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Investigations of Elastic Vibration Periods of Reinforced Concrete Moment-Resisting Frame Systems with Various Infill Walls

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ABSTRACT

The fundamental period of vibration is a crucial characteristic in assessing the dynamic performance of reinforced concrete (RC) buildings because it is not only directly related to the mass and stiffness of the structure, but also to the lateral actions applied, e.g. earthquakes and winds. In this study, the RC moment-resisting frame (MRF) systems designed under gravity and wind loading have been evaluated by utilising 3D FE modelling incorporating eigen-analysis to obtain the elastic periods of vibration. The parameters considered include the number of storeys, the number and length of bays, plan configurations, mechanical properties of infill walls, and the presence of openings in the uncracked and cracked infill walls. These analyses provide a sound basis for further investigating the effects of these parameters and exploring the possibility of proposing new formulas for predicting the fundamental vibration period by utilising regression analyses on the obtained results. The proposed numerically based formula for vibration periods of bare RC frame models reasonably agrees with some cited formulas for vibration period from design codes and standards due to disregarding contributions of infills' stiffness towards the structural systems. Meanwhile, the proposed formulas for RC MRF buildings with uncracked infills agree well with most cited experimentally based formulas and some numerically based ones. However,

the proposed formulas for RC MRF buildings with cracked infills only reasonably agree with some cited numerically based formulas.

Keywords Reinforced concrete; Moment-resisting frame; Infills; Vibration period; Wind load

1. Introduction

The period of vibration has been widely evaluated with regard to the effect of seismic forces on reinforced concrete (RC) buildings by using the equivalent lateral load analysis. However, evaluating such dynamic characteristics of these buildings under wind loading has attracted less attention than earthquake loading due to the life threating situations caused under the latter. In conventional structural analysis, the bare frame is only taken into account with all other structural elements, while the effect of non-structural elements such as infill walls is neglected due to uncertainty on the behaviour of these non-structural elements [1-4]. However, the inclusion of the infills in assessing the behaviour of RC buildings with infills has become of interest due to their significant contributions to the enhancement of the lateral stiffness of bare frame buildings under seismic loading, and then to the alternations of the dynamic properties of these buildings by the added mass and stiffness of the infills [5-7].

The behaviour of RC buildings with infills, i.e. linear or nonlinear, is related to the intensity of the applied lateral loads where there are two distinguished phases, namely local and global behaviours, depending on the interactions between the infill panels and the surrounding frame elements. In the past few decades, these two phases have been evaluated using experimental investigations and analytical simulations, and the latter approach has been calibrated with the former in many studies to evaluate different behaviours of RC framed buildings with infills. Much attention has been paid to analytical methods to simulate the behaviour of RC frame structures with infills, including two common techniques, namely macro-modelling and micro-modelling. The former technique simulates the infills as equivalent diagonal struts, while the latter technique models the infill materials individually using the finite element method [8]. Thus, the macro-modelling approach is widely used to simulate the global behaviour of RC framed buildings with infills, e.g. in determining the period of vibration due to the simplicity of the model. The micro-modelling technique is used to simulate the local behaviour of these buildings, such as local forces and failure modes, and is made possible by the accurate and intensive simulation of the interactions of the infilled panels with the surrounded frame elements in the numerical analysis [3,9,10].

In general, the evaluation of seismic forces on RC buildings relies on the period of vibration of the structures. In the event of moderate to high intensity earthquakes, the pre-existing cracking in RC concrete members can be more influential, resulting in larger periods of vibration and lower levels of stiffness of buildings. Therefore, the use of either numerically or experimentally based formulae for predicting the periods of vibration of RC buildings under moderate to high intensity earthquakes may significantly overestimates the periods of vibration under other types of low amplitude motion, e.g. wind [11]. Thus, the design of buildings according to their behaviour under earthquakes will be very conservative due to the assumption of cracking and degraded stiffness. However, the design of buildings under wind loading is more economical than earthquake design due to the linear elastic assumptions of structural members. In particular, the design of tall buildings is governed by wind loading with respect to lateral drift and human comfort. In regions with low seismic events, therefore, the design of mid-rise to high-rise buildings is dominated by wind loading as the leading variable action. Some numerical studies [5,12–15] have simulated low-rise to mid-rise infilled RC buildings by evaluating the period of vibration based on their popularity and easiness, and some experimental investigations have been conducted on these types of buildings for comparison. Most studies so far have been conducted on the design of RC buildings under gravity loads only, or by considering seismic forces.

In this study, RC moment-resisting frame (MRF) systems were strictly designed under the effects of gravity, imposed and wind loads to structural Eurocodes [16-20], and then these systems were modelled in a parametric study to investigate the effects of several design parameters on the dynamic response of the models in terms of the elastic period of vibration. The parameters investigated included the number of storeys, the number and length of bays, plan configurations, mechanical properties of infill walls, and the presence of openings in the infill walls. These parameters are vital in the investigation due to their direct and indirect effects on the dynamic performance of RC buildings. Nowadays, with the availability of powerful computers, numerical analysis using the Finite Element (FE) method becomes a reliable approach of accurately evaluating the dynamic response of this type of buildings due to the involvement of the mass and stiffness of the model. Correspondingly, this can help to establish the formulas for estimating the periods of vibration experienced by the addressed buildings. The deduced formulas will be compared with those cited from the literature and in the design codes and standards to check their accuracy in evaluating this dynamic property.

2. Formulas for determining the vibration period of RC buildings

To evaluate the dynamic characteristics in terms of the fundamental period of vibration and the damping ratio for RC MRF systems, gravity loads are normally considered together with lateral loads, e.g. earthquake. Many design standards and codes recommend assessing the period of vibration solely with respect to the building height. In the presence of infills or shear walls, the wall area and length in the considered directions are normally considered. Therefore, many studies have utilised code approaches by applying different parametric studies on the obtained data experimentally or numerically to establish formulas and evaluate the periods of vibration of RC MRF systems. These studies have investigated MRF systems with different configurations, e.g. bare frames, frames with solid masonry infills of clay bricks and concrete blocks, or with the presence of openings. However, these systems have not been comprehensively analysed under wind loading, and their height limit is narrowed down to low-rise to mid-rise buildings due to the popularity of these structures in the cities around the world. This has attracted much attention onto RC MRF buildings.

The proposed formulas in these studies have created huge scatters in relation to different key issues for comparison with the formulas proposed in the design standards and codes. These issues include the amplitude of motion sources in the experimental studies, the cracking or damage of structural components in the tested buildings, the presence of other structural components, e.g. shear walls, the presence of infills and the variety of their mechanical properties, the height of the building, the assumptions adopted in the numerical modelling, etc. In particular, the classification of excitation sources has expanded from weak to strong amplitudes, which causes variations of the response of the structure. Even under weak motions, nonlinear response can be observed, i.e. a variation of the elastic properties due to nonlinear elastic phenomena as stated in the study by Guéguen et al. [21], or due to atmospheric loading, e.g. temperature or/and wind as stated in the studies by Clinton et al. [22], Herak and Herak [23], and Mikael et al. [24]. They can therefore be used to evaluate the dynamic characteristics of buildings, including their fundamental vibration periods and damping ratios. Thus, this paper focuses on those prior studies which used low intensity sources to carry out ambient microtremor vibration tests to obtain the fundamental vibration period. The formulas presented in the design codes/standards and suggested by other researchers are summarised, and then compared with those proposed in the present study based on the numerical results generated.

2.1. Design codes and standards for buildings

The American Standard ASCE 7-10 [25] states that for wind design, the proposed formulas for predicting the fundamental frequency under earthquakes may lead to non-conservative values for wind. The frequency will be higher than the actual one and yield lower gust effect factor and design wind pressures. Thus, the standard recommends that such formulas are used in predicting the fundamental frequency under wind loading, but some limitations need to be verified with respect to the height and effective width of the buildings with regular plans. Hence, the cited design formulas are presented in terms of the period of vibration, T, rather than the fundamental frequency (the inverse of T), and the International System of Units (SI) is used. Most codes and standards evaluate the vibration periods or frequencies of buildings in terms of the building height H as follows, based on the regression analyses on the prior test results:

$$T = \alpha H^{\beta} \tag{1}$$

where α and β are empirical coefficients. The upper-bound formula provided for concrete moment-resisting frame buildings is based on the study by Goel and Chopra [26]. The standard also provides another upper-bound expression obtained from analytical analyses for wind tunnel tests, which can be applied to all buildings with height below 122 m, regardless of their material types. The empirical coefficients used for Eq. (1) are listed in Table 1.

The Eurocode BS EN 1991-1-4 [20] and the Australian and New Zealand Standard AS/NZS 1170.2 [27] recommend a similar formula for obtaining the fundamental period of vibration for all types of buildings, which was first derived by Ellis [28], with the corresponding empirical coefficients cited in Table 1. Fig. 1 illustrates the recommended formulas in the standards and codes mentioned above which are to be used in evaluating the fundamental vibration periods of RC buildings in terms of the height of the building. It is notable that huge differences exist between the ASCE standard and other codes.

2.2. Experimental studies

As stated above, the evaluation of seismic forces is related to the estimation of vibration periods of buildings. This dynamic property has been evaluated by using different methods for various building configurations around the world since 1936. The first surveying study on the use of ambient vibration method to obtain the vibration periods of specified buildings in the US was done by Carder [29]. After that, this method has been developed and adopted for obtaining the dynamic characteristics of buildings, e.g. vibration period and damping ratio due to its simplicity, reliability and reasonable cost on performing in comparison with other tests. The vibration sources of this method are of low amplitude of motion such as wind, traffic and low intensity earthquakes, which have been reflected in many studies such as those by Trifunac [30], Kobayashi et al. [31,32], Lagomarsino [33], Hong and Hwang [34], Guler et al. [35], Gallipoli et al. [36], Michel et al. [37], and Pan et al. [38]. Due to the focus of the current study on the ambient vibration tests by using the vibration sources of low amplitude motion and on the RC MRF systems, the proposed formulas for vibration period from most of the recent studies are listed in Table 2 in a similar manner to the design codes and standards with the general form for vibration period as Eq. (1).

2.3. Numerical studies

Amanat and Hoque [5] used 3D FE modelling to numerically evaluate the fundamental vibration periods of regular RC framed buildings with infills. A number of parameters were taken into account in a sensitivity analysis including the number of storeys, the floor height, the number of spans, the stiffness of columns and beams, and the amount and distribution of infill panels. The infills were modelled as diagonal struts based on a previously proposed model [39]. They found that the fundamental period of vibration is significantly affected by column stiffness for bare frames. However, by including infill walls this effect is largely reduced and can be ignored due to the dominant influence of the lateral stiffness of the infill walls. The same trends were illustrated for other parameters. Their proposed formula for the fundamental vibration period was based on a Bangladesh National Building Code (BNBC) and took into account the most effective parameters including the span length, the number of spans, and the number of infill panels, which is expressed as:

$$T = \alpha_1 \alpha_2 \alpha_3 \, C_t \, H^{3/4} \tag{2}$$

where α_1 to α_3 are the modification factors for the span length, the number of spans, and the amount of infill walls, C_t is a factor given as 0.073 for RC MRF buildings, and H is the height of the building in metres.

Crowley and Pinho [12] performed 2D fibre-modelling FE analysis on eleven existing RC MRF buildings, ranging from 2 to 8 storeys, in five European countries to establish formulas for evaluating the fundamental period of vibration. Their study included some parameters, e.g. the gross and yield stiffness of the RC sections, the presence of infills, and the presence of openings. The simulated models were bare frames, frames with full infills but no openings, and frames with infills and openings. The simulated buildings were designed under gravity loads only, and the infill walls were modelled as diagonal struts. The results they obtained for different uncracked and cracked frames were weighted as averages based on the percentages of these frames in the Turkish buildings' stock. However, this simplification underestimates the periods of vibration for the bare frames and overestimates the periods of vibration for other frames in comparison with the numerically proposed formulas for each frame type. The proposed averaged formulas for buildings with uncracked and cracked infills are shown in Table 3.

Similarly, Kose [13] carried out a parametric study on RC framed buildings and took into account the effects of the height of the building, the number of bays, the ratio of shear walls, the ratio of infills with openings, and the types of frame, respectively, on the fundamental period of vibration. The simulation of the infills in the numerical analyses was in the form of a diagonal strut based on the FEMA-306 recommendation [40]. Three types of symmetrical building plans were modelled: bare frames, frames with infills and an open first floor, and frames with infills but no open first floor. The significant effects of the studied parameters on the fundamental period were ranked in the order of significance as the height of building, the ratio of shear walls, the ratio of infills, the number of bays, and the type of frame, respectively. The number of bays and the frame type were regarded as the least influential parameters. Using a multiple linear regression analysis on the obtained results by disregarding the effect of the less influential parameters, the following formula was proposed as:

$$T = 0.1367 + 0.0301\,H - 0.1663\,S - 0.0305\,I \tag{3}$$

where *H*, *S*, and *I* are the height of building, the area ratio of the shear walls to the total floors, and the area ratio of the infills to the total number of panels, respectively.

Masi and Vona [14] performed a parametric study on various classes of European RC buildings by utilising 3D FE analysis where the chosen parameters were the number of storeys of the buildings, the irregularity in their elevation, and the presence and configurations of infills. The simulated buildings were designed under gravity loads only. These models were classified as bare frames, exterior frames with regularly and irregularly arranged masonry infills, and exterior frames without masonry infills on the ground floor (Pilotis), or with partial infills on the ground floor. Uncracked and cracked RC elements were adopted to establish expressions for evaluating the vibration periods of the buildings. Hence, the infills were modelled as diagonal struts based on the model proposed by Mainstone [41], and applied to the exterior frames only. The respective adopted expressions for uncracked RC bare frame buildings with infill mass, uncracked RC frames with infills, and cracked RC frames with infills are illustrated in Table 3.

Ricci et al. [15] investigated the influence of infills on the lateral stiffness of moment-resisting frame systems in terms of the elastic period of vibration in a parametric study. The included parameters were the number of storeys, the plan dimensions, the bay lengths, the infill characteristics and the presence of the openings. The simulated buildings were low-rise to mid-rise in height with regular plans, and had been designed under gravity loads only. The numerical modelling of the infills was simulated using two distinct approaches taken from the literature in terms of uncracking and cracking infills. They stated that the correlation of the period with the height and dimension of buildings in the evaluated direction did not largely influence the prediction. However, the proposed formulas displayed high correlations with the height of the buildings, the shear modulus, and the area ratio of the infills to the floors. The proposed formulas with high correlations for frames with uncracked and cracked external infills in terms of the height of buildings (without separate consideration of the orthogonal directions) are indicated in Table 3 to compare with those proposed in the literature and in the current study.

3. Numerical modelling of infills

3.1. Infills without openings

The effects of infills have been simulated since the 1950s due to the complex behaviour of infill materials in connecting with the surrounding frame, particularly under lateral loading [1,42]. In the prior literature, many studies [1,2,43] have considered the behaviour of infilled panels at different stages. In particular, the behaviour of infill panels is divided into four categories: (i) the initial shear behaviour of the panel, (ii) the first cracking (equivalent strut formulation), (iii) the softening response corresponding to the maximum applied force, and

(iv) the final constant residual resistance. Thus, the initial stiffness of the infill, K, can be evaluated as follows [8]:

$$K = \frac{G_{\mathrm{m}}\, t\, l}{h} \tag{4}$$

where G_{m} is the tangential shear modulus of the masonry infill, and t, l and h are the thickness, length and height of the infilled panel, respectively. Recently, only Ricci et al. [15] considered the initial stiffness of infills in a numerical analysis based on the cited formula. They used the equivalent diagonal strut models with a secant modulus of elasticity to represent cracked infills, in line with the behaviour of RC buildings with infills under the effect of moderate to strong amplitude motions. However, for these RC buildings under the effect of low amplitude motions, e.g. wind, the modelling of infills should be simulated in the elastic range (initial stiffness) to decrease the huge scatters on the fundamental period of vibration in the experimental results.

In the prior literature, two main distinctive approaches have been taken to model cracked infills. The first is called micro-modelling, where each component of the infill panel is modelled separately. This approach can be quite complex and will require huge memory and extensive computing time, but it can also reproduce the local behaviour of a building with infills through an accurate simulation of infill panel details. However, the second approach, macro-modelling, has attracted many researchers since the 1960s due to its comparative simplicity and possible correlations with actual behaviour of infills in a way which requires less effort than the former approach. The latter approach can simulate the global behaviour of buildings with infills in relation to the stiffness and strength of the infill materials used [3,44]. This approach represents the infills as equivalent diagonal struts based on the real behaviour of the infills under lateral loads. The first recognised study considering this behaviour of infills and the surrounding frames was done by Polyakov [45]. This approach has then been developed in many studies such as those by Holmes [46], Stafford Smith [47], and Stafford Smith and Carter [48]. The most popular models in the literature were proposed by Mainstone and Weeks [49] and Mainstone [50], and have been adopted in many studies, codes and guidelines such as FEMA-273 [51] and FEMA-306 [40]. These models will be utilised in the present study for simulating the infill walls and their equivalent widths based on the mechanical and geometrical properties of the infills and the surrounding frame panel elements as follows [40]:

$$a = 0.175\left(\lambda_1 h_{\mathrm{col}}\right)^{-0.4} r_{\mathrm{inf}} \tag{5}$$

with

$$\lambda_1 = \sqrt[4]{\frac{E_{\mathrm{m}}\, t_{\mathrm{inf}} \sin 2\theta}{4 E_{\mathrm{f}}\, I_{\mathrm{col}}\, h_{\mathrm{inf}}}} \tag{6}$$

and

$$\theta = \tan^{-1}\left(\frac{h_{\mathrm{inf}}}{L_{\mathrm{inf}}}\right) \tag{7}$$

where

a is the equivalent width of the diagonal strut,

λ_1 is the relative stiffness parameter,

h_{col} is the height of the surrounding columns,

r_{inf} is the diagonal length of the infill panel,

E_{m} is the elastic modulus of the infill material,

t_{inf} is the thickness of the infill,

E_{f} is the elastic modulus of the frame element material,

I_{col} is the moment of inertia of the surrounding columns,

h_{inf} is the clear height of the infill panel,

L_{inf} is the length of the infill panel,

θ is the tangent angle of the equivalent diagonal strut with the horizontal direction.

Ultimately, there is a need in the prior studies considering a wide range of mechanical properties of infill materials and their effects on the global behaviour of RC MRF buildings. Therefore, the current study considers the variations in the shear and elastic moduli of infill materials, together with other parameters.

3.2. Infills with openings

The presence of openings in the infill panels is an avoidable pattern intended to provide access to enclosed spaces, to allow light and air into rooms, and/or to be for aesthetic reasons, especially on external façades. Thus, the presence of openings should be taken into account in evaluating the lateral stiffness of infills. In general, the openings will decrease the lateral stiffness of infills, and many studies have taken into account the presence of openings in buildings with infills through experimental tests or analytical simulations. In this study, in terms of the initial stiffness of the infill, only the cross-sectional area of the infill is reduced to take into account the effect of openings, as stated in some studies [1,15]. However, in terms of the equivalent width of diagonal struts, the stiffness reduction factor for the presence of openings proposed by Asteris [52,53] is used in the numerical analyses and multiplied by the equivalent width of the diagonal strut for the solid infills. The stiffness reduction factor is determined based on Eq. (8) as follows:

$$\lambda = 1 - 2\alpha_{\mathrm{w}}^{0.54} + \alpha_{\mathrm{w}}^{1.14} \tag{8}$$

where λ is the infill wall stiffness reduction factor, and α_{w} is the wall-opening ratio defined as the ratio of the area of opening to the area of the solid infill wall.

4. Simulations of the models

The simulated models in this study include bare frames, frames with full exterior infills, and frames with exterior infills and openings. The models were designed under gravity and wind loads according to relevant Eurocodes [16–20]. In addition to the ultimate limit state, the models were also verified against a serviceability limit state to limit the maximum lateral drift to 1/500 of the total height of the building under wind load [54]. Hence, the RC elements were assumed to be in a full linear elastic range without considering the cracking effect, as the wind load is classified as low amplitude motion and the dynamic behaviour of buildings like the fundamental periods of vibration is expected to be in the linear elastic range. Accordingly, the designed RC column sections varied with the height of the buildings due to the applied loads and the plan configurations of the buildings. The studied parameters were therefore the height of the buildings, the bay length, the number of spans and the mechanical properties of the infills. The number of storeys was taken as 5, 10, 15 and 20. The bay length was taken as 5 and 6 metres, while the number of spans ranged from 1 to 7 based on the building area and the length of span, so the plan aspect ratio (long direction dimension to short direction dimension) ranged between 1 and 7. As an example, the models were square and rectangular in plan due to the assumed number of spans as shown in Fig. 2. Several researchers, including Fardis and Panagiotakos [2], Guler et al. [35], Bal et al. [55], Skafida et al. [56], and Akhoundi et al. [57], utilised different values for the elastic modulus or shear modulus of infills. These variations resulted in different formulas for the fundamental periods of vibration due to the direct influence of these properties of the infill materials on the global mass and stiffness of buildings. Also, the stiffness of these infills would be varied due to the various adopted mechanical properties of infills. Thus, in this study, an assumed range for these properties was adopted to cover the popular values in the literature. The used elastic modulus ranged between 1.5 and 6.5 GPa. Moreover, based on the recommendation by Eurocode 6 [58], the shear modulus of infill material was taken as 0.4 times the elastic modulus, resulting in 0.6 to 2.6 GPa for the shear modulus. The density of brick masonry was assumed to be 2100 kg/m^3 [59]. Commercial software SAP2000 [60] was used for simulating the models, where the RC elements, such as the columns and beams, were modelled as two-node beam elements with six degrees of freedom for each node, while the slabs were assumed to behave as rigid diaphragms in plane with a thickness of 0.2 m. For the initial stiffness of the infills, the diagonal linear link elements were used to connect the two opposite corners of each panel with assigned initial stiffness based on Eq. (4). However, for the cracked infills, diagonal pin-jointed elements with released moments at both ends were used, with an assigned equivalent diagonal strut width based on Eq. (5), and the thickness of the element taken as the thickness of the infill wall at 0.2 m. To account for the presence of openings, the area of the uncracked infill walls was reduced based on Eq. (4), while for the cracked infills the stiffness reduction factor based on Eq. (8) was evaluated by multiplying the equivalent diagonal width of the solid infill walls. Hence, the ratio of the openings was assumed to be 20% [12,55]. Based on the conducted parameters, a total of 4400 models were simulated.

5. Analysis and discussion of the results

In this section, the results of the fundamental vibration periods obtained from the 3D FE eigen analyses are presented and discussed based on the studied parameters. Correspondingly, new vibration period formulas are proposed based on the single or multiple regressions performed on the obtained results. Hence, the regression analyses were performed on the basis of log-log scales for the considered parameters to obtain the empirical coefficients for each proposed formula. More details can be found in the study by Goel and Chopra [26]. The regression analyses on the obtained results were performed using the software Matlab [61]. Also, the results are categorised in terms of the types of models, i.e. bare frames and frames with infills, in relation to the elastic and shear moduli of the infills, and frames with infills and the presence of openings.

5.1. Modelling of bare frames

First, a sensitivity analysis was performed only on the obtained results from the square and rectangular bare frame models for the assumed plan aspect ratios (ARs), to investigate the effect of the number of spans and the bay length on the fundamental periods of vibration. The results for bare frames models with a plan aspect ratio of one are shown in Fig. 3 and the results for other plan aspect ratios are shown in Fig. 4. These models were designed under combined gravity and wind loads while assessing the effect of the infills as a dead load in relation to the external panels only. The fundamental period of vibration increases with the height of the buildings, indicating a direct relationship between them illustrated in the formulas proposed in the literature. This also indicates that the stiffness of models decreases with the height of the buildings due to the decrease in columns sections with the height of the buildings. Moreover, the reason why the second period of vibration for the square models is not stated is the similarity between the two periods due to the regular plan of the buildings and the uniform distributions of mass and stiffness for each model. Also, the scattered results for the same height are due to the different bay lengths adopted, which when increased also led to an increase in the period of vibration due to the decrease in the stiffness of the panels. The fitted power trend curves for the adopted span lengths of 5 m and 6 m are displayed in Fig. 3 with dashed red and dotted green lines, respectively. However, with the increase in the plan aspect ratio, the stiffnesses of the models become different in two principal horizontal directions, transverse and longitudinal, so the vibration period versus building height relationships follow slightly different trends, see the dotted red and dashed green power trend lines in Fig. 4. Based on the obtained results, a single regression was performed on the separated results in terms of plan aspect ratio to indicate the significant relationship between the fundamental period of vibration and the height of the building. For the square models, the regression analysis provided an averaged formula relating to the height of the building with a high correlation coefficient R^2 = 0.9575 and the corresponding root mean squared error, termed as *RMSE*, is 0.0802, as shown in the solid black fitted power trend line in Fig. 3. Meanwhile, a multiple regression analysis was performed on the results which took into account the dimension of the buildings in the evaluated direction as another parameter, but the correlation coefficient was not improved significantly, as Table 4 indicates. The latter evidence is consistent with previous studies [13,36–38]. However, this finding may be related to the symmetric plan of the buildings, and the dimension of buildings may largely affect rectangular models with different plan aspect ratios. For the bare frames, two formulas were used to predict the fundamental periods of vibration, i.e. for the first mode, with the parameters considered as follows:

$$T_{\mathrm{f}} = a\,H^{\mathrm{b}} \tag{9}$$

$$T_{\mathrm{f}} = a\,H^{\mathrm{b}}\,D^{\mathrm{c}} \tag{10}$$

where T_{f} is the fundamental period of vibration, a, b and c are empirical coefficients based on the regression analysis of the obtained results, and H is the building height, and D is the dimension of the buildings corresponding to the evaluated direction.

The same procedure was utilised for rectangular frame models to investigate the sensitivity of the vibration periods of these models to the adopted plan aspect ratios. The dotted red and dashed green fitted power trend lines for the transverse and longitudinal directions are shown in Fig. 4 and their empirical coefficients are illustrated in Table 5, together with the solid black fitted power trend line and their values for the combined directions. It is worthwhile to mention that the difference between the proposed formulas between the square and rectangular models is 5% on average. Thus, the evaluation of periods of vibration will be based on the combined square and rectangular models with different plan aspect ratios. The dimension of the buildings in the evaluated direction was disregarded here due to its low influence on the predicted fundamental periods of vibration. Therefore, a single regression analysis was performed on the complete results for the bare frames models with considering different plan aspect ratios and including the two principal horizontal directions. The fitted power trend lines are illustrated in Fig. 5 and the obtained empirical coefficients are listed in Table 6, with the power trend lines of the same colours and patterns as stated above.

5.2. Modelling of infilled frames without openings in terms of the shear modulus

In this subsection, the effect of shear modulus of infill walls on the vibration period is evaluated. The modelling of infills was simulated in terms of the shear modulus of elasticity based on Eq. (4). Fig. 6 represents the obtained fundamental periods of vibration against the height of the frame buildings with infills but no openings, with the fitted power trend lines in dotted red and dashed green for the orthogonal directions and in solid black for the combined results. Additionally, Figs. 7(a) and (b) illustrate the fundamental periods of vibration versus the shear modulus of the infills in the transverse and longitudinal directions for different building heights, respectively. In comparison with the results for the bare frames, Fig. 6 indicates the contributions of the lateral stiffness of the infills on the lateral stiffness of the bare frame alone. For example, the fundamental vibration periods decrease by 6% to 76% in comparison to the bare frames models. This is related to the increase in the shear modulus of the infills. Thus, the lateral stiffness of RC frames with infills can significantly alter the dynamic response of RC bare frames as the prior studies have also observed [5,12,14,35]. Hence, the infills in the present research are considered to act on the external façades only. Here, a single regression analysis was performed on the results for the frames with infills to provide a unique formula governing the buildings' height, as demonstrated in Table 7. However, to increase the accuracy of prediction, two multiple regressions were performed on the obtained data without including the results for the bare frames. The first included the building height and the dimension of the buildings in the evaluated direction, while the second included the shear modulus of the infills in addition to the building height and the dimension of the buildings in the considered direction. In the second multiple regression, an extra fitted power formula form Eq. (11) was utilised in combination with the previously suggested Eqs. (9) and (10). The bare frames models were disregarded to make the prediction more accurate in considering the shear modulus of the infills.

$$T_{\mathrm{f}} = \mathrm{a}\, H^{\mathrm{b}}\, D^{\mathrm{c}}\, G^{\mathrm{d}} \tag{11}$$

where G is the shear modulus of the infill material in GPa, and a, b, c and d are empirical coefficients.

The improved prediction illustrated in Table 7 indicates the significant effect of including the mechanical properties of the infill material on the evaluation of the fundamental vibration periods of buildings with infills in addition to the building height. Further, the inclusion of the dimensions of the building in the regression analysis improved the prediction accuracy in relation to the length of the infill walls and the corresponding lateral stiffness. However, in the longitudinal direction, the length of the buildings did not largely affect the vibration periods in line with the increase in the building length despite use of different spans. The vibration periods in the transverse direction are longer than those in the longitudinal direction, due to the dominant influence of rectangular plan models over those with square plans and corresponding to the lower lateral stiffness provided by the infills. Hence, for tall buildings with high plan aspect ratios, i.e. slender ones, the longitudinal vibration mode came after or was coupled with the torsional vibration mode due to the slenderness effect of the building height and plan and the distribution of stiffness. Further, these two vibration modes were too close, so the proposed formula for the longitudinal direction can also be applied to the torsional mode of vibration.

5.3. *Modelling of frames with infills and openings in terms of the shear modulus*

The effect of openings was simulated by considering the shear modulus of infills based on Eq. (4), and the percentage of openings was taken as 20% as applied in previous studies [12,55]. Obviously, the presence of openings in infills decreases the lateral stiffness contributed by the infills towards the overall lateral stiffness of the bare frames, leading to increased fundamental periods of vibration as indicated in Table 8 in comparison with the data in Table 7. This effect has been addressed in many studies in relation to the presence and position of openings in infill walls [52,53]. Assuming an approximate power form in the proposed formula with one variable, i.e. the building heights in Tables 7 and 8, the average increase in the fundamental period of vibration is 14.47%, which closely agrees with the results presented by Ricci et al. [15]. Similar to the previous subsection, by utilising single and multiple regressions on the obtained results, the proposed formulas were in the same forms as those utilised in the previous sub-sections, and the corresponding empirical coefficients are specified in Table 8. Here, the openings were assumed to be in the middle of the infill walls.

5.4. *Modelling of frames with infills but no openings in terms of the elastic modulus*

Similarly, the infills were simulated as equivalent diagonal struts based on Eq. (5) by taking into account the properties of the infills and the surrounding frame elements. Fig. 8 illustrates the fundamental periods of vibration as the function of the building height for the frames with infills but no openings, represented by the dotted red, dashed green and solid black power trend lines for the orthogonal directions and the combined results, respectively. Figs. 9(a) and (b) represent the relationships between the fundamental vibration period and the elastic modulus of infills in the transverse and longitudinal directions for different building heights. The proposed formulas were in the same forms as those including the elastic modulus of the infills instead of the shear modulus, and the corresponding empirical coefficients are indicated in Table 9. In terms of the heights of the buildings, a very good prediction was obtained from the regression analysis. However, by including the elastic modulus of infills and the dimension of buildings in the evaluated direction, the prediction could be further improved. This indicates that the mechanical properties of infills play an effective role in enhancing the lateral stiffness of frame buildings with infills in comparison with bare frames buildings. Moreover, the differences in the fundamental periods of vibration between bare frames and frame buildings with infills ranged from 1% to 50% in relation to adopting the elastic modulus and number of the infills in the considered direction. For example, the effect of the infills decreases with an increase in the plan aspect ratio in the transverse direction. Also, the effect of infills can be disregarded in that direction for slender models. However, in the longitudinal direction, the lateral stiffness from infills increases due to the longer and stiffer infills in this direction. These values reasonably agree with other numerical studies [5,12,14].

5.5. Modelling of frames with infills and openings in terms of the elastic modulus

The frames with infills and openings were modelled in terms of the elastic modulus by taking into account 20% openings based on Eq. (8), and then multiplied by the solid infill equivalent width in Eq. (5). In addition to the decrease in the infills' lateral stiffness due to cracking, the presence of openings in the infills further decreases the infills' lateral stiffness towards the lateral stiffness of bare frames. This leads to an increase by up to 34% in the fundamental periods of vibration for various building heights as indicated in Table 10 and compared with Table 9. It is worthwhile to mention that the effect of infills on low-rise and mid-rise buildings is more pronounced than that on high-rise buildings in relation to the shear behaviour of the infills. However, the lateral stiffness of the frames is more dominant in higher storeys in relation to the flexural behaviour of frame structures. The proposed formulas have the same forms as those utilised in previous sub-sections, and the corresponding values of the empirical coefficients are listed in Table 10.

6. Comparison between the proposed formulas and those in the prior literature

In this section, the proposed formulas for evaluating the vibration periods of RC MRF buildings are compared with those cited in the prior studies. Different models were simulated and compared on RC bare frames only, frames with uncracked infills, and frames with cracked infills, taking into account the openings in the infilled models. As most of the cited formulas are mainly dependent on the building height, here only the proposed formulas in terms of the height of buildings are used for comparisons with those cited in the literature. Further, the proposed formula for bare frame models in combined directions (Table 6), $T = 0.1304\ H^{0.6826}$, is compared with the recommended empirical equations [20,25,27] and the numerically cited equations [12,14] for uncracked RC buildings as shown in Fig. 10. It can be seen that the proposed formula for the bare frame models is in a reasonable agreement with the recommended empirical expressions in ASCE7-10 [25] (Table 1), particularly for low-rise to mid-rise buildings, and that the degree of agreement decreases with the increase in the height of the buildings. However, the proposed formula has not largely agreed with the recommended expression in BS EN 1991-1-4 [20] and AS/NZS 1170.2 [27] (Table 1). The proposed numerical formula has reasonably agreed with the expression proposed by Crowley and Pinho [12] (Table 3) for uncracked low-rise to mid-rise RC buildings with various plan configurations and heights of the buildings. It should also be mentioned that their equation was based on the averaged results for three different models: bare frames, frames with full infills, and frames with infills and openings. Thus, if only bare frames models are considered in their situations there will be a huge scatter in comparison with the proposed formula. On the other hand, the proposed formula has not largely agreed with the formula proposed by Masi and Vona [14] (Table 3) for uncracked RC bare frames. In addition, the scattered results in their numerical studies come from the fact that the current models were designed for gravity and lateral loads, under which the lateral stiffness will be higher than those buildings designed under gravity loads only.

For the frames with infills with or without openings, the current proposed formulas are compared with the empirically based expressions from prior studies [20,25,27,34–36] and the numerically based expressions from others [12,14,15]. The reason for including the expressions from the design standards and codes was to examine the difference between the bare frames and the frames with infills. Figs. 11(a) and (b) present the comparison between the proposed formulas for the frames with uncracked infills with and without openings in relation to the shear modulus of infills, and the cited empirical expressions. For the RC buildings with uncracked infills and without openings, the proposed vibration period formula, $T = 0.0159\ H^{1.0320}$, is largely different from the formulas in ASCE 7-10 [25] (see Table 1) which significantly overestimated the vibration periods of RC buildings with infills, as these formulas have a reasonable agreement with bare frames models. However, it is interesting to note that the proposed formula has shown relatively smaller differences from the recommended expression [20,27] in Table 1 than the previously mentioned code expressions [25]. Further, the proposed formula has agreed well with the formula proposed by Gluer et al. [35] (Table 2), with an average difference of 3.23%. In fact, this is related to the mixed methods involved in the experimental and numerical analyses used to obtain their proposed formula. Similarly, the proposed formula shows a consistent agreement with other cited expressions [34,36] (see Table 2), in particular for low-rise to mid-rise RC buildings. This indicates that the effect of modelling the infills based on the initial stiffness, as has been stated [15], can be comparable to those periods obtained from the ambient vibration tests of low amplitude motions. For the RC frames with uncracked infills and openings, similar trends are observed in Fig. 11(b) but with the increased vibration periods due to the decrease in the lateral stiffness of the RC buildings in the presence of openings.

Figs. 12(a) and (b) show the differences between the proposed formulas for RC buildings with cracked infills which either have no openings or have openings, $T = 0.0503\ H^{0.8369}$ and $T = 0.0776\ H^{0.7582}$, and those proposed in the literature. It is clear that the cracked infills largely reduce the contribution to the lateral stiffness of the whole frame systems due to the use of the secant elastic modulus for infills instead of the shear modulus where the buildings with infills sustain purely linear shear under lateral loading. However, the flexural behaviour of the frames will govern their global behaviour in the nonlinear stage and in the cracked infills. Thus, the huge scatters between the proposed numerical formulas and the cited empirical formulas for low amplitude motion [34–36] are related to the assumed cracked infills. Despite that assumption, large differences are still evident in comparison with the formulas recommended in ASCE7-10 [25]. In contrast, the proposed formula, $T = 0.0503\ H^{0.8369}$, shows less differences from the corresponding formula in the design code and standard [20,27], as shown in Fig. 12(a). In addition, the same trend is indicated in Fig. 12(b) by taking into account openings in infills, where the vibration periods will be increased further due to the reduced lateral stiffness caused by the openings in infills with reduced equivalent widths of diagonal struts for the infills. This results in reasonable agreement between the proposed formula, $T = 0.0776\ H^{0.7582}$, and the cited formula from analytical analyses for wind-tunnel testing [25], in particular, for low-rise to mid-rise RC buildings.

Figs. 13(a) and (b) show a comparison between the proposed formulas for RC buildings with uncracked infills with and without openings and the cited numerical formulas from the literature [12,14,15]. The proposed expressions, $T = 0.0159\ H^{1.0320}$ and $T = 0.0182\ H^{1.0067}$, are largely different from the cited ones for uncracked RC buildings [12,14] due to the assumed cracked infills despite the uncracked RC elements assumed in these studies. However, for low-rise to mid-rise buildings, in particular, the proposed formulas for RC buildings with uncracked infills have a good agreement with the cited ones [15] in Table 3, as shown in Figs.

13(a) and (b), due to the adoption of the shear modulus in the current study. Further, the degree of agreement decreases with the increased height of buildings. Hence, the buildings studied by Ricci et al. [15] were designed under gravity loads only. The lateral stiffness of frame buildings with infills can be evaluated based solely on the contribution of the lateral stiffness of the infills due to the dominant effect of the infills on the elements of RC frames.

Similarly, Figs. 14(a) and (b) demonstrate the comparison between the proposed formulas for the RC buildings of cracked infills with and without openings and the cited numerically based formulas. Despite using the popular formula for modelling the cracked infills, there are still huge differences in comparison of the proposed expression, $T = 0.0503\ H^{0.8369}$, with those cited formulas [12,14]. However, the proposed expression agrees well with the previously cited formula for RC buildings due to the assumption adopted in relation to the uncracked RC elements [12], in particular for low-rise RC buildings. Further, in particular for low-rise RC buildings, the inclusion of openings increases the fundamental vibration periods, largely reducing the differences between the proposed formula, $T = 0.0776\ H^{0.7582}$, and the equation for uncracked RC buildings [12], as shown in Fig. 14(b). In addition, the currently proposed expression, $T = 0.0503\ H^{0.8369}$, for RC buildings with cracked infills and without openings displays a consistent agreement with the corresponding cited formula [15] for the low-rise to mid-rise RC buildings, as shown in Fig. 14(a). The same trend is observed for RC buildings with cracked infills and openings in comparison with the cited formula [15]. However, the proposed expression, $T = 0.0776\ H^{0.7582}$, is largely different from the cited formula for buildings with cracked infills and openings [15], particularly in the case of high-rise buildings designed under gravity and lateral loads. In the previous study, the buildings were designed under gravity loads only and a range of low-rise to mid-rise RC MRF buildings were studied.

7. Conclusions

In this study, the elastic vibration periods of RC buildings designed under gravity and wind loads were numerically evaluated by utilising FE modelling. A number of influencing parameters were investigated, including the height of the building, the length and number of spans, the mechanical properties of the infills, and the presences of openings in the infills. Based on these numerical analyses and comparisons with the proposed formulas obtained experimentally and numerically, and cited in the literature, the following conclusions can be drawn:

- For bare frames buildings, the obtained numerical results reasonably agree with some formulas cited in the design standards and codes, thus indicating the possibility of applying these formulas in evaluating the vibration periods of bare frames only.
- For RC frames buildings with infills, the lateral stiffness provided by the infills as non-structural elements can significantly affect the lateral stiffness of bare frames. Also, the lateral stiffness of these buildings can be represented by the lateral stiffness of the infills alone due to the disregarded effect on the RC frame elements' stiffness.
- The most influential parameters on the elastic periods of vibration are the height of the buildings and the mechanical properties of the infills, while the number and length of spans are less influential.
- The percentage of openings is another factor which should be taken into account in relation to its direct influence on the lateral stiffness of the infills, and then on the elastic vibration periods of the buildings.
- The proposed formula by adding the dimension of the buildings in the considered direction for the bare frames does not significantly improve the prediction of elastic periods of vibration.

- The proposed formulas considering the height and dimension of buildings and the mechanical properties for infills of RC frame models have high correlations with other formulas cited in the literature due to the inclusion of the geometrical and mechanical properties of infills in the proposed formulas.
- The proposed formulas for vibration periods can be applied for low-rise to high-rise regular bare MRF buildings and buildings with infills for plan aspect ratios between 1:1 and 1:7.
- The RC MRF buildings designed under gravity and wind loads possess higher stiffness than those designed under gravity loads only, hence reducing the vibration periods.

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List of Tables:

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

Codes/Standards	Empirical coefficients		Remarks
	α	β	
ASCE7-10 [25]	0.0670	0.9	Adopted from [26]
	0.0437	1	From comprehensive analyses on wind-tunnel testing
EN1991-1-4 [20]/ AS/NZS 1170.2 [27]	0.0217	1	Also in [28]

Table 2 Empirical coefficients in the formulas for vibration period from recent experimental studies on infilled RC MRF buildings

References	Empirical coefficients		Remarks
	α	β	
Hong and Hwang [34]	0.0294	0.804	Period for combined directions
Guler et al. [35]	0.026	0.9	Period for combined directions
Gallipoli et al. [36]	0.016	1	Period for combined directions
Michel et al. [37]	0.013	1	Period for combined directions
Pan et al. [38]	0.0244	0.8840	For firm soil and combined direction

Table 3 Empirical coefficients in the formulas for vibration period from recent numerical studies on RC MRF buildings

References	State of RC elements/infills	Empirical coefficients		Remarks
		α	β	
Crowley and Pinho [12]	uncracked	0.038	1	Bare + infills w/o openings
	cracked	0.055	1	
Masi and Vona [14]	uncracked	0.086	1	Bare frames
	uncracked	0.050	1	Fully infilled
	cracked	0.055	1	
Ricci et al. [15]	uncracked	0.022	0.85	Infills without openings
	uncracked	0.025	0.85	Infills with openings
	cracked	0.031	1	Infills without openings
	cracked	0.041	1	Infills with openings

Table 4 Empirical coefficients in the proposed formulas for vibration period of square bare RC MRF buildings

Empirical coefficients			R^2	*RMSE*	Remarks
a	***b***	***c***			
0.1193	0.7199		0.9575	0.0802	Fundamental periods for combined directions
0.0775	0.7199	0.1274	0.9632	0.0772	

Table 5 Empirical coefficients in the proposed formulas for vibration period of rectangular bare RC MRF buildings

Empirical coefficients			R^2	*RMSE*	Remarks
a	***b***	***c***			
0.1253	0.7065		0.9567	0.0788	Transverse period
0.1095	0.7065	0.0517	0.9632	0.0729	
0.1411	0.6421		0.8224	0.1564	Longitudinal period
0.0778	0.6421	0.1731	0.8330	0.1523	
0.1330	0.6743		0.8710	0.1356	Period for combined directions

Table 6 Empirical coefficients in the proposed formulas for vibration period of square and rectangular bare RC MRF buildings

Empirical coefficients		R^2	*RMSE*	Remarks
a	***b***			
0.1242	0.7089	0.9567	0.0789	Transverse period
0.1368	0.6563	0.8370	0.1517	Longitudinal period
0.1304	0.6826	0.8842	0.1290	Period for combined directions

Table 7 Empirical coefficients in the proposed formulas for vibration period of RC MRF buildings with uncracked infills and without openings

Empirical coefficients				R^2	*RMSE*	Remarks
a	***b***	***c***	***d***			
0.0182	1.0440			0.8817	0.1993	Transverse period
0.0339	1.0440	-0.1910	-0.2693	0.9766	0.0887	
0.0138	1.0200			0.8057	0.2610	Longitudinal period
0.0128	1.0200	0.0579	-0.3388	0.8870	0.1992	
0.0159	1.0320			0.7705	0.2934	Period for combined directions
0.0389	1.0320	-0.2900		0.8446	0.2414	
0.0434	1.0320	-0.2900	-0.3040	0.9054	0.1884	

Table 8 Empirical coefficients in the proposed formulas for vibration period of RC MRF buildings with uncracked infills and openings

Empirical coefficients				R^2	*RMSE*	Remarks
a	***b***	***c***	***d***			
0.0209	1.0162			0.8928	0.1835	Transverse period
0.0358	1.0162	-0.1631	-0.2548	0.9762	0.0865	
0.0158	0.9972			0.7996	0.2602	Longitudinal period
0.0132	0.9972	0.0874	-0.3281	0.8794	0.2020	
0.0182	1.0067			0.7740	0.2833	Period for combined directions
0.0411	1.0067	-0.2644		0.8390	0.2392	
0.0456	1.0067	-0.2644	-0.2914	0.8980	0.1905	

Table 9 Empirical coefficients in the proposed formulas for vibration period of RC MRF buildings with cracked infills and without openings

Empirical coefficients				R^2	*RMSE*	Remarks
a	***b***	***c***	***d***			
0.0553	0.8417			0.9451	0.1058	Transverse period
0.0768	0.8417	-0.0495	-0.1516	0.9775	0.0677	
0.0457	0.8321			0.8130	0.2080	Longitudinal period
0.0335	0.8321	0.1741	-0.2240	0.8730	0.1716	
0.0503	0.8369			0.8270	0.1994	Period for combined directions
0.0752	0.8369	-0.1305		0.8515	0.1847	
0.0956	0.8369	-0.1305	-0.1878	0.8894	0.1595	

Table 10 Empirical coefficients in the proposed formulas for vibration period of RC MRF buildings with cracked infills and openings

Empirical coefficients				R^2	*RMSE*	Remarks
a	***b***	***c***	***d***			
0.0803	0.7696			0.9721	0.0680	Transverse period
0.0879	0.7696	-0.0010	-0.0683	0.9790	0.0590	
0.0749	0.7467			0.8505	0.1632	Longitudinal period
0.0485	0.7467	0.1694	-0.1133	0.8762	0.1486	
0.0776	0.7582			0.8800	0.1458	Period for combined directions
0.0948	0.7582	-0.0649		0.8878	0.1410	
0.1065	0.7582	-0.0649	-0.0908	0.8993	0.1336	

List of Figures:

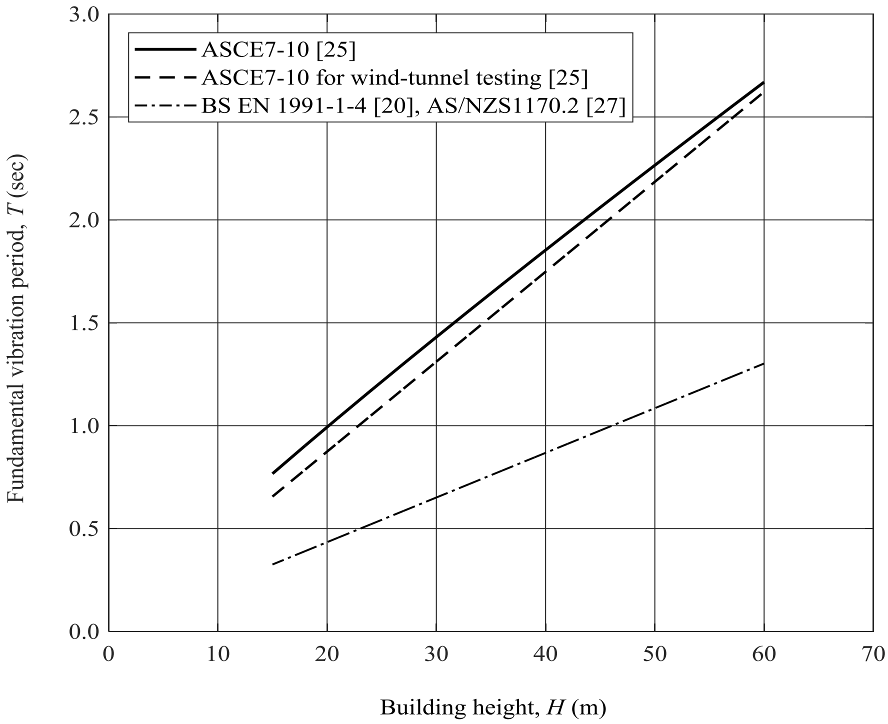
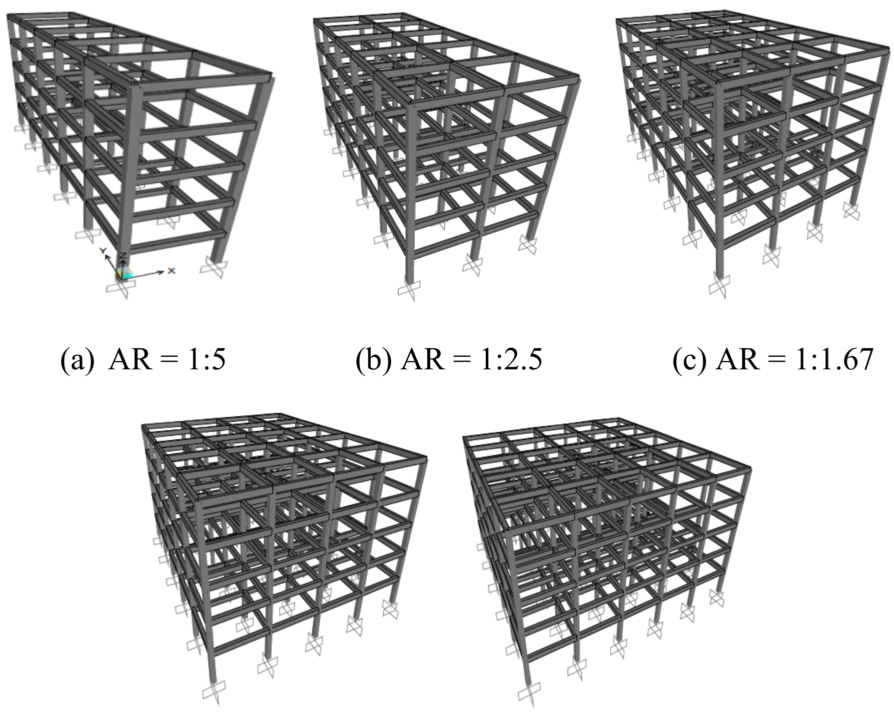


Fig. 1 Fundamental vibration period versus building height in the design codes and standards



(a) AR = 1:5 (b) AR = 1:2.5 (c) AR = 1:1.67

(d) AR = 1:1.25 (e) AR = 1:1

Fig. 2 Three-dimensional views of five-storey RC building models with various plan aspect ratios

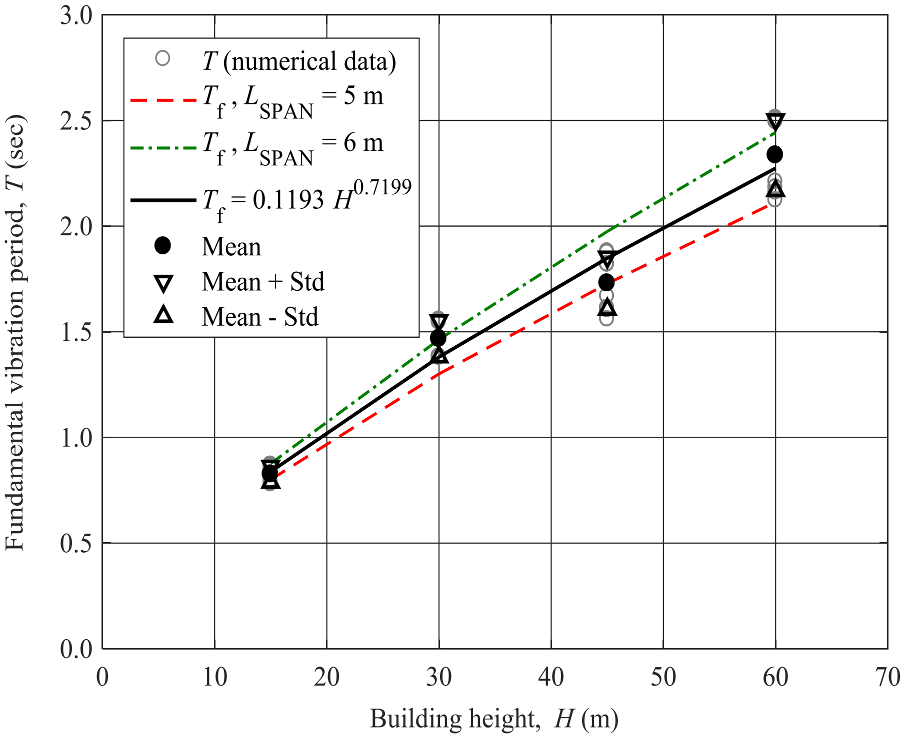


Fig. 3 Fundamental vibration period versus building height for bare RC MRF buildings of L = 5 m and 6 m with AR = 1:1

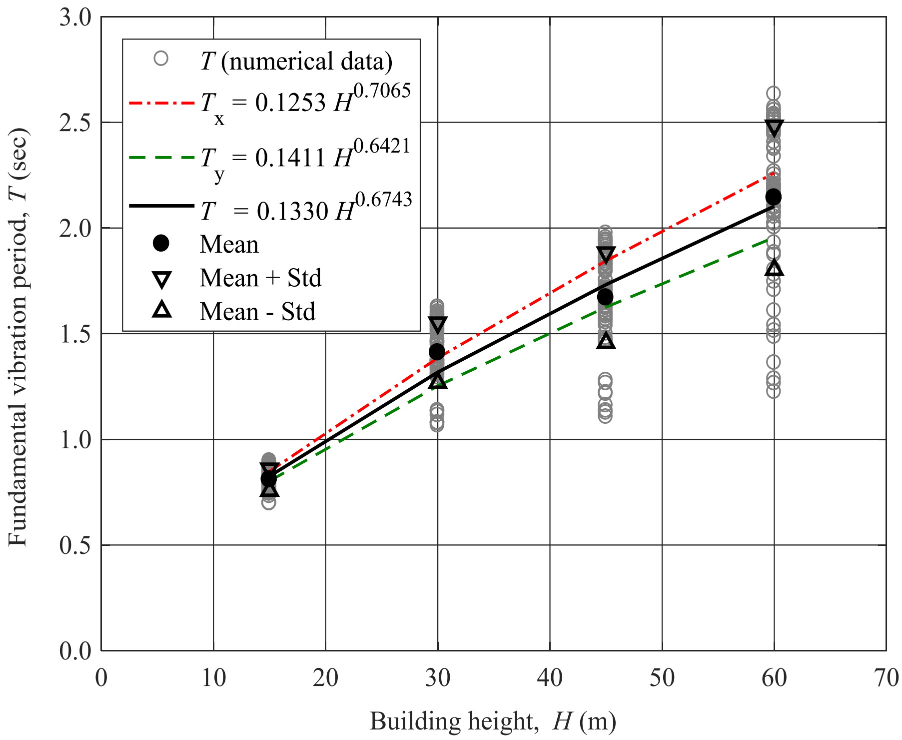


Fig. 4 Fundamental vibration period versus building height for bare RC MRF buildings with AR = 1:1.16 to 1:7

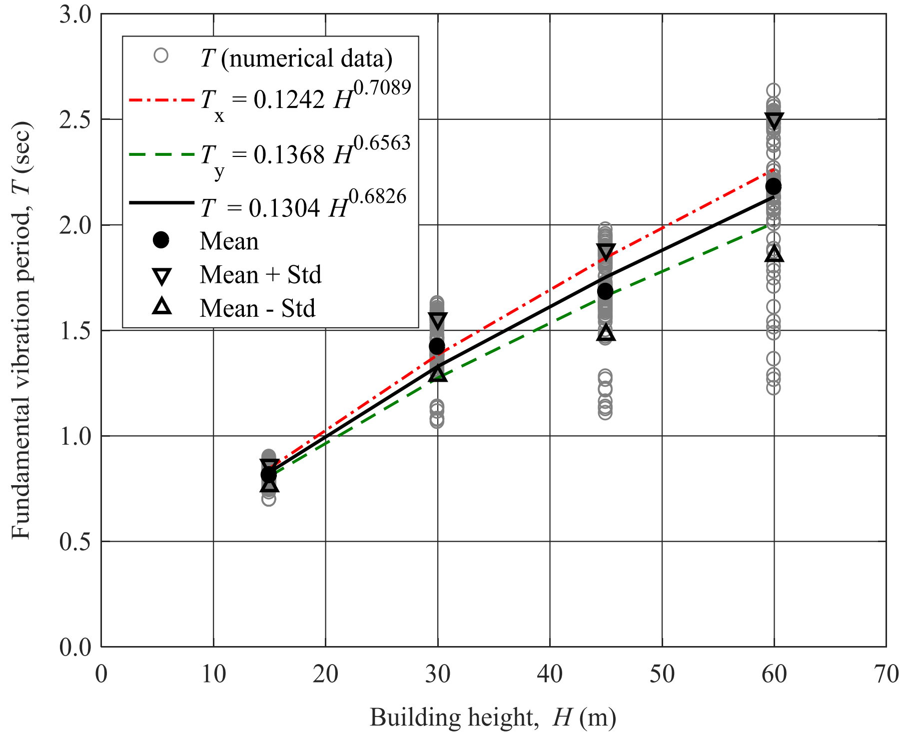


Fig. 5 Fundamental vibration period versus building height for bare RC MRF buildings with AR = 1:1 to 1:7

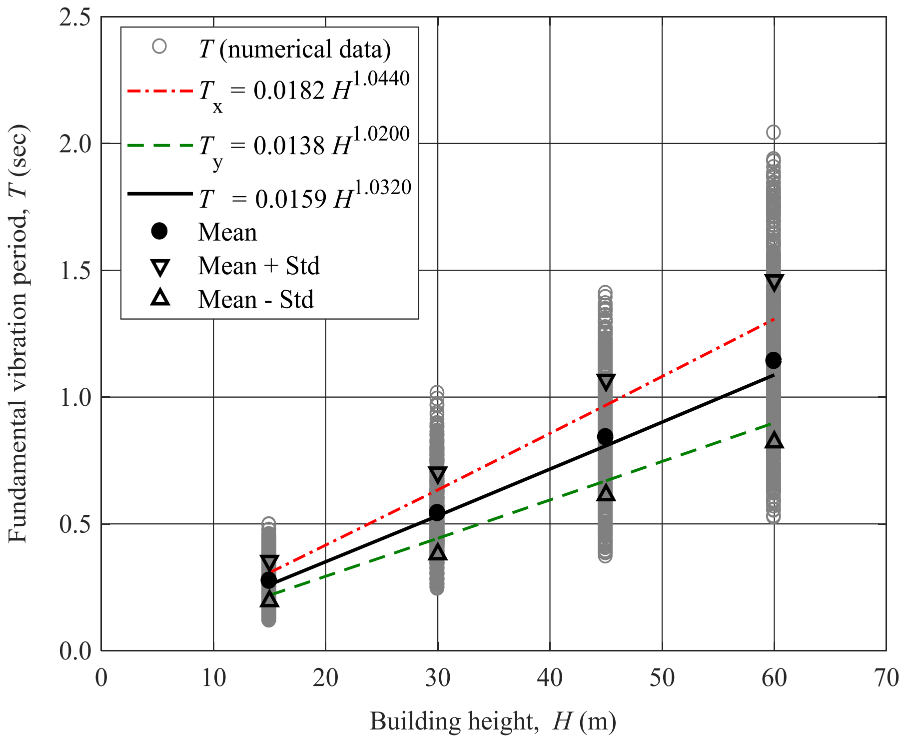
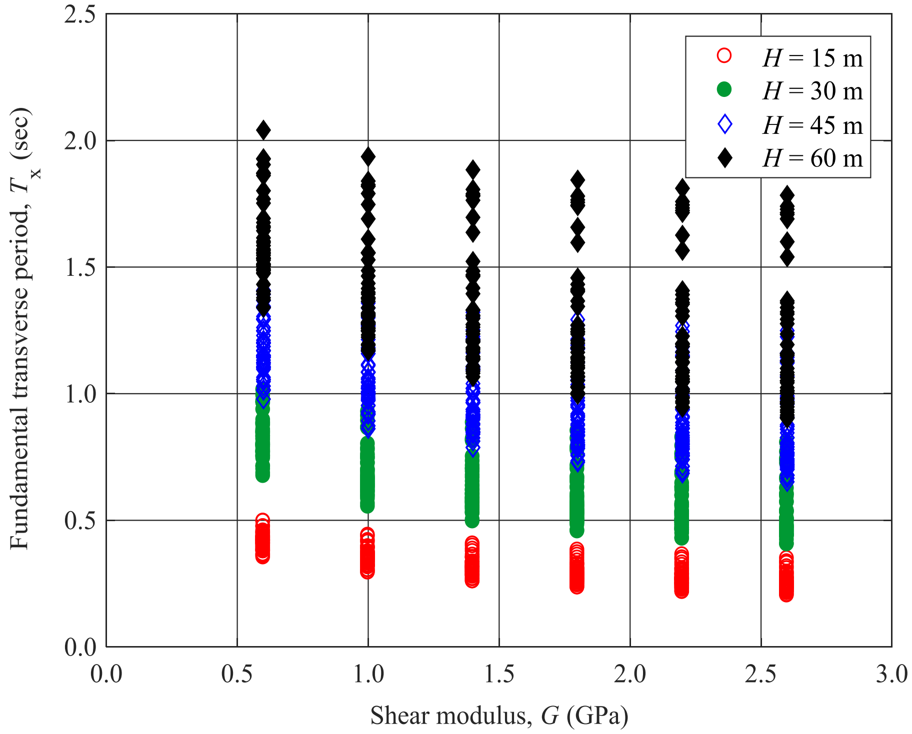
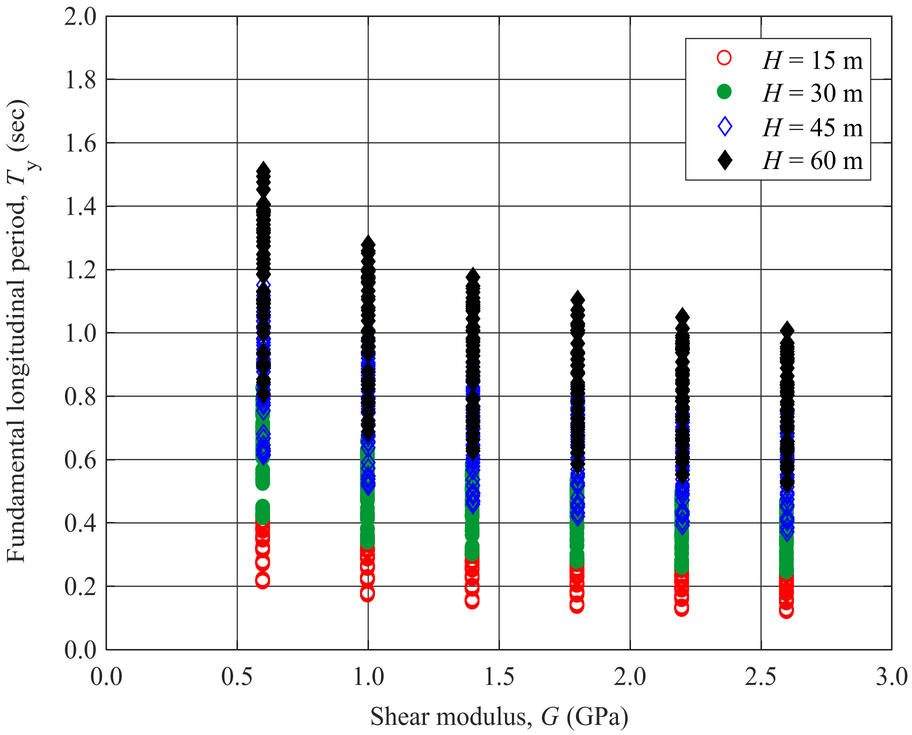


Fig. 6 Fundamental vibration period versus building height for RC MRF buildings with uncracked infills and without openings



(a) T_x versus G



(b) T_y versus G

Fig. 7 Fundamental vibration period versus shear modulus of infills for RC MRF buildings of different heights with uncracked infills and without openings in x and y directions

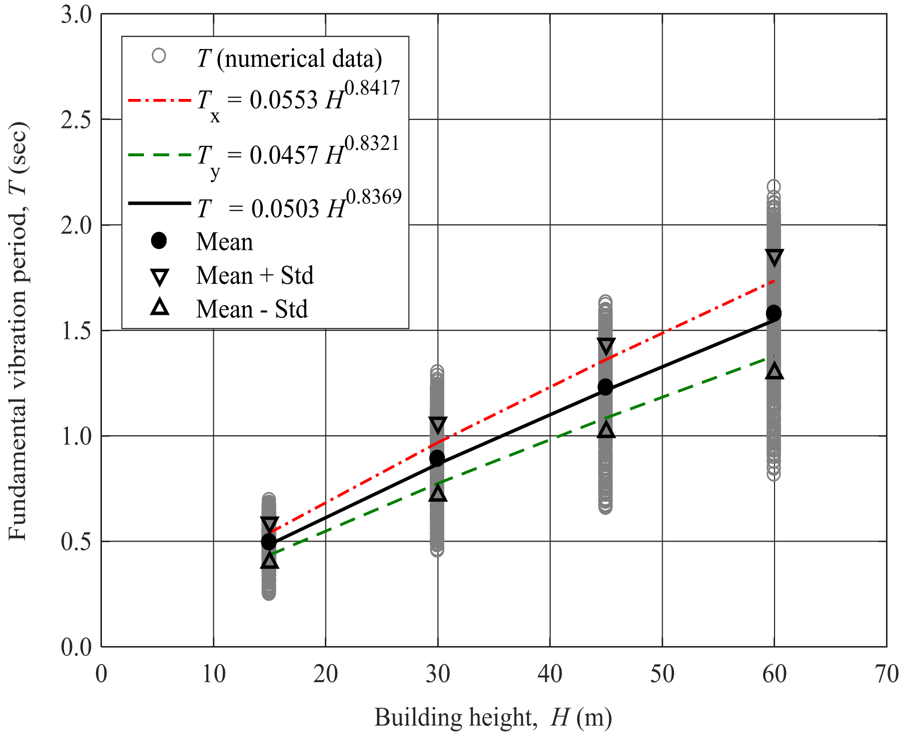
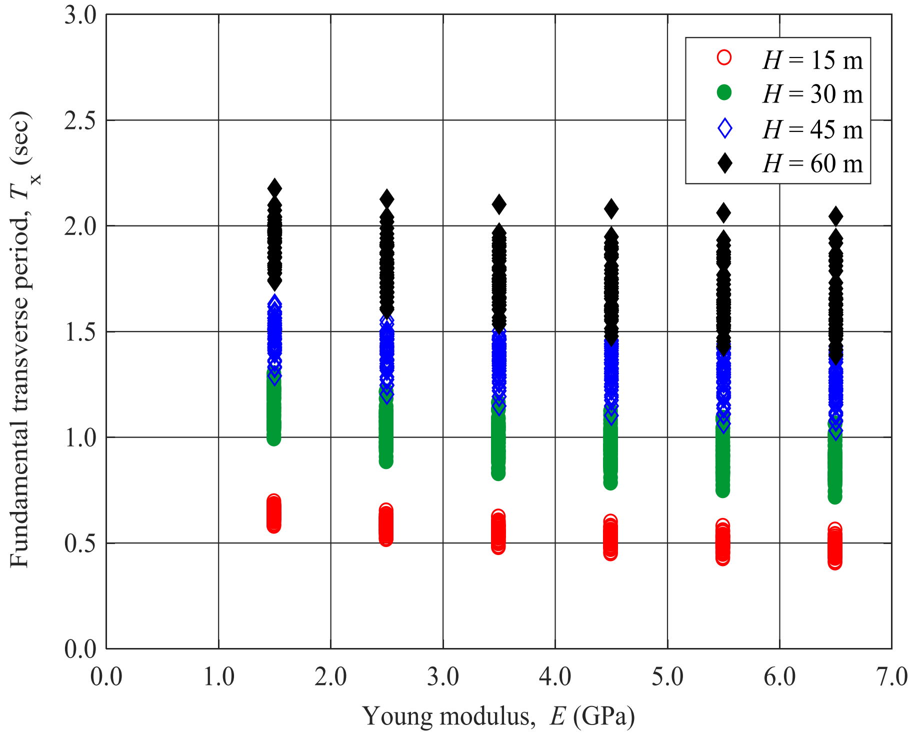
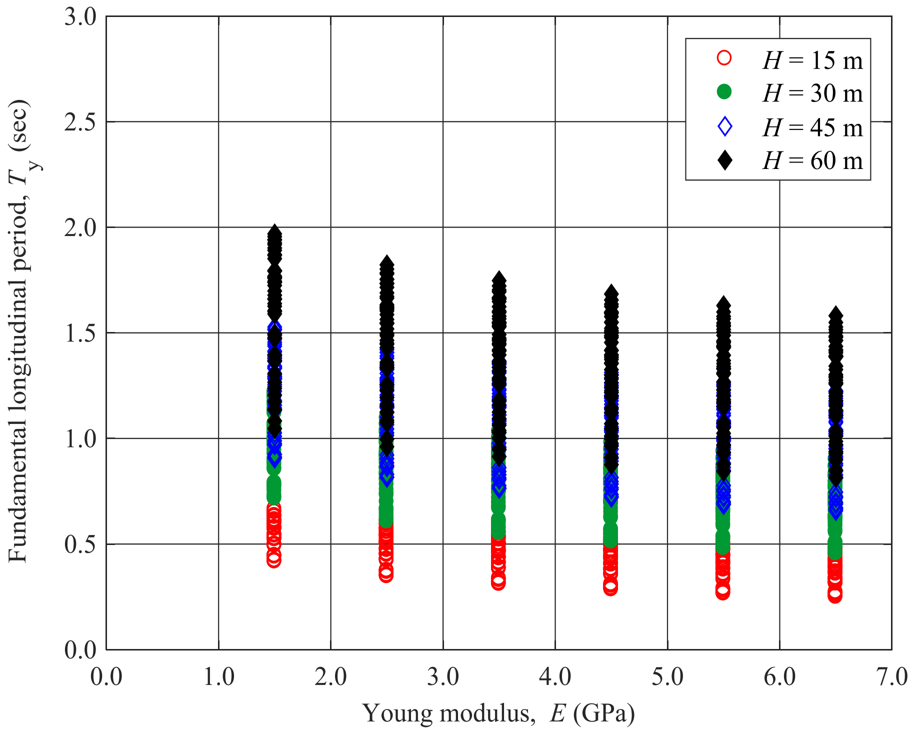


Fig. 8 Fundamental vibration period versus building height for RC MRF buildings with cracked infills and without openings



(a) T_x versus E



(b) T_y versus E

Fig. 9 Fundamental vibration period versus Young modulus of infills for RC MRF buildings of different heights with cracked infills and without openings in x and y directions

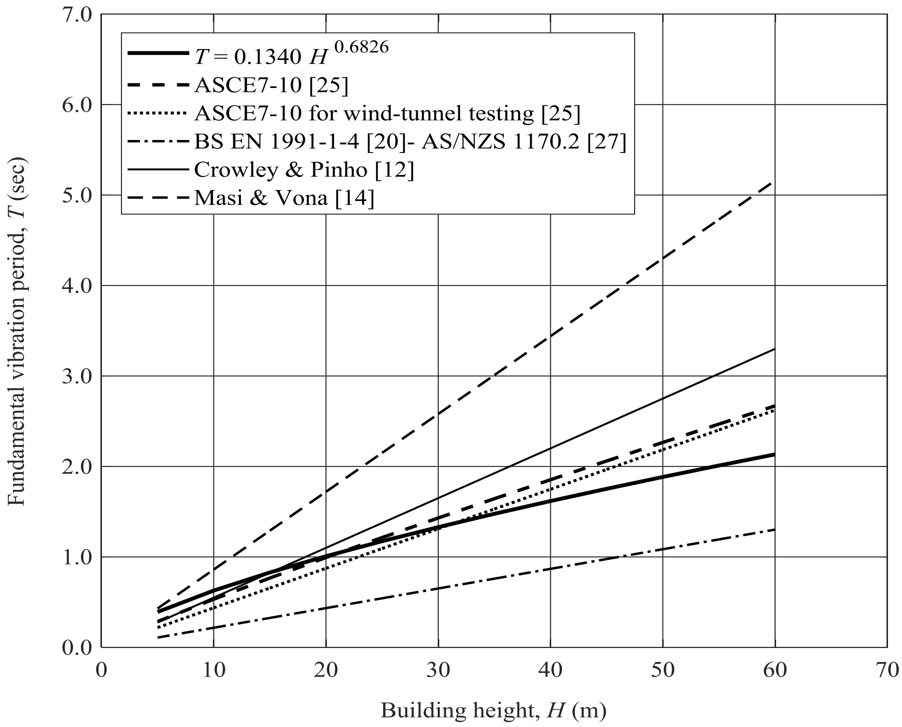
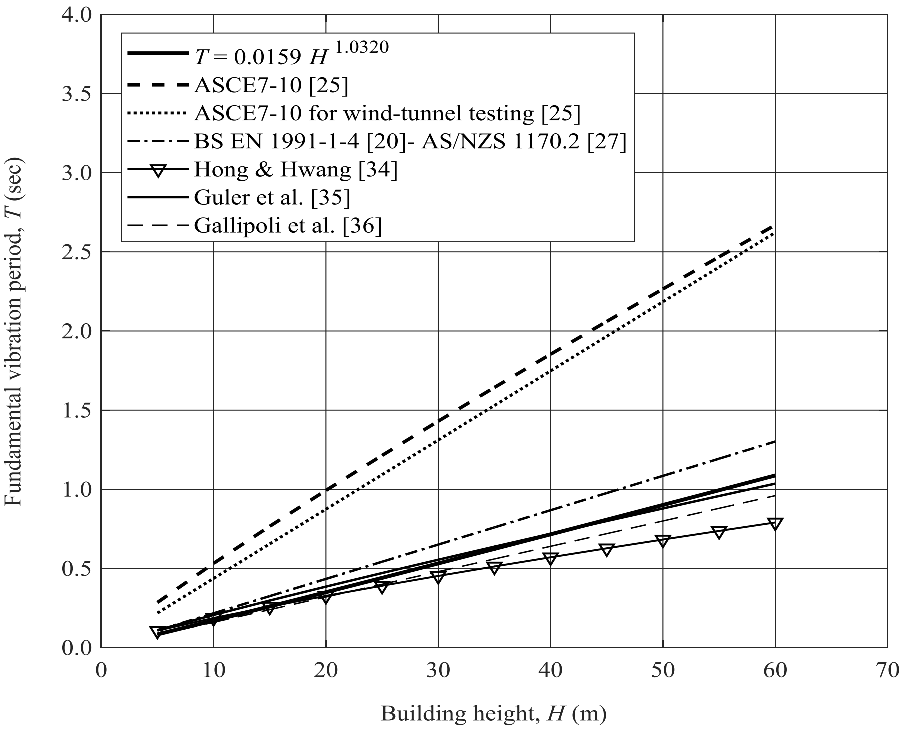
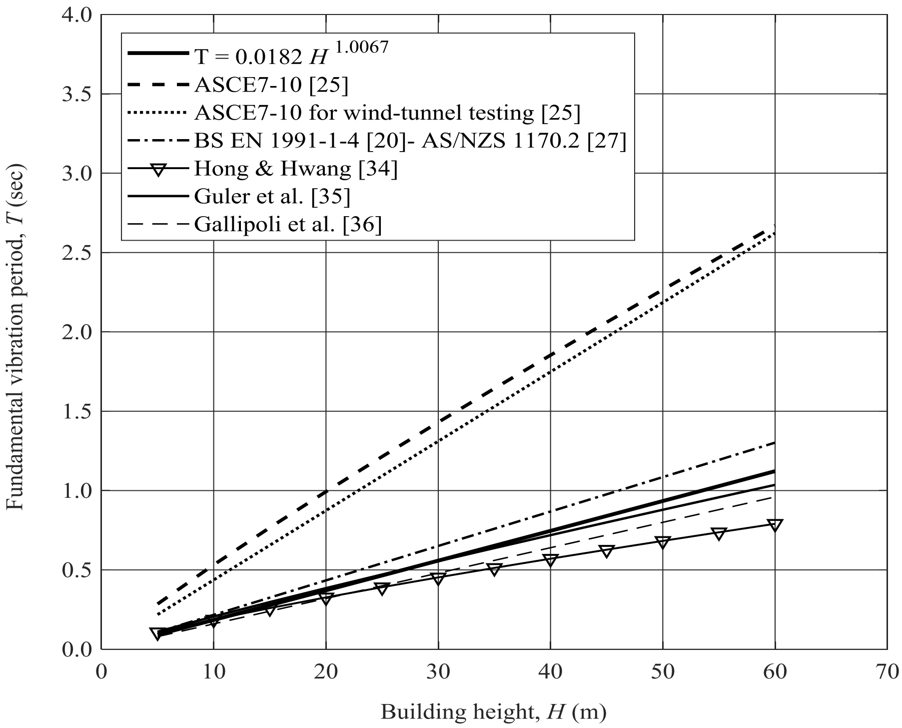


Fig. 10 Comparison of the proposed formula for fundamental vibration period with those proposed in the literature for bare RC MRF buildings of different heights

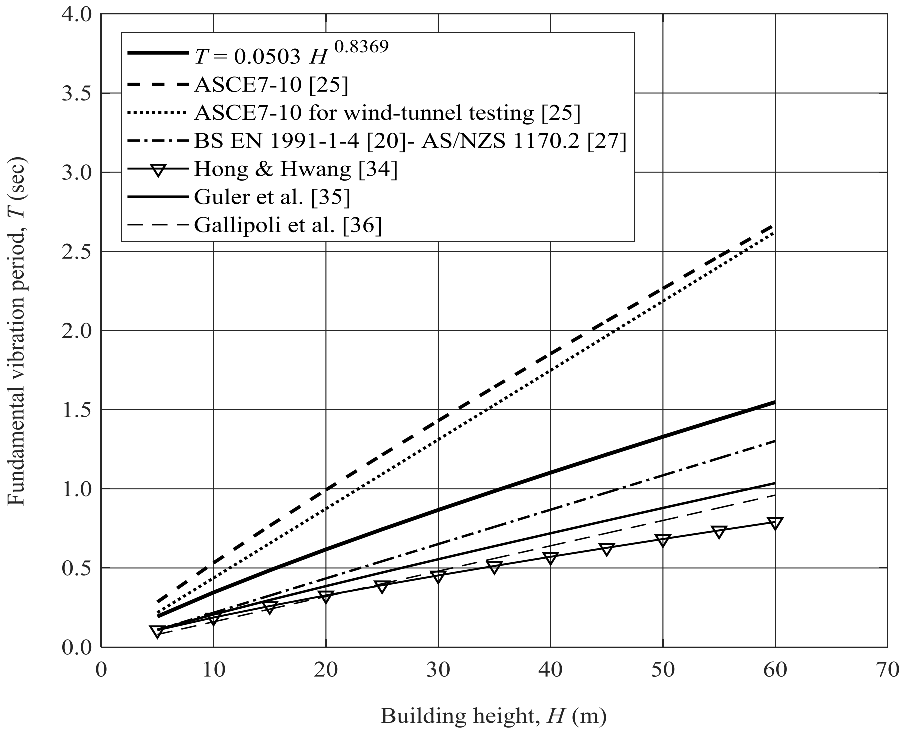


(a) For infills without openings

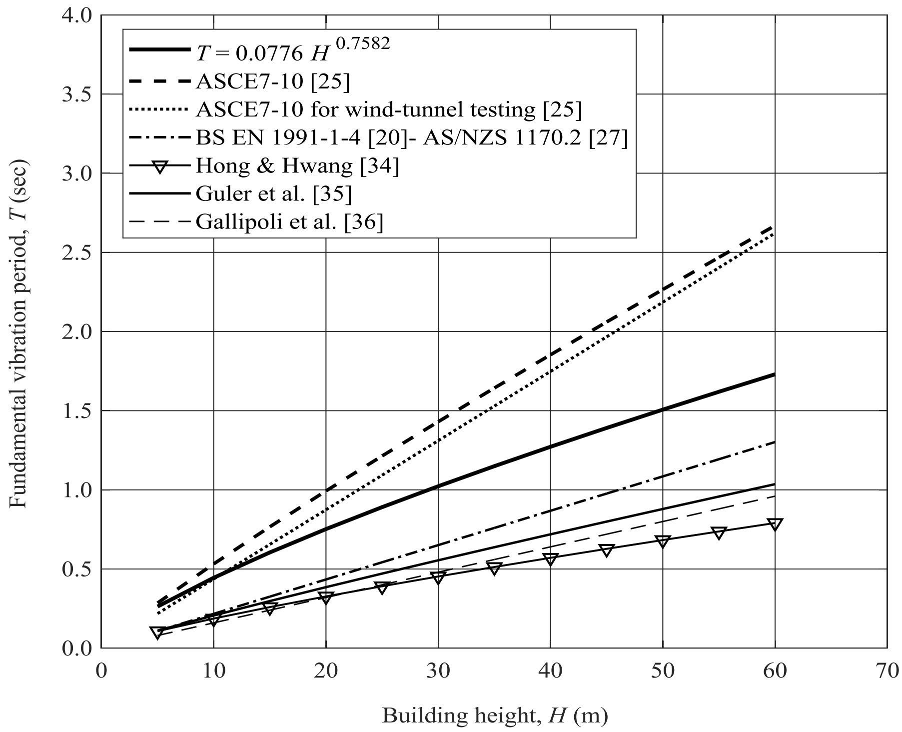


(b) For infills with openings

Fig. 11 Comparison of the proposed formulas for fundamental vibration period with those in prior studies for RC MRF buildings of different heights with uncracked infills

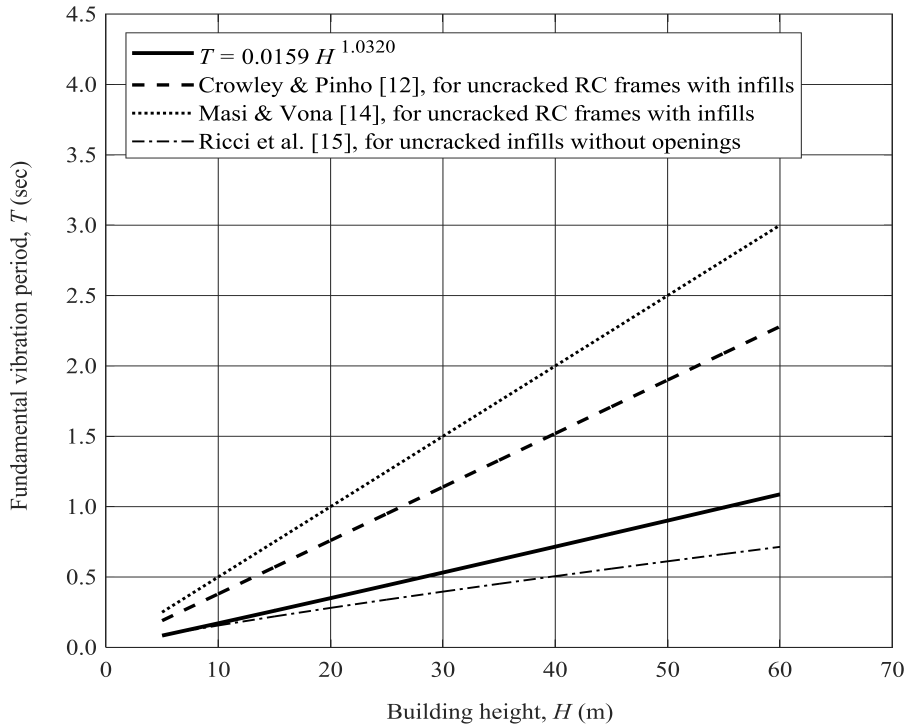


(a) For infills without openings

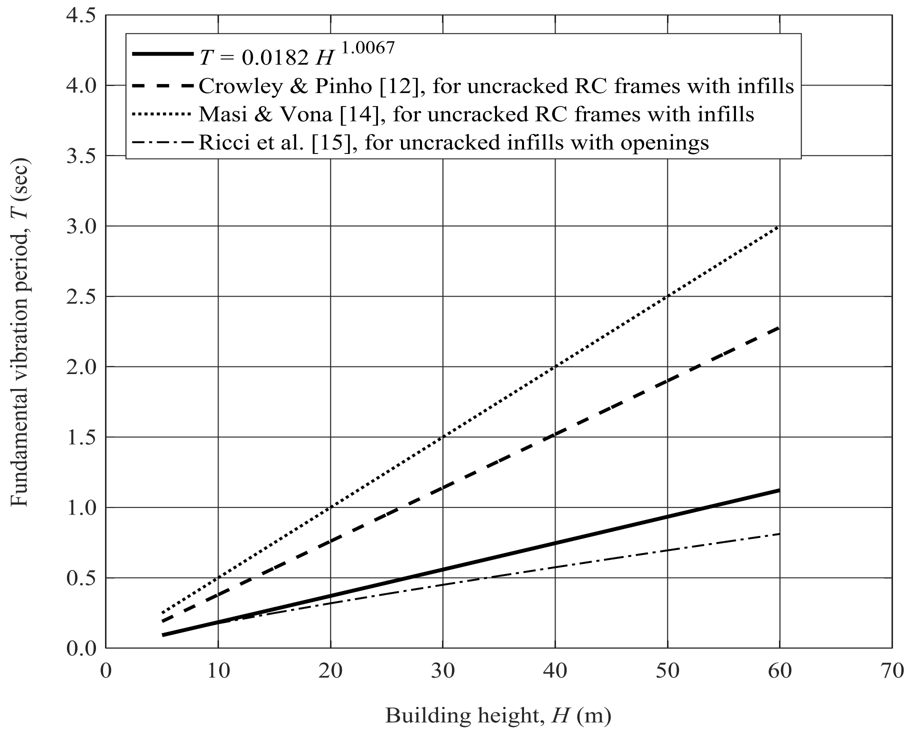


(b) For infills with openings

Fig. 12 Comparison of the proposed formulas for fundamental vibration period with those in prior studies for RC MRF buildings of different heights with cracked infills

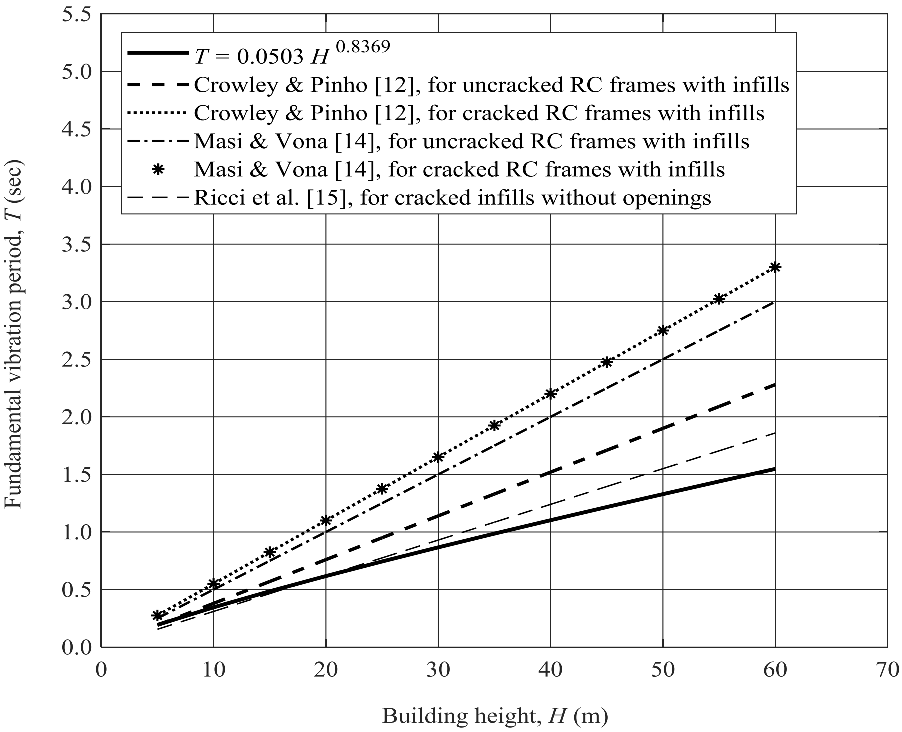


(a) For infills without openings

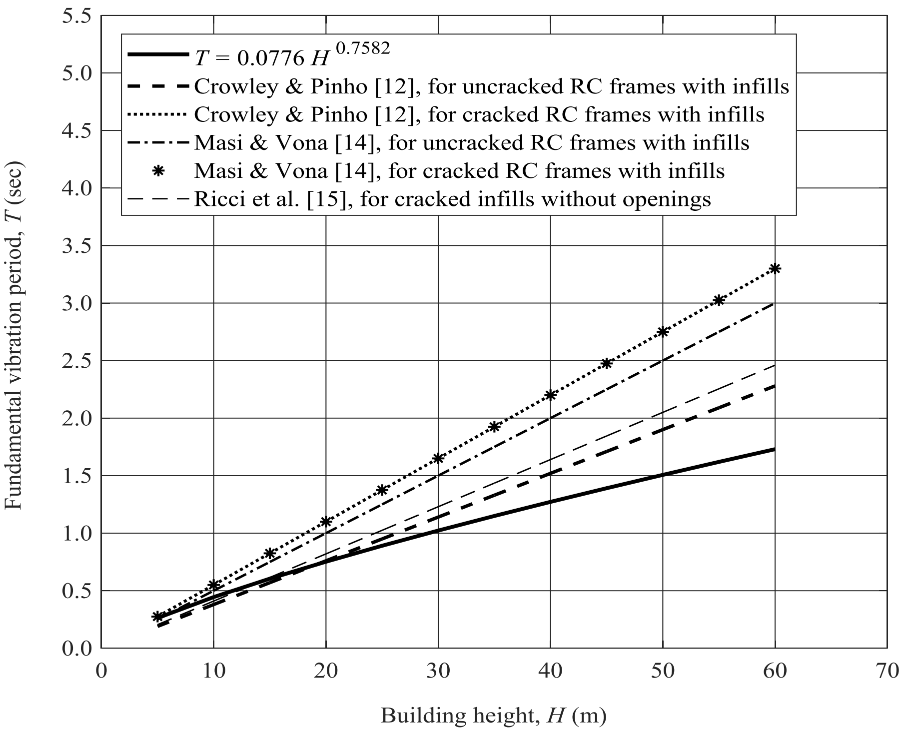


(b) For infills with openings

Fig. 13 Comparison of the proposed formulas for fundamental vibration period with those in prior studies for RC MRF buildings of different heights with uncracked infills



(a) For infills without openings



(b) For infills with openings

Fig. 14 Comparison of the proposed formulas for fundamental vibration period with those in prior studies for RC MRF buildings of different heights with cracked infills