SEISMIC DESIGN AND PERFORMANCE ASSESSMENT OF MASONRY INFILLED R/C FRAMES

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SUMMARY

The study evaluates the effect of applying the Eurocode 8 provisions to multistorey reinforced concrete frames with different arrangements of masonry infill walls (uniformly infilled or with an open ground storey). The behaviour of the infilled frames to a number of input motions is studied using a newly developed element for the masonry infills, and it is found to be generally equally satisfactory to that of the corresponding bare frame, even when infills are discontinued at the ground storey. A simple design procedure for infilled frames is tentatively proposed, based on a dual stiffness assumption for the infills, one for calculating design forces and a smaller one for calculating the action effects for the various members.

INTRODUCTION

It is now widely recognised that masonry infill panels used for cladding and/or partition in reinforced concrete (R/C) buildings, significantly alter their seismic response, and their effect should be duly accounted for in the design. However, related code provisions hardly include any detailed guidance as to how this should be done. Given the rather detailed provisions included for bare frames and other common R/C structural systems, and the advanced analytical tools currently available to designers, the paucity of code provisions for infilled frames should be primarily attributed to the incomplete understanding of their role, as well as to the numerous uncertainties involved in modelling the effect of infills, particularly their interaction with the surrounding frame. It is indeed surprising that there still appears to be no consensus among code developers as to whether the effect of common masonry infills is generally favourable or unfavourable from the seismic performance point of view. Most codes recognise that irregular arrangement of infill walls (in plan and/or in elevation) has an unfavourable effect on the R/C system, and sometimes penalty factors are specified for the regions where the irregularity occurs. On the other hand, there are clearly different approaches to the treatment of masonry walls arranged in an essentially regular (uniform) fashion. Even in countries like New Zealand, where isolation of infills from the surrounding frame has been used with a view to maintaining the favourable structural characteristics resulting from the application of capacity design procedures, there is an apparent change of attitude, in favour of interacting infills [Paulay and Priestley, 1992], as it is difficult and expensive to maintain lateral (out-of-plane) stability of the isolated panels.

Eurocode 8 [CEN 1994] is (to the authors' best knowledge) the first code that introduced rather detailed provisions for the design of infilled R/C frames (and 'frame-equivalent' dual structures), mainly along the lines of penalising them with respect to corresponding bare frames. The effect of these provisions on the seismic performance has not been studied yet; only as part of a recently completed study [Fardis 1997], the effect of applying some of the relevant EC8 provisions (increase of design actions in soft-storey columns), has been investigated with the aid of nonlinear analysis of a series of R/C infilled frames with different heights.

The objectives of the present study are to evaluate the effect of applying the full set of EC8 provisions to a multistorey R/C frame with different arrangements of masonry infill walls, and to propose possible refinements to the design procedure, including explicit consideration of the infills with appropriate stiffness values.

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DESIGN CONSIDERATIONS

The reference structure studied is a ten-storey, three-bay R/C frame (Fig. 1), previously designed according to the EC8 ductility class M (‘medium’) provisions for bare frames [Kappos 1997], for a design ground acceleration of 0.25g. The structure has been assessed both as a bare frame and as an infilled one [Kappos et al. 1998]; in the latter case infill walls were considered only as gravity loads at the design stage, according to standard practice. It was found that the seismic performance of the structure was generally satisfactory for various limit states, but a significant amount of damage was predicted for the ground storey columns when infill walls were discontinued at that storey (‘pilotis’ building).

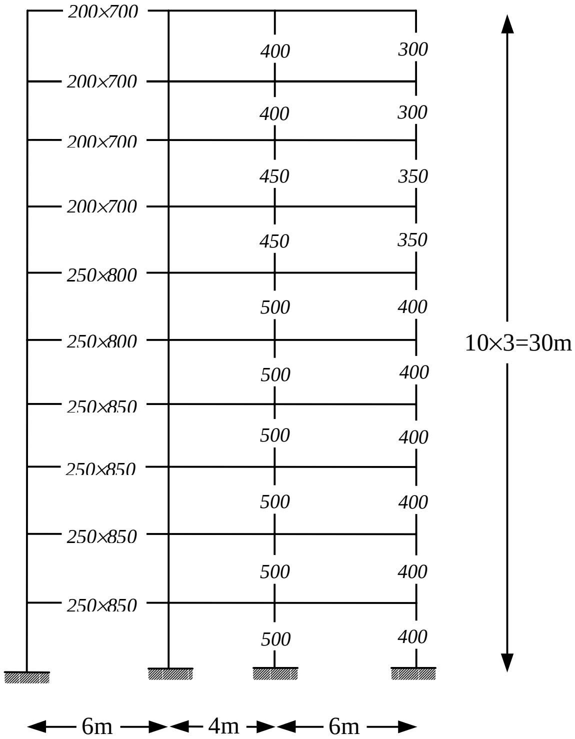

Figure 1. Geometry of bare frame and dimensions of member cross-sections (in mm)

In the framework of the present study the ten-storey frame was redesigned for various scenarios of infill wall arrangement, and the provisions of EC8 Part 1-3 §2.9 were applied in full. The scenarios considered were as follows:

- Fully infilled frame, or pilotis (i.e. with open ground storey)
- ‘Weak’ or ‘strong’ brick masonry infill; in line with common S. European practice, the former had a compressive strength of 1.5 MPa and the latter of 3.0 MPa (shear stresses at cracking of 0.19 and 0.27 MPa, respectively).
- Thickness of infill wall 90 mm (partition) or 200 mm (cladding or strong partition).

The first design implication of the presence of infills is that they increase the fundamental period of the structure, which typically leads to increased seismic actions. EC8 specifies that these actions should correspond to a period T_1 equal to the average of that of the bare frame and the infilled frame (it is understood that both refer to the elastic range). A series of empirical formulae are included in EC8 Part 1-3 §2.9.4 for the calculation of T_{1i}, the period of the infilled frame. Table 1 lists the periods calculated according to the code procedures as well as from more rigorous analysis, using appropriate models for the infill panels, as described later. Note that T_{1b} refers to the fundamental period of a bare frame, T_{1i} to that of an infilled frame, while the code value $T_{1,des} = (T_{1b} + T_{1i})/2$.

Table 1. Fundamental natural periods of bare and infilled frames

Frame Type	Infill Panel Thicknes	Weak Masonry T_{1i} (s)	Strong Masonry T_{1i} (s)	Bare Frame T_{1b} (s)	Code Value T_{1i} (s)	Code Value $T_{1,des}$ (s)
Bare Frame	-	-	-	0.98	0.96	-
Fully infilled	90	0.64	-	-	0.61	0.78
Fully infilled	90	-	0.53	-	0.48	0.72
Fully infilled	200	0.51	-	-	0.45	0.71
Fully infilled	200	-	0.42	-	0.35	0.65
Pilotis	90	0.65	-	-	0.61	0.78
Pilotis	90	-	0.56	-	0.48	0.72
Pilotis	200	0.55	-	-	0.45	0.71
Pilotis	200	-	0.47	-	0.35	0.65

The first remark from Table 1 is that the empirical code formulae generally provide reasonable predictions of the infilled frame period; the larger discrepancy was found for the pilotis frame with strong infill walls of 200mm thickness (code value 25% lower than the ‘exact’ one). The most important remark, though, refers to the large difference between the period of the bare frame (0.98sec) and that of the infilled frame; the latter is from 34% to 57% smaller, which is equivalent to increases in stiffness of 127% to 444%. It is clear that, conditional upon the spectral characteristics of the design earthquake, the dynamic behaviour of the two systems (bare vs. infilled frame) can be dramatically different. EC8 bases the calculation of seismic actions for infilled frames on the average periods ($T_{1,des}$ in last column of Table 1), but displacements should be calculated considering the bare frame alone. Depending on the thickness of infill panels, the calculated base shear coefficient for the 10-storey frame was 22% to 30% higher than that for the bare frame.

The second important EC8 provision for infilled frames refers to the effect of irregularities in elevation, resulting when part of, or all, infill panels are missing at certain storeys. The action effects (member forces) in these storeys are then increased by a factor $\alpha = (1 + \Delta V_{Rw} / \Sigma V_{Sd})$ where ΔV_{Rw} is the total reduction of the masonry panel resistance (compared to the more infilled storey closest to it) and ΣV_{Sd} is the sum of seismic shear forces in the vertical members of the storey with missing infills. In the structure studied, the foregoing provision applies to the ground storey when a pilotis system is considered. In this case the calculated α–factor was equal to 1.32. EC8 is not very clear as to whether this should multiply both beam and column action effects, but experience has shown and recent analytical studies [Fardis 1997] have verified that the members affected are the columns of the weak storey, hence the 32% increase was applied to these members of the ground storey (beams were kept the same as in the fully infilled frame).

The effect on the R/C member design of applying the EC8 provisions for masonry infilled frames can be evaluated by comparing the reinforcement provided in the members for the bare frame case and the fully infilled case; this is done for the members of the first and second storeys in Table 2. It is seen that the required reinforcement increases substantially in the case of columns (for both longitudinal and transverse reinforcement), whereas increases in beam reinforcement are moderate (up to about 25%). It is pointed out that increases are generally less significant in the upper part of the structure, particularly for beams. The dimensions of R/C members in the infilled frames were kept as similar as feasible to those of the bare frame. They were changed only whenever the required reinforcement ratio exceeded about 3% (limit selected on the basis of construction ease as well as ductility considerations). This resulted in increasing by 50 mm the size of the columns (shown in Fig. 1) in the top two storeys of all infilled frames, and of the ground storey in the pilotis buildings. It is worth pointing out that according to EC8 the stringent confinement provisions applicable to critical regions of columns (typically located at their ends), apply to the entire length of the columns in the storeys where infills are significantly reduced, as well as to all corner columns. It is clear from all the previous considerations that application of the EC8 provisions for masonry infilled frames results in a substantial increase in the cost of the R/C structural system, hence it is worth investigating whether this additional cost results into a commensurate improvement in their seismic performance.

Table 2. Reinforcement increase between bare frame and fully infilled frame

Member Location	Bare frame	Fully infilled frame	% Increase
Ext. beam top reinf.,	804	1005	25
Ext. beam, bottom	603	770	27
Ext. beam, shear reinf.	ϕ6/100	ϕ6/100	0
Int. beam, top reinf.	957	1206	26
Int. beam, bottom reinf.	756	864	14
Int. beam, shear reinf.	ϕ6/100	ϕ6/100	11
Ext. column, long. reinf.	2272	5323	134
Int. column, long. reinf.	4032	8035	99
Ext. column, shear	double ϕ10/90	triple ϕ10/70	180
Int. column, shear reinf.	triple ϕ10/110	quatriple ϕ10/90	113

ASSESSMENT OF SEISMIC PERFORMANCE

Modelling assumptions

The two types of infilled frames (fully infilled and pilotis) designed as described in the previous section were then assessed by analysing their inelastic response to a number of input motions. Standard member-by-member modelling, with concentrated plastic hinges at the ends, was adopted for the R/C beams and columns [Kappos 1997, Kappos et al. 1998]. The bilinear version of the well-known Takeda model incorporated in DRAIN-2D/90 [Kappos 1996] was used for all R/C members, except the exterior columns, for which a simpler bilinear model but with ability to account for yield moment-axial load interaction was preferred. Masonry infills were modelled using the four-node shear-only element developed by Kappos et al. [1998], each of the four nodes connecting to a beam-column joint of the surrounding frame. The key feature of the element is its refined hysteretic behaviour, incorporating strength and stiffness degradation, and pinching effects. Moreover, the use of this simple finite element instead of the commonly used pair of diagonal struts permits accounting for the effect of inelastic behaviour in one direction on the behaviour in the other direction, a feature disregarded in recently developed equivalent diagonal strut models [Fardis 1997].

Input motions

To account for the sensitivity of calculated inelastic response to input motion characteristics, four accelerograms were used, recorded in Greece during two of the most damaging earthquakes occurred in the last twenty years (Thessaloniki, 1978, Kalamata 1986). The motions were recorded at sites close to the earthquake epicentre (10 to 30 km). All records were first normalised to the spectrum intensity of the design earthquake, using a modified Housner technique suggested by Kappos [1991]. The resulting scaling factors were higher than the ratios of design acceleration (0.25g) to peak acceleration of each record. Further scaling of the records was carried out when analysing the response of the structures to the serviceability earthquake (taken as 0.4 times the design one, following the EC8 recommendations) and the ‘survival’ or ‘non-collapse’ earthquake (taken as twice the design one). The durations considered for analysis were up to the release of 95% of the input energy of each record.

Main results

Probably the most critical response quantity is the interstorey drift, which in this case is also equal to the shear strain in the infill panels. The peak interstorey drifts calculated along the height of the two infilled frames (fully infilled and pilotis) with 90 mm partitions, subjected to various earthquake intensities, are shown in Fig. 2; similar patterns were found for the 200 mm panel structures. Note that the values shown are the average over the four earthquakes, hence the effect of variability to specific characteristics of the input (which was relatively important) is essentially filtered out.

It is seen that under the serviceability earthquake (0.10g) the interstorey drifts remain below 0.2% along the entire height for both buildings. Bearing in mind that the cracking strain of the brick masonry infills was 0.08% (assuming good wedging against the surrounding frame) and the strain at peak load 0.36%, it follows that for this limit state (LS) cracking of the panels is expected in the lower ¾ of the height, but otherwise they remain well below ultimate conditions. It is interesting to note that application of the EC8 procedure leads to a very satisfactory performance of the pilotis structure, essentially the same as that of the fully infilled frame. No R/C column yielding was recorded at this LS, and beam yielding was sparse and almost negligible.

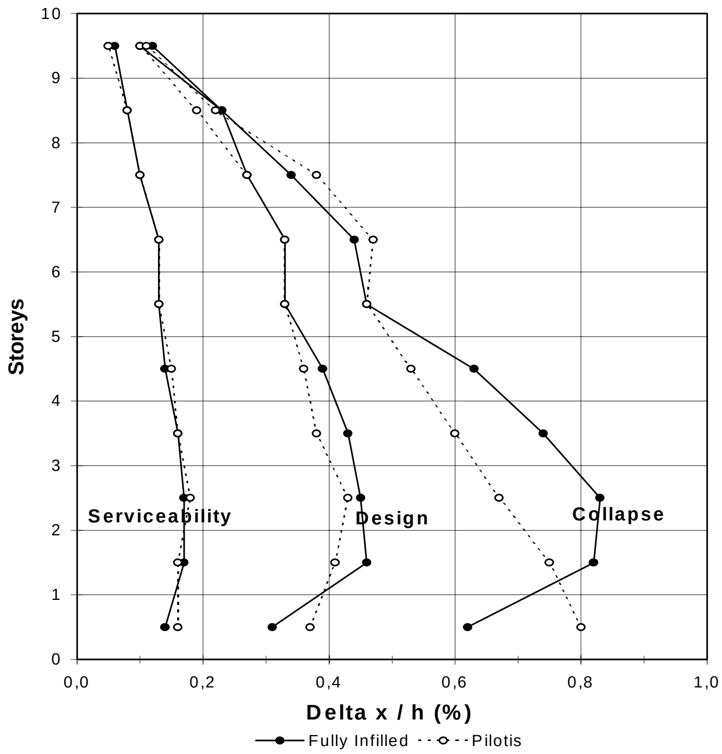

Figure 2. Drift patterns at different limit states for infilled frames.

Under the design and 'survival' LS, drifts increase as anticipated, but there is no marked difference in their distribution. It is now clear that the pilotis building has a larger drift at the ground storey but, importantly, this is not larger than the peak drift recorded in the fully infilled frame (typically just above the ground storey). Hence the performance of both frames appears to be essentially the same, which is in sharp contrast with the conclusions of previous studies [Kappos et al. 1998], where it was found that pilotis structures not strengthened to EC8 provisions (i.e. designed as bare frames) show dramatic increases in the ground storey drift with respect to corresponding bare and fully infilled structures. Under the design LS earthquake these pilotis had an average drift of 1.3%, which should be contrasted to the just over 0.4% shown in Fig. 2 for the EC8-designed structure. Even under the survival LS earthquake, peak drifts for both infilled frames remained below about 0.8%, which is in fact a very satisfactory seismic performance. Note that masonry infills reach at this stage strains about twice those corresponding to peak stress, which is accompanied by a moderate reduction in strength, but the panels are still quite far from failure.

As far as R/C members are concerned, no failure due to flexural ductility (i.e. exceedance of plastic rotation capacity) was recorded for any structure studied. Fig. 3 compares average values of required and available peak plastic hinge rotations in the interior columns of the infilled frames subjected to the design earthquake intensity, and it is seen that the safety margins are more than satisfactory. Under the survival earthquake (0.5g), the requirements almost doubled but they were still well below the corresponding capacities. In the case of columns this was even more the case, since, due to capacity design, plastic hinging was very limited in these elements (essentially confined to the ground storey); safety factors against ductility failure were at least 2.5, even under the survival earthquake and no trend for column sidesway mechanisms was detected. It is pointed out that corresponding safety factors were found to be higher in the case of the bare frame, the main difference being the axial loading in the columns. Due to the presence of infill walls, both tension and compression axial loads are higher, hence leading to earlier yielding and (more importantly) to lower available ductility of the columns. However, as shown previously [Kappos et al. 1998], if analysis includes the vertical compliance of the foundation, axial loads in columns of infilled frames are much closer to those in corresponding bare frames.

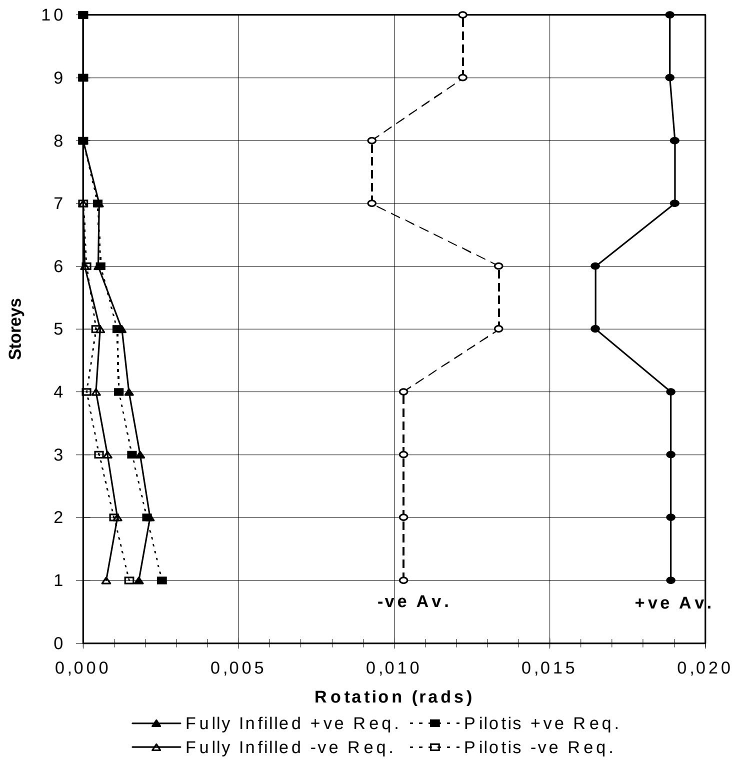

Figure 3. Required and available plastic rotations at the interior beam supports

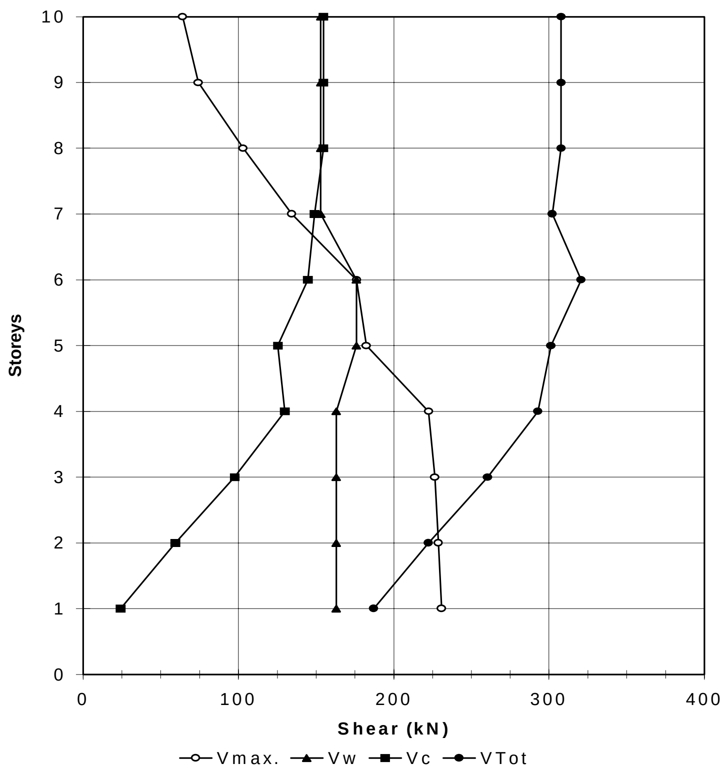

Figure 4. Required and available shear capacities at the interior beams - Pilotis frame

Some concern arises, though, when the shear capacities of the R/C members are evaluated. Fig. 4 compares the capacities $V_{tot}=V_c+V_w$ of the most critical beams with the corresponding maximum shears recorded from the dynamic analysis; V_w is the contribution to shear resistance of the transverse reinforcement, while V_c is the 'concrete' contribution, calculated with due account for the effect of inelastic flexural response [Kappos 1997]. V_{max} refers to the average over the four earthquakes, scaled to the design intensity. It is seen that in the lower two storeys, shear resistance of the beams is lower than the corresponding capacity, the main reason being the substantial drop in V_c, which is negligible, when flexural ductility exceeds about 4. For the survival earthquake shear force demands increased by about 10%, hence the problem was more acute in this case. It is noted, though, that this is not a deficiency particular to infilled frames; it was detected previously [Kappos 1997] as a problem in bare frames as well, and it is currently considered by the committee responsible for converting EC8 into a European standard (EN) to rectify this by introducing capacity design procedures for shear in DC 'M' beams.

A very useful global picture of the seismic performance of the studied infill frames can be obtained by referring to the energy dissipated by each component of the structural system, shown in Fig. 5 as a function of the earthquake intensity considered. It is clear that at the serviceability level over 95% of the energy dissipation is taking place in the infill panels (subsequent to their cracking), whereas at higher levels the R/C members start making a significant contribution. The latter concerns mainly the yielding beams, but in the pilotis frame columns play also a role in dissipating energy, particularly at the survival earthquake level; for this high intensity the infills dissipated only 40% of the seismic energy, the rest 60% being dissipated by R/C members, mainly the beams This is a clear verification of the remark that masonry infill panels act as a first line of defence in a structure subjected to earthquake attack, while the R/C system is crucial for the performance of the structure to stronger excitations (beyond the design earthquake).

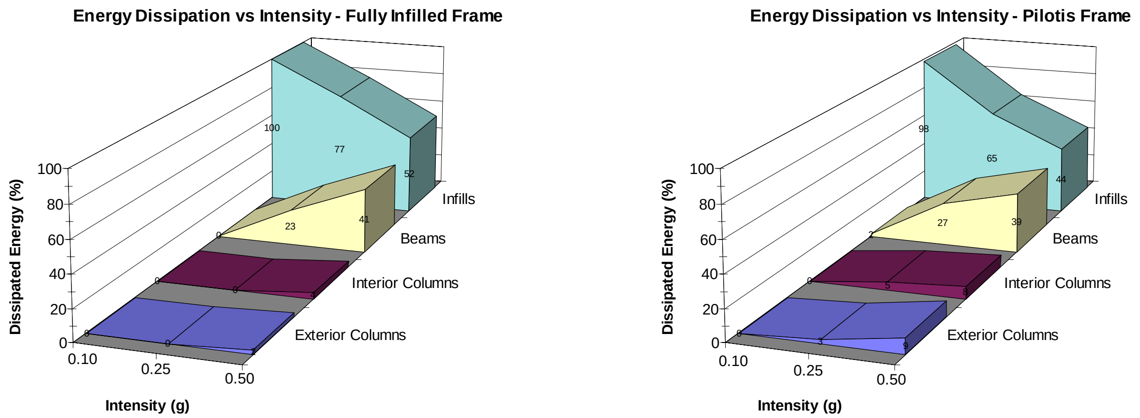

Figure 5. Energy dissipation in each component of the infilled frames

RECOMMENDATIONS FOR DESIGN

On the basis of the results of this and previous [Kappos et al. 1997, 1998] studies, it is believed that it is now time to explicitly account for the effect of infill walls when designing R/C frames. This means that the analytical model used in the design process should directly incorporate the infills, rather than being based on the analysis of the bare frame with essentially empirical corrections (magnification) of the design actions, as suggested in EC8. A model such as that used by the authors or, alternatively, an equivalent strut approach, can be used to this purpose. The critical issue is the assumption to be made regarding the stiffness of the infills. It is clear from the results presented here (and also from other studies) that depending on the LS considered, this stiffness may be quite different, and it is important to consider more than one performance level (or limit state), depending on the design parameter to be estimated.

In the light of the above, it is proposed to design infilled frames on the basis of a model including beam, column, and panel (or strut) elements, using at least two different stiffness assumptions for the latter. The basis for selecting these stiffnesses should be the trilinear sceleton curve (break points at cracking and peak stress, which is followed by a descending branch) adopted for the infill panel model proposed by Kappos et al. [1998] for brick masonry walls, or equivalent models pertinent to other types of infills. Design forces (base shear) should be calculated assuming the secant stiffness at peak load for the infill panels; this should give an appropriate level of strength to the structure (neither overconservative nor underconservative). These forces should then be distributed to the members of the structural system assuming a lower stiffness for the infills, typical to that found in this study when considering the ‘survival’ LS. It is provisionally (and conservatively) suggested to base this stiffness at the secant value corresponding to three times the strain at peak loading. This will result in a relatively small part of the base shear being assigned to the infills, while the rest would go to the R/C members. Note that in this analysis the effect of irregular arrangement of infills is accounted for in a straightforward way. Calibration studies are currently in progress to verify and refine this approach, and will be reported in the future.

CONCLUSIONS

The present study, concerning multistorey R/C frames with masonry infills, has clearly demonstrated that if this type of structure is designed according to EC8 provisions, it will perform better than the corresponding bare frame at the serviceability LS, and equally satisfactorily at the design and survival LS. The dramatic increase in column ductility requirements at the open storeys of pilotis buildings found in infilled frames designed ignoring the effect of infill walls, is not present in the EC8-designed structures. However, this improvement comes at a significant cost, since longitudinal and transverse reinforcement in the R/C members (especially the columns) is significantly increased with respect to the bare frame. Importantly, this is not true only for the irregular (pilotis) frames, but also for the regular, uniformly infilled ones. Therefore, it appears that the EC8 procedure is overconservative, since it penalises infilled structures by increasing design action effects but completely disregards the contribution of infills in carrying these increased effects. Such a conservatism might be justified at this stage, due to lack of appropriate alternative procedures and pending a probabilistic assessment of the performance of infilled frames (such an effort is currently being undertaken by the first author and his co-workers).

Looking towards the future, it appears that the only feasible way to account for the negative effects of irregularly arranged infill walls, but also for the positive effects of the infills which, as shown here, act effectively as a ‘first line of defence’ in the structure, is to directly include the infill walls in the analytical model used for design. Simple models are currently available to this purpose, and one of them has been used and tested in the course of the programme, part of which is the work presented in this paper. A simple design procedure for infilled frames is tentatively proposed here, based on a dual stiffness assumption for the infills, one for calculating design forces and a smaller one for calculating the action effects for the various members.

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