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Asteris, PG, Repapis, CC, Cavaleri, L et al. (2 more authors) (2015) On the fundamental period of infilled RC frame buildings. Structural Engineering and Mechanics, 54 (6). pp. 1175-1200. ISSN 1225-4568

https://doi.org/10.12989/sem.2015.54.6.1175

On the Fundamental Period of Infilled RC High Rise Buildings

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ABSTRACT

In this study, the fundamental periods of vibration of a series of high rise buildings are studied using finite element modelling and modal eigenvalue analysis. As a base study, a 14 storey designed and non-designed RC building has been considered. Several parameters studied including the number of spans; the influence of span length in the direction of motion; the influence of infill stiffness in the structure; the infill panel percentage opening and; the soft storey position. The time periods obtained from the eigenvalue analysis were also compared against the period obtained from codes as well as other researchers. From the results analysis it was found the span length of the panel, the stiffness of the infill wall panel and the location of the soft storey in the structure are some of the important parameters influencing the natural period. This study also shows that varying the number of spans does not have a significant effect on the period. Instead, an increase a change in the span length will significantly contribute to the period. 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. The findings of the study shows that for the 14 storey RC frame, the location of the soft sorey at the first floor will not necessarily result in higher fundamental period.

KEYWORDS: Fundamental period, infilled frames, masonry, reinforced concrete buildings

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1 INTRODUCTION

In the context of seismic risk assessment and mitigation, a trustworthy expression 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 that influence the vibration period of buildings are: the structural regularity, the height of the building, the provision of shear walls, the number of storeys and bays, the dimensions of the member sections, the amount of infill, the position of load, the soil flexibility etc. The complexity of evaluation the above parameters and their interactions make the estimation of the fundamental period of a building a difficult task.

Worldwide, several earthquake design codes provide formulas for estimating the fundamental period of buildings. Typically, such formulas derived from regression analysis of values obtained from observed periods of real buildings during past earthquakes. Despite the fact that several 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. Also, the periods calculated based on these expressions revealing large discrepancies. Today, with the use of sophisticated computational methods of analysis, it is possible to determine the natural period of buildings by means of the exact eigenvalue analysis or by other rational methods (i.e. Rayleigh's method). Values obtained from such methods have found to be significantly larger than the observed period of the buildings. 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 building performance [1], [2], [41].

The rationale behind neglecting infill walls is partly attributed to: a) incomplete knowledge of the behavior of quasi-brittle materials, such as unreinforced masonry (URM); b) the composite behavior of the frame and the infill; as well as c) the lack of conclusive experimental and analytical results to substantiate a reliable design procedure for this type of structures, despite the extensive experimental efforts [3] – [9] and analytical investigations [10] – [19]. Moreover, due to the large number of interacting parameters, if the infill wall is to be considered in the analysis and design stages, a modeling 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 changes completely their behavior, 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 behavior 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 only applicable to the case of infill walls without openings on the diagonal of the infill panel [20], [21], [22], [23].

The aim of the work presented herein is to investigate the fundamental period of vibration of 14 storey RC bare and infilled-frame buildings by means of a finite-elements modeling under various geometric and other parameters, including the number of spans, the influence of span length, the influence of infill stiffness, the influence of the infill panel percentage opening and the influence of 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 Eqn. (1):

$$\mathbf{T = C_t \cdot H^{3/4}} \quad (1)$$

, where H is the total height of the building 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 [24] 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) [25] and the Uniform Building Code (UBC) [26], among others, adopt the same expression as ATC3-06 for the evaluation of fundamental period of vibration. The Jordanian National Building Code [27] also uses eq. (1) for the evaluation of the fundamental period of vibration and suggests a value for C_t equal to 0.04, merely based on expert judgment, as noted by Al-Nimry et al. [28]. Also, the New Zealand Seismic Code (NZSEE) [29] adopts the period-height relation for the fundamental period where the coefficient C_t is given as 0.09 for reinforced concrete frames, 0.14 for structural steel and 0.06 for other type of structures. Further, the Israeli Seismic Code (SI-413) [30] provides a value of 0.049 for the coefficient $C_{t.}$

The UBC proposed formula has been updated in FEMA 450 [31] based on the study by Goel and Chopra [32] and the measured period of concrete moment-resisting frames buildings, monitored during California earthquakes (including the 1994 Northridge earthquake). Based on the lower bound of the data presented by Goel and Chopra, FEMA proposed the expression of Eqn. (2) for RC frames that provides a conservative estimate of the base shear:

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

where C_r is given as 0.0466 and x as 0.9.

The National Building Code (NBC) [33] of Canada adopts the expression of Eqn. (3) that relates the fundamental period of building with the number of stories, N, above the exterior base, as follows:

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

Similarly, the Costa Rican Code [34] gives the expression:

$$T = 0.08N \tag{?}$$

The aforementioned empirical expressions are very 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) [35] takes into account the influence of shear walls on the fundamental period of the RC buildings as shown in Eqn. (4):

$$T = 0.09\,\frac{H}{\sqrt{L}}\sqrt{\frac{H}{H+\rho L}} \tag{4}$$

, where L is the width of structure in the seismic direction under consideration (in meters) and ρ is the ratio of the areas of shear wall section along a seismic action direction to the total area of walls and columns.

Other building codes (i.e. the Indian Seismic Code [36] , the Egyptian Code [37] and the Venezuelan Code [38]), in addition the building's height H, take into consideration the total base dimension, d, of the masonry in-filled reinforced concrete frame. Such expression for the estimation of the fundamental period of vibration is given by Eqn. (5):

$$\mathbf{T = \frac{0.09h}{\sqrt{d}}} \tag{5}$$

The French Seismic Code [39] recommends using the most unfavorable of Eqn. (5) and Eqn. (6) that is specified for masonry buildings:

$$\mathbf{T = 0.06\frac{h}{\sqrt{d}}\sqrt{\frac{h}{2d+h}}} \tag{6}$$

The Algerian Seismic Code [40] adopts two expressions for the fundamental period, the simple height-related Eqns. (1) and Eq. (6) and prescribes that the smallest value should be used.

Also, the Eurocode 8, besides the general height-related expression (Eqn. 1), provides a more exact expression for the calculation of the coefficient C_t, for masonry in-filled reinforced concrete frames (Eqn. 7):

$$\mathrm{C_t} = \frac{0.075}{\sqrt{\mathrm{A_c}}} \text{ and } \mathrm{A_C} = \sum \mathrm{A_i}\left(0.2 + \frac{\mathrm{l_{wi}}}{\mathrm{h}}\right) \tag{7}$$

where, $\mathrm{C_t}$ is the correction factor for masonry in-filled reinforced concrete frames, $\mathrm{A_C}$ is the combined effective area of the masonry in-fill in the first storey, A_i is the effective cross-sectional area of the wall in the first storey and $\mathrm{l_{wi}}$ is the length of wall in the first story in the considered direction.

2.2 *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. Crowley and Pinho [42] 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. Further in 2006, Crowley and Pinho [43] 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 use in the assessment of existing RC buildings where the presence of masonry infills was also taken into account.

In Guler et al [44], the fundamental periods of some RC buildings, considering the effects of infill walls, were computed using ambient vibration tests and elastic numerical analyses. A period-height relationship relevant to Turkish RC moment-resisting frames 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) [32]
$T = 0.0294H^{0.804}$	Hong and Hwang (2000) [45]
$T = 0.067H^{0.9}$	Chopra and Goel (2000) [46]
$T = 0.1H$	Crowley and Pinho (2004) [42]
$T = 0.055H$	Crowley and Pinho (2006) [43]
$T = 0.026H^{0.9}$	Guler et al. [44]

Fig. 1 presents a comparison of some of the aforementioned height-related expressions for the evaluation of fundamental period of vibration. 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 [45] underestimates the value of fundamental period (the period value is below 0.5 sec even for total building height of 30 m). On the other hand, the equation by Crowley and Pinho [42] seems to overestimate the value of fundamental period, especially in cases of buildings with total height of 30 m. Besides the extreme boundaries, other proposals show similarities in calculating the fundamental period when considering only the building total height.

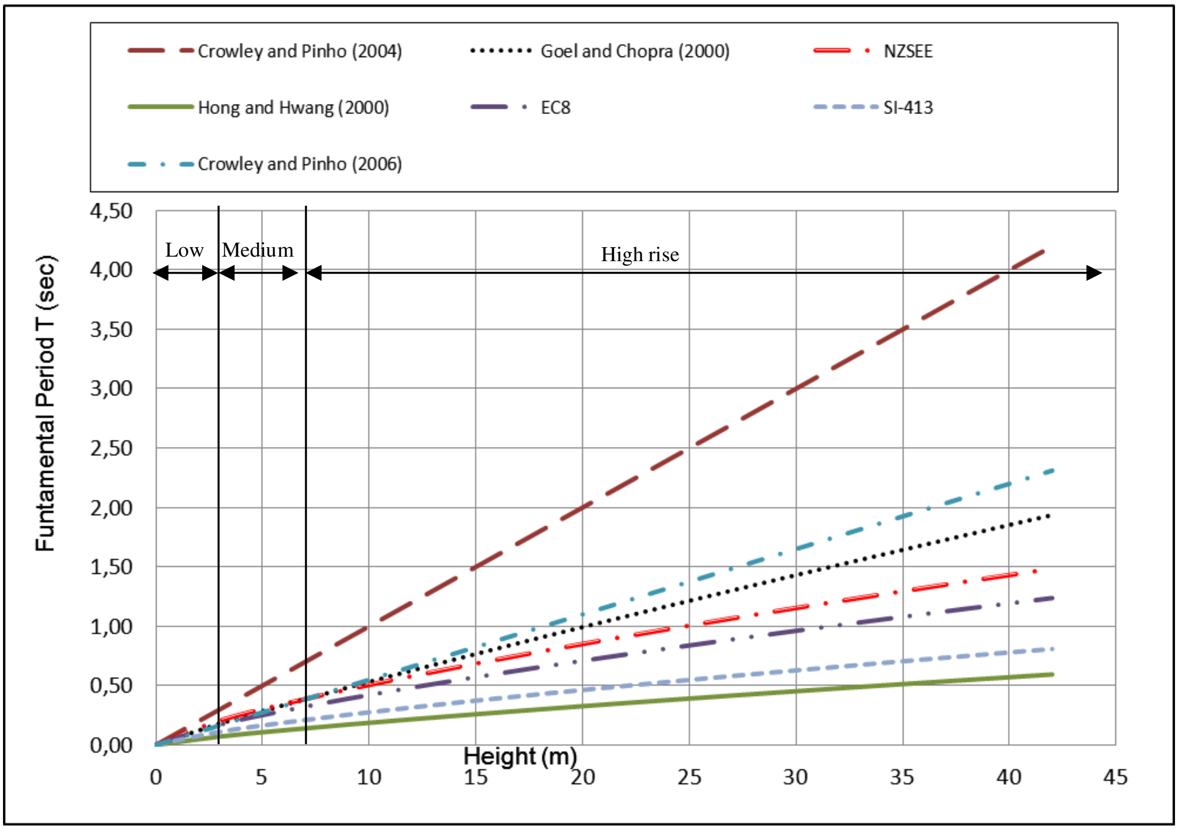


Fig. 1. Comparison of equations for the evaluation of the fundamental period

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 [47]; Amanat and Hoque [48]). 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 that take into consideration more than the type and height of the structure (Amanat and Hoque [48]; Crowley and Pinho [42]; Goel and Chopra [32]; Hong and Hwang [45]) and that will be discussed in the following sections.

Crowley and Pinho [49] by taking into account the presence of infills, have proposed an expression for RC moment resisting frames with rigid infills based on the simple period-height formula:

$$\mathrm{T} = \frac{0.09\mathrm{H}}{\sqrt{\mathrm{D}}} \tag{8}$$

, where D is the dimension of the building at its base in the direction under consideration.

Amanat and Hoque [48] have studied the fundamental periods of vibration of a series of regular RC framed buildings using a 3D finite-element modeling and modal eigenvalue analysis and have identified that the span length, number of spans and amount of infills a significantly influence the fundamental period. The proposed equation based on the study, is given by Eqn. (9):

$$\mathbf{T} = \boldsymbol{\alpha_1 \alpha_2 \alpha_3 C_t h^{3/4}} \tag{9}$$

, where C_t = 0.073 for RC buildings, the factor $\boldsymbol{\alpha_1}$is the modification factor for span length of infill panel, $\boldsymbol{\alpha_2}$ is the modification factor for number of spans and $\boldsymbol{\alpha_3}$ is the modification factor for amount of infill.

Hatzigeorgiou and Kanapitsas [50] have proposed an empirical expression for the fundamental period of frames 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 based on a database of 20 real RC buildings which have already been constructed in Greece and is as follows:

$$\mathbf{T} = \frac{\mathbf{H^{c_1} L^{c_2} (c_3 + c_4 W)}}{\mathbf{\left[1 - \exp\left(c_5 k_s^{c_6}\right)\right] \sqrt{(1 + c_7 \rho)}}} \tag{10}$$

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 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. Ignoring the influence of infill and concrete shear walls, soil flexibility and length of the building, Hatzigeorgiou and Kanapitsas [50] give a simpler expression for the fundamental period which is very similar to the formula by Eurocode 8:

$$T = 0.073H^{0.745} \tag{11}$$

Kose [2] investigated the fundamental period of vibration of RC frame buildings using a computational iterative modal analysis in 3D. He evaluated the effect of building height, frame type and the presence of infill walls, among other parameters and he proposed the expression of Eqn. (12) for the prediction of the fundamental period of reinforced concrete moment resisting frames:

$$T = 0.0935 + 0.0301H + 0.0156B + 0.039F - 0.1656S - 0.0232I \tag{12}$$

where H = height of building in meters, B = number of bays, F = frame type equal to 1 for frames with infills, 2 for frames with open first floor and 3 for bare frames, S - ratio in percentage of shear walls to total floor area, I = area ratio of infill walls to total 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 [2] 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 area ratio of infill walls to total panels:

$$T = 0.1367 + 0.0301H - 0.1663S - 0.0305I \tag{13}$$

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. Further, 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, infill height-to-length aspect ratio, the percentage of infill opening, the relative panel-to-frame-stiffness and the presence of soft story. Such parameters need further investigation, in order to assess their importance and impact of the fundamental period and propose refined expressions for a more accurate period evaluation.

3 OMPUTATIONAL STRUCTURAL MODELING

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).

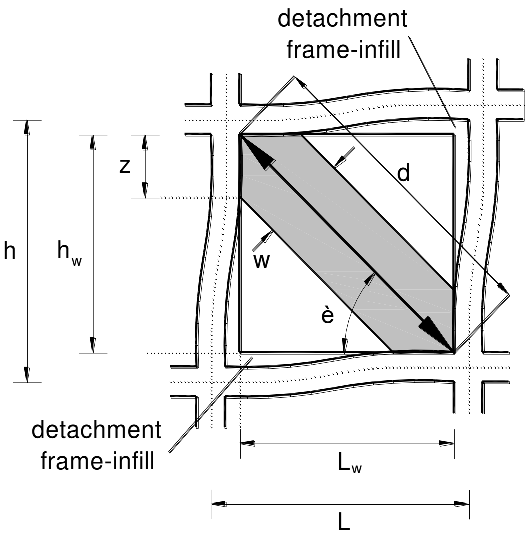


Fig. 2. Masonry infill frame sub-assemblage

Early research on the in-plane behavior 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 behavior and confirmed its highly indeterminate nature in terms solely of the normal parameters of design [51]-[53]. On the basis of these few tests a purely empirical interaction formula was later tentatively suggested by Wood [54] for use in 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, he short-circuited a mass of confusing detail and he recognized the desirability of a higher load factor where strengths were most dependent on the infills.

3.1 *Infill Walls Modeling*

In the early sixties, Polyakov [55] suggested the possibility of considering the effect of the infilling in each panel as equivalent to diagonal bracing, and this suggestion was later adopted by Holmes [56], 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 defined by

$$\frac{w}{d} = \frac{1}{3} \tag{14}$$

where d is the diagonal length of the masonry panel. The "one-third" rule was suggested as being applicable irrespective of the relative stiffness of the frame and the infill. One year later, Stafford Smith [57], 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. On the second half of the sixties Stafford Smith and his associates using additional experimental data [3], [58], [59] 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 [60]. They proposed the evaluation of the equivalent width as a function of the relative panel-to-frame-stiffness parameter, in terms of

$$\lambda_h = h \sqrt[4]{\frac{E_w t_w \sin 2\theta}{4EIh_w}} \tag{15}$$

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

$$\theta = \tan^{-1}\left(\frac{h_w}{L_w}\right) \tag{16}$$

in which L_w is the length of infill panel (all the above parameters are explained in Fig. 2). Based on experimental and analytical data Mainstone [61] proposed an empirical equation for the calculation of the equivalent strut width, given by

$$\frac{w}{d} = 0.16\,\lambda_h^{-0.3} \tag{17}$$

Mainstone and Weeks [62] and Mainstone [63], also based on experimental and analytical data, proposed an empirical equation for the calculation of the equivalent strut width:

$$\frac{w}{d} = 0.175\,\lambda_h^{-0.4} \tag{18}$$

This formula was included in FEMA-274 (Federal Emergency Management Agency 1997) [31] for the analysis and rehabilitation of buildings as well as in FEMA-306 (Federal Emergency Management Agency 1998) [64], 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 usually have oversized openings, recent research has mainly focused on the simple case of infill wall without openings. 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 much 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 in the lateral stiffness of masonry infill walls a finite element technique proposed by Asteris [15], [18] has been used herein. The basic characteristic of this analysis 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. 3a). 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 neighboring 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. 3h).

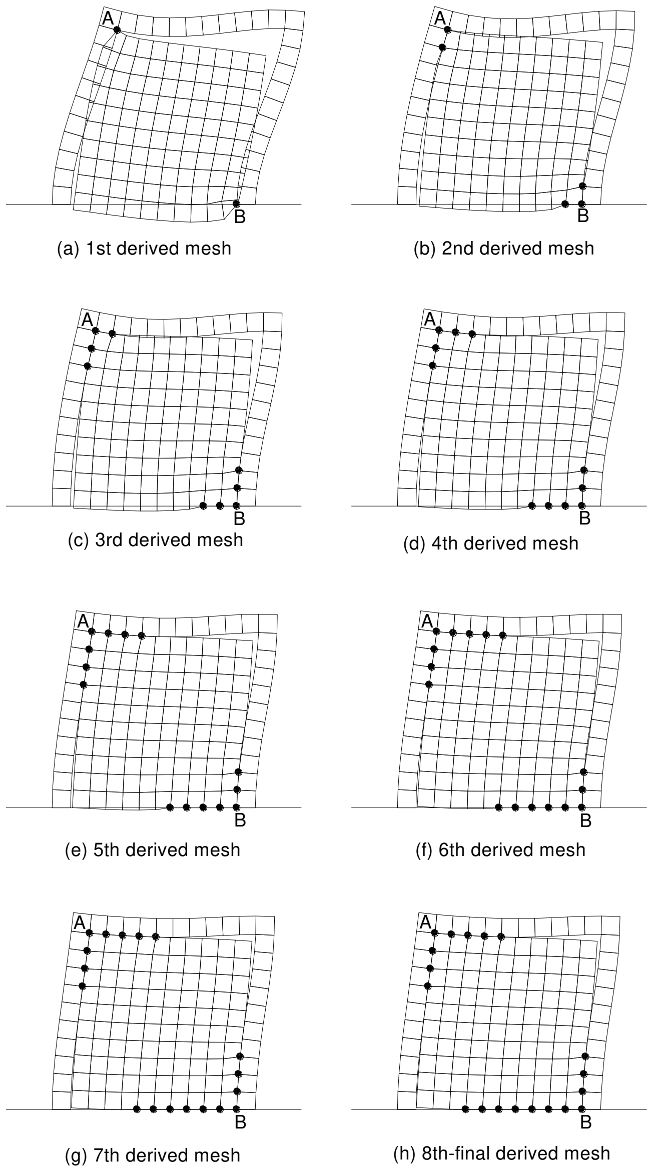


Fig. 3. Deformed meshes of an one-storey one-bay infilled frame using the finite element technique proposed by Asteris [15], [18].

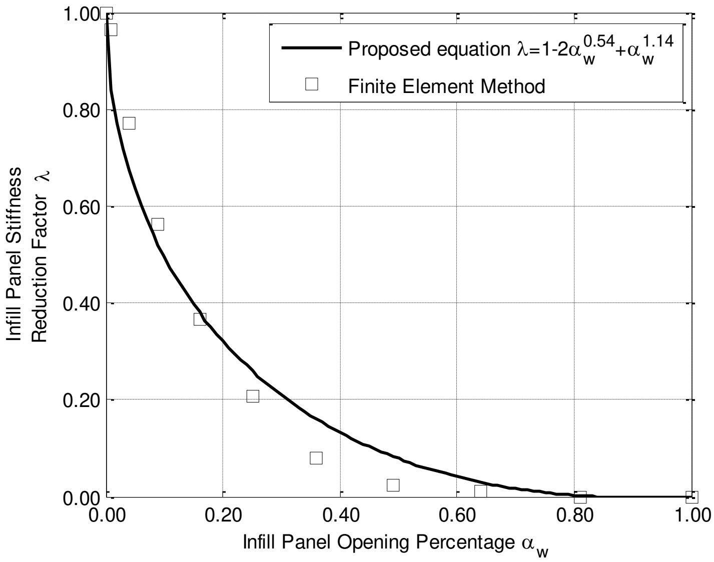


Fig. 4. Infill panel stiffness reduction factor in relation to the opening percentage

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 present parametric study using the finite-element method, lead to the following relationship for the infill wall stiffness reduction factor λ:

$$\lambda = 1 - 2\alpha_w^{0.54} + \alpha_w^{1.14} \tag{19}$$

in which α_w is the infill wall opening percentage (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 results of Eqns 14, 17 and 18 above. It can also be used to modify the equations of the Crisafulli model, which is described below.

4 DESCRIPTION OF THE INVESTIGATED STRUCTURES

4.1 *Building forms*

The fundamental period of high rise RC structures are examined in this study. Buildings considered are frame systems regular in plan, comprised by beams and columns. Thus, only one frame is finally considered and planar analysis is done. The buildings are cast-in-place reinforced concrete structures with beams cast monolithically with slabs and supported by columns.

Buildings examined have 14 storeys in order to examine the influence of the number of storeys. The storey height for all buildings is 3.0 m. The number of spans ranges form 2, 4 and 6. For each case, three different span lengths are examined, namely 3.0 m, 4.5 m and 6.0 m. In the perpendicular direction the bay size is 5 m, which is common for all buildings.

4.2 *Influence of infill panels*

The influence of infill walls is examined analyzing both bare frame structures as well as structures with fully or partially unreinforced masonry infilled frames with or without openings. Various parameters are considered for each case. Infill panels are 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. That is fully infilled walls (0% openings), infill walls with small and large openings (25%, 50% and 75% openings) and bare frames (100% openings).

Finally, three values for the masonry strength were adopted to represent weak, medium and strong clay brick masonry, namely 1.5 MPa, 3.0 MPa and 4.5 MPa. These values are assumed to cover the most common cases for masonry infill condition.

The building parameters are listed in Table 2.

Table 2. Building parameters

Concrete strength	25 MPa

Modulus of elasticity of concrete, E_c	31 Gpa
Steel tensile yield strength	500 MPa
Size of beams	25/50 (bay size 3.0m), 25/60 (bay size 4.5 m, 6.0m)
Slab thickness	15 cm
Dead loads	1.50 kN/m^2 + 0.90 kN/m^2
Live loads	3.50 kN/m^2
Number of floors	2, 4, 6, 8, 10, 12, 14
Building height	14 m
Bay size	3.0 m, 4.5 m, 6.0m
Number of bays	2, 4, 6
Masonry compressive strength, f_m	1.5 MPa, 3.0 MPa and 4.5 MPa
Modulus of elasticity of masonry, E_m	1.5 GPa, 3.0 GPa and 4.5 GPa
Thickness of infill panel, t_w	25 cm
Infill wall opening percentage	0% (fully infilled, 25%, 50%, 75%, 100% (bare frame)

4.3 *Structural design of structures*

The frames are designed according to Eurocodes using the software FESPA [65]. Modal response spectrum analysis is performed. The frames are designed for seismic zone I with reference peak ground acceleration on type A ground, a_{gR} = 0.16 g. The importance factor γ_I is 1.0 and ground type is B with soil factor S = 1.2. Frames are designed for medium ductility class (DCM) and the behaviour factor, q is 3.45 for both horizontal directions. Concrete strength class C25/30 was used for beams and columns, while steel grade B500c was used for reinforcement steel bars. Dead loads are self-weight of the structure, 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 are 15 cm thick for all cases. Beams are 250/500 [mm] for frames with 3.0 m bay size and 250/600 [mm] for frames with 4.5 and 6.0 m bay sizes. Columns are rectangular for all frames. Columns dimensions vary from 650x650 [mm] at the ground floor to 500x500 [mm] at the roof for the 14storey frame with 6.0 m bay size. On the contrary, the 3 storey frame with 3.0 m bay size has column dimensions 350x350 [mm] at both storeys. Column longitudinal reinforcement ratio was kept low and ranges between 1.0% and 1.5%, with most cases being under 1.15%. Column dimensions for all frames are shown in detail in **Error! Reference source not found.**. Column dimensions were kept the same for buildings with the same number of storeys, same bay size but different number of bays.

Table 3. Side dimension (cm) of rectangular columns

Storey	Column's Dimensions (cm)		
	Bay size 3.0 m	Bay size 4.5 m	Bay size 6.0 m
14	40	40	50
13	45	45	55
12	45	45	55
11	45	50	55
10	50	50	55
9	50	50	55
8	50	50	60
7	50	55	60
6	50	55	60
5	50	60	65
4	55	60	65
3	55	60	65
2	55	60	65
1	55	60	65

4.4 *Modelling of structures*

All buildings are modelled as plane frames using Seismostruct [66]. 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. [67] model, later modified by Martinez-Rueda and Elnashai [68], has been assumed for the core and the unconfined concrete, while Menegotto-Pinto steel model has been adopted for the reinforcement steel [69]. Concrete compressive strength is 25 MPa and yield strength of steel is 500 MPa. Mass is calculated from seismic load combination, namely dead loads + 30% live loads.

Masonry is modelled using the inelastic infill panel element. This is an equivalent strut nonlinear cyclic model proposed by Crisafulli [70] 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. 5). This latter strut only acts across the diagonal that is on 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 [70].

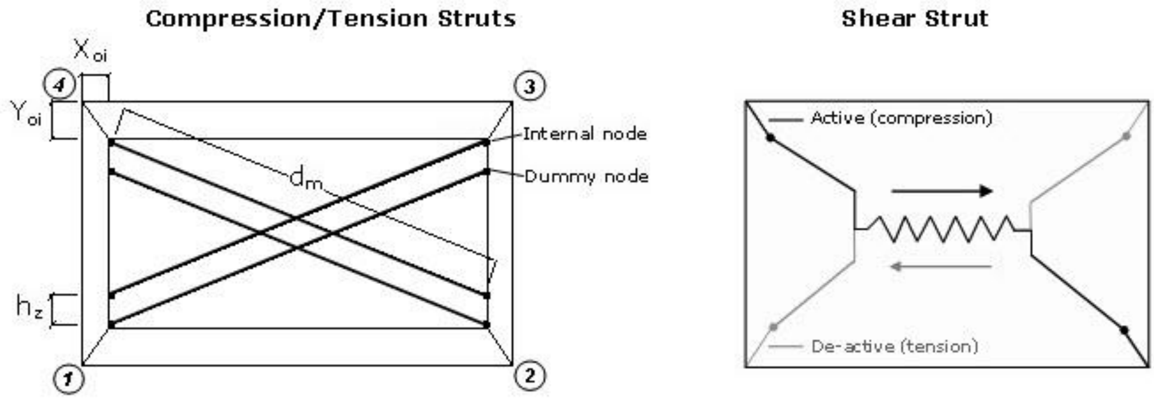


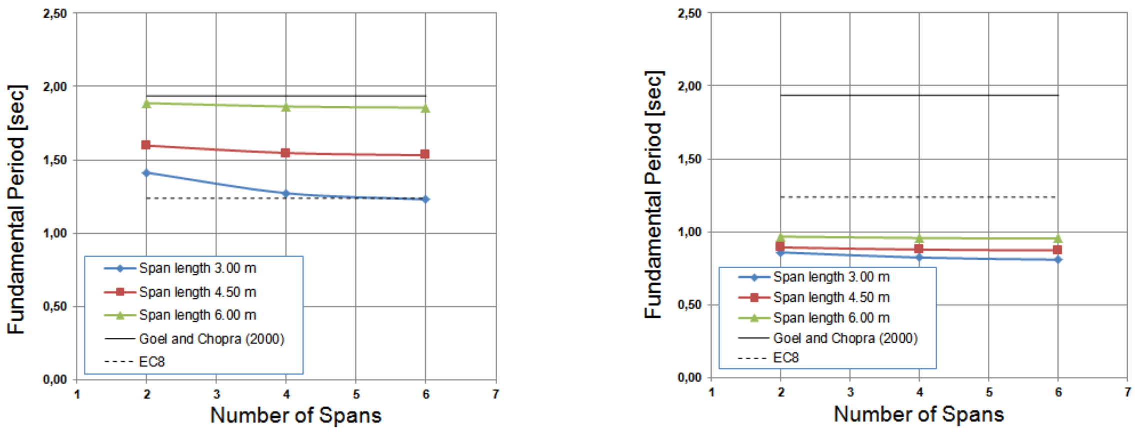
Fig. 5. Infill panel element proposed by Crisafulli [70]

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. 5). All the internal forces are transformed to the exterior four nodes where the element is connected to the frame.

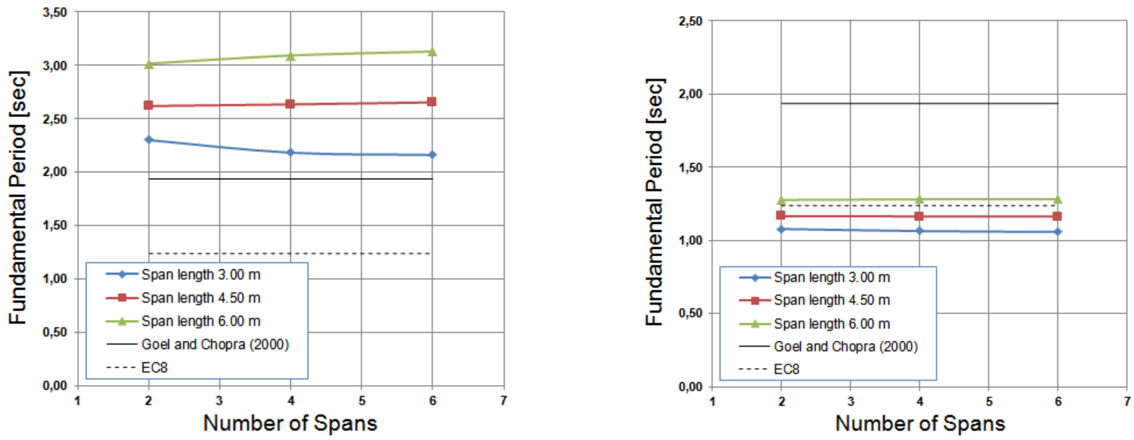
5 RESULTS AND DISCUSSION

5.1 *Influence of number of spans on the fundamental period*

Figure 6 shows the relationship between the determined fundamental period 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 Goel and Chopra (2000). From Figure 6 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 3m to 6 m is not influenced by number of spans (Fig. 6b & 6d). 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. 6a & 6c). However, the span length does affect the fundamental period of the building. For the bare frame with two spans and span length equal to 3 m (Fig 6a & 6c), the fundamental period is higher when compared to the same frame with four number of 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 have some general ideal of the characteristics of the RC frame even in these extreme conditions. Code equations are not capable to reflect the effect of the number of spans and span length. The equations from EC8 and that of Goel and Chopra (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. 6a, 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 Goel and Chopra (2000). On the other hand, from Fig. 6b & 6d, 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 Goel and Chopra (2000). However, from Fig. 6c, for the non-designed fully infilled frame, the values of the fundamental period determined using modal analysis, are higher than those estimated from the code equations and Goel and Chopra (2000).



a) Designed bare frame b) Designed fully infilled frame



c) Non Designed bare frame   d) Non-Designed fully infilled frame

Fig. 6. Influence of Number of spans on Fundamental Period of a 14-storey RC frame

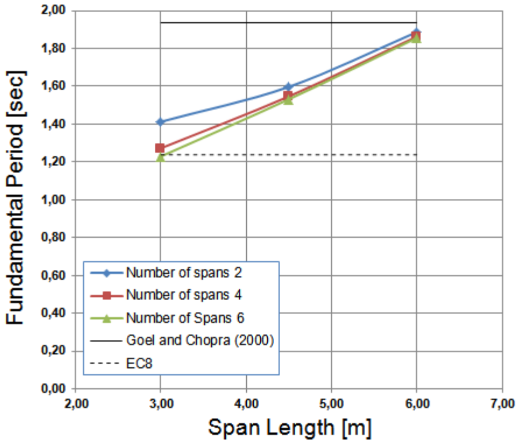
Table 4: Fundamental Period of a 14-storey concrete frame

Case	Number of Spans	Bare Frame			Fully Infilled Frame		
		Span Length			Span Length		
		3.0	4.5	6.0	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

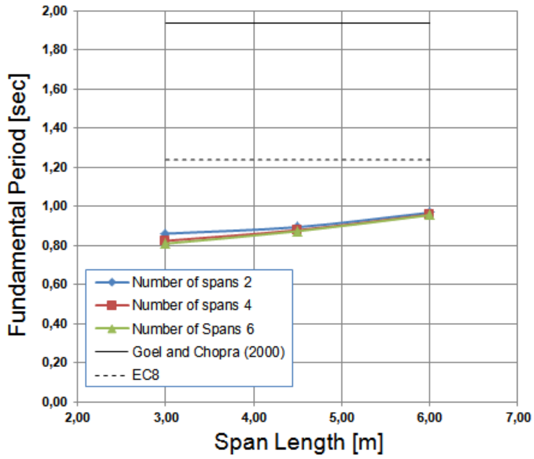
Note: Masonry wall Modulus of Elasticity E=1500 Mpa;
Masonry wall Thickness t=0.15 m;
Masonry Wall Stiffness Et=2.25E+05 kN/m

5.2 *Influence of span length on the fundamental period*

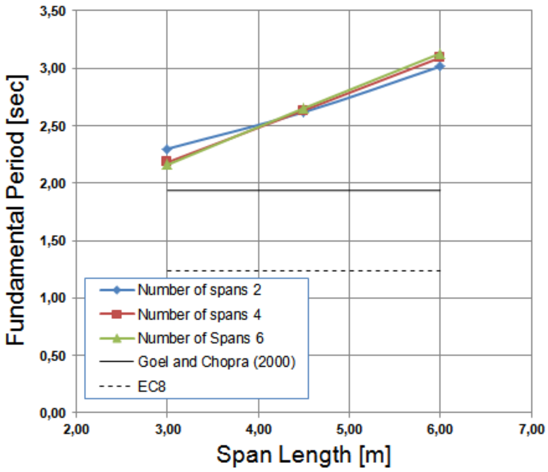
Figure 7 shows the relationship between the determined fundamental period versus the spans length for both the designed and non-designed bare and fully infilled 14 storey RC frames. Similarly, the time periods obtained from the eigenvalue analysis are also compared against the period obtained from code equations and that from Goel and Chopra (2000). From Figure 7 and Table 4, increasing the span length decreases the period of the RC building. This 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 Goel and Chopra (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. 7a), the values of the determined fundamental period falling within the range of the values suggested by the code equations as well as that from Goel and Chopra (2000). But, this is not the case for the rest of the cases studied. From, Fig. 7a & Fig.7d, it can be observed that at 6 m span, the period is about the same as that obtained from the code equation. The period decreases for smaller spans. This is due to the fact, that for longer spans, the stiffness contribution of the infill decreases (Madan et al. 1997). In the future, further works will be undertaken to investigate the case when the span length is 7.5 m. From Figure 8a, it can be seen that the period increases by about 45 % for each 3 m change in span from the reference value of 3m for the non-designed bare frame and by 31% for the designed bare 14 storey RC frame with six spans. Similarly, from figure 8b and for the fully infilled frame, the period increases by about 21% for each 3 m change in span from the reference value of 3 m for the non-designed frame and by 8% for the designed 14 storey RC frame with six spans.



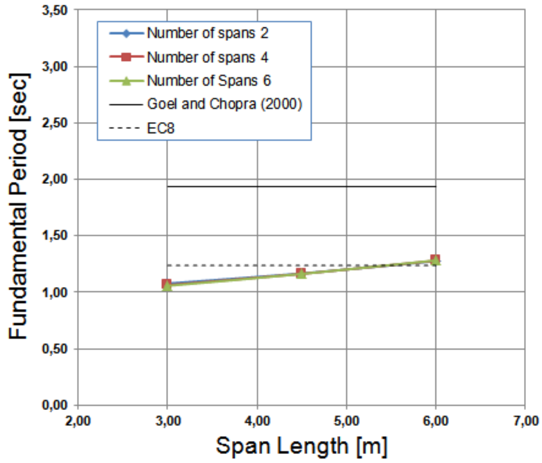
a) Designed bare frame



b) Designed fully infilled frame

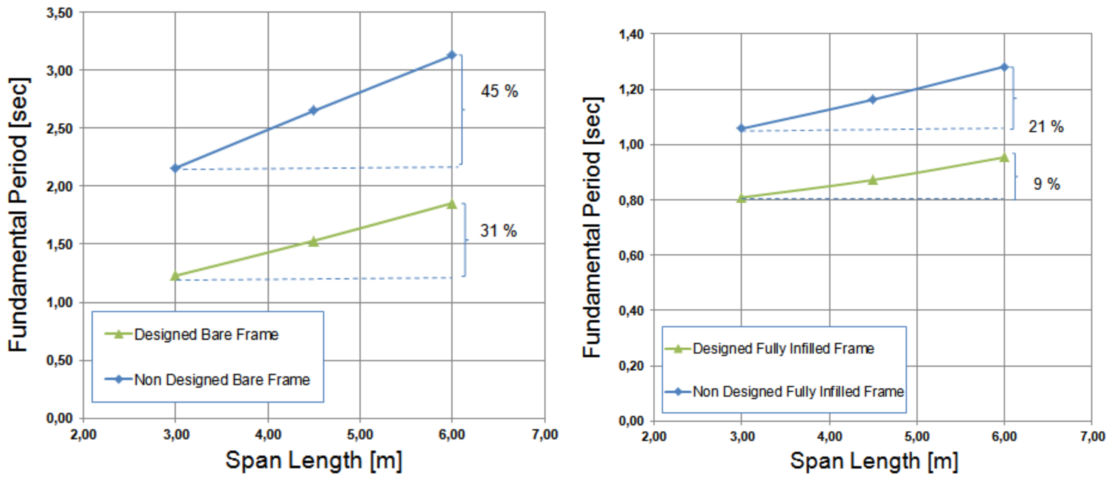


c) Non Designed bare frame



d) Non Designed fully infilled frame

Fig. 7. Influence of Number of spans on Fundamental Period of a 14-storey concrete frame

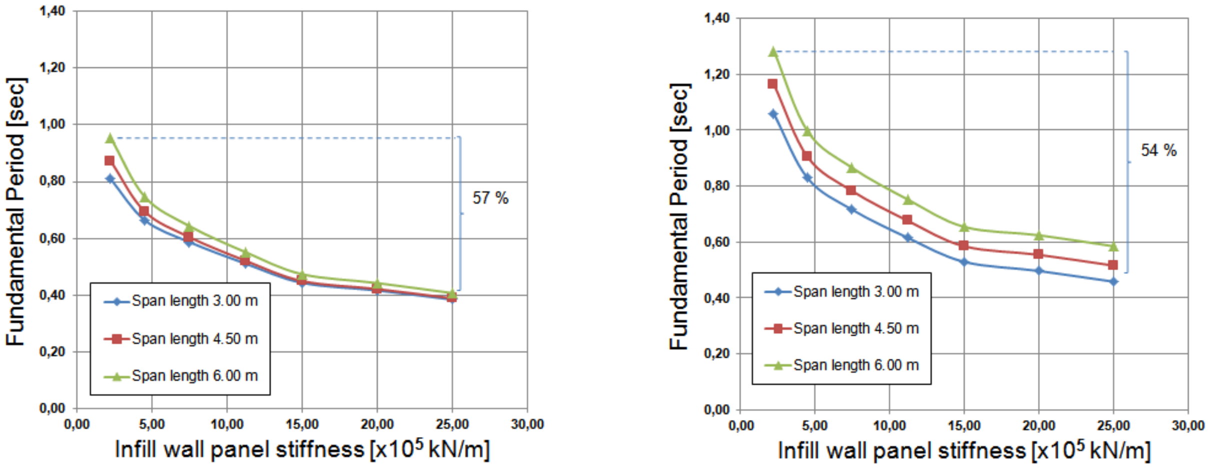


a) Bare frame b) Fully in-filled

Fig. 8. Influence of design on Fundamental Period of a 14-storey concrete frame (Number of spans 6)

5.3 *Influence of infill masonry panel stiffness on Fundamental Period ή πλαίσιο*

Figure 9 and Table 6 shows the determined fundamental period versus the column stiffness (EI/L) for both the designed and non-designed 14 storey infilled RC frame for six of spans with lengths ranging from 3 to 6 m. The mechanical characteristics of the masonry infill panels is shown in Table 6. From Figure 9, the period is highly sensitive to the infill wall panel stiffness. Infills act as diagonal bracing and resist lateral deflection. So, increasing the infill wall panel stiffness, increases the lateral deflection and reduces the fundamental period. For the design RC frame (Fig. 9a), it seems that for the same infill wall stiffness, the change in span length does not vary the fundamental period. However, this is not the case for the non-designed frame (Fig 9b), where for the same stiffness of the infill, the fundamental period increases proportionally with the span length. Finally, from Figure 9a, 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 x 10^5 to 25 x 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.



a) Designed b) Non-designed

Fig. 9. Influence of masonry stiffness on Fundamental Period of a 14-storey fully infilled concrete frame (Number of spans 6)

Table 5: Mechanical Characteristics of Masonry Infill panels

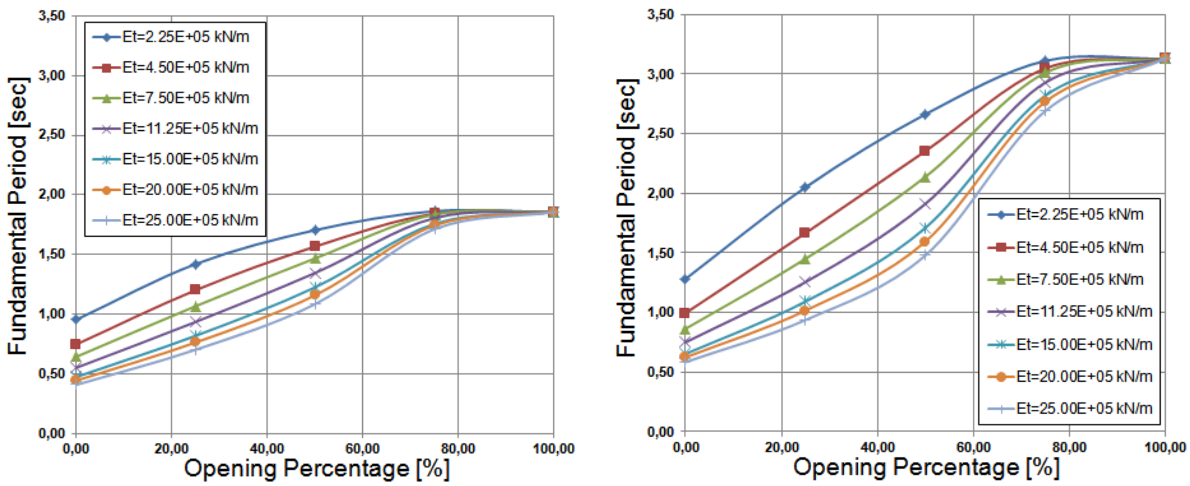
Case of infill panel	**Modulus of Elasticity E (Mpa)**	**Thickness t (m)**	**Stiffness Et [x10^5 kN/m]**
1	1,500	0.15	2.25
2	3,000	0.15	4.50
3	3,000	0.25	7.50
4	4,500	0.25	11.25
5	10,000	0.15	15.00
6	8,000	0.25	20.00
7	10,000	0.25	25.00

Table 6: Fundamental Period of a six-span-14-storey fully infilled concrete frame for different span lengths

Masonry Wall Stiffness Et [x10^5 kN/m]	**Designed Frame**			**Non Designed Frame**		
	Span Length			**Span Length**		
	3.0	4.5	6.0	3.0	4.5	6.0
2.25	0.809	0.872	0.954	1.058	1.163	1.283
4.50	0.665	0.695	0.747	0.828	0.904	0.996
7.50	0.587	0.604	0.643	0.715	0.782	0.865
11.25	0.511	0.521	0.551	0.614	0.675	0.751
15.00	0.444	0.450	0.474	0.529	0.584	0.654
20.00	0.417	0.422	0.443	0.496	0.554	0.624
25.00	0.385	0.389	0.408	0.458	0.515	0.584

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

Figure 10 shows the influence of opening percentage on fundamental period of a 14-storey fully infilled designed RC frame with six number of spans and span length equal to 6 m. From Figure 10a, as the opening percentage decreases from full infill to 80%, the fundamental period increases almost linearly. However, when the opening percentage is above 80% up to the 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 designed frame (see Figure 10b). More specifically, for the non-designed frame, as the opening in the infill panel increases from a full infill to bare 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 and for values of Et ranging from 2.25 to 25 x 10^5 kN/m, the fundamental perid ranges from 0.4 to 1.8. However, for the non-designed frame, the fundamental period varied from approximately 0.6 to 3.1.



a) Designed

b) Non designed

Fig. 10. Influence of opening percentage on Fundamental Period of a 14-storey fully infilled concrete frame (Number of spans 6; Span length=6.00 m)

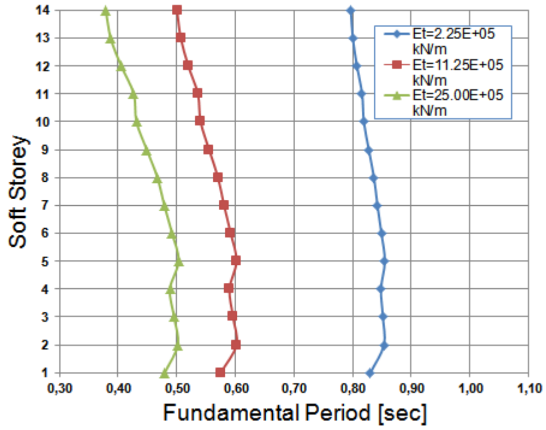
Table 7: Fundamental Period of a 14-storey partially infilled concrete frame

Case	Masonry Wall Stiffness Et [x10^5 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**	

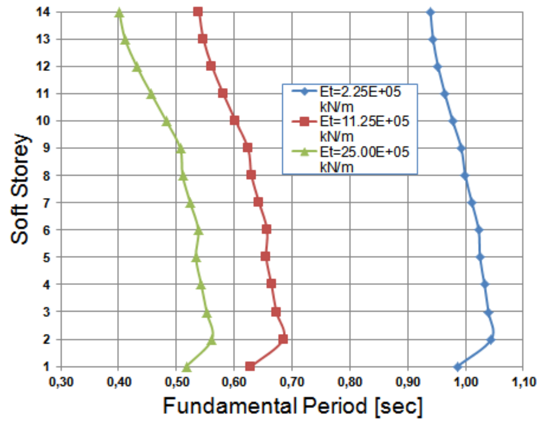
Note: Number of spans =6; Span length=6.00 m

5.5 *Influence of soft storey on the fundamental period*

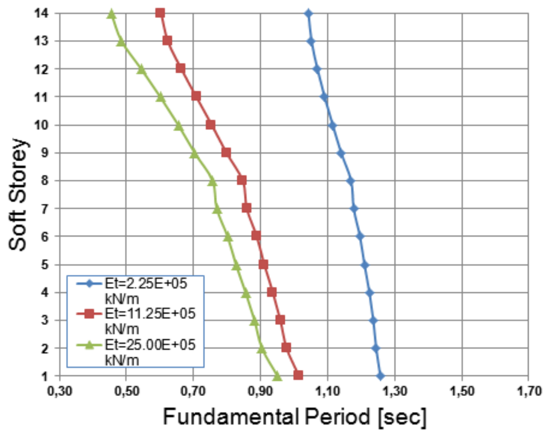
Figure 11 shows influence of soft storey position on the fundamental period of a 14-storey fully infilled concrete frame with six spans. From Figure 11b and 11c we can see that for the non designed frame, the higher fundamental period occurs when the soft storey is at the first floor. From Figure 11a and for the designed RC frame with span length equal to 3 m, the fundamental period is high when the soft storey is located at the second and fifth floor of the building. From Figure 11b, and 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. Also, from Table 8, the change of the fundamental period when the soft storey is at the first floor and when it is at the fourteenth floor increases from 7 to 40%. Also, this effect becomes less for lower values of stiffness.



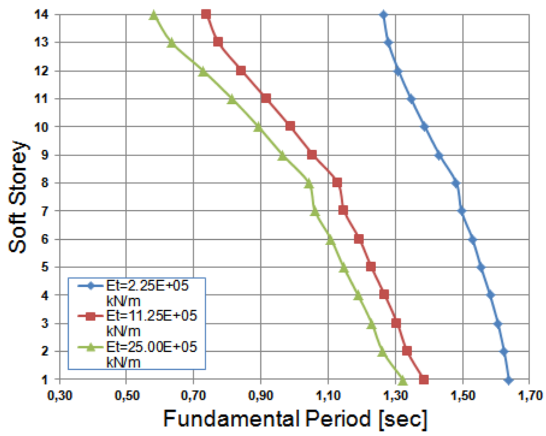
a) Designed, Span length=3.00 m



b) Designed Span length=6.00 m



c)Non-designed, Span length=3.00 m



d) Non-designed, Span length=6.00 m

Fig. 11. Influence of soft storey position on Fundamental Period of a 14-storey fully infilled concrete frame (Number of spans 6)

Table 8: Fundamental Period of a 14-storey partially infilled concrete frame for different position of soft storey

Case	Span length [m]	Masonry Wall Stiffness Et [x10^5 kN/m]	Soft Storey														Increase %
			1	2	3	4	5	6	7	8	9	10	11	12	13	14	
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

6 CONCLUSIONS

An investigation has been performed on the fundamental natural period of vibration of high rise 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. Some 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 percentage opening 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 was found the span length of the panel, the stiffness of the infill wall panel and the location of the soft storey in the structure are some of the important parameters influencing the natural period. However, code equations do not take into account the above parameters and inaccurately predict the natural period of a structure. This study also shows that varying the number of spans from three to six does not have a significant effect on the period. Instead, an increase a change in the span length will significantly contribute to the period. More specifically, from the sensitivity analysis it was found:

- Increasing the span length decreases the period of the RC building;
- An increase of the infill wall panel stiffness will increase the lateral deflection and reduce the fundamental period by approximately 57% for a designed frame and for wall stiffness ranging from 2.5 x 10^5 to 25 x 10^5 KN/m;
- Fort he designed frame, as the opening decreases 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;
- 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 high 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

In order to undertake a more generalized suggestion regarding the determination and influence of the buildings period, in the future the fundamental period of vibration of a 2, 4, 6 8 and 10 m height building and their sensitivity to the above studied parameters will be investigated.

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