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REVISITING EUROCODE 8 FORMULAE FOR PERIODS OF VIBRATION AND THEIR EMPLOYMENT IN LINEAR SEISMIC ANALYSIS

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ABSTRACT

The period of vibration is a fundamental parameter in the force-based design of structures as this parameter defines the spectral acceleration and thus the base shear force to which the building should be designed. This paper takes a critical look at the way in which seismic design codes around the world have allowed the designer to estimate the period of vibration for use in both linear static and dynamic analysis. Based on this review, some preliminary suggestions are made for updating the clauses related to the estimation of the periods of vibration in Eurocode 8.

KEYWORDS

Eurocode 8, period of vibration, RC frames, linear analysis.

1 INTRODUCTION

Provisions for the linear static and dynamic design of reinforced concrete buildings are included in almost all seismic design codes around the world. Although the names of these procedures vary from code to code, the basic principles are the same. The linear static (or lateral force) method allows engineers to predict the fundamental period of vibration in a simplified manner and calculate the design base shear force from the response spectrum (see Figure 1 for an example Eurocode 8 spectrum [CEN, 2004]), this base shear force is then distributed along the height of the building in a linear manner. Period-height relationships which have been obtained for different building typologies from the measured periods of vibration during earthquake ground shaking are generally used, though Rayleigh analysis is also often allowed. The linear dynamic (or modal response spectrum) method requires a simple analytical model of the structure to be produced (often using structural sections of reduced stiffness) and the periods of vibration and modal shapes of a number of significant modes to be calculated. The forces resulting from each mode are applied to the building using the appropriate modal shape and the seismic actions resulting from these forces are combined using specified combination rules.

As can be deduced from Figure 1, the period of vibration is a fundamental parameter in the force-based design of structures as this parameter defines the spectral acceleration and thus the base shear force to which the building should be designed. For the usual range of structural periods, higher periods of vibration lead to lower design forces. This paper takes a critical look at the way in which seismic design codes around the world have allowed the designer to estimate the period of vibration for use in both linear static and dynamic analysis. The influence of the period of vibration on the design will be briefly discussed and some preliminary proposals for updating the periods of vibration in linear analysis in Eurocode 8 will be made.

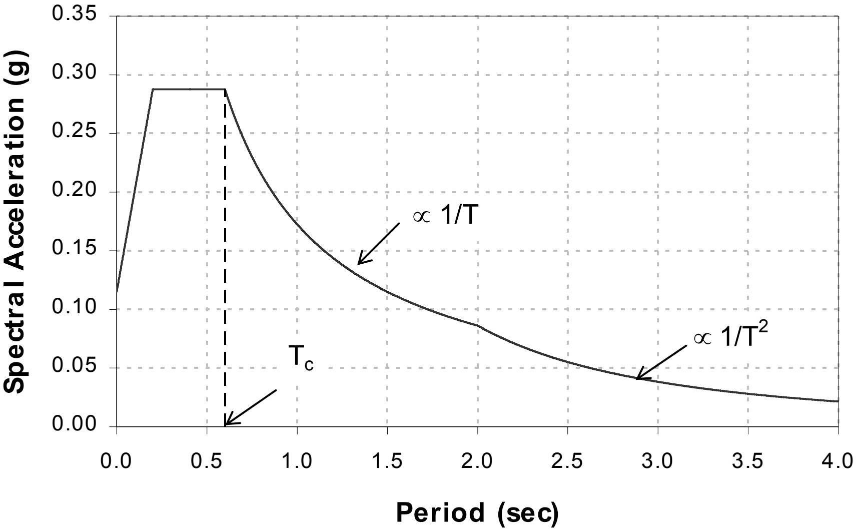

Figure 1. Type 1 acceleration response spectrum in Eurocode 8 for a peak ground acceleration of 0.1g and site condition C [CEN, 2004].

2 PERIOD OF VIBRATION IN DESIGN CODES

2.1 Period-height relationships in seismic design codes for moment resisting frames

The fundamental period of vibration required for the simplified design of reinforced concrete structures has been calculated for many years using a simplified formula relating the period to the height of the building. One of the first formulae of this type was presented almost 30 years ago in ATC3-06 [ATC, 1978] and had the form:

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

where C_t was a regression coefficient and H represented the height of the building in feet.

As discussed in Goel and Chopra [1997], the particular form of Eq. (1) was theoretically derived by assuming that the equivalent static lateral forces are linearly distributed over the height of the building and the distribution of stiffness with height produces a uniform storey drift under the linearly distributed lateral forces. Furthermore in ATC3-06 [ATC, 1978] the base shear force was assumed to be inversely proportional to $T^{2/3}$ and thus these two assumptions led to Eq. (1), as shown in the workings below.

The period of vibration (T) of a single degree of freedom oscillator can be obtained from Eq. (2) where m is the mass of the oscillator and k is the stiffness:

$$T = 2\pi\sqrt{\frac{m}{k}} \tag{2}$$

The stiffness of the oscillator can be obtained from the base shear (V) divided by the lateral displacement (Δ). From the response spectrum in early design codes, the base shear for the usual range of periods of structures was taken as inversely proportional to the period to the power of two-thirds, with the coefficient of proportionality defined as C_1 herein. As mentioned previously, if one assumes that the distribution of stiffness with height produces a uniform storey drift under the linearly distributed lateral forces, then the lateral displacement, Δ is given by the interstorey drift, ϑ, multiplied by the height, H:

$$k = \frac{V}{\Delta} = \frac{C_1}{T^{2/3}\Delta} = \frac{C_1}{T^{2/3}\vartheta H} \tag{3}$$

By replacing Eq. (3) in Eq. (2), the relationship shown in Eq. (1) between period and height can be obtained, as outlined in the workings of Eq. (4) to (6):

$$T = 2\pi\sqrt{\frac{mT^{2/3}H\vartheta}{C_1}} \tag{4}$$

$$T^2 = (2\pi)^2 \frac{mT^{2/3}H\vartheta}{C_1} \tag{5}$$

$$T^{4/3} = C_t H \tag{6}$$

In ATC3-06 [ATC, 1978], the coefficient C_t in Eq. (1) was given equal to 0.025 for reinforced concrete moment resisting frames. This coefficient was identified from a study by Gates and Foth [1978] based on the measured periods of vibration of reinforced concrete frames during the 1971 San Fernando earthquake. A subsequent re-evaluation by SEAOC-88 [SEAOC, 1988] found that a value of Ct=0.03 was more appropriate for reinforced concrete buildings. The coefficient Ct was generally calibrated such that the derived fundamental period would underestimate the period by approximately 10-20% at first yield to obtain a conservative estimate for the base shear [Goel and Chopra, 1997].

Bertero *et al.* [1988] studied in greater detail the fourteen buildings considered by Gates and Foth [1978] and found that four of the buildings and the longitudinal direction of a fifth building could not be considered as moment-resisting frame (MRF) structures and they thus excluded them from the database. They further identified two buildings with structural damage and two others that had non-structural damage; they discuss how damage leads to stiffness degradation and thus an increase in the period of vibration. Gates and Foth [1978] did not relate the building damage to the period of vibration and thus Bertero *et al.* [1988] re-evaluated the time histories of building response for the moment-resisting frame structures and identified times where a sudden increase in the period of vibration took place, which was then correlated to the onset of non-structural and structural damage. The period of vibration at the second increase in period was considered to be the stage when the non-structural components were no longer contributing significantly to the stiffness and interpreted as the period at which the building was essentially vibrating as a bare structural frame. They added a further four buildings to the database and evaluated the bare frame period of vibration for all buildings in a uniform manner at the aforementioned second increase in period. The conclusions of their study was that the formula of Eq. (1) with C_t equal to 0.03 does not constitute a reliable estimate of the during earthquake period of reinforced concrete moment-resisting frames, and a better fit was found with C_t =0.04 (0.097 with H in metres). For a lower bound estimate of the period, Bertero *et al.* [1988] recommend the use of C_t = 0.035 (0.085 with H in metres).

In the buildings used in the Bertero *et al.* [1988] study, the partition walls were generally plaster board/dry walls whilst the outer infill was often constructed with glass curtain walls with spandrels built into the outer frames. Bendimerad *et al.* [1991] found that the participation of these non-structural components on the stiffness of the building was minimal compared to the stiffness of the frame and thus had practically no effect on the building period beyond the first 5 seconds of earthquake motion. Hence, the use of the equation derived from these buildings for the design of "moment resisting frame systems of reinforced concrete in which the frames resist 100 percent of the required seismic force" appears to be justified.

The use of the form of period-height equation shown in Eq. (1), along with the SEAOC-88 recommended 0.03 coefficient, has been adopted in many design codes since 1978, for example in UBC-97 [UBC, 1997], in SEAOC-96 [SEAOC, 1996], in NEHRP-94 [FEMA, 1994] and in Eurocode 8 [CEN 1994; 2004]. In Eurocode 8, the C_t coefficient has simply been transformed considering that the height is measured in metres, leading to $C_t = 0.075$. In a similar way that the use of a 475 year return period to define the seismic actions was adopted in seismic design codes around the world [see e.g. Bommer and Pinho, 2006], the period-height equation of Eq. (1) with C_t=0.075 has also spread around the world (see for example the design codes of the following countries: Algeria '88; Cuba '94; El Salvador '89; Israel '95; Korea '88; Panama '94; Philippines '92) [IAEE, 1996; 2000].

Goel and Chopra [1997] used the fundamental period of vibration of buildings measured from their motions recorded during eight California earthquakes from 1971 to 1994 to update the period-height formula in UBC-97. They found that the best fit lower bound curve (i.e. the mean –1 standard deviation) for reinforced concrete frames was given by:

$$T = 0.0466H^{0.9}\,(H \text{ in metres}) \tag{7}$$

This period-height formula has recently been included in ASCE 7-05 [2006]. As can be noted from Eq. (7), Goel and Chopra [1997] decided to move away from the 0.75 power regression and found the best-fit regression to be 0.9. Simplified period-height equations can only be applied in Eurocode 8 for buildings up to 40 metres, and thus the period of vibration obtained with such equations will generally be within the range of inverse proportionality between base shear and period (see Figure 1). By repeating the calculations in Eq. (2) to (6) with the base shear force inversely proportional to T (i.e. $V=C_1/T$), the form of the period-height equation becomes linear:

$$T = C_t H \tag{8}$$

The reason that Goel and Chopra [1997] did not arrive at a linear relationship between period and height is probably because they did not just focus on the buildings whose base shear should be inversely proportional to period, but included taller buildings in the range of "non-proportionality". Based on the fact that an updated period-height equation for RC frames has now been proposed, as well as the possibility that a linear equation might be more valid for frames designed to Eurocode 8, an examination of the periods of vibration of newly designed European reinforced concrete bare frames is thus warranted; this issue is considered further in Section 2.3.

2.2 *Period-height relationships in seismic design codes for structures with reinforced concrete or masonry walls*

The first period-height relationship for buildings with concrete shear walls had the form presented in Eq. (9) and, as with the previous equations, was also calibrated using the measured motions of buildings recorded during the 1971 San Fernando earthquake:

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

where D is the dimension of the building at its base in the direction under consideration. With the height and dimension D of the building measured in feet, C_t was proposed in ATC3-06 [ATC, 1978] as 0.05 (which would be 0.09 with these dimensions measured in metres). This formula comes from the equation of the frequency of vibration of a cantilever (considering shear deformations only), with the thickness of the wall considered to be more or less constant and thus only the width/length of the building is an input parameter, as demonstrated in Eq. (10):

$$T = 4\sqrt{\frac{m}{\kappa G}}\frac{H}{\sqrt{A}} = \frac{\alpha H}{\sqrt{A}} = \frac{\alpha H}{\sqrt{D t_w}} = \frac{\alpha_1 H}{\sqrt{D}} \tag{10}$$

where m is the mass per unit length, G is the shear modulus, κ is the shape factor to account for non uniform distribution of shear stresses, D is the length of the cantilever and t_w is the thickness. This formula is used in many design codes around the world, but the type of structure to which it is applied varies from code to code, as illustrated in Table 1; it is noted that the text used in each code to describe the structures to which Eq. (9) applies has been maintained. Some codes use this formula specifically for buildings with both frames and shear walls, some use the equation for reinforced concrete MRF with masonry infill panels, but many specify it for use with any building except moment resisting space frames.

Table 1. Buildings to which Eq. (9) is applied in different codes from around the World [IAEE, 1996; 2000].

Country, Year	Type of Structure to which Eq. (9) is applied
Albania, 1989	RC framed structures with brick masonry infill walls participating in seismic force resistance
Algeria,1988	Steel or RC moment resisting frames with infilled masonry and partial or total RC shear walls, braced frames, and masonry walls
Canada, 1995	Other structures (i.e. not moment resisting frames)
Colombia 1984	Other structures (i.e. all except framed structures where the frame is not braced by rigid elements that tend to impede the free deflection)
Cuba, 1995	RC buildings with frames and shear walls
Egypt, 1988	All buildings except moment resisting space frames
El Salvador, 1989	All buildings except frames
Ethiopia, 1983	All buildings except those with moment resisting space frames capable of resisting 100% of the required lateral forced and not enclosed by or adjoined by more rigid elements
India, 1984	Other structures (not moment resisting frames without bracing or shear walls for resisting lateral loads)
Iran, 1988	All buildings except moment resisting frames, if other elements do not create an obstacle to the movement of the building frame
Israel, 1975	Multi-storey structures in which horizontal forces are carried by RC frames
Italy, 1986 & 1996	For framed structures (N.B. C_t from Eq. (9) is taken as equal to 0.1)

Country, Year	Type of Structure to which Eq. (9) is applied
Peru, 1977	For buildings whose structural elements are exclusively open frames and shear walls of the elevator, without other elements for providing rigidity to the structure
Venezuela, 1982	Structures consisting of frames and structural walls of reinforced concrete or braced frames

The UBC-97 code [UBC, 1997] did not use Eq. (9) for shear wall buildings, but instead reported empirical equations of the form of Eq. (1), where C_t was taken equal to 0.02 with the height measured in feet and 0.05 with the height measured in metres. This formula has become the "other structures" formula in Eurocode 8 [CEN, 2004] and is also present in the Algeria 1988 code as another possible formula to be used for steel or RC moment resisting frames with infilled masonry and partial or total RC shear walls, braced frames, and masonry walls. The Algerian code is much more explicit about the coefficient to be used in Eq. (1) for RC buildings with and without infilled frames; it is believed by the authors that many designers following Eurocode 8 would use Eq. (1) with C_t equal to 0.075 for all reinforced concrete buildings regardless of the details of the masonry infills. Hence, considering that many reinforced concrete buildings in Europe are constructed with stiff masonry infill panels which are often not isolated from the RC frame, the period of vibration is probably being overestimated by the designers and thus the forces are subsequently being underestimated. This issue is considered further in Section 3.

Another value of C_t to be used in Eq. (1) was permitted for buildings with shear walls in the UBC-97 and SEAOC-96 [SEAOC, 1996] documents, based on the following formula:

$$C_t = \frac{0.1}{\sqrt{A_c}} \tag{11}$$

where A_c, the combined effective area (in square feet) of the shear walls is defined as:

$$A_c = \sum_{i=1}^{NW} A_i \left[0.2 + \left(\frac{D_i}{H} \right)^2 \right]; \; D_i / H \leq 0.9 \tag{12}$$

in which A_i is the horizontal cross-sectional area (in square feet); D_i is the dimension in the direction under consideration (in feet) of the ith shear wall in the first storey of the structure; and NW is the number of shear walls. The numerator of Eq. (11) becomes 0.075 when the dimensions of the structure are measured in metres. This equation has also found its way in Eurocode 8 [CEN, 2004] as the formula to be used for structures with concrete or masonry shear walls; however, the formula for A_c, given below in Eq. (13), appears to be slightly different to the original formula in Eq. (12) which the authors believe might be due to an editing error.

$$A_c = \sum_{i=1}^{NW} A_i \left[\left(0.2 + \left(\frac{D_i}{H} \right) \right)^2 \right] \tag{13}$$

Goel and Chopra [1998] and Lee *et al.* [2000] both show how Eq. (9) and (11) are too conservative for shear wall buildings when compared with the measured periods of vibration of buildings during earthquakes. Goel and Chopra [1998] discuss how there is little correlation between the $H/\sqrt{D}$ value of Eq. (9) and the period of vibration. This could be because the shear walls do not extend for the whole dimension "D" of the buildings, but for just a small proportion. On the other hand, Eq. (11) which includes explicitly the dimensions of the walls, appears to be better correlated to the period of vibration, but was nevertheless found to be too conservative. Both Goel and Chopra [1998] and Lee *et al.* [2000] decided to calibrate an equation for the period of vibration of shear wall structures by considering the fundamental period of a uniform cantilever beam with both flexural and shear deformations. Figure 2 shows the period of vibration of a cantilever with both flexural and shear deformations (based on Dunkerley's method – see Inman [1996]) divided by the period of vibration of a pure-flexural cantilever as a function of the ratio of height to depth (H/D). This plot shows that the period approaches the period of a pure-shear cantilever as the height to depth ratio becomes smaller, and the period of a pure-flexure cantilever as the ratio increases. For shear walls with H/D ratios between 0.2 and 5, the contribution of both flexure and shear to the period of vibration should be considered.

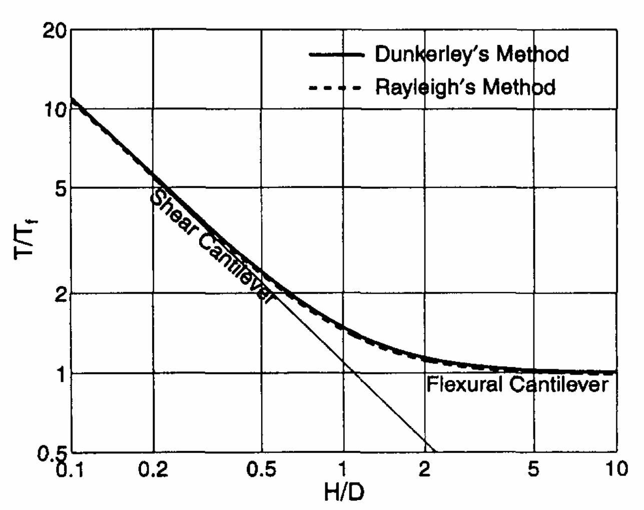

Figure 2. Fundamental period of cantilever beams as a function of height to depth ratio [Goel and Chopra, 2008].

Goel and Chopra [1998] calibrated Dunkerley's equation using the measured periods of vibration of shear wall buildings and obtained the formula which is shown in Eq. 14; this equation has recently been included in ASCE 7-05 [2006].

$$T = \frac{0.0019}{\sqrt{C_w}} H$$

$$\text{where } C_w = \frac{100}{A_B} \sum_{i=1}^{NW} \frac{A_i}{\left[1 + 0.83\left(\frac{h_i}{D_i}\right)^2\right]} \qquad (14)$$

2.3 *Periods of vibration of case study buildings*

The period-height relationships derived in the past, and described in the previous sections, have been obtained from the measured period of vibration of buildings built over a long period of time to different design codes; this variation in the age of the buildings, and thus design regulations, might be part of the reason for the large scatter in the results. Buildings designed to more recent seismic design codes with higher lateral force requirements and capacity design principles are more likely to have larger columns and are thus likely to be stiffer. A number of European moment resisting reinforced concrete frames were modelled by Crowley [2003] using SeismoStruct [SeismoSoft, 2008], a fibre-element finite elements structural analysis package, and Eigenvalue analysis was carried out to compute their fundamental periods of vibration. The results showed that there was a large difference in the stiffness of the buildings designed pre- and post-1980 (Figure 2), most likely due to the changes in design philosophy mentioned previously. The Eurocode 8 equation [CEN, 2004] which was obtained by converting Eq. (1) from feet to metres appears to match well the period of vibration of these newer European buildings, though as mentioned in Section 1 there have been a number of criticisms of the post-processing of the data which led to this equation and it was found to be too conservative. However, considering that more recent European buildings are stiffer than older pre-1980 buildings, the equation now appears to be more reliable. The recently proposed upper and lower bound equations by Goel and Chopra [1997] which were obtained from a number of buildings subjected to earthquakes in California from 1971 to 1994 are also shown in Figure 2. The characteristics of the buildings in California may not have changed so extensively during the last 40 years as they appear to have done in Europe, and so the scatter in the Goel and Chopra data may not be due to age, but it would be an interesting study to see if the scatter could be reduced by including this parameter.

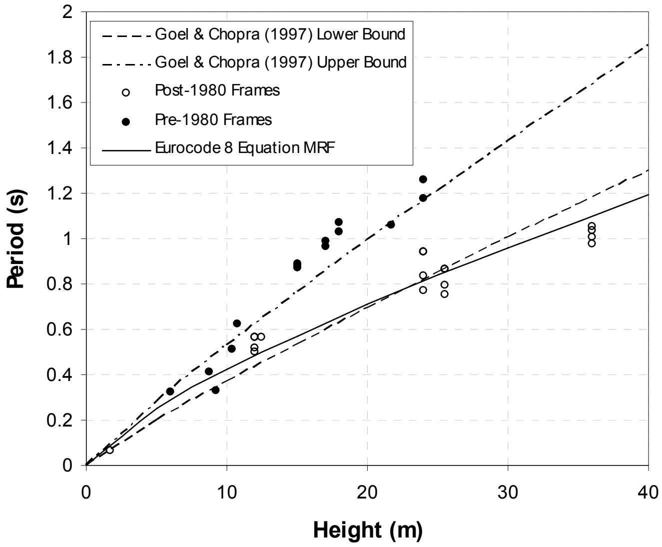

Figure 3. Upper and lower bound period-height formulae by Goel and Chopra [1997] and the EC8 [CEN, 2004] for bare MRF compared with the gross stiffness periods of vibration from European bare MRF designed pre- and post-1980 [see Crowley and Pinho, 2004; Crowley, 2003].

In Europe, it is very common to add rigid masonry infill panels to reinforced concrete moment resisting frames, both internally and externally; these non-structural elements will influence both the lateral stiffness and strength of the building. Eurocode 8 allows the designer to ignore the contribution of the infill panels to the strength of the building (by considering them as non-structural elements), and the authors believe that many designers following the recent Eurocode 8 regulations would also ignore their contribution to the stiffness. Eurocode 8 states in the modelling section that "*infill walls which contribute significantly to the lateral stiffness and resistance of the building should be taken into account*" but there is no guidance on how they should be modelled. If the designers use the linear static method to design the building then they may mistakenly use a moment resisting space frame equation to calculate the period of vibration, or otherwise use the Rayleigh method, where the deformed shape is obtained from a numerical model where the infill panels are not included. For what concerns the linear dynamic method, where Eigenvalue analyis is required to calculate the periods of a number of modes of vibration, again it is probable that these rigid elements are not being included in the numerical model.

In order to consider the influence of this type of rigid masonry infill on the period of vibration of reinforced concrete moment resisting frames, Angel [2007] added equivalent struts to the post-1980 European frame models mentioned previously; the thickness of the infill was taken as either 100mm or 250mm and the panels were modelled as either fully infilled, or with a number of openings. The periods of vibration of these four types of infilled frames are shown in Figure 3, together with the period of vibration of the bare frames.

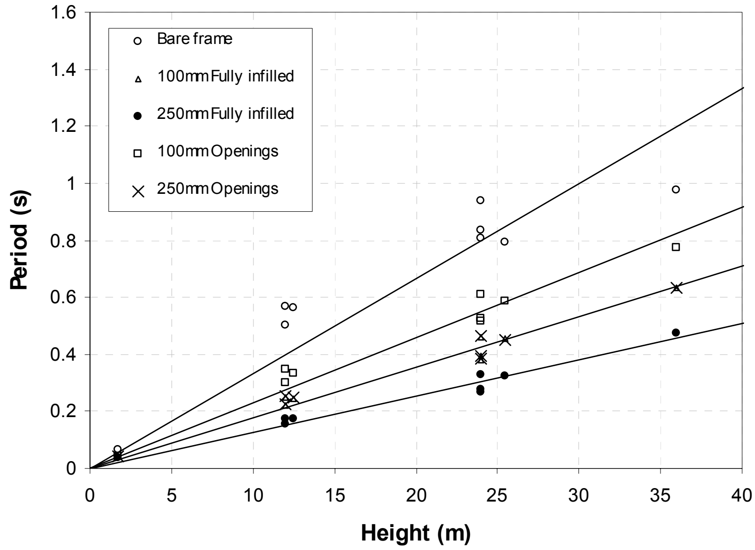

Figure 4. Periods of vibration of post-1980 reinforced concrete moment resisting frames modelling with infill panels with varying characteristics.

The distribution of the infill panels will vary within a building with some frames fully infilled, some with openings, whilst some frames will remain bare. Bal *et al.* [2008] studied the characteristics of Turkish reinforced concrete buildings and found that the proportion of bare frames, fully infilled frames and partially infilled frames in a sample of Turkish buildings to be 34%, 28% and 38%, respectively. These ratios have been used to calculate a weighted mean period of vibration for each frame presented in Figure 3, considering 50% with 100mm infills and 50% with 250mm infills. The period of vibration of the same frames has also been calculated using Eq. (9) and with the formula for "other structures" in Eurocode 8 (Eq. (1) with C_t equal to 0.05); a comparison of these code equations with the analytically calculated periods of vibration is given in Figure 4.

The results in Figure 4 show that the periods of vibration for these infilled post-1980 European buildings from numerical analysis match well both Eq. (9) and Eq. (1) (with the C_t coefficient suggested in Eurocode 8 for "other" structures). The reason that these infilled buildings agree with these equations, whereas other authors have found that they are too conservative for shear wall structures, is probably due to the low height to width ratios which arise when you have infill walls in all bays of the structure. A low height to width ratio implies a higher shear deformation contribution to the period of vibration and so this may be the reason why Eq. (9) works well.

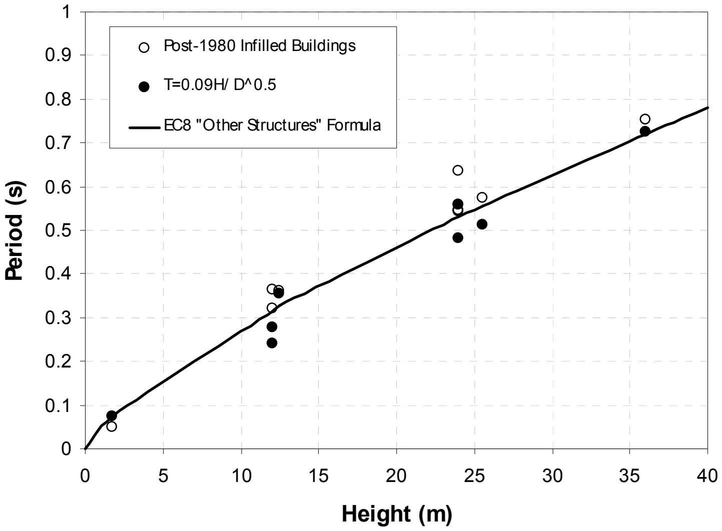

Figure 5. Periods of vibration of post-1980 “infilled buildings” based on a weighted mean of the results in Figure 2, compared with the periods predicted with Eq. (9) and the formula in Eq. (1) with C_t equal to 0.05.

3 PERIODS USED IN LINEAR STATIC AND DYNAMIC ANALYSES IN EC8

Eurocode 8 [CEN, 2004], as with most design codes, allows linear analyses to be carried out for the design of new structures. Linear analysis includes the lateral force method (static) and the modal response spectrum (dynamic) method: both types of analysis use smooth response spectra specified in the code in order to characterise the earthquake actions (see Figure 1). The modal response spectrum analysis is applicable for all types of buildings, whilst the lateral force method of analysis has many restrictions on its use due to the ‘fear’ that it would provide unconservative results in certain conditions; however, in spite of this disadvantage the method is still widely used due to its ease of application.

Essentially, the lateral force method of analysis may be applied to buildings whose response is not extensively affected by the contribution of higher modes of vibration. According to Eurocode 8, such buildings follow two conditions: firstly, the fundamental period of free vibration in the principal directions is smaller than either four times the corner period ($4T_c$, where T_c varies from 0.25s to 0.8s) or two seconds; and secondly, the building is regular in elevation. The lateral force method allows the use of the simplified relationships described in Section 2 for the estimation of the period of vibration of the building under design. For those buildings for which the lateral force method cannot be applied, in many cases it is possible to apply the modal response spectrum method. In order to calculate the periods of a number of modes of vibration, the designer would generally produce a numerical model of the building and carry out Eigenvalue analysis.

Recent studies [Angel *et al.*, 2006; Pinho *et al.*, 2007] have shown through a number of examples how the application of these two methods to a given building leads to different design base shear forces and shear force profiles. Examples of the shear force profiles obtained with the two methods for two post-1980 European frames are given in Figure 5.

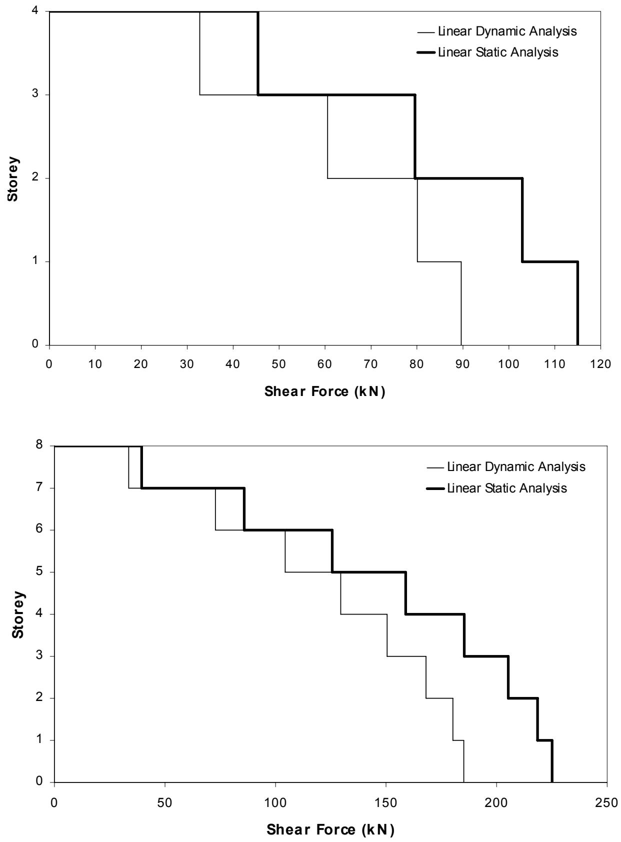

Figure 6. Shear force profiles for two reinforced concrete buildings designed to Eurocode 8 obtained with the linear dynamic and static procedures (Pinho *et al.*, 2007).

The main reason for the differences in the base shear force in the plots is Figure 5 is the disparity between the fundamental period of vibration from the period-height equation and the period of vibration from Eigenvalue or Rayleigh analysis of a bare frame model, where the stiffness of the sections may be reduced by up to 50% [CEN, 2004]. Many codes recognise that the period of vibration from the simplified period-height equation is more realistic, having been directly obtained from the measured periods of vibration of buildings subject to earthquake ground motions, but that when higher modes are important (in tall and/or irregular structures) the modal response spectrum method gives a more realistic profile of the lateral forces. Hence, these codes (e.g. ASCE [2006]; NBCC [2005]) require the designer to check whether the modal base shear force is less than 85% of the base shear force from the equivalent lateral force method. If this is the case then the modal forces, but not the drifts, should be multiplied by $0.85V/V_t$ where V is the base shear from the lateral force method and V_t is the base shear from the required modal combination.

Even when higher modes are not important and the designers are allowed to use the linear static method, but they decide to calculate the period of vibration from the Rayleigh method, many codes apply an upper bound to the period of vibration from the Rayleigh method. This is another procedure which is used to safeguard against unrealistically high periods of vibration used in the design to lower the base shear forces. In ASCE 7 (ASCE, 2006) the period of vibration of the building calculated by the designer (perhaps with a numerical model with members of reduced stiffness) should not be higher than C_u multiplied by the period from the standard period-height equations such as those presented in Section 1. This coefficient C_u varies from 1.4 to 1.7 as a function of the design spectral response acceleration at 1 second; the higher the spectral acceleration the lower the coefficient C_u. The empirical equations have been derived using buildings in areas with higher lateral force requirements and the aforementioned variation of the coefficient C_u is intended to reflect the likelihood that buildings in areas with lower lateral force requirements will be more flexible and hence it is not necessary to be so stringent on the period to be used for design. Furthermore, it results in less dramatic changes from present practice in lower risk areas [FEMA, 2003].

Eurocode 8 [CEN, 2004] does not include either of these clauses, which the authors believe provide a rational method to avoid unrealistically low seismic design forces, whilst at the same time allowing the drifts and the higher mode effects to be realistically modelled.

4 RECOMMENDATIONS FOR FUTURE DRAFTS OF EUROCODE 8

The main recommendation of this paper is that future revisions of Eurocode 8 [CEN, 2004] should consider the following points related to the periods of vibration and forces obtained in linear analysis:

- When presenting the period-height equations for reinforced concrete moment resisting frames (MRF) there should be two formulae which depend on whether (a) the infill panels are to be isolated from the MRF or (b) the infill panels are to be rigidly connected to the MRF.
- The period-height equation for bare MRF that is currently in Eurocode 8 appears to be reasonable and should not necessarily be updated with the Goel and Chopra [1997] equation, especially considering that this equation is based on buildings constructed over many years and thus to different design codes.
- The period-height equation for "other structures" that is currently included in EC8 [CEN, 2004] could be used for the period of vibration of the MRF will rigid infill panels, as could Eq. (9) with $C_t = 0.09$.
- The error which the authors appear to have found in the period of vibration equation for shear wall buildings should be rectified and this equation could be updated using the more recent equation proposed by Goel and Chopra [1998]. However, it would be interesting to compare this equation with analytically calculated periods of vibration for modern shear wall buildings; this is something which the authors hope to carry out in future research.
- The period of vibration calculated with the Rayleigh method (in linear static analysis) should be limited based on the period of vibration from the period-height equations.
- The base shear force obtained with the modal response spectrum method should be scaled up using the base shear force from the lateral force method; this will safeguard against low forces from the use of analytical models with unrealistically high periods of vibration.

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