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# SIMPLIFIED EQUATIONS FOR ESTIMATING THE PERIOD OF VIBRATION OF EXISTING BUILDINGS

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## SUMMARY

Currently, seismic design of new European buildings follows a force-based approach, whilst the assessment of existing buildings is moving towards a displacement-based philosophy. In force-based design, conservative estimates of the period of vibration should be produced such that the base shear force will be conservatively predicted from an acceleration spectrum, and thus the use of gross section (uncracked) stiffness in analytical calculations is perhaps acceptable. For the assessment of buildings, the use of the uncracked stiffness in the determination of the period is certainly inappropriate considering cracking of critical elements such as beams generally occurs under gravity loading alone. Even if cracking is not found to have occurred before the design seismic level of excitation (considered unlikely as this level of excitation would with all probability have been preceded by a number of lower intensity events), it will occur early on in the response to excitation and thereafter the stiffness will reduce rapidly leading to the loss of the tension stiffening effect of the concrete. Thus, the reliable stiffness of the members of an existing RC frame can only be confidently taken as the yield/cracked stiffness. The uncracked and yield period of existing European reinforced concrete buildings of varying height is analytically calculated herein using eigenvalue analysis. A simplified equation is proposed to relate the yield period of vibration of existing buildings to their height for use in large-scale vulnerability assessment applications.

## 1. INTRODUCTION

The determination of the natural period of vibration of a reinforced concrete structure is an essential procedure in earthquake design and assessment. An improved understanding of the global demands on a structure under a given seismic input can be obtained from this single characteristic. This property is dependent on the mass, strength and stiffness of the structure and is thus affected by many factors such as structural regularity, number of storeys and bays, section dimensions, infill panel properties, axial load level, reinforcement ratio and extent of concrete cracking. For the seismic design of a reinforced concrete (RC) frame, the period of vibration will not be known *a priori* and thus simplified equations are employed in the seismic design codes to relate the fundamental period to the height of the frame. These equations have traditionally been obtained by regression analysis on the periods of vibration measured during earthquakes. The first semi-empirical formulae employed in seismic design codes or guidance documents, such as the ATC3-06 [ATC, 1978], had the form:

$$T = C_t H^{0.75} \text{ ($H$ in feet)} \qquad (1)$$

where $C_t$ was taken to be 0.03 for reinforced concrete moment-resisting frames and $H$ represents, for this equation only, height defined in feet (in all other expressions herein $H$ corresponds to height in metres). This

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particular form was theoretically derived using the Rayleigh's method with the following assumptions; the equivalent static lateral forces are distributed linearly over the height of the building, the distribution of stiffness with height produces a uniform storey drift under the linearly distributed lateral forces, the base shear is proportional to $1/T^{2/3}$, and the deformations are controlled by the drift limit-state. The numerical value of the constant $C_t$ was obtained from the measured periods of buildings during the 1971 San Fernando earthquake.

In the European seismic design regulation, Eurocode 8 [CEN, 2003], the period height relationship shown in Eq. (2) is specified for force-based design of moment resisting frames. This equation is very similar to Eq. (1) given above, the only difference being that the coefficient $C_t$ has been conveniently adapted from feet to metres (0.03 × 3.2810.75 = 0.073 ≈ 0.075).

$$T = 0.075H^{0.75} \tag{2}$$

Goel and Chopra [1997], who collected data measured from eight Californian earthquakes, starting with the 1971 San Fernando earthquake and ending with the 1994 Northridge event, showed that Eq. (1) tends to underestimate the periods of vibration measured from a total of 27 reinforced concrete frames, particularly those above sixteen storeys. Hence, these researchers proposed alternative period-height formulae, with the best-fit minus 1 standard deviation recommended for conservative force-based design, whilst the best-fit plus 1 standard deviation was recommended for displacement-based assessment [Chopra and Goel, 2000].

Simplified period vs. height equations are also fundamental for analytical loss assessment methodologies wherein the period of vibration of classes of buildings needs to be predicted. This is the case for the displacement-based earthquake loss assessment (DBELA) methodology which is currently being developed [Crowley *et al.*, 2004]. The period vs. height relationship currently employed in DBELA has been analytically derived for bare RC frames by Crowley and Pinho [2004]; a code-based equation was not employed because the latter leads to a conservative estimation of the period for force-based design, whilst for displacement-based assessment a conservative estimation of the period would lead to an underestimation of the displacement demands. Crowley and Pinho [2004] recognised the need to account for the influence of infill panels on the fundamental period of RC frames and thus the aim of the study presented herein is to update the aforementioned equation through the inclusion of infill panel stiffness.

## 2. CASE STUDY BUILDINGS

The RC frames considered in this study correspond to actual buildings from five different European countries exposed to earthquake action (Greece, Italy, Portugal, Romania, ex-Yugoslavia), the vast majority of which were designed and built between 1930 and 1980, thus following outdated seismic regulations, where seismic action was either ignored or inadequately accounted for. Table 1 summarises the general characteristics of the 11 case studies considered in the present study, indicating also the publications from where the necessary modelling data has been obtained, and to which readers can refer to for further information and construction/design details [see also Crowley, 2003].

**Table 1: Summary of case studies**

| No. | Country | Year | Height | Geometric Characteristics | Reference |
|---|---|---|---|---|---|
| 1 | Italy | 70's | 6 m | 3 storeys, 3 bays, asymmetric, 2/3 scale model | Moratti [2000] |
| 2 | Italy | 60's | 9 m | 2 storeys, 5 bays, asymmetric | FIB [2003] |
| 3 | Portugal | 60's | 11 m | 4 storeys, 3 bays, asymmetric | Carvalho *et al.* [1999] |
| 4 | Romania | 30's | 15 m | 5 storeys, 5 bays, symmetric | Bostenaru [2003] |
| 5 | Greece | 50's | 15 m | 5 storeys, 6 bays, symmetric | Zeris *et al.* [2002] |
| 6 | Greece | 50's | 17 m | 5 storeys, 6 bays, symmetric, "pilotis" | Zeris *et al.* [2002] |
| 7 | Italy | 60's | 18 m | 6 bays, symmetric | Moratti [2000] |
| 8 | Romania | 30's | 18 m | 6 bays, symmetric | Bostenaru [2003] |
| 9 | Ex-Yugoslavia | 80's | 22 m | 7 storeys, 2 bays, symmetric | ACI [1984] |
| 10 | Italy | 60's | 24 m | 8 storeys, 3 bays, symmetric | Moratti [2000] |
| 11 | Italy | 70's | 24 m | 8 storeys, 2 bays, symmetric | Cosenza [2000] |

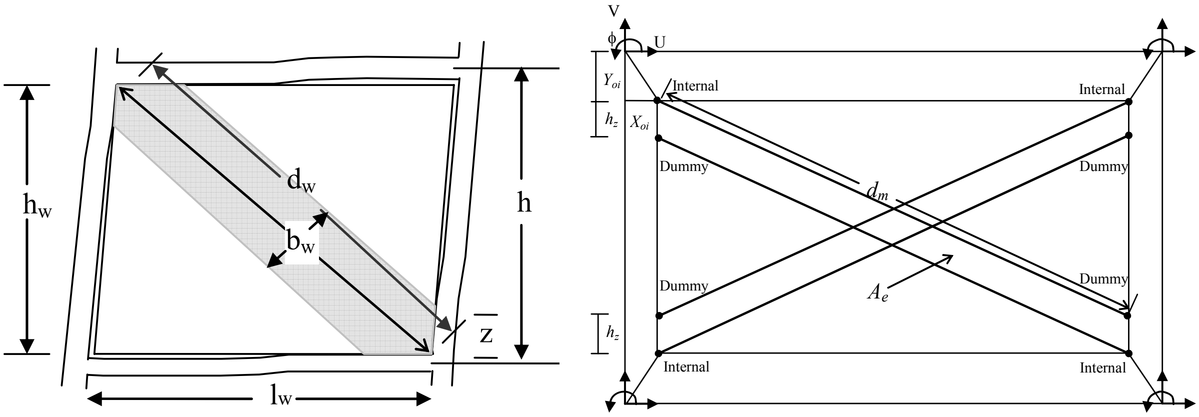
The total height of the frames varies between 2 and 24 metres, corresponding to buildings featuring two to eight storeys, a range that covers a great proportion of seismically exposed old structures in earthquake-prone European countries. The material properties for each of the considered structures were also found to be vastly diverse from one case to the other, with unconfined concrete compressive stress ($f_c$) ranging from 15 to 29 MPa, whilst the yield strength ($f_y$) of the reinforcement typically varied from 200 to 380 MPa. Only frames featuring non-embedded (i.e. emergent) beams have been considered in the present study; preliminary studies indicate that embedded-beam frames tend to have markedly higher periods of vibration due to their increased flexibility, thus justifying a separate study.

## 3. PERIOD OF VIBRATION OF INFILLED BUILDINGS

The aim of this study is to produce a period vs. height equation in order to predict the fundamental period of vibration of reinforced concrete buildings with moment resisting frames and infill panels. Such buildings, however, are considered in this study by analytically modelling representative 2D frames. These frames have already been employed in the period-height study for bare frames by Crowley and Pinho [2004]. For bare RC buildings, these 2D frames can be used to adequately estimate the period of vibration of the whole building in the principle direction in which these frames act. However, in a real building with infill panels, some of the frames are bare, some are fully infilled and some have infill panels with openings for windows and doors. Hence, the influence of infill panels cannot be simply modelled using a single 2D frame and should ideally be calculated using 3D models of real structures. A simplified solution to this problem has been adopted herein such that the 2D frames which have already been modelled for the Crowley and Pinho [2004] study can be employed. The 2D frames are modelled as bare frames, fully infilled frames and infilled frames with openings and a weighted average of the period of vibration of these three types of frames is then calculated by taking into account the their frequency of occurrence within a typical building stock. A detailed study of the characteristics of Turkish reinforced concrete building stock in the Marmara region has been carried out by Bal *et al.* [2006] and it has been found that the proportion of each frame in a building stock of 30 buildings has been found to be 34% bare frames, 28% fully infilled frames and 38% infilled frames with openings. In the frames with openings, the ratio of area of the openings compared to the area of the bays (within a single storey) has been found to have a mean value of 20%. The size of these openings ranges between (1.0-1.7) × (1.3-2.4) m for windows and between (0.9-1.2) × (2.1-2.3) m for doors.

All the eigenvalue analyses described in the subsequent sections of this paper were carried out with SeismoStruct [SeismoSoft, 2006], a fibre-modelling Finite Elements program for seismic analysis of framed structures, which can be freely downloaded from the Internet. The program is capable of predicting the large displacement behaviour of space frames under static or dynamic loading, taking into account both local (beam-column effect) and global (large displacements/rotations effects) geometric nonlinearities as well as material inelasticity. The spread of the latter along the member length and across the section area is explicitly represented through the employment of a fibre-modelling approach, implicit in the formulation of the inelastic beam-column frame elements employed in the analyses. Full details on this computer package can be found in its accompanying manual. Structural members were subdivided into 4 or 5 beam-column elements, the latter being smaller at frame member ends so as to ensure that material inelasticity could be accurately modelled. Beams and columns were modelled as extending from the centre of one beam-column joint to the centre of the next, in order to take into account, albeit in an indirect and empirical manner, joint flexibility that could be induced by shear distortion, yield penetration and/or bar slippage (particularly relevant for the case of old existing buildings, often designed and constructed with the employment of smooth bars).

SeismoStruct models the masonry infill panels using a macro model that represents the infill panel as a diagonal strut (see Figure 1a) with a given equivalent area/stiffness, resistance and nonlinear behaviour. The diagonal element consists of two compression struts in each diagonal of the panel, as shown in Figure 1b. The element has 4 external nodes, 4 internal nodes (connected to the frame corner) and 4 dummy nodes (located at a distance $h_z$ from the frame corner); such formulation is based on the proposals of Crisafulli [1997] and Crisafulli *et al.* [2000]. The dummy nodes are used to represent the local effect between the infill and the frame and are placed at a distance equal to a third of the contact length, *z* (illustrated in Figure 1a). The internal nodes represent the frame-infill contact at the exterior part of the beam and column and the internal forces are then transformed to the 4 exterior nodes where the element is connected to the frame. The hysteretic material model which represents the nonlinear behaviour of the strut has been defined using the default values suggested/calibrated by Blandon-Uribe [2005] and Smyrou *et al.* [2006]. The geometric and material properties of the diagonal struts have been defined using the values and formulae reported in Table 2.



**(a)** **(b)**

**Figure 1: (a) Diagonal strut representation of an infill panel [Paulay and Priestley, 1992], (b) Description of implemented infill panel [adapted from Crisafulli, 1997]**

**Table 2: Values and formulae used to define the properties of the diagonal masonry strut**

| **Parameters required to define masonry strut** | **Adopted value/formula** | **Reference** |
|---|---|---|
| Mean diagonal compressive strength, $f'_{m\theta}$ | 1.5MPa | Calvi *et al.* [2004]<br>Bal *et al.* [2006] |
| Elastic Young's Modulus, $E_{m\theta}$ | $1000f'_{m\theta} = 1500$MPa | Crisafulli [1997]<br>Calvi *et al.* [2004] |
| Thickness of infill panel, $t_w$ | 0.1 m and 0.25 m | Calvi *et al.* [2004]<br>Bal *et al.* [2006] |
| Length of strut, $d_w$ | varies, calculated from geometry of frame | See Figure 1a |
| Relative stiffness parameter, $\lambda$ | $\lambda = \sqrt[4]{\frac{E_{m\theta} t_w \sin(2\theta)}{4E_c I_p h_w}}$<br>$E_c$ = elastic modulus of concrete<br>$\theta$ = angle of diagonal strut with beam<br>$I_p$ = moment of inertia of column<br>$h_w$ = height of infill panel | Stafford Smith and Carter [1969] |
| Width of compressive strut, $b_w$ | $b_w = \frac{0.95 h_w \cos\theta}{\sqrt{\lambda h_w}}$<br>(varies between approx. $0.1d_w$ and $0.4d_w$) | Liauw and Kwan [1984] |
| Reduction coefficient for openings | $r_{ac} = 0.78e^{-0.322\ln A_a} + 0.93e^{-0.762\ln A_e}$<br>$A_a$ = opening area/infill area = 20%<br>$A_e$ = opening length/infill length = 25%<br>$\therefore r_{ac} \approx 0.4$ | Bertoldi *et al.* [1994] |
| Initial area of strut, $A_{ms1}$ | $A_{ms1} = d_w b_w$, fully infilled<br>$A_{ms1} = d_w b_w r_{ac}$, infills with openings | - |
| Residual strut area, $A_{ms2}$ | $A_{ms2} = A_{ms1}(b_{wcracked}/b_{wuncracked})$ | Decanini and Fantin [1986]<br>See Figure 2 |
| Contact length, z | $z = \frac{\pi}{2\lambda}$ | Stafford Smith and Carter [1969] |
| Horizontal offset, $X_0$ | varies, calculated from geometry of frame | See Figure 1b |
| Vertical offset, $Y_0$ | varies, calculated from geometry of frame | See Figure 1b |

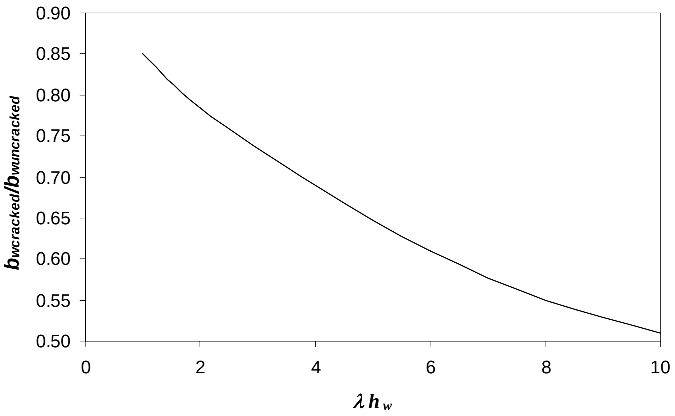


**Figure 2: Reduction factor to be applied to the equivalent width of diagonal struts due to cracking of masonry panel [Decanini and Fantin, 1986]**

Two "boundary" values of infill panel thickness have been adopted (0.1m and 0.25m), as this parameter has a large influence on the stiffness of the diagonal struts and varies from building to building. The material properties adopted for the masonry infill aim to reflect the characteristics of highly perforated clay blocks typically used in European earthquake-prone countries. The elastic modulus of this type of infill panel has a large coefficient of variation [Calvi *et al.*, 2004]; however, a mean value has been adopted herein and further studies will be carried out in the future to study the influence of this variation on the period of vibration of the infilled frames.

## 4. GROSS STIFFNESS VS. HEIGHT

Eigenvalue analyses considering the gross stiffness section properties for the bare reinforced concrete (RC) frames, the fully infilled RC frames and the RC frames with openings have been carried out and the period of vibration has been plotted against the height of the building, as presented in Figure 3. Regression analysis has then been carried out, for each analysis case, in order to derive a best-fit line to the analytically computed data of the form T=αH. The fully infilled frames and the infilled frames with openings can be seen to have a larger amount of scatter than the bare frame plot and this is due to the two values of panel thickness which have been adopted. The use of additional models with variations in the value of the elastic modulus of the masonry would lead to even larger scatter in these plots and thus until such studies have been carried out, an indication of the scatter/variability in the reported equations will not be given.

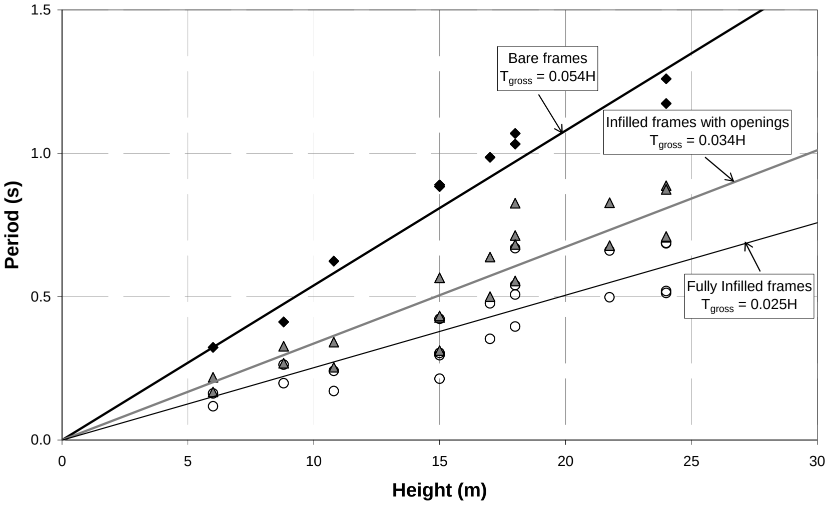


**Figure 3: Analytical period-height relationships for uncracked RC frames obtained with gross stiffness eigenvalue analyses for bare frames, fully infilled frames and frames with openings**

The mean period of vibration of European buildings can be assumed to fall between the bare and fully infilled frame period-height curves presented in Figure 3, which can be viewed as possible bounds to the actual period of vibration. As mentioned previously, the proportion of bare frames, fully infilled frames and partially infilled frames in a sample of Turkish buildings has been studied by Bal *et al.* [2006] and the ratios have been found to be 34%, 28% and 38%, respectively. These ratios have been used to calculate a weighted mean period of vibration for each frame, as presented in Figure 4.

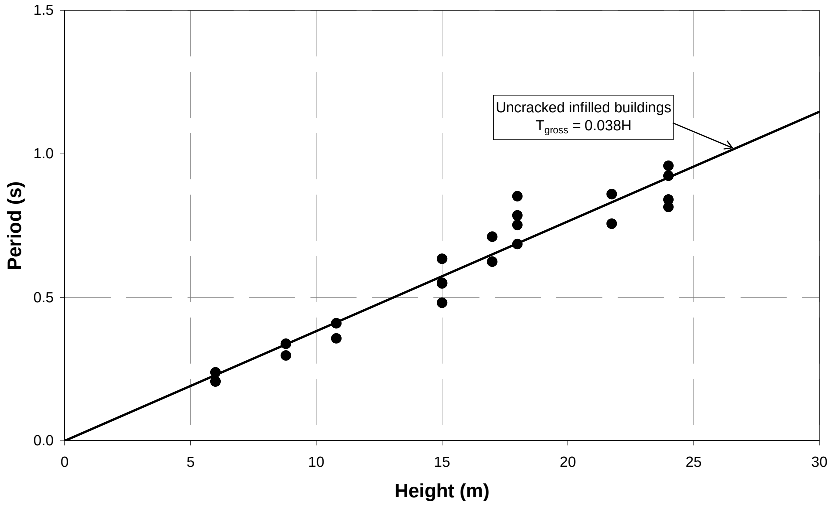


**Figure 4: Analytical period-height relationship for uncracked infilled RC buildings obtained by calculating a weighted average of the results in Figure 3**

The period-height relationship for infilled RC buildings derived in Figure 4 has been compared with the EC8 period-height formula (Eq. 2) and the best-fit curve proposed by Goel and Chopra [1997], as presented in Figure 5. Although Figure 5 would appear to suggest that the building characteristics of European and US building construction are similar, this is not quite the case as the EC8 [CEN, 2003] and Goel and Chopra [1997] equations are based on the measured periods of buildings during earthquakes where at least a limited amount of cracking of the reinforced concrete and masonry infill panels is to be expected. The analytical formula for European buildings that has been derived herein is based on the gross section stiffness properties and thus these buildings would have even higher periods of vibration due to cracking, as discussed further in Section 5.

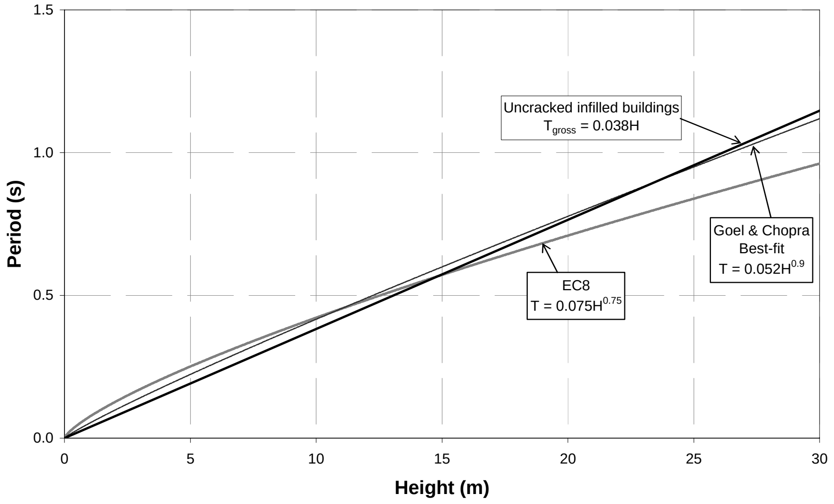


**Figure 5: Analytical period-height relationship for uncracked infilled RC buildings compared with the EC8 [CEN, 2003] equation and the equation proposed by Goel and Chopra [1997]**

## 5. YIELD STIFFNESS VS. HEIGHT

The use of gross section (i.e. uncracked) stiffness in the computation of elastic vibration periods of reinforced concrete structures is inadequate because it would lead to an underestimation of the displacement demands to the structure. Indeed, as reported by Priestley [2003], amongst others, cracking of critical elements such as beams generally occurs under gravity loading alone. And even in those cases where cracking is not found to have occurred before the design seismic level of excitation (considered unlikely as this level of excitation would with all probability have been preceded by a number of lower intensity events), it will occur early on in the response to excitation and thereafter the stiffness will reduce rapidly. Since the tension stiffening effect of the concrete is lost following the initiation of seismic excitation and will not be regained without post-earthquake intervention, the stiffness of RC members can only be reliably estimated if taken as the yield stiffness. The study by Crowley and Pinho [2004] considered five different procedures (using eigenvalue analysis, pushover analysis and dynamic analysis) to estimate the yield stiffness of RC frames and found that all five approaches led to very similar period-height predictions, as presented in Figure 6. The dependency of the yield stiffness on the strength of the RC members [see for e.g. Priestley, 2003] was the main assumption that was applied in all methods to calculate the yield period of the RC frames.

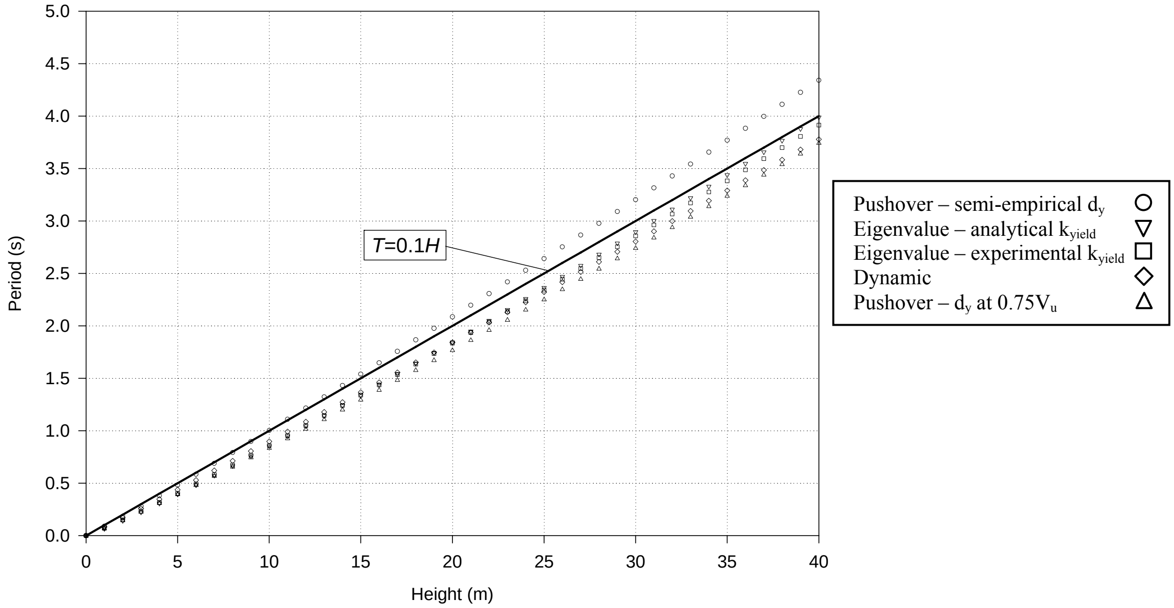


**Figure 6: Analytical yield period-height relationships for bare frames derived using various methods to define the yield stiffness [Crowley and Pinho, 2004]**

Considering the similarity of the five methods used in the Crowley and Pinho [2004] study, only one of these methods needs to used herein, hence the approach suggested by Priestley [1998] has been adopted to extend the study to calculate the yield period of vibration of infilled buildings. Priestley [1998] carried out an analytical parametric study whereby moment curvature curves of a reinforced concrete member with varying levels of longitudinal reinforcement ($\rho_l$) and axial load ($N/f_cA_g$) were plotted. The attained results led to the development of the dimensionless stiffness ratio curves presented in Figure 7 which allow the effective stiffness ($EI$) of reinforced concrete members to be obtained as a function of their respective gross stiffness ($EI_{gross}$). Priestley [1998] notes that slight errors may be expected when small section dimensions are considered, since these curves were derived for a large section column, and that the employment of such curves in cases where very high strength concrete or reinforcement are used may required careful evaluation of results. Reduction factors have been calculated using Figure 7 for all members in the considered frames taking into account their respective level of axial load and section reinforcement ratio. The eigenvalue analyses for the bare frames were then simply repeated using the reduced values of elastic modulus ($E$) and the results are shown in Figure 8.

The same models with reduced member stiffness were used to create the fully infilled frames and the infilled frames with openings. To take into account the cracking of the masonry, the residual strut area, $A_{sm2}$, has been calculated for each strut using the reduced strut width obtained from Figure 2. The yield period-height results for these two types of frames are presented in Figure 8, and again the weighted average period for infilled buildings has been calculated using the ratios presented previously, leading to the curve presented in Figure 9.

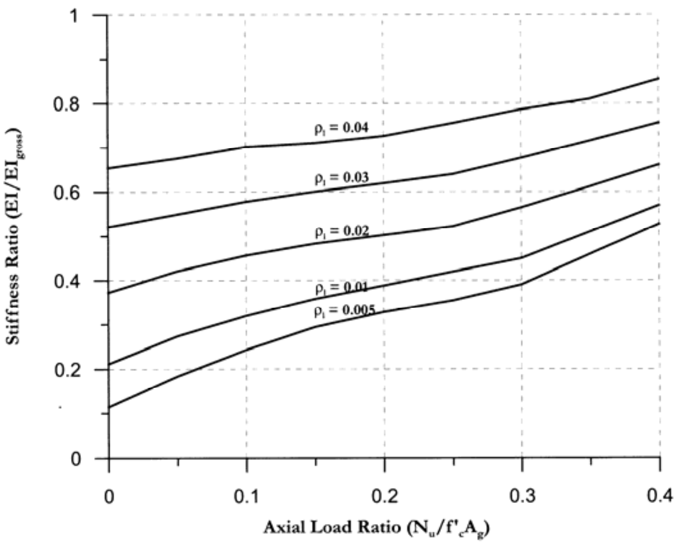


Figure 7: Effective stiffness of reinforced concrete members [Priestley, 2003]

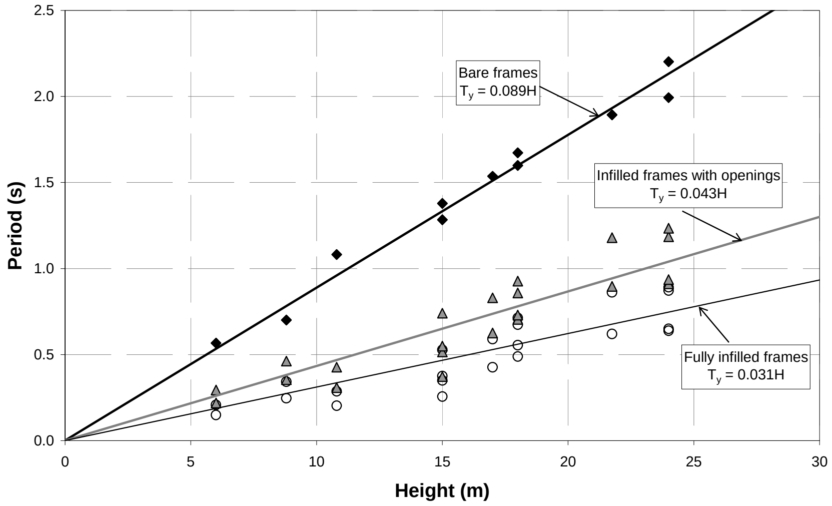


Figure 8: Analytical yield period-height relationships for cracked RC buildings obtained with yield stiffness eigenvalue analyses for bare frames, fully infilled frames and frames with openings

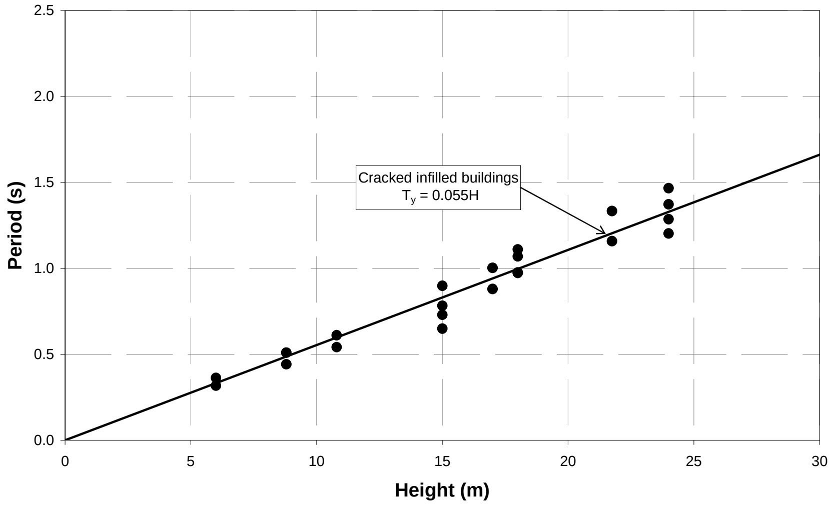


Figure 9: Analytical yield period-height relationship for cracked infilled RC buildings obtained by calculating a weighted average of the results in Figure 8.

## 6. CONCLUDING REMARKS

The study presented herein has led to a simplified period-height equation for use in the assessment of existing RC buildings, taking due account of the presence of infill panels:

$$T_y = 0.055H \tag{3}$$

This equation is compared in Figure 10 with the empirical formula currently employed in EC8 [CEN, 2003] for the assessment of reinforced concrete buildings; the large difference between these curves would suggest that the current study could be used as a starting point for a revisiting exercise of the empirical equations used in EC8 to estimate the period of vibration of existing European buildings. In order to do so, the formula proposed herein should be verified for other European countries (Turkish typical building properties were considered in this study) using appropriate material properties to model the masonry panels and considering the proportions of bare, fully infilled and partially infilled frames that can be found in a sample of the building stock. Furthermore, comparisons of the period estimates made with the formula proposed herein should be made with the results of detailed analyses on a limited number of 3D analytical models of infilled reinforced concrete buildings.

The period of vibration which has been derived herein represents the period of the first mode of vibration of a fixed-base building model, thus the rocking and horizontal translation periods of the building as it moves as a rigid body on flexible soil are ignored, though they may of importance. Future studies could thus aim to include the soil-structure interaction in the prediction of the period of vibration.

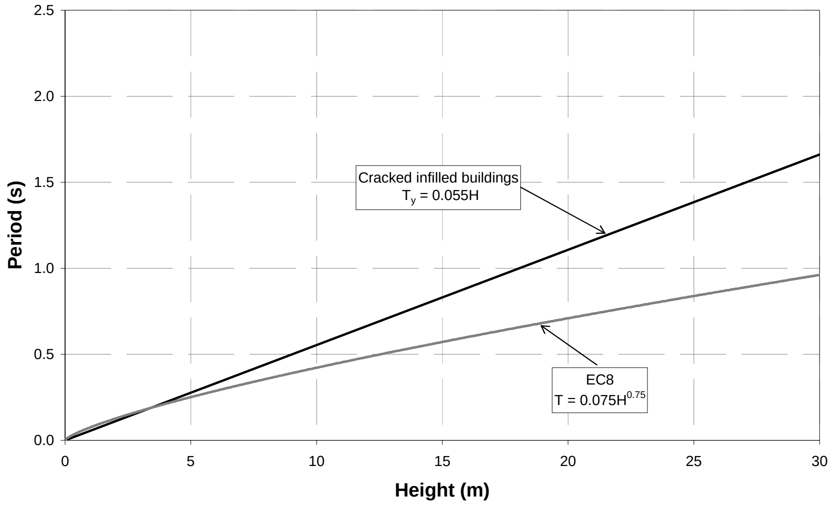


**Figure 10: Analytical yield period-height relationship for cracked infilled RC buildings compared with the EC8 empirical equation**

## 7. ACKNOWLEDGEMENTS

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