, Cavaleri *et al.* 2005) and analytical investigations (Liauw and Kwan 1984, Dhanasekar and Page 1986, Chrysostomou 1991, Saneinejad and Hobbs 1995, Chrysostomou *et al.* 2002, Asteris 2003, 2005, 2008, Moghaddam 2004, Kakaletsis and Karayannis 2009, Cavaleri and Papia 2003, 2014), the rationale behind neglecting infill walls is partly attributed to: a) incomplete knowledge of the behaviour of quasi-brittle materials, such as unreinforced masonry (URM); b) the composite behaviour of the frame and the infill; and c) the lack of conclusive experimental and analytical results to substantiate a reliable design procedure for this type of structures. Moreover, due to the large number of interacting parameters, if infills are to be considered in the analysis and design stages, a modelling problem arises because of the many possible failure modes that need to be evaluated with a high degree of uncertainty. This is compounded by the presence of openings in the infills, which change completely their behaviour and the large variety of infill walls and their dependence on local construction practices. In addition, the non-structural nature of infills, may result in their removal in the case of building renovations, during which heavy masonry infills may be replaced by light partitions and hence change the overall behaviour of the structural system with possible detrimental effects. Therefore, it is not surprising that no consensus has emerged leading to a unified approach for the design of infilled frame systems in spite of more than six decades of research. However, it is generally accepted that under lateral loads an infill wall acts as a diagonal strut connecting the two loaded corners, an approach that is preferred in the case of infill walls without openings on the diagonal of the infill panel (Moghaddam and Dowling 1987, Asteris *et al.* 2011, 2013, Chrysostomou and Asteris 2012).

The aim of the present work is to investigate the fundamental period of vibration of a 14-storey bare and infilled RC frame building by means of finite-element modelling under various geometric and other parameters, including the influence of the number of spans, span length, infill stiffness, infill panel opening percentage as well as the soft storey position.

2. Estimation of fundamental period for RC buildings

2.1 Building design codes

The most common expression for the calculation of the fundamental period of vibration (T) is given by Eq. (1)

$$T = C_t \cdot H^{3/4} \tag{1}$$

where H is the total height of the building (in meters) and C_t is a numerical coefficient. Such expression derived using Rayleigh's method by assuming that the horizontal forces are linearly distributed over the height of the building; the mass distribution is constant; the mode shape is linear; and the base shear is inversely proportional to $T^{2/3}$. The above expression was first adopted by ATC3-06 (1978) for reinforced concrete moment-resisting frames. The coefficient C_t was obtained through regression analysis based on the period of buildings measured during the San Fernardo (1971) earthquake and determined as 0.075. The European seismic design regulations (Eurocode 8 2004) and the Uniform Building Code (UBC 1997), among others, adopt the same expression as ATC3-06 for the evaluation of fundamental period of vibration. The Jordanian National Building Code (2005) also uses Eq. (1) for the evaluation of the fundamental period of vibration and suggests a value for C_t equal to 0.04 for infilled RC frame buildings, merely based on expert judgment, as noted by Al-Nimry *et al.* (2014). Also, the New Zealand Seismic Code (NZSEE 2006) adopts the period-height relation for the fundamental period of vibration where the coefficient C_t is given as 0.09 for reinforced concrete frames, 0.14 for structural steel and 0.06 for other types of structures. Furthermore, the Israeli Seismic Code (SI-413 1994) provides a value of 0.073, 0.085 and 0.049 for the coefficient C_t for concrete, steel and other buildings, respectively.

The UBC proposed formula has been updated in FEMA 450 (Federal Emergency Management Agency 2003) based on the study by Goel and Chopra (1997) and the measured period of concrete moment-resisting frame buildings, monitored during California earthquakes (including the 1994 Northridge earthquake). Based on the lower bound of the data presented by Goel and Chopra (1997), FEMA proposed the expression of Eq. (2) for RC frames that provides a conservative estimate of the base shear

$$T = C_r H_n^x \tag{2}$$

where H_n the height of the structure (in meters), C_r is given as 0.0466 and x as 0.9.

The National Building Code (NBC 1995) of Canada adopts the expression of Eq. (3) that relates the fundamental period of buildings with the number of stories, N, above the ground, as follows

$$T = 0.1\mathrm{N} \tag{3}$$

Similarly, the Costa Rican Code (1986) gives the expression

$$T = 0.08\mathrm{N} \tag{4}$$

The aforementioned empirical expressions are simple as the only parameter considered is the total height of the building or the number of stories. However, other parameters such as the presence of shear walls are also influencing the fundamental period of vibration of buildings. The Greek Seismic Code (EAK 2003) takes into account the influence of shear walls on the fundamental period of the RC buildings as shown in Eq. (5)

$$T = 0.09\,\frac{H}{\sqrt{L}}\sqrt{\frac{H}{H+\rho L}} \tag{5}$$

where H the height of the structure (in meters), L is the width of structure in the direction of analysis (in meters) and ρ is the ratio of the areas of shear wall sections along the direction of analysis to the total area of walls and columns.

Other seismic building codes including the Indian Seismic Code (IS-1893 2002), the Egyptian Code (1988) and the Venezuelan Code (1988), in addition to the building's height h (in meters), take into consideration the total base dimension, d (in meters), of the masonry infilled RC frame. The expression for the estimation of the fundamental period of vibration from the aforementioned seismic codes is given by Eq. (6)

$$T = \frac{0.09\mathrm{h}}{\sqrt{\mathrm{d}}} \tag{6}$$

The French Seismic Code (AFPS-90 1990) recommends that for the estimation of the fundamental period, the lower value between Eq. (6) and Eq. (7) should be used

$$T = 0.06\frac{\mathrm{h}}{\sqrt{\mathrm{d}}}\sqrt{\frac{\mathrm{h}}{2\mathrm{d}+\mathrm{h}}} \tag{7}$$

The Algerian Seismic Code (1988) adopts two expressions for the fundamental period, the simple height-related Eq. (1) and Eq. (7) and prescribes that the smallest value should be used.

Also, the Eurocode 8 (2004), besides the general height-related expression (Eq. (1)), provides a more exact expression for the calculation of the coefficient C_t, for masonry infilled reinforced concrete frames (Eq. (8))

$$\mathrm{C_t} = \frac{0.075}{\sqrt{\mathrm{A_c}}} \text{ and } \mathrm{A_C} = \sum \mathrm{A_i}\left(0.2 + \frac{\mathrm{l_{wi}}}{\mathrm{H}}\right)^2 \tag{8}$$

where, C_t is the correction factor for masonry infilled reinforced concrete frames, A_C is the combined effective area of the masonry infill in the first storey, A_i is the effective cross-sectional area of the wall *i* in the direction considered in the first storey and l_{wi} is the length of the walls in the first storey in the direction under consideration. An extensive and in-depth state-of-the-art report on the Code approaches about the fundamental period of masonry-infilled RC frames can be found in Morales (2000), Kaushik *et al.* (2006), Dorji (2009).

Empirical and semi-empirical expressions derived from FE modelling

Several researchers have proposed refined semi-empirical expressions for the fundamental period of RC frame structures based on the height related formula, as given in Table 1. In 2004, Crowley and Pinho (2004) highlighted that it is important to develop region-specific simplified period-height formulae. They focused on period-height relationships for the seismic assessment of existing, rather than the design of new, RC buildings and proposed a period-height formula for displacement-based design drawn from the results of eigenvalue, push-over and non-linear dynamic analyses carried out on 17 RC frames representative of the European building stock. The simple relationship presented in Table 1 is valid for RC buildings without masonry infills. The RC frames considered corresponded to actual buildings from five different south European countries designed and built between 1930 and 1980 according to older design codes. Later in 2006, Crowley and Pinho (2006) studied the elastic and yield period values of existing European RC buildings of varying height using eigenvalue analysis. Such studies led to a simplified period-height expression for the assessment of existing RC buildings, where the presence of masonry infills was also taken into account.

Guler *et al.* (2008) computed the fundamental periods of RC buildings, considering the effects of infill walls, using ambient vibration tests and elastic numerical analyses. A period-height relationship was derived for a fully elastic condition.

Table 1 Expressions for the evaluation of fundamental period of vibration

Expression	Reference
$T=0.053H^{0.9}$	Goel and Chopra (1997)
$T=0.029H^{0.804}$	Hong and Hwang (2000)
$T=0.067H^{0.9}$	Chopra and Goel (2000)
$T=0.1H$	Crowley and Pinho (2004)
$T=0.055H$	Crowley and Pinho (2006)
$T=0.026H^{0.9}$	Guler *et al.* (2008)

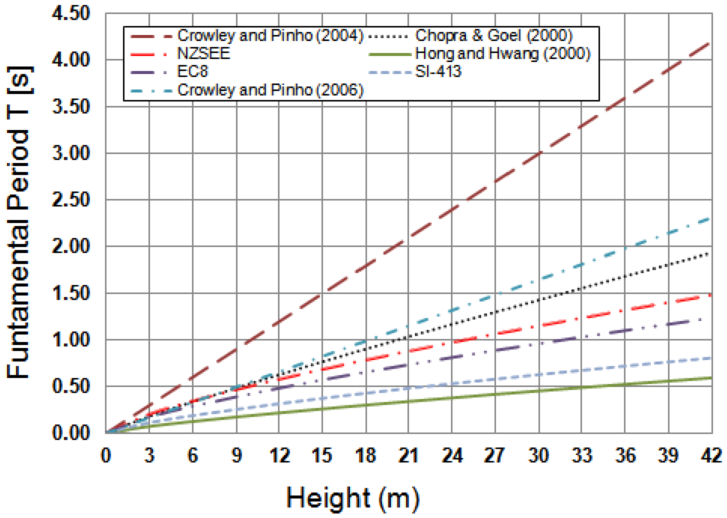


Fig. 1 Comparison of equations for the evaluation of the fundamental period

Fig. 1 presents a comparison between some of the aforementioned height-related expressions for the evaluation of fundamental period of vibration. From Fig. 1, it is clear that the value of the fundamental period calculated based on these expressions show a significant spread, revealing the need for further investigation and refinement of the proposals. In particular, the expression by Hong and Hwang (2000) underestimates the value of fundamental period (the period value is below 0.5 s even for a total building height of 30 m). On the other hand, the equation by Crowley and Pinho (2004) which has been proposed for existing buildings, results in larger values for the fundamental period and is the upper bound of all codes and research proposals. A reviewed equation by Crowley and Pinho (2006), taking into account the masonry infill contribution, has lowered the values to almost half compared to their previous proposal. However, Crowley and Pinho (2004, 2006) calculate the yield period of vibration of existing buildings, for use in vulnerability assessment applications, using yield/cracked stiffness of the members. Besides the extreme boundaries, other proposals show similarities in calculating the fundamental period when considering only the building total height.

Studies have shown that numerical analyses usually return values for the fundamental period that are significantly different than those evaluated using the code period-height expressions, for example Masi and Vona (2010); Amanat and Hoque (2006). Usually, the fundamental period determined by the computational methods is longer than the period obtained by the code equations due to the elimination of the effects of non-structural members in the computational methods. The presence of infill walls and their connectivity to the frame has been identified as the main reason for this discrepancy. Of course, there are some proposed equations for the prediction of the fundamental period of frames (Amanat and Hoque 2006, Crowley and Pinho 2004, Goel and Chopra 1997, Hong and Hwang 2000) that take into consideration more than the type and height of the structure and that will be discussed in the following sections.

Crowley and Pinho (2010), by taking into account the presence of infills, have proposed an expression for RC moment resisting frames, designed according to recent seismic codes, with rigid infills based on the simple period-height formula

$$T = \frac{0.09\mathrm{H}}{\sqrt{\mathrm{D}}} \tag{9}$$

where D is the dimension of the building at its base in the direction under consideration (in meters).

Amanat and Hoque (2006) have studied the fundamental periods of vibration of a series of regular RC framed buildings using a 3D finite-element modelling and modal eigenvalue analysis. They have identified that the span length, the number of spans and the amount of infills significantly influence the fundamental period. The proposed equation based on this study is given by Eq. (10)

$$T = \alpha_1 \alpha_2 \alpha_3 C_t h^{3/4} \tag{10}$$

where C_t=0.073 for RC buildings, the factor α_1 is the modification factor for span length of infill panels, α_2 is the modification factor for the number of spans and α_3 is the modification factor for the amount of infills.

Hatzigeorgiou and Kanapitsas (2013) have proposed an empirical expression for the fundamental period of frame buildings which takes into account simultaneously the soil flexibility, the effect of shear walls, and the influence of external and internal infill walls. The proposed expression was estimated using a database of 20 full scale RC buildings which have been constructed in Greece. The proposed expression based on this study is given by Eq. (11)

$$T = \frac{\mathrm{H}^{\mathrm{C}_1}\mathrm{L}^{\mathrm{C}_2}(\mathrm{c}_3+\mathrm{c}_4\mathrm{W})}{\left[1-\exp\left(\mathrm{c}_5\mathrm{k}_\mathrm{S}^{\mathrm{c}_6}\right)\right]\sqrt{(1+\mathrm{c}_7\rho)}} \tag{11}$$

where height H and length L in meters, ρ the ratio of the areas of shear wall sections along a seismic action direction to the total area of walls and columns, k_s is the subgrade modulus of soil (in $\mathrm{MN/m^3}$), W a parameter related to the influence of infill walls on the fundamental period. The coefficients c_1–c_7 were determined by nonlinear regression analysis using analytical values of the fundamental period. Ignoring the influence of infill and concrete shear walls, soil flexibility and length of buildings, Hatzigeorgiou and Kanapitsas (2013) derived a simpler expression for the estimation of the fundamental period of buildings which is very similar to the formula provided by the Eurocode 8

$$T = 0.073\mathrm{H}^{0.745} \tag{12}$$

Kose (2009) investigated the fundamental period of vibration of RC frame buildings using a computational iterative modal analysis in 3D. Kose (2009) evaluated the effect of building height, frame type and the presence of infill walls, among other parameters and he proposed the expression of Eq. (13) for the prediction of the fundamental period of reinforced concrete moment resisting frames

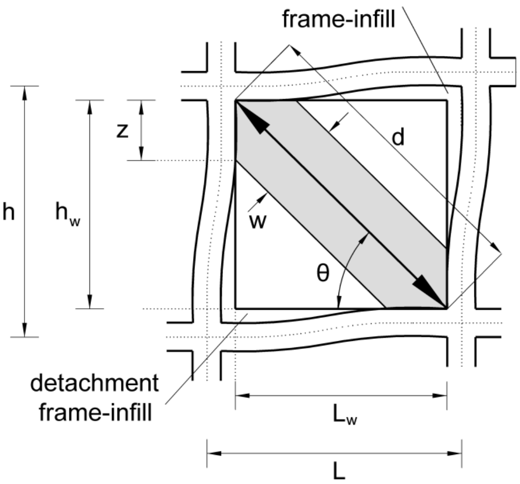


Fig. 2 Masonry infill frame sub-assemblage

$$T = 0.0935 + 0.0301\mathrm{H} + 0.0156\mathrm{B} + 0.039\mathrm{F} - 0.1656S - 0.0232\mathrm{I} \tag{13}$$

where *H* is the height of building in meters, *B* is the number of bays, *F* is a coefficient for the frame type and it is equal to 1 for frames with infills, 2 for frames with open first floor and 3 for bare frames, *S* is the percentage ratio of shear walls to total floor area, *I* is the ratio of the area of infill walls to total area of the panels. For the infilled frames, the fundamental period of vibration was found to be 5 to 10% lower than that of RC frames without infill walls, regardless of the presence of shear walls. Based on a sensitivity analysis undertaken by Kose (2009) and since the fundamental period is not that sensitive to the number of bays and frame type, a more convenient formula was derived taking into account only the building height and the ratio of area of the infill walls to total area of the panels

$$T = 0.1367 + 0.0301\mathrm{H} - 0.1663\mathrm{S} - 0.0305\mathrm{I} \tag{14}$$

It is evident from the aforementioned review that the proposed empirical expressions for the evaluation of fundamental period of vibration present similarities, in terms of the parameters used to express the period value, e.g., building total height and length, percentage of infill and span length of infill but at the same time provide values for the fundamental period with significant spread. Crowley and Pinho (2010) noted that period-height relationships proposed by several researchers have been obtained from buildings constructed over a long period of time. They believe that the large scatter in the results of different relationships may be due to the variation in the age of the buildings and thus design regulations. They also highlighted that buildings constructed before 1980 according to older design codes have usually larger period and the equations should be calibrated. Furthermore, a trustworthy expression for the evaluation of fundamental period must simultaneously consider, besides the total height and length of the RC frame, other parameters such as the frame height-to-length aspect ratio, the infill height-to-length aspect ratio, the percentage opening of the infills, the relative panel-to-frame-stiffness and the presence of soft storey. Such parameters need further investigation, in order to assess their importance and impact on the fundamental period and propose refined expressions for a more accurate period evaluation.

3. Overview of computational strategies for modelling RC infilled frame structures

Since the first attempts to model the response of the composite infilled-frame structures, experimental and conceptual observations have indicated that a diagonal strut with appropriate geometrical and mechanical characteristics could possibly provide a solution to the problem (Fig. 2). In Fig. 2 w is the width of the diagonal strut, d is the diagonal length of the masonry panel, L is the distance between the centres of two columns and z is the contact length of the diagonal strut to the column.

Early research on the in-plane behaviour of infilled frame structures undertaken at the Building Research Station, Watford (later renamed Building Research Establishment, and now simply BRE) in the 1950s served as an early insight into this behaviour and confirmed its highly indeterminate nature in terms solely of the normal parameters of design (Thomas 1953, Wood 1958, Mainstone 1962). On the basis of these few tests a purely empirical interaction formula was later tentatively suggested by Wood (1958) for the design of tall framed buildings. By expressing the composite strength of an infilled frame directly in terms of the separate strengths of the frame and infill, Wood (1958) short-circuited a mass of confusing detail and recognized the desirability of a higher load factor where strengths were most dependent on the infills.

3.1 Infill walls modelling

In the early sixties, Polyakov (1960) suggested the possibility of considering the effect of the infill in each panel as equivalent to diagonal bracing. This suggestion was later adopted by Holmes (1961) who replaced the infill by an equivalent pin-jointed diagonal strut made of the same material and having the same thickness as the infill panel and a width, w, defined by

$$\frac{w}{d} = \frac{1}{3} \tag{15}$$

where, d is the diagonal length of the masonry panel (Fig. 2). The “one-third” rule was suggested as being applicable irrespective of the relative stiffness of the frame and the infill. One year later, Smith (1962), based on experimental data from a large series of tests using masonry infilled steel frames, found that the ratio w/d varied from 0.10 to 0.25. In the second half of the sixties Smith and his fellow researchers using additional experimental data (Smith 1966, 1967 Smith and Carter 1969) related the width of the equivalent diagonal strut to the infill/frame contact lengths using an analytical equation, which has been adapted from the equation of the length of contact of a free beam on an elastic foundation subjected to a concentrated load (Hetenyi 1946). They proposed the evaluation of the equivalent width as a function of the relative panel-to-frame-stiffness parameter, in terms of

$$\lambda_h = h \sqrt[4]{\frac{E_w t_w \sin 2\theta}{4EIh_w}} \tag{16}$$

where E_w is the modulus of elasticity of the masonry panel, EI is the flexural rigidity of the columns, t_w the thickness of the infill panel and equivalent strut, h the column height between centerlines of beams, h_w the height of infill panel, and θ the angle, whose tangent is the infill height-to-length aspect ratio, being equal to

$$\theta = \tan^{-1}\left(\frac{h_w}{L_w}\right) \tag{17}$$

where L_w is the length of infill panel. All the above parameters are explained in Fig. 2. Based on experimental and analytical data, Mainstone (1971) proposed an empirical equation for the calculation of the equivalent strut width, given by

$$\frac{w}{d} = 0.16\lambda_h^{-0.3} \tag{18}$$

Later, Mainstone and Weeks (1970), Mainstone (1974) based on experimental and analytical data, proposed an empirical equation for the calculation of the equivalent strut width as follows

$$\frac{w}{d} = 0.175\lambda_h^{-0.4} \tag{19}$$

Eq. (19) was included in FEMA-274 (Federal Emergency Management Agency 1997) for the analysis and rehabilitation of buildings as well as in FEMA-306 (Federal Emergency Management Agency 1998), as it has been proven to be the most popular over the years. This equation was accepted by the majority of researchers dealing with the analysis of infilled frames.

3.2 Effect of openings on the lateral stiffness of infill walls

Although infill walls have often oversized openings recently research has mainly focused on the simple case of infill walls without openings. However, research on infill walls with openings is mostly analytical, restricted to special cases, and as such cannot provide rigorous comparison to actual cases because of its focus on specific materials used and specific types of openings. It is worth noting that the contribution of the infill wall to the frame lateral stiffness is reduced when the structure is subjected to reversed cyclic loading, as in real structures under earthquake conditions.

In order to investigate the effect of openings on the lateral stiffness of masonry infill walls a finite element technique proposed by Asteris (2003, 2008) has been used herein. The basic characteristic of this technique is that the infill/frame contact lengths and the contact stresses are estimated as an integral part of the solution, and are not assumed in an ad-hoc way. In brief, according to this technique, the infill finite element models are considered to be linked to the surrounding frame finite element models at two corner points (only), at the ends of the compressed diagonal of the infill (points A and B in Fig. 3(a)). Then, the nodal displacements are computed and checked whether the infill model points overlap the surrounding frame finite elements. If the answer is positive, the neighbouring points (to the previously linked) are also linked and the procedure is repeated. If the answer is negative, the procedure is stopped and the derived deformed mesh is the determined one (Fig. 3(h)).

Using this technique, analytical results are presented on the influence of the opening size on the seismic response of masonry infilled frames. Fig. 4 shows the variation of the λ factor as a function of the opening percentage (opening area/infill wall area), for the case of an opening on the compressed diagonal of the infill wall (with aspect ratio of the opening the same as that of the infill). As expected, the increase in the opening percentage leads to a decrease in the frame's stiffness. Specifically, for an opening percentage greater than 50% the stiffness reduction factor tends to zero. The findings of the parametric study by Asteris (2003) using the finite-element method, lead to the following relationship for the infill wall stiffness reduction factor λ

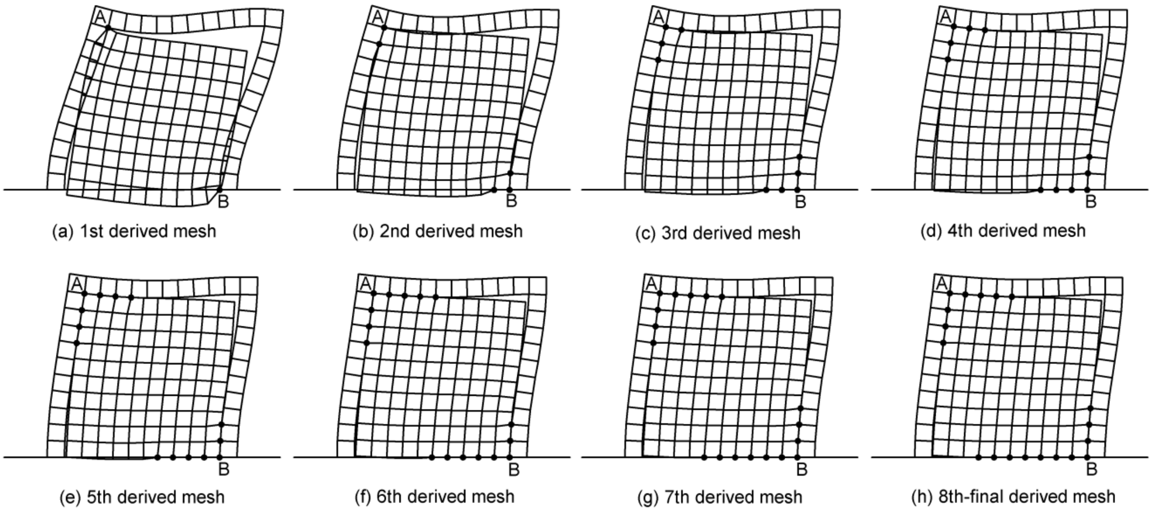


Fig. 3 Deformed meshes of an one-storey one-bay infilled frame using the finite element technique proposed by Asteris (2003, 2008)

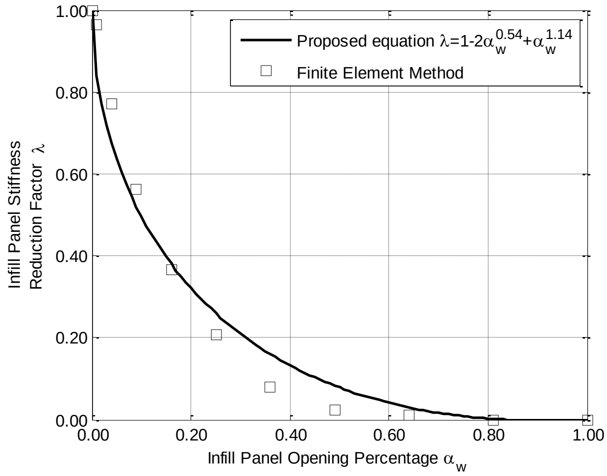


Fig. 4 Infill panel stiffness reduction factor in relation to the opening percentage

$$\lambda = 1 - 2\alpha_w^{0.54} + \alpha_w^{1.14} \tag{20}$$

where α_w is the ratio of the area of opening to the area of infill wall. The above coefficient could be used to find the equivalent width of a strut for the case of an infill with opening by multiplying the width obtained using Eqs. (15), (18) and (19) by the relevant reduction factor. It can also be used to modify the equation of the Crisafulli model, which is described in section 4.4.

4. Description of the investigated structures

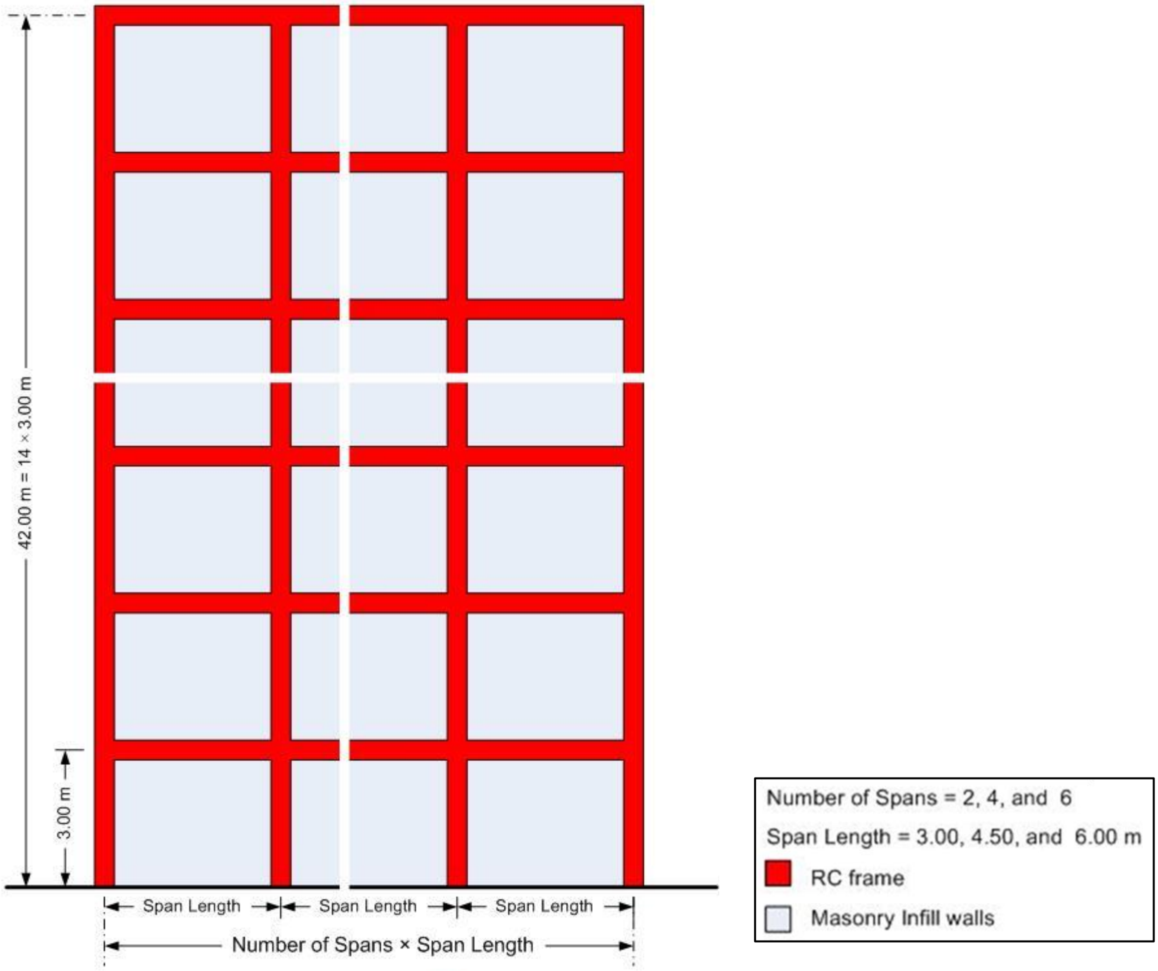


Fig. 5 Cross section details of 14-storey RC infilled frame

4.1 Building forms

The fundamental periods of RC structures are investigated in this study. Buildings considered are frame systems, regular in plan, comprised of beams and columns and only one frame is finally considered. The buildings are cast-in-place reinforced concrete structures with beams cast monolithically with slabs and supported by columns.

Buildings investigated have 14-storeys (Fig. 5). The storey height for all buildings is kept constant and equal to 3.0 m. The number of spans varied between 2, 4 and 6. For each case, three different span lengths were considered, namely 3.0 m, 4.5 m and 6.0 m. In the perpendicular direction the bay size has been kept constant and equal to 5 m for all cases.

4.2 Influence of infill panels

The influence of infill walls is examined analysing both bare frame structures as well as

Table 3 Side dimension (mm) of square columns

Storey	Column's Dimensions (mm)		
	Bay size 3.0 m	Bay size 4.5 m	Bay size 6.0 m
14	400	400	500
13	450	450	550
12	450	450	550
11	450	500	550
10	500	500	550
9	500	500	550
8	500	500	600
7	500	550	600
6	500	550	600
5	500	600	650
4	550	600	650
3	550	600	650
2	550	600	650
1	550	600	650

structures with fully or partially unreinforced masonry infilled frames with or without openings. Various parameters are considered for each case. Infill panels are either 0.15 or 0.25 m thick, following the conventional construction of single and double leaf walls. The influence of infill wall openings is also examined. Infill wall openings are given as a percentage of the panel area. Five different cases for infill wall openings are studied. These are: fully infilled walls (0% openings), infill walls with small and large openings (25%, 50% and 75% openings) and bare frames (100% openings).

Moreover, five different values for the masonry panel strength were adopted to represent weak, medium and strong masonry, namely 1.5 MPa, 3.0 MPa, 4.5 MPa, 8.0 MPa and 10.0 MPa. These values are assumed to cover the most common cases for masonry infill condition in Europe.
The building parameters used for the development of the model are listed in Table 2.

4.3 Structural design of structures

The frames are designed according to Eurocode standards using the software FESPA (2013). Modal response spectrum analysis was also performed. The frames designed for seismic zone I with reference peak ground acceleration on type A ground, a_{gR}=0.16 g. The importance factor γ_I was taken as 1.0 and the ground type as B with soil factor S equal to 1.2, according to Eurocode 8. Frames designed for medium ductility class (DCM) and the behaviour factor, q assumed to be 3.45. Concrete strength class C25/30 was used for beams and columns, while steel grade B500c was used for the reinforcement steel bars. The dead load was 1.50 kN/m^2 plus 0.90 kN/m^2 to include interior partition walls in the mass of the building. Live load is 3.5 kN/m^2.

Slabs were 150 mm thick for all cases. Beams were 250/600 mm for all frames. Square column sections were used for all frames. For the fourteen storey frame with 6.0 m span length, columns had dimensions ranging from 650×650 [mm] at the ground floor to 500×500 [mm] at the roof. Column longitudinal reinforcement ratio was kept low and ranged between 1.0% and 1.5%, with most cases being under 1.15%. Column dimensions for all frames are shown in detail in Table 3. Column dimensions were kept the same for buildings with the same number of storeys, same bay size but different number of bays.

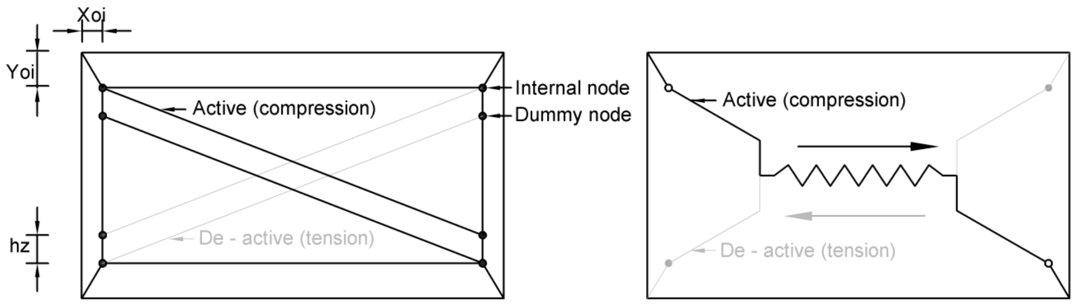
According to the seismic design for new structures, a change in building geometry or dimensions will cause a change in the column dimensions. This adds another parameter that affects the fundamental period of the structure, making the problem even more complex. For example, increasing the height of the ground floor would normally make the building more flexible, causing an increase of its period. That would be the case if the column dimensions stayed the same. However, according to the seismic design of this new building, column dimensions of the ground floor that is now higher may be larger and thus, the period may not be increased as expected. In order to examine the influence of certain parameters and changes in building geometry without being influenced by the changes in column dimensions, due to the seismic design, another building form is considered for comparison reasons. These frames are not designed according to codes. All columns dimensions were chosen to be 300×300 [mm] constant for all floors, independently of the number of bays or the bay size. These buildings will be referred as "non-designed" differently from the previous ones described referred as "designed".

4.4 Modelling of structures

All buildings were modelled as plane frames using Seismostruct (Seismosoft 2013). A plastic-hinge element has been adopted for beams and columns, with concentrated inelasticity within a fixed length at each member's end. The Mander et al. (1988) model, later modified by Martinez-Rueda and Elnashai (1997), has been assumed for the core and the unconfined concrete, while Menegotto-Pinto steel model has been adopted for the reinforcement steel (1973). Concrete compressive strength was equal to 25 MPa and the yield strength of the steel equal to 500 MPa. Mass was calculated using the seismic load combination, namely dead loads plus 30% of the live loads.

Masonry is modelled using the inelastic infill panel element. This is an equivalent strut nonlinear cyclic model proposed by Crisafulli (1997) for the modelling of the nonlinear response of infill panels in framed structures. Each panel is represented by six strut members. Each diagonal direction features two parallel struts to carry axial loads across two opposite diagonal corners and a third one to carry the shear from the top to the bottom of the panel (Fig. 6). Struts are active only in compression, hence its "activation" depends on the deformation of the panel. The axial load struts use the masonry strut hysteresis model, while the shear strut uses a dedicated bilinear hysteresis rule, as described by Crisafulli (1997).

Four internal nodes are employed to account for the actual points of contact between the frame and the infill panel (i.e., to account for the width and height of the columns and beams, respectively), whilst four dummy nodes are introduced with the objective of accounting for the contact length between the frame and the infill panel (Fig. 6). All the internal forces are transformed to the exterior four nodes where the element is connected to the frame. In Fig. 6 x_{oi} and y_{oi} are the horizontal and the vertical distance between the external corner nodes (middle of the beam-column joint) and the internal ones (point of contact of the masonry with the RC frame), accordingly. In the same Figure h_z is the equivalent contact length. Seismostruct (2013) suggests that reasonable results are obtained for values equal to 1/3 of the actual contact length (z), defined by Smith (1966) as equal to $0.5\,\pi(\lambda_h/h)^{-1}$, ($\lambda_h$ according to Eq. (16)). Infill walls with openings are modelled with the same element but reduced stiffness, according to Eq. (20).



(a) (b)

Fig. 6 Infill panel element proposed by Crisafulli (1997) (a) Compression/tension struts, (b) Shear strut

The fundamental periods are calculated with eigenvalue analyses based on the linear model with initial stiffness conditions for all components.

5. Results and discussion

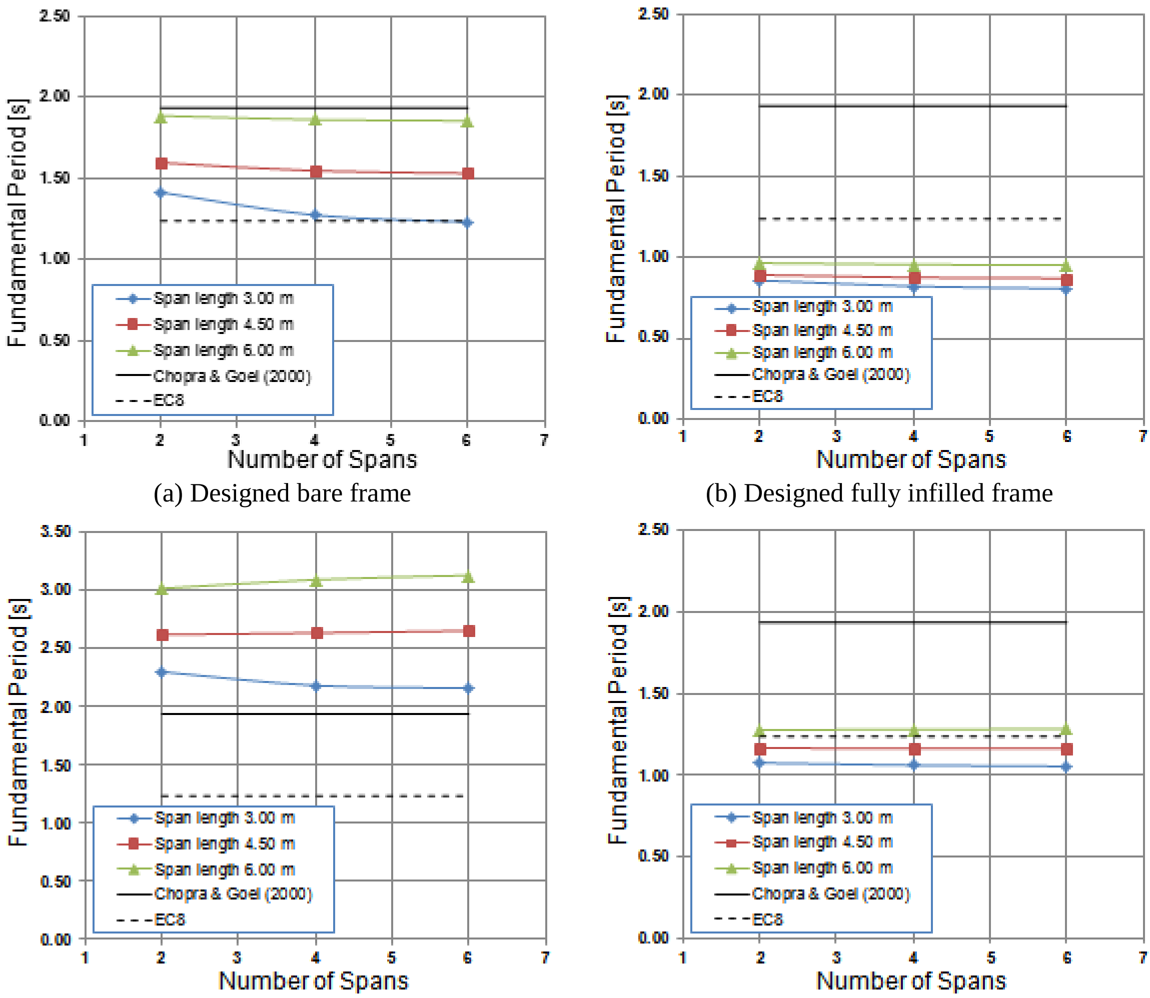
5.1 Influence of the number of spans on the fundamental period

Fig. 7 shows the relationship between the determined fundamental period of vibration versus the number of spans for both the "designed" and "non-designed" bare and fully infilled 14-storey RC frames. The time periods obtained from the eigenvalue analysis are also compared against the period obtained from EC8 and that from Chopra and Goel (2000). From Fig. 7 and Table 4, it is shown that the fundamental period obtained from modal analysis for both the "designed" and "non-designed" 14-storey RC infilled frames with span lengths ranging from 3 m to 6 m is not influenced by the number of spans (Fig. 7 (b), (d)). Also, the fundamental period for both the "designed" and "non-designed" 14-storey RC bare frames with span lengths ranging from 4.5 to 6 m is not influenced by the number of spans (Fig. 7 (a), (c)). This is in agreement with the findings of Amanat and Hoque (2006). However, the span length affects the fundamental period of the building. For the bare frame with two spans and span length equal to 3 m (Fig. 7 (a), (c)), the fundamental period is higher when compared to the same frame with four spans. The reference building is fourteen storeys. Thus, when there are two spans, the building becomes relatively slender and more flexible, since a cantilever action comes into effect against lateral sway, resulting in longer period. It may be mentioned that although a span length equal to 3 m is not common in practice, such theoretical span is used herein to better comprehend the characteristics of the RC frame even in these extreme conditions. Code equations are not capable to reveal the effect of the number of spans and span length. The equations from EC8 and that of Chopra and Goel (2000) do not have any provision to incorporate the effect of the number of spans in determining the time period, since there is no parameter relevant to span in the code equations. Moreover, from Fig. 7(a), the modal analysis for the "designed" bare frame resulted in periods falling within the region of those estimated by the code equations and that of Chopra and Goel (2000). On the other hand, from Fig. 7 (b), (d), for the "designed" fully infilled frame and that of the "non-designed" infilled frame, it is apparent that the values of the determined fundamental period are lower than those obtained from EC8 as well as that from Chopra and Goel (2000).

Table 4 Fundamental period of a 14-storey concrete frame

Case	Number of Spans	Bare Frame Span Length (m) 3.0	4.5	6.0	Fully Infilled Frame Span Length (m) 3.0	4.5	6.0
Designed Frame	2	1.413	1.597	1.887	0.860	0.893	0.967
	4	1.273	1.547	1.863	0.823	0.878	0.958
	6	1.230	1.532	1.856	0.809	0.872	0.954
Non Designed Frame	2	2.300	2.619	3.017	1.078	1.167	1.277
	4	2.182	2.635	3.093	1.064	1.164	1.281
	6	2.161	2.653	3.130	1.058	1.163	1.283

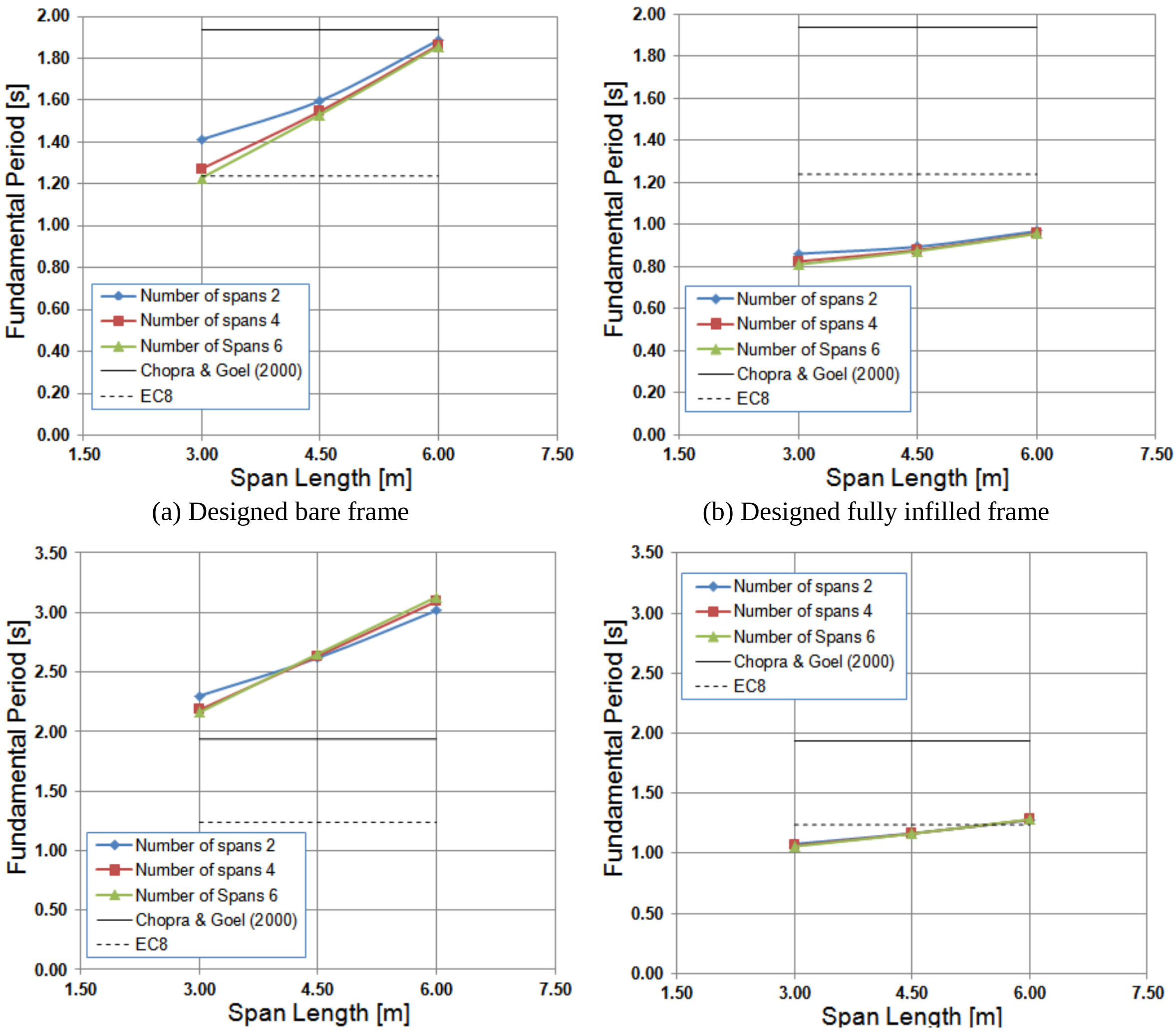
Note: Masonry wall modulus of elasticity E=1500 MPa;
Masonry wall thickness t=0.15 m;
Masonry wall stiffness $Et=2.25\cdot10^5$ kN/m



(a) Designed bare frame (b) Designed fully infilled frame

(c) Non-designed bare frame (d) Non-designed fully infilled frame

Fig. 7 Influence of number of spans on fundamental period of a 14-storey RC frame



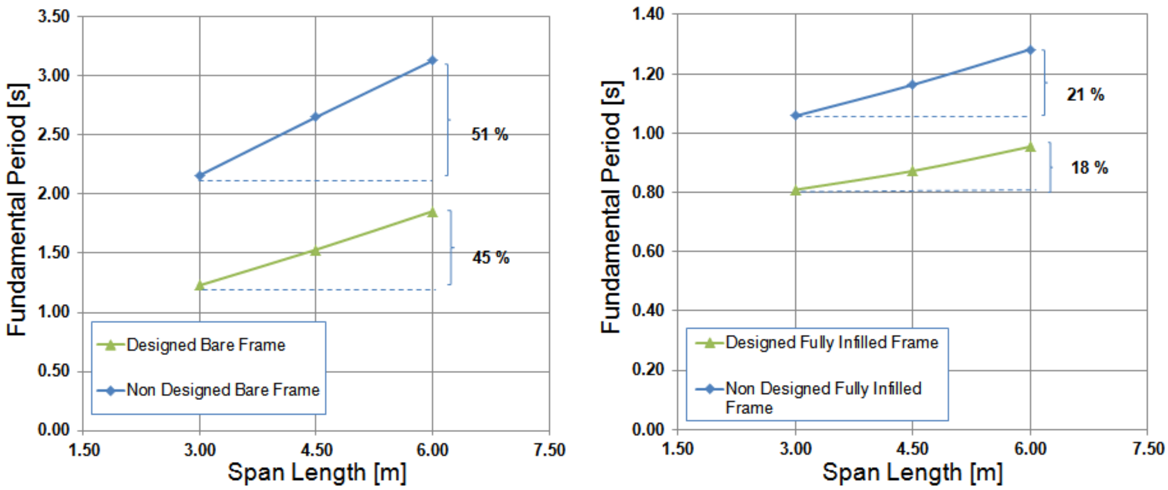
(a) Designed bare frame (b) Designed fully infilled frame

(c) Non-designed bare frame (d) Non-designed fully infilled frame

Fig. 8 Influence of span length on the fundamental period of a 14-storey RC frame

5.2 Influence of the span length on the fundamental period

A series of sensitivity studies has been undertaken to investigate the influence of span length on the fundamental period of the 14-storey RC framed building. The height of the interstorey has been kept constant and equal to 3 m while the span length taken as 3.0, 4.5 and 6.0 m. Fig. 8 shows the relationship between the determined fundamental period versus the span length for both the “designed” and “non-designed” bare and fully infilled 14-storey RC frames. Similarly, the time periods obtained from the eigenvalue analysis are also compared against the period obtained from the code equations and that from Chopra and Goel (2000). From Fig. 8 and Table 4, it can be seen that if the span length increases, the period of the RC building also increases. This is observed for both “designed” and “non-designed” bare and fully infilled 14-storey RC frames. Also, for the estimation of the time period of a building, both the code equations and the relationship derived from Chopra and Goel (2000) do not have any provision to incorporate the effect of span lengths in the direction of motion. Therefore, the periods predicted by these equations are the same for all values of span length studied. However, for the designed bare frame (Fig. 8(a)), the values of the determined fundamental period fall within the range of the values suggested by the code equations as well as that from Chopra and Goel (2000). But, this is not the case for the rest of the cases studied. Further work will be undertaken, in the future, to investigate the case when the span length is 7.5 m. It is important to mention that the influence of the span length corresponds to infill length-to-height aspect ratio equal to 1.0, 1.5 and 2.0 accordingly. From Fig. 8(a), it can be seen that the period increases by about 45% for 3 m increase of the span compared to the reference value of 3 m for the "designed" bare frame and by 51% for the "non-designed" bare 14-storey RC frame with six spans. Similarly, from Fig. 8(b) and for the fully infilled frame, the period increases by about 18% for 3 m increase of the span compared to the reference value of 3 m for the "designed" frame and by 21% for the "non-designed" 14-storey RC frame with six spans.

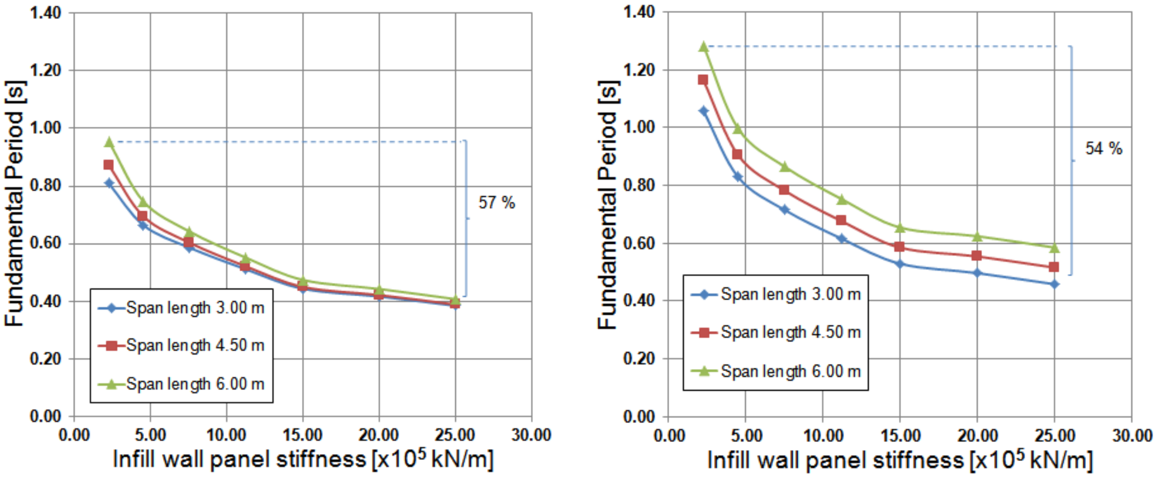


(a) Bare frame (b) Fully infilled frame

Fig. 9 Influence of design on the fundamental period of a 14-storey RC frame (Number of spans 6)

Table 5 Fundamental period of a six-span-14-storey fully infilled concrete frame for different span lengths

Masonry Wall			Designed Frame			Non Designed Frame		
Modulus of Elasticity *E* (MPa)	Thickness *t* (m)	Stiffness *Et* [$\times 10^5$ kN/m]	Span Length (m)			Span Length (m)		
			3.0	4.5	6.0	3.0	4.5	6.0
1,500	0.15	2.25	0.809	0.872	0.954	1.058	1.163	1.283
3,000	0.15	4.50	0.665	0.695	0.747	0.828	0.904	0.996
3,000	0.25	7.50	0.587	0.604	0.643	0.715	0.782	0.865
4,500	0.25	11.25	0.511	0.521	0.551	0.614	0.675	0.751
10,000	0.15	15.00	0.444	0.450	0.474	0.529	0.584	0.654
8,000	0.25	20.00	0.417	0.422	0.443	0.496	0.554	0.624
10,000	0.25	25.00	0.385	0.389	0.408	0.458	0.515	0.584



(a) Designed frame (b) Non-designed frame

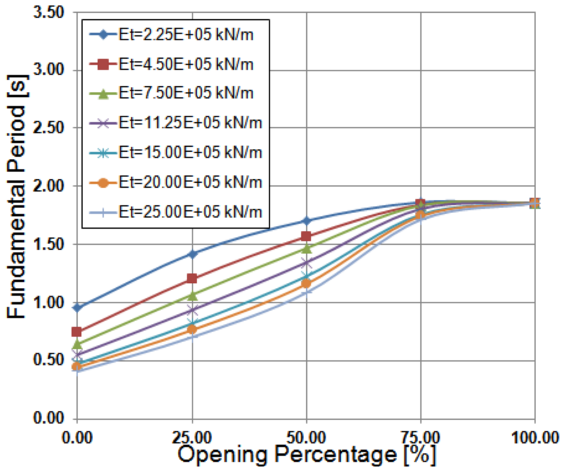
Fig. 10 Influence of masonry stiffness on the fundamental period of a 14-storey fully infilled RC frame (Number of spans 6)

5.3 Influence of infill masonry panel stiffness on the fundamental period

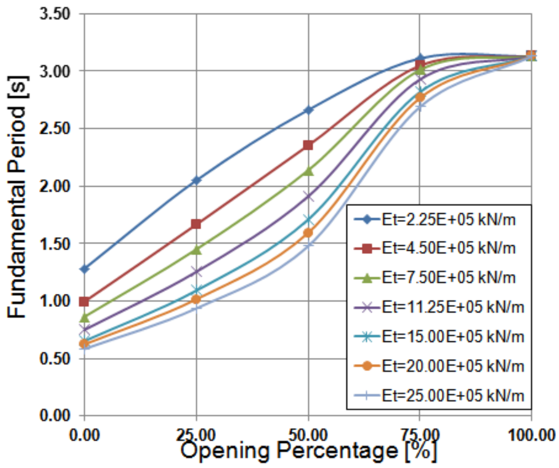
The mechanical characteristics of the masonry infill panels are shown in Table 5. The same Table and Fig. 10 show the determined fundamental period versus the infill masonry panel stiffness *Et* (E: modulus of elasticity, t: thickness of the masonry panel) for both the "designed" and "non-designed" 14-storey infilled RC frames for six spans with lengths ranging from 3 to 6 m. From Fig. 10, it can be seen that the period is highly sensitive to the infill wall panel stiffness. Infills act as diagonal bracing and resist lateral deflection. So, if the infill wall panel stiffness increases, the lateral deflection decreases and the fundamental period decreases. For the designed RC frame (Fig. 10(a)), it seems that for the same infill wall stiffness, the change in span length does not affect the fundamental period. However, this is not the case for the "non-designed" frame (Fig. 10(b)), where for the same stiffness of the infill, the fundamental period increases proportionally with the span length. Finally, from Fig. 10(a), it can be seen that the fundamental period of the 14-storey RC frame decreases by about 57 % for a change in infill wall stiffness from each 2.5×10^5 to 25×10^5 kN/m for the designed frame and by 54% for the non-designed frame with six spans. We can conclude that the decrease of the fundamental period as a result of the influence of infill masonry panel stiffness is almost the same for the "designed" and "non-designed" 14-storey RC frame.

5.4 Influence of infill openings percentage on the fundamental period of infilled frames

Fig. 11 shows the influence of infill opening (in terms of infill opening percentage with respect to total area of masonry wall) on the fundamental period of a 14-storey fully infilled designed RC frame with six spans and span length equal to 6 m. The values of the fundamental period of this 14-storey infilled frame with openings are shown in Table 6. From Fig. 11(a) and Table 6, it can be seen that as the infill opening percentage increases from full infill (i.e., no opening. 0% infill opening percentage) to 80% infill opening, the fundamental period increases almost linearly.



(a) Designed frame



(b) Non-designed frame

Fig. 11 Influence of opening percentage on the fundamental period of a 14-storey fully infilled RC frame (Number of spans 6; Span length=6.00 m)

Table 6 Fundamental period of a 14-storey partially infilled concrete frame

Case	Masonry Wall Stiffness *Et* [$\times10^5$ kN/m]	Opening Percentage					Reduction [%]
		0.00	25.00	50.00	75.00	100.00	
Designed Frame	2.25	0.954	1.421	1.705	1.863	1.856	48.57
	4.50	0.747	1.202	1.568	1.841	1.856	59.72
	7.50	0.643	1.068	1.468	1.837	1.856	65.33
	11.25	0.551	0.936	1.346	1.806	1.856	70.29
	15.00	0.474	0.820	1.226	1.758	1.856	74.45
	20.00	0.443	0.763	1.159	1.744	1.856	76.12
	25.00	0.408	0.703	1.086	1.713	1.856	78.03
	Reduction [%]	57.28	50.52	36.32	8.06	0.00	
Non Designed Frame	2.25	1.283	2.051	2.664	3.112	3.130	59.01
	4.50	0.996	1.667	2.355	3.049	3.130	68.17
	7.50	0.865	1.452	2.139	3.012	3.130	72.37
	11.25	0.751	1.256	1.912	2.930	3.130	76.00
	15.00	0.654	1.093	1.711	2.820	3.130	79.11
	20.00	0.624	1.018	1.596	2.769	3.130	80.06
	25.00	0.584	0.938	1.479	2.691	3.130	81.32
	Reduction [%]	54.44	54.27	44.46	13.53	0.00	

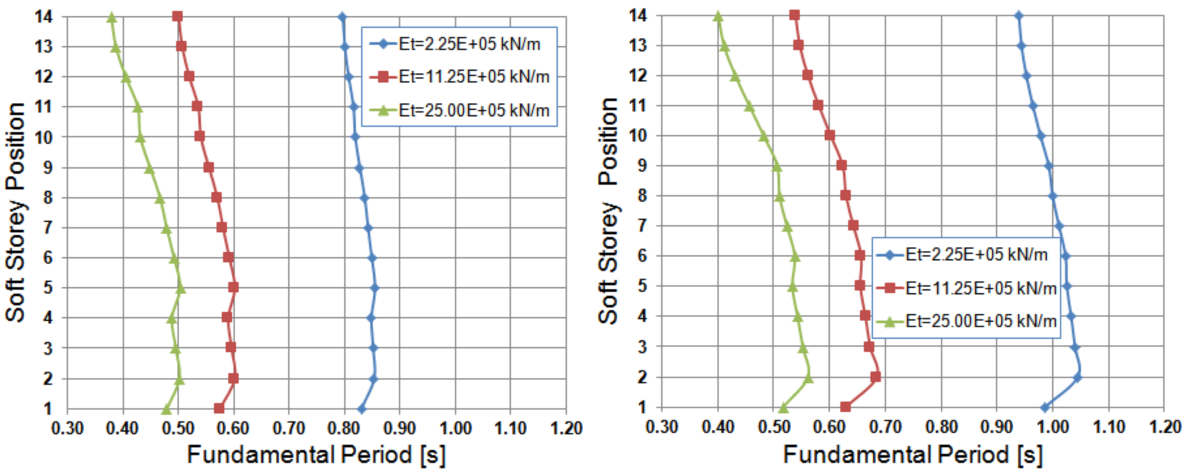
Note: Number of spans=6; Span length=6.00 m

However, when the infill opening percentage is ranging from 80% to 100% (i.e., bare frame), the opening does not affect the fundamental periods. This is due to the fact that when the opening is above 85%, the mass and stiffness of the infill does not contribute to the fundamental period. However, this is not the case for the “non-designed frame” (see Fig. 11(b)). More specifically, for the “non-designed” frame, as the opening in the infill panel increases from a full infill to bare

Table 7 Fundamental period of a 14-storey partially infilled concrete frame for different position of soft storey

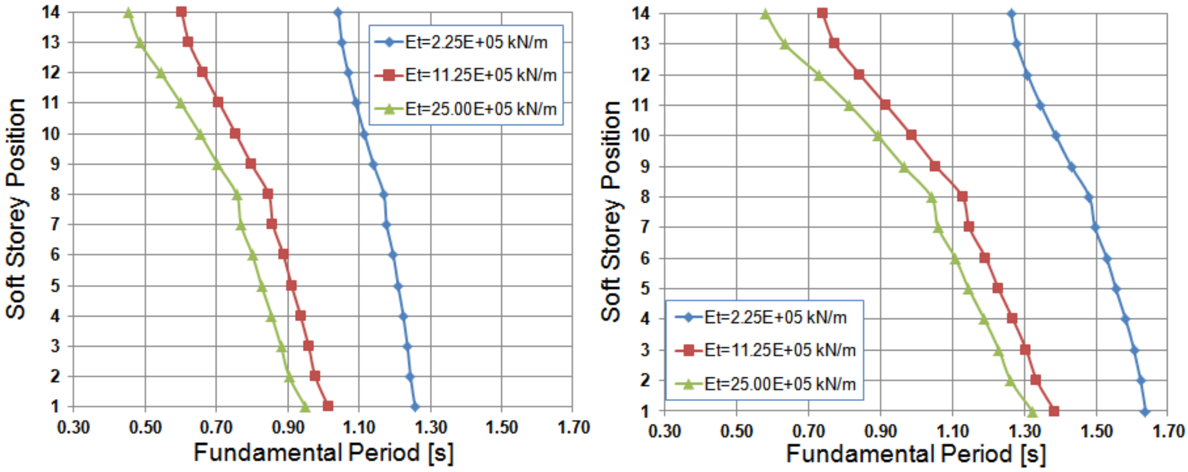
Case	Span length [m]	Masonry Wall Stiffness Et [$\times 10^5$ kN/m]	Soft Storey 1	2	3	4	5	6	7	8	9	10	11	12	13	14	Increas %
Designed	3	2.25	0.830	0.853	0.851	0.848	0.855	0.849	0.842	0.836	0.827	0.819	0.816	0.807	0.800	0.797	7.28
		11.25	0.575	0.602	0.596	0.590	0.602	0.592	0.581	0.570	0.555	0.541	0.536	0.520	0.507	0.501	20.22
		25.00	0.479	0.502	0.495	0.488	0.503	0.492	0.479	0.467	0.449	0.431	0.427	0.405	0.387	0.379	32.79
	6	2.25	0.986	1.045	1.039	1.033	1.025	1.023	1.011	1.000	0.993	0.978	0.964	0.952	0.944	0.939	11.23
		11.25	0.629	0.685	0.673	0.664	0.655	0.657	0.643	0.630	0.624	0.602	0.581	0.561	0.547	0.539	27.00
		25.00	0.517	0.561	0.552	0.543	0.534	0.538	0.524	0.511	0.506	0.482	0.456	0.431	0.411	0.401	40.06
Non Designed	3	2.25	1.257	1.244	1.236	1.224	1.210	1.196	1.178	1.170	1.140	1.115	1.091	1.069	1.052	1.042	20.59
		11.25	1.015	0.977	0.959	0.936	0.911	0.888	0.859	0.847	0.798	0.754	0.708	0.664	0.623	0.603	68.24
		25.00	0.951	0.904	0.881	0.855	0.826	0.801	0.769	0.757	0.703	0.654	0.601	0.544	0.485	0.454	109.29
	6	2.25	1.638	1.624	1.606	1.582	1.555	1.530	1.497	1.481	1.431	1.388	1.346	1.309	1.279	1.264	29.57
		11.25	1.385	1.335	1.306	1.269	1.229	1.193	1.148	1.128	1.055	0.988	0.916	0.843	0.774	0.739	87.47
		25.00	1.323	1.263	1.230	1.189	1.146	1.109	1.061	1.042	0.964	0.893	0.814	0.728	0.634	0.582	127.45

frame, the fundamental period of the structure increases. Finally, for both the "designed" and "non-designed" frames with the same opening, the higher the masonry stiffness, the lower the fundamental period. For the designed frame for various values of infill opening percentages and for values of masonry infill wall stiffness (Et) ranging from 2.25 to 25×10^5 kN/m, the fundamental period ranges from 0.4 to 1.8.



(a) Designed, Span length=3.00 m (b) Designed, Span length=6.00 m

Fig. 12 Influence of soft storey position on the fundamental period of a 14-storey fully infilled RC frame (Number of spans 6)



(c) Non-designed, Span length=3.00 m (d) Non-designed, Span length=6.00 m

Fig. 12 Continued

5.5 Influence of soft storey on the fundamental period

In earthquake resistant design, irregular building configurations due to architectural features are a significant source of serious earthquake damage. The influence of the location of the soft storey irregularity (absence of infill walls) on the fundamental period of the 14 storey RC frame with six spans is presented herein. According to ASCE/SEI 7-10 (2010), the soft storey irregularity (or flexible storey) is characterised as "Irregularity Type 1" and refers to the existence of a building floor that presents a significantly lower stiffness than the others. In this study, a series of sensitivity studies were undertaken for the 14 storey RC frame with six spans. In particular, the location of the soft storey varied from the first floor (case of Pilotis) to the fourteenth floor. Also, the masonry infill panel stiffness's varied from 2.25, 11.25 and 25 x 10^5 kN/m while the span lengths varied from 3 to 6 m. In total, 168 different cases were analysed and the results are presented in Table 7. Fig. 12 shows the influence of soft storey position on the fundamental period of a 14-storey fully infilled RC frame with six spans. From Fig. 12 (b)-(c) it can be seen that, for the "non-designed" frame, the higher fundamental period occurs when the soft storey is at the first floor. From Fig. 12(a), for the designed RC frame with span length equal to 3 m, the fundamental period is higher when the soft storey is located at the second and fifth floor of the building. From Fig. 12(b), for the designed frame with a span length equal to 6 m, the higher fundamental period occurs when the soft storey is at the second floor. The fundamental period depends on the location of the soft storey across the height of the building. Lower values of the period result when the soft storey is at the top floor, closer to the value of the fully infilled frame. Higher values of the period are evaluated for a soft storey at lower storeys but not at the first storey. From Table 7 it can be seen that the difference between the minimum and maximum fundamental period for different positions of the soft storey is 7% for the building with masonry wall stiffness 2.25×10^5 kN/m and 3.0 m span length and 40% for the building with masonry wall stiffness 25×10^5 kN/m and 6.0 m span length. This effect becomes smaller for lower values of stiffness.

6. Conclusions

An investigation has been performed on the calculation of the fundamental natural period of vibration of RC bare and infilled frame buildings by means of a finite-elements modelling. As a base study, a 14-storey “designed” and “non-designed” RC building has been considered. A sensitivity analysis has been undertaken to study the influence of geometric and stiffness parameters on the fundamental period of the structure. More specifically, the parameters investigated include: a) the number of spans; b) the influence of span length in the direction of motion; c) the influence of infill stiffness in the structure; d) the infill panel opening percentage and e) the soft storey position. The time periods obtained from the eigenvalue analysis were also compared against the period obtained from EC8 and that from other researchers including Goel and Chopra (2000). From the results analysis it has been found that the span length of the panel, the stiffness of the infill wall panel and the location of the soft storey in the structure are the important parameters influencing the natural period. However, code equations do not take into account the aforementioned parameters and inaccurately estimate the natural period of a structure. This study also shows that increasing the number of spans from two to six does not have a significant effect on the period. Instead, a change in the span length will significantly contribute to the period. More specifically, from the sensitivity analysis it has been found that:

• Increasing the span length increases the fundamental period of the infilled RC framed building;

• Similarly, increasing the infill length-to-height aspect ratio also increases the fundamental period of the infilled RC framed building;

• An increase of the infill wall panel stiffness from 2.25×10^5 to 25×10^5 kN/m reduces the fundamental period by approximately 57% for a designed frame;

• For the designed frame, as the opening increases from full infill to 80% opening, the fundamental period of the structure increases almost linearly. However, when the opening percentage is 80% and above, the increase of the opening does not affect the fundamental period of the structure since it is almost the same with the period of the bare structure;

• For both the designed and non-designed frames with the same opening, the higher the masonry stiffness, the lower the fundamental period;

• For the non-designed frame, the higher fundamental period occurs when the soft storey is at the first floor;

• The location of the soft storey in the structure and the length of the span in the direction of motion significantly affect the fundamental period of the structure. For the designed frame with span length equal to 3 m, the fundamental period is higher when the soft storey is located at the second and fifth floor of the building. For the designed frame with a span length equal to 6 m, the higher fundamental period occurs when the soft storey is at the second floor

Based on all the above, it is clear that the fundamental period of a building cannot be predicted using only the height of the building. The problem is more complex and various parameters influence the results. Nevertheless, the majority of the code equations as well as the equations proposed by many researchers take into account only the height of the building for the evaluation of the fundamental period. In order to undertake a more generalized suggestion regarding the determination and influence of the building’s period in the future, the fundamental period of vibration of a 2, 4, 6, 8 and 10 storey building and their sensitivity to the above studied parameters will be investigated in more detail and depth. In addition, in the future, apart from the influence of the location of the soft storey irregularity (absence of infill walls), the effect of setback on the fundamental period of RC framed buildings will be investigated. This has been investigated extensively by Young and Adeli (2013). They analyzed 12 irregular concentrically braced steel frame structures. From their results it was shown that current code equations are unable to accurately predict the effect of horizontal and vertical irregularities on the fundamental period. It was also shown that structures with vertical irregularities, like a setback, tend to have shorter fundamental period than the regular structures and a new equation was suggested.

Our expressed future goal is the development of a more reliable analytical equation for the estimation of the value of the fundamental period of infilled framed structures. Especially we will try to achieve it by means of regressive techniques and using as data all the above studied parameters and corresponding results.

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