DESIGN PROVISIONS FOR MASONRY-INFILLED RC FRAMES

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SUMMARY

Conclusions of recent European-Commission-sponsored research on the seismic response and design of infilled RC structures are summarized and their implications for the revision of Eurocode 8 are presented. The research, mainly analytical/numerical, covers regularly or irregularly infilled structures, focusing on the effects of infills on global response. For regularly infilled structures the overall effect of infills is found to be beneficial and designing the structure as bare suffices. For the design of structures with infills irregularly arranged in plan or elevation, more economic and effective alternatives to the current Eurocode 8 rules are proposed.

INTRODUCTION

Infilling RC structures with masonry walls is quite common in Europe, especially in the seismic prone Southern countries. Masonry infills are typically built with hollow bricks, although hollow concrete blocks are not uncommon. Dimensions and void ratios of masonry units vary considerably throughout Europe, and so does the in-plane strength and other mechanical properties of infills which are of importance for seismic loading. As far as the in-plane strength is concerned, what matters most is the shear strength of continuous bed joints, which is typically measured on wallettes in diagonal compression. Values of shear strength range from around 0.2MPa to over 1.0MPa, depending on the void ratio of the blocks, the direction of the perforations relative to the bed joints, the filling or not of head joints, etc. Nevertheless, in Europe masonry infills seem to be on the average considerably stronger and heavier than in North America. Therefore their role in earthquake-resistant structures is considered as very inportant.

Eurocode 8 (Part 1-3) is the first seismic design code to include specific rules for RC structures with masonry infills: Sect. 2.9.3.1 refers to strongly irregular arrangement of infills in plan and calls for including them explicitly in a 3D model for the calculation of the seismic action effects; Sect. 2.9.3.2 gives rules for increasing the resistance of structural members in storeys with less infills than in the neighbouring ones; Sect. 2.9.4 requires increasing the design seismic action effects of infilled structures due to the stiffening influence of the infills; finally Sect. 2.9.5 gives design and detailing rules for RC elements against the adverse local effects of infills. This paper summarizes the prenormative research on infilled RC structures sponsored in recent years by the European Commission.

SDOF IN-PLANE SEISMIC RESPONSE OF INFILLED ELASTIC FRAME

For the purposes of understanding the fundamental dynamic behaviour of infilled frames under in-plane seismic loads, the simplest possible infilled frame, consisting of a 5%-damped elastic frame acting in parallel (i.e. having the same horizontal displacement, acceleration, etc. at the top and sharing the base shear) with a nonlinear infill panel, was modeled and analysed as a SDOF system. Parametric studies of the effect of: a) the uncracked stiffness, b) the cracking and ultimate strength of the infill, c) the nonlinear model used for the infill panel, and d) the input motion, were performed on this SDOF nonlinear system, by constructing its acceleration and

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displacement response spectra for each parameter value or case. Alternative cyclic nonlinear models used for the infill panel were: A) a rigid-perfectly plastic model in primary loading, unloading rigidly to the horizontal axis and then horizontally (i.e. with zero resistance) to the origin, and following a similar behaviour in reloading in the opposite direction; B) a realistic model with a multilinear primary loading or envelope curve (including "uncracked", "cracked", "post-ultimate" and "residual strength" behaviour) and multilinear inverted-S-shaped hysteresis loops with strength and stiffness degradation due to cycling. Model A corresponds to an equivalent viscous damping ratio of $1/\pi$=32%, while for the parameter values producing the best fit to test results, model B gives a damping ratio around 25% in the first post-cracking cycle or 4-5% in subsequent ones.

The parametric studies show the following [Fardis, 1996], [Fardis & Panagiotakos, 1997b]: a) The peak response of the infilled system is affected mainly by the strength of the infill panel, less (and depending on motion characteristics) by the damping inherent in the hysteresis, even less by its post-ultimate behaviour and very little by its uncracked stiffness. b) The details of the hysteretic behaviour of the infill, as reflected by its model, are important under long duration multi-cycle motions conforming fully to a smooth code-like spectrum, but become unimportant under shock-type excitations with few large cycles driving the infills into large post-cracking excursions. c) The predominant period of nonlinear response is not shortened appreciably by the presence and stiffness of the infills: for long-duration motions fully conforming to smooth response spectra, the effective period of the response, T_{eff}, starts decreasing with respect to that of the elastic frame, T_{fr}, only for T_{fr}>0.8sec and decreases appreciably only for $T_{fr}\geq$2.0sec. For such excitations and considering for simplicity that $T_{eff}\approx T_{fr}$, the infill simply causes a reduction of the peak displacement and acceleration response of the elastic frame, equivalent to that effected by an increase in damping ratio from ζ=5% to:

$$\zeta(\%) = 5 + 2.5(2.5 + 5T_{fr} - T_{fr}^2)\frac{f_{inf}}{mS_a} \qquad [1]$$

In Eq.[1] f_{inf} is the infill ultimate strength, m is the system mass and S_a the 5%-damped spectral acceleration of the input motion in the constant-acceleration range. If the elastic spectrum of the motion has large peaks and valleys, the effect of infilling on a SDOF structure cannot be attributed simply to an increase in energy dissipation of the type of Eq.[1], as even small differences between the periods of the elastic frame T_{fr} and the effective one of the nonlinear response. T_{eff}, may affect significantly the response.

REGULARLY INFILLED MULTISTOREY BUILDINGS UNDER BIDIRECTIONAL MOTION

For the purposes of studying the effects of regular infilling on the global seismic response of multistorey buildings, as well as the possibility that out-of-plane expulsion of infill panels may render the infilling irregular, twelve RC frame structures in 3D were subjected to nonlinear dynamic analyses. Excitation consisted of four pairs of bidirectional synthetic motions, conforming fully to the design spectrum and scaled to the design ground acceleration of 0.15g or 0.3g, times an intensity factor of 1.0 or 2.0. The 12 buildings comprise three groups of four buildings each: a group of 4-storey, another of 12-storey and a third of 3-storey buildings, all designed in detail as bare structures according to the provisions of Eurocodes 2 and 8 [Fardis & Panagiotakos, 1997a]. The buildings in each group of four were designed to different combinations of Ductility Class (DC) and ground acceleration. All structures are regular and symmetric in plan and elevation, except of the 3rd storey of the 3-storey ones, which is shorter and supports a steel roof. In the nonlinear analyses they were considered either as bare or with their exterior frames fully infilled in all storeys. The total infill strength in a storey in these combinations of buildings and infills is equivalent to a storey shear coefficient ranging from 0.035 to 0.17, compared to a design base shear coefficient of the bare structure between 0.08 and 0.195. Infill panels were assumed solid (without openings) with a height-to-thickness ratio (slenderness) equal to 23 in the 4-storey and 12-storey buildings, or ranging from 29 to 36 in the 3-storey ones.

A member-type one-component lumped inelasticity model is used to model beams and columns, with Takeda-type hysteresis rules and pre-yield stiffness computed from the secant stiffness to yielding of both ends in antisymmetric bending and with a large width of the slab considered as contributing to the flange of the beams in tension and compression. In columns the coupling and interaction of nonlinearity between the two directions of bending, as well as the effect of the variation of axial load, were neglected. The model used for the infill panels consists of: a) diagonal struts for the in-plane behaviour, following the realistic degrading cyclic force-deformation model B of Section 2, and b) a SDOF elastic-brittle model for the out-of-plane dynamic response and failure due to out-of-plane excitation of the panel by the transverse motion of the surrounding frame. The work-equivalent mass and stiffness of this SDOF system are defined so that the total lateral force on the panel is reproduced: As described in detail in [Fardis 1996] and [Fardis et al. 1998], comparison with measurements of stiffness, natural frequency and failure load of infill panels in the out-of-plane direction, shows that their elastic force-deformation properties and strength can be computed on the basis of the two-way arching action models by [Bashandy et al. 1995].

To study the vulnerability of panels to out-of-plane expulsion and the influence of panel out-of-plane vibration on global response, two sets of analyses of the infilled structures were performed: In one the infill out-of-plane vibration was activated and in the other it was not. This gives a total of 2 (motion intensities) x 4 (bidirectional motions) x 12 (buildings) x 3 (cases) = 288 nonlinear dynamic analyses in 3D.

The conclusions of the nonlinear analyses are the following [Fardis, 1996], [Fardis & Panagiotakos, 1997b]: a) The frequency content of the response is controlled by the RC structure, as in most storeys infills crack and separate from the frame early in the response. b) Most of the energy dissipation takes place in the infills and structural damage in beams and columns is very low. c) Global seismic forces and shears in the infilled structures are not higher than those in the bare structure, as the beneficial effect of energy dissipation in the infills prevails over the adverse stiffening effect. d) Serviceability limits on storey drifts are widely violated in the bare structures, but are respected in the infilled ones. e) In infilled structures lateral drifts tend to concentrate in the bottom storey; nevertheless even at twice the design ground acceleration ground-storey column deformations are well below those corresponding to soft-storey formation. f) In-plane deformation and damage of infills increase from the top of the building to the bottom, but the proximity to out-of-plane collapse follows the reverse pattern. g) Out-of-plane vibration of infill panels is well below the threshold of out-of-plane expulsion and has a beneficial effect on global response, as it reduces the participating mass in the fundamental modes of vibration and hence all global response measures. h) The effect of infills is often so dominant, that the Ductility Class or the ground accleration for which the bare frame was designed play a minor role in the response. i) The stronger are the infills, the larger their effect on global and local response. Conclusions a) to e) are in full compliance with the results of a pseudodynamic test on a 4-storey two-by-two-bay fully infilled frame structure at the ELSA laboratory of the European Commission in Ispra (I), under a spectrum-compatible unidirectional motion with a peak ground acceleration of 0.45g [Negro et al. 1997].

An effort was made to predict the peak inelastic member deformations in the most critical ground storey of the fully infilled structures, from the elastic deformations of a bare structure with viscous damping given by Eq. [1] and static analysis under equivalent lateral forces with inverted triangular distribution [Panagiotakos, 1998]. This calculation was found to underpredict inelastic deformation by up to 60%. Comparison of the peak deformations of the 4-storey ELSA structure, tested bare and fully infilled (with estimated infill strength about 8% fo the total weight of the building and T_{fr}=1.16sec), leads to the same conclusion. It seems that in multistorey buildings infills have overall a more beneficial effect on the demands on structural members than in one-storey SDOF-like structures. This is more so in the upper storeys.

The most important implication of the above for the design of regularly infilled RC structures is that it is safe enough and indeed conservative, to design such structures as bare, ignoring the effect of infills on the global response. This conclusion is fully in line with current displacement-based thinking for seismic design. The requirement of Sect. 2.9.4 of EC8, Part 1-3, to base calculation of seismic internal forces on the average of the periods of the bare RC structure and of the infilled with uncracked RC members and infills, and to neglect any contribution of the infills to the resistance and energy dissipation capacity, overly penalizes infilled structures and is not supported by tests, or by detailed analyses such as those of this work, or by experience from strong earthquakes.

The only concern for regularly infilled RC buildings that seems founded is that regarding the tendency for concentration of interstorey drifts and damage in the ground storey and the possibility of soft-storey development there. Nevertheless this does not appear to represent a major threat, except for extremely strong ground motions, which are anyway expected to cause major damage to a similar non-infilled RC structure. Creation of irregularities due to out-of-plane expulsion of infill panels also seems unlikely, especially in the most critical lower storeys. However this possibility cannot be excluded for infill panels with openings, which is an issue not studied enough so far, neither experimentally nor numerically.

RC BUILDINGS WITH HEIGHTWISE IRREGULAR INFILLING

Although there is still no universal agreement on whether infills have overall beneficial or adverse effect on the seismic performance and safety of RC buildings, wide consensus exists that their detrimental effects are typically associated with irregularities in their arrangement in the structure. Particularly disconcerting is the absence of infills in a lower storey, usually the bottom and most critical one.

To study in depth the global response of infilled RC structures with an open ground storey, an extensive parametric analysis was performed on the 4-storey test structure mentioned in Section 3 and tested pseudodynamically in an open-ground-storey configuration at the ELSA reaction wall facility. The analyses employed four spectrum-compatible synthetic motions scaled to a ground acceleration ranging from the design value of 0.3g up to 0.9g, and three different cases of infills in the three upper storeys, with strength and stiffness properties those of the test structure, or higher or lower by a factor of 2.0. The main conclusions of the parametric studies are: a) As in the fully infilled structures, the predominant period of the response is controlled by the frame and very little influenced by the stiffness of infills. b) Although damage is concentrated in the columns of the open storey, at design acceleration levels this concentration is not much higher than in the bare structure, despite the fact that these columns had not been designed against the associated soft-storey effect according to Sect. 2.9.3.2 of EC8, Part 1-3; only at excitation intensities much beyond the design level, some of the open-storey columns do approach failure. c) Structural damage in all storeys above the open one and in the beams of that storey is always minor and much less than in the bare structure. d) The higher the strength and stiffness of infills in the upper storeys, the more severe the concentration of deformations and damage in the open 1st storey. Conclusions a) to c) are in agreement with the results of the pseudodynamic testing of the open-ground-storey structure at the ELSA facility under a unidirectional motion with a peak acceleration of 0.45g.

A second series of parametric seismic response analyses with a wider scope was performed on the twelve buildings of Section 2, all considered in an open-ground-storey configuration. For simplicity the out-of-plane vibration of infill panels was neglected. The four synthetic motions were applied separately in the two horizontal directions. The twelve structures were considered in three design alternatives each: In the first, the resistance of all beams and columns of the open storey was increased to make up for the absence of infills there according to Sect. 2.9.3.2 of EC8, Part 1-3, i.e. by multiplying seismic moments, shears and axial forces by $\alpha=1+\Delta R_{inf}/V_E$, with ΔR_{inf} denoting the deficit in infill ultimate strength and V_E the design seismic shear of the storey. This amounts to designing these elements for the maximum total shear that can develop in the overlying storey (the infills included). The multiplicative α-factor assumes values between 1.22 (for the Ductility Class Medium, 0.3g 12-storey structure) and 2.77 (for the same Ductility Class but for 0.15g and 4-storey). In the second design alternative, only the required resistance of the columns of the open ground storey was multiplied by the α-factor, but not that of the beams. In the third alternative beams and columns of the open storey are designed as in the bare (or fully infilled) structures of the previous Section, without any precautions due to the infilling. 576 nonlinear dynamic analyses were performed with unidirectional input motions. Conclusions of these analyses, in addition to a) of the previous paragraph, are: a) Because of overstrength from various sources and of the reduction in force demands due to cracking and other causes of softening, the response of the open ground storey structures to the design ground motion is almost elastic. b) The concentration of deformations and structural damage in the bottom storey at twice the design ground acceleration increases with the infill strength of the upper storeys, expressed as a fraction of building weight. This concentration is almost negligible in the 12-storey structures, in which infills represent a minor fraction of storey strength. c) The high shear stiffness of the infilled second storey prevents large shear distortions of that storey and limits end chord-rotations in the beams of the open storey directly below it. (As the storey average shear distortion is equal to the sum of: a) the average column chord rotation in the storey, b) the average chord rotation in the floor beams above and below the storey, and c) the average shear distortion in beam-column joints, if the storey shear distortion is low due to the infills, so are the chord rotations at beam ends as well). Therefore, increasing the resistance of these beams as required by EC8, Part 1-3, Sect. 2.9.3.2, is unnecessary. It is even counterproductive for the columns of the open storey, as inelastic chord rotations and energy dissipation at the ends of these beams slightly relieves the first storey columns. d) The EC8 requirement to increase the seismic moment and shear demands on the soft storey columns by the α-factor, works well in the direction of reducing structural damage to these columns under twice the design ground acceleration; in some cases though it leads to column over-reinforcement with adverse effects on local ductility capacity and damage.

In view of the above the α-factor method in Sect. 2.9.3.2 of EC8 Part 1-3 for design against the effects of heightwise irregularity of infills needs to be revisited. An alternative less demanding approach was proposed for these columns, on the basis of capacity design concepts, not of the soft storey as a whole in shear (as in the rule of EC8, Sect.2.9.3.2, Part 1-3) but of the columns in bending at beam-column joints [Fardis 1996, Fardis et al. 1999a]. This capacity-design proposal is based on the observation that due to the high shear stiffness of the infilled storey next to the open one and to the large shear force in the infill itself, the moment and the shear in a column of the infilled storey have opposite sign and direction relative to those of the columns of the neighbouring open storey. (Because of the low shear distortion of the infilled storey, column chord rotations there have also opposite sign relative to those in the beams). Then, if subscript “soft” is used for the column of the less-infilled storey and “inf” for that of the adjacent normally infilled one, the capacity design check of flexural strengths M_R of columns (c) and beams (b) (γ_{Rd}=overstrength factor): $M_{Rc,soft}+M_{Rc,inf}\geq\gamma_{Rd}\sum M_{Rb}$, should be replaced by:

$$M_{Rc,soft} \geq \gamma_{Rd} \sum M_{Rb} + M_{c,inf} \quad [2]$$

or in a more relaxed form along the line of Sect. 2.8.1.1.1(4) of EC8, Part 1-3:

$$M_{Rc,soft} \geq \gamma_{Rd} (1+\gamma_{Rd}-\delta)\delta \sum M_{Rb} + M_{c,inf} \quad [3]$$

with $\delta=|\sum M_{Sb}|/\sum M_{Rb}$ being the beam moment reversal factor (ratio of beam seismic moments, $\sum M_{Sb}$, from the seismic analysis to the corresponding flexural capacity, $\sum M_{Rb}$). Taking in Eqs. [2] and [3] $M_{c,inf}$ equal to $M_{Rc,inf}$ is unrealistic, as the column of the infilled storey is not expected to reach ultimate strength in the same sense of bending as the beams and opposite to that of the column of the open storey. However this conservative assumption offers coverage against beam overstrength in negative bending (tension at the top) due to the contribution of slab steel in tension. A more realistic alternative is to take in Eqs. [2] and [3] $M_{c,inf}$ equal to the corresponding column moment from the seismic analysis of the bare structure $|M_{Sc,inf}|$.

Eq. [2] with $M_{c,inf}=M_{Rc,inf}$ and (3) with $M_{c,inf}=|M_{Sc,inf}|$ were applied separately for the design of the two 2-bay frames of a 3-storey RC structure constructed and tested pseudodynamically in open-ground-storey configuration at the ELSA reaction wall facility in Ispra (I), under a spectrum-compatible motion scaled to 1.5 times the design PGA of 0.3g [Negro et al. 1997, Fardis et al. 1999a]. Test results and pre- and post-test nonlinear dynamic analyses validated the proposed design concept, in that: a) Bending of the columns of the infilled second storey is in the same sense as that of beams and opposite to that of columns of the open ground storey. b) Structural damage in the open storey columns is not much more severe than that predicted when these columns are proportioned according to the α-factor method of Sect. 2.9.3.2 of EC8, Part 1-3, despite the much heavier reinforcement resulting from such proportioning. c) Plastic hinging at the top of the open ground storey columns is prevented; in a cyclic test to failure of the open-1st-storey configuration of the test structure, involving open-storey drift ratios of about 6%, such plastic hinging was preceded and prevented by hinging in the beams framing in the same joint and by shear failure of the joint core (instead of plastic hinging of the columns of the adjacent infilled storey, in the same sense of bending as in the beams). Parametric analyses in [Fardis et al. 1999a] show that, although the design concept expressed by Eqs. [2] and [3] does not take into account the properties of the infills, it is effective over a wide range of the values of these properties.

An alternative to Eqs. [2] or [3] is to proportion columns of open storeys on the basis of peak deformation demands at their ends i.e. within displacement-based seismic design. For this purpose upper characteristic (i.e. 95%-fractile) values of the peak inelastic deformation demands at column ends under the design seismic action can be obtained from the corresponding deformations of the elastic structure with the infills considered everywhere as rigid and the RC members with their secant stiffness at yielding of both ends in antisymmetric bending. This rule was developed from comparisons of the results of various alternative elastic calculations with those of the 576 nonlinear dynamic analyses of the 12 open-ground-storey buildings and applies best to the version of these buildings in which only the columns and not the beams of the open-storey are proportioned for extra protection against the soft-storey effect [Panagiotakos, 1998]. A multiplicative factor of 1.3 needs to be applied to the elastic deformations (chord rotations) at column tops for the calculation of upper characteristic values, whereas no such factor is needed at column bottoms. Then the transverse reinforcement of the open-storey-columns should be proportioned to provide a 5% lower characteristic value of the chord rotation at least equal to:

$$\theta_{uk,0.05} = 0.0085(0.24)^{\nu}\left(\frac{L_s}{h}\right)^{4/9} f_c^{0.3}\omega_w^{1/8} \quad [4]$$

In Eq. [4] $\nu=N/A_c f_c$ is the normalised axial force, L_s/h is the shear span ratio of the column (with L_s equal to half the clear column height), f_c is in MPa and ω_w is the mechanical ratio of the column transverse reinforcement in each horizontal direction. Eq. [4], fitted to the results of over 650 tests, includes the effects of cycling and of fixed-end rotation due reinforcement pull-out from joints.

Both alternatives presented above for the proportioning of open-storey-columns have the advantage of not requiring knowledge of infill properties at the design stage.

The design implications of the studies of RC structures with heightwise irregular infilling can be summarized as follows: a) With the exception of the columns of open (or in general less-infilled) storeys, all RC members can be designed for earthquake resistance as if the structure was bare (i.e. without the reduction in natural period required in Sect. 2.9.4 of EC8, Part 1-3 and without the increase in seismic action effects required by Sect. 2.9.3.2 therein for the beams between the less infilled storey(s) and the neighbouring ones): b) The columns of the open (or less-infilled) storey(s) can be designed for additional strength either according to the α-factor method of Sect. 2.9.3.2 of EC8, Part 1-3, or according to capacity design at beam-column joints following Eqs. (2) or (3), i.e. with the columns of the more infilled adjacent storeys considered to work with the beams against the columns of the less-infilled storey, instead of with the latter against the beams. Alternatively, the confinement ends of these columns can be proportioned within the framework of displacement-based design for inelastic deformation demands estimated from those of an elastic structure with the secant stiffness of RC members at yielding in both ends and with the infills considered as rigid.

RC STRUCTURES WITH INFILLS ARRANGED IN PLAN ON TWO ADJACENT SIDES

Structures with infills arranged only on two adjacent sides of the plan and with the two other sides open are cited in Sect. 2.9.3.1. of EC8 Part 1-3 as an extreme case of infill irregularity in plan, the design of which requires an analysis in 3D with the infills properly included in the structural model. However, if the infills do not crack, they act as essentially rigid and the seismic response of such an irregularly infilled structure to unidirectional or bidirectional motions is simple: it consists mainly of twisting about the common corner of the two infilled sides. Moreover, in the special but characteristic case of a structure which is symmetric in plan and has the same lateral stiffness in both horizontal directions and storey masses distributed in plan in the same way as the lateral stiffness, the twisting response about the corner of the two infilled sides has the same fundamental period as the translational response of the bare structure in the two horizontal directions. Under unidirectional motion the peak translational displacements of the all-around-free column at the opposite corner in both horizontal directions will be equal to that in the bare structure, but will take place simultaneously in the two directions. Under bidirectional motions consisting of two uncorrelated components conforming to the same spectrum, the simultaneous peak displacement demands of the corner column in the two horizontal directions will exceed those of the bare structure due to the individual components of the motion by about 25%. This thinking has led to the definition of a bidirectional shake table test on a two-storey RC frame structure, with one bay in each direction and two adjacent sides infilled in both storeys (the infill of the second storey had a slenderness ratio of 31 to study the possibility out-of-plane expulsion under the bidirectional motion). Test results [Franchioni, 1997], pre- and post-test numerical simulations using the nonlinear models described in Sections 2 and 3, confirm the above thinking: predominant period of twisting equal to that of the translational response of a companion bare test structure; peak displacements of the free column in the two horizontal directions about equal to those in the bare companion structure, but occurring simultaneously; structural damage in the corner column and its vicinity less than in the bare specimen [Fardis et al. 1999b]. Post-test parametric studies show also that these conclusions are not very sensitive to infill properties.

The design implication is that in RC structures with a structural system and a mass distribution which are symmetric in plan in both horizontal directions, the effect of infilling along only two adjacent sides in plan can be taken into account by designing the columns near the corner opposite to the two infilled sides for simultaneous occurrence of the full peak internal force and displacement demands due to the two horizontal components of the seismic action. In any other respect, the structure can be designed for earthquake resistance as if it was bare.

CONCLUSIONS

Infills with regular distribution in plan and elevation affect the response and performance of RC building structures under strong ground motions mainly through their strength and not through their stiffness; their main effect is on energy dissipation and not on the predominant period of the response. Accordingly in fully and regularly infilled structures lateral drifts are well below those of the corresponding bare structures and often peak storey shears are also lower. It follows then that the provisions of EC8 requiring calculation of seismic action effects in the members of fully infilled structures on the basis of the average period of the infilled and of the uncracked bare structure, are too conservative and penalise unduly infilled structures (even when the period of the infilled structure is calculated considering the secant stiffness of infills to ultimate strength). The only adverse effect of regularly arranged infills on the global response is a tendency for drift concentration in the bottom storey. This is not a matter of serious concern though, as, even well beyond the design ground motion, deformations in that storey are well below those required for a soft-storey mechanism. Loss of infills in the bottom storey and creation of open storey there may take place due to a combination of in-plane and out-of-plane demands under bidirectional ground motions. For panels with slenderness as high as 30 but without openings, this does not seem to be a major threat. However this issue requires further studies for panels with openings.

In infilled structures with the ground storey open the concentration of drift and structural damage in the open storey becomes significant at ground motions above the design level. Beams around the open storey are protected from large inelastic deformations by the increased shear stiffness and the low shear distortion of the adjacent storey, but columns of the open storey need additional protection against the localisation of drifts and damage. The relevant EC8 rule, calling for the design seismic shear of these columns to be increased by the shear strength of infills in the overlying storey, works well, except when the combination of structure and infills is such that this provision results in over-reinforcement and reduction of ductility of these columns. One rational, effective and more economic alternative is to design the open-storey columns on the basis of capacity-design moments at the joints, with the column moments in the adjacent storey acting in the opposite sense relative to those in the open storey. Two different versions of this alternative design concept, which offer different levels of protection against soft-storey formation in the open storey, were implemented in a 3-storey test structure and validated through a combination of pseudodynamic testing and parametric analyses. A third alternative is displacement-based design of columns of open-storeys. To this end, the upper characteristic value of peak inelastic deformations of these columns under the design seismic action can be estimated as a multiple of the corresponding elastic deformations, computed from a static elastic analysis for the seismic action, with the infills above the open ground storey considered rigid and the RC members with their secant stiffness at yielding of both ends in antisymmetric bending. Chord rotation demands from such an analysis can be met by proper proportioning of the column confining reinforcement.

In RC structures with stiffness and mass distribution which are symmetric in plan in both horizontal directions, full infilling along two adjacent sides in plan, with the other two sides open, turns the response to bidirectional ground motion into quasi-rotational about the common corner of the two infilled sides, with predominant period approximately equal to the average of those of the translational responses of the bare structure in the two horizontal directions. Columns near the opposite corner in plan experience simultaneously their peak displacements in the two horizontal directions, these displacements being approximately equal to those of the bare RC structure under the same ground motions. These columns need to be proportioned for simultaneous occurrence of peak seismic force or displacement demands due to the two components of the action. In all other respects, design of the RC structure with the irregularity of infills in plan can be as if it was bare.

ACKNOWLEDGEMENTS

The European Commission supported this work: under contract No. CHRX-CT92-0011 for the PREC8 project of the HCM (Human Capital & Mobility) program of DG XII, contract No. FMRX-CT96-0022 (DG12-RSRF) for the ICONS project of the TMR (Training & Mobility of Researchers) program of DG XII, contract No. 10196-94-05 F1ED ISP I with the Joint Research Centre in Ispra (I), and contract No. 11710-96-03 F1ED ISP GR under the “Support to the Commission” program of DG III. Individuals who contributed to this work are (in alphabetic order) Messrs S. Bousias, M. Calvi, G. Franchioni, P. Negro, T. Panagiotakos, A. Pavese and G. Verzeletti.

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