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STUDIES FOR THE DEVELOPMENT OF CODE PROVISIONS FOR INFILLED RC FRAMES

M.N. FARDIS(1), G.M. CALVI(2) and T.B. PANAGIOTAKOS(1)

(1) Department of Civil Engineering, University of Patras, P.O. Box 1424, GR26500, Patras, Greece.
(2) Dipartimento di Meccanica Strutturale, Universitá di Pavia, Via Abbiategrasso 211, 27100 Pavia, Italia.

ABSTRACT

The objectives, the current status and the achievements to-date of European research efforts for improvement of seismic design of RC frame structures with masonry infills, are presented. These efforts comprise coordinated experimental and analytical studies. The out-of-plane behavior of infill panels under their own inertia loads is studied by a combination of shaking-table tests, completed or underway, of monotonic, static cyclic or dynamic out-of-plane loading of infill panels and of modeling of out-of-plane infill dynamics and ultimate strength. The in-plane seismic behavior of infills and its effects on the global response are studied through nonlinear dynamic response analyses of several infilled RC structures. Results of the parametric studies contribute to the improvement of the understanding of the seismic behavior of infilled structures, to the identification of the important parameters and of their effects and to the assessment of current Eurocode 8 provisions for taking into account in seismic design the presence and the irregularity in the spatial arrangement of infills. Early conclusions allow the formulation of suggestions for improvement of code provisions.

KEYWORDS: Eurocode 8, inelastic seismic response, infilled frames, masonry infills, shaking table tests.

EC8 PROVISIONS FOR INFILLED RC FRAMES AND NEEDS FOR IMPROVEMENT

The importance of the influence of infills on the seismic behavior of frame structures, esp. of reinforced concrete ones, is widely recognised by the European earthquake engineering community. Section 2:9 "Additional design measures for masonry infilled frames" of Part 1.3 of Eurocode 8 reflects current thinking of this community on this important issue, as well as today's State-of-the-Art. Although pioneering in its very presence, this Section of EC8 is essentially limited to: a) general principles regarding the need to appropriately consider the effects of infills; b) the requirement to take into account the effects of infills with irregular arrangements in plan through appropriate analysis in 3D, including explicitly the infills in the model (due to the incompleteness and immaturity of the State-of-the-Art, application rules on how this can be accomplished are not given); c) specific and rational application rules regarding how the effects of infill irregularities in elevation can be taken into account in the design of the frame; d) very specific application rules aiming at increasing the seismic action effects (design internal forces) for which the frame should be designed, due to the shifting of the elastic natural period of the structure effected by the infilling; e) general and specific application rules, to take into account the adverse local effects of infills on the adjacent structural members.

On the basis of recent research and of observations from past earthquakes, some comments on these provisions can be made. Firstly the application of the rules under point "d" seems to lead to overconservative designs and penalises infilled frames, to a point not justified by research results or by observed earthquake damage. Second, infills irregularly arranged in plan do not seem to considerably increase the global vulnerability of the structure. Hence, development of very detailed rules to assist the application of the principles mentioned under point "b" above, does not seem to be of top priority. On the contrary, infill irregularities in elevation invalidate the concepts of overall ductility and capacity design, which form the backbone of the seismic design philosophy, not only of EC8 but also of every modern seismic design code, and may pose a major threat to the global integrity of otherwise properly designed RC structures. The EC8 application rules mentioned under point "c" above are a step in the right direction. Nevertheless, they lack the necessary support of in-depth research. So it is mainly regarding points b to d that prenormative research for improvement of EC8 is needed. A further issue for investigation is the possibility of development of infill irregularities in plan or in elevation, through loss of infill panels due to their out-of-plane dynamic response to the transverse to them seismic motion.

OUT-OF-PLANE SEISMIC RESPONSE OF INFILL PANELS

Overview of experimental work

Shaking table tests at EERC, Univ. of Bristol (Taylor and Ndamage, (1995)): Six shaking table tests were performed on 2m-long by 1.6m-tall one-bay single-story steel frames, infilled with masonry of either low-strength (2.3MPa) lightweight (0.7t/m^3) 100mm-thick blocks, or with 65mm-thick normal-weight (2.3t/m^3) high strength (25MPa) solid blocks, and of mortar with nominal strength 1, 3 or 16 MPa. Specimens were subjected to separate or simultaneous in-plane and out-of-plane motions, of the sinusoidal (ramp-dwell), band-limited random-noise, or EC8-spectrum-compatible earthquake, type. Out-of-plane natural frequencies of the panels were measured to be between 32Hz and 40Hz before damage and between 3.5Hz and 30Hz in the damaged condition, depending on damage level. In-plane cracking of panels started at input peak accelerations between 0.2g and 0.9g, depending on the strength of the masonry, but main diagonal cracking took place around 0.5g to 0.6g. In general, table accelerations in excess of 1g were required in both horizontal directions to cause any noticeable out-of-plane damage, and base accelerations between 2g and 3g were incapable of causing partial or total out-of-plane collapse of the panels, despite the almost complete disintegration of the mortar in the infill-frame interface joints at the top and the vertical sides of the panels and the local crushing of masonry block corners along the top. Overall, the results suggest very little dynamic amplification of the imposed out-of-plane seismic motions by the panel itself and extremely high out-of-plane resistance and good performance of a large variety of infill panels. It is noteworthy that the Bashandy et al (1995) model for out-of-plane strength predicts collapse at an out-of-plane lateral pressure of about $3x10^{-3}f_{cv}$ for the lightweight blocks, or of about $0.4x10^{-3}f_{cv}$ for the normal weight ones, i.e. for an out-of-plane acceleration (in g's) of $4f_{cv}$ or of $0.03f_{cv}$ respectively. For the values of the wall strengths, f_{cv}, estimated from the block and mortar strengths, the predicted accelerations required for collapse of the lightweight block panels are of the order of 10g, which is consistent with the fact that the panels withstood out-of-plane accelerations in excess of 1g. However, the normal weight block panels withstood without collapse, and despite the complete disintegration of the mortar in the joints, peak base accelerations of 2g to 3g, i.e. well in excess of the predicted collapse accelerations of only 0.6g.

Shaking table tests at LNEC - Lisbon: The purpose of shaking table testing of infilled RC specimens at LNEC is: a) to investigate the behavior and possible failure of infill panels under simultaneous in-plane and out-of-plane seismic excitation and the impact on the global response of the test structure; and b) to study the magnitude and the consequences of the torsional response caused by a strongly irregular arrangement of the infills in plan. The test structures are approximately 3m by 4m in plan with one bay in each direction and have one 3m high story. Column and beam cross-sections are relatively small, to avoid large member overstrengths due to minimum reinforcement and to achieve a structure with enough flexibility that the presence of the infills is strongly felt. Slender, 70mm thick, brick walls are constructed between corner columns on only 3 sides of the structure. So, the effect of the large eccentricity in plan which is produced in one direction, is partly counteracted by the torsional rigidity of the pair of parallel walls in the other direction. The slenderness ratio, H/t, of the infills in the vertical direction is over 35. For these values of panel slenderness and aspect ratios and for horizontally perforated bricks with a horizontal-to-vertical E-modulus ratio of about 2.0, the Bashandy et al (1995) model predicts out-of-plane failure of the panel without in-plane damage at a lateral pressure of about $0.3x10^{-3}$ times the masonry strength in the vertical direction, f_{cv}, i.e. for f_{cv}=3.5MPa, at an infill out-of-plane peak acceleration of above 0.75g. The table will be excited simultaneously in both horizontal directions by two different historical records from the same seismic event. The records will be selected to maximize the spectral acceleration demands on the infilled structures, within the maximum velocity and displacement restrictions of the table. Preliminary nonlinear dynamic response analyses of the infilled specimens suggest that for simultaneous bidirectional excitation with so-selected components a) there is possibility for out-of-plane failure of infill panels; and b) the torsional response is limited, despite the eccentricity in one direction.

Out-of-plane load tests on infill panels of the full scale 4-story ELSA test structure: Static (monotonic or repeated cyclic) and low-amplitude dynamic tests on individual infill panels of the infilled 4-story ELSA test structure at the Joint Research Center of the European Commission in Ispra, were performed. Within the framework of the coordinated research effort on infilled RC structures, the 4-story structure was pseudodynamically tested to an input motion 1.5 times its EC8-spectrum-compatible design ground motion of 0.3g effective peak acceleration. It was tested first bare, then with its two exterior frames infilled in all 4 stories and finally in an open (soft) 1st-story configuration. Detailed results are given by Negro and Taylor (1996). Most infill panels were damaged in the pseudodynamic testing parallel to their in-plane direction. The out-of-plane natural frequency of small-amplitude forced vibrations of the panels was found to be around 40Hz in the undamaged condition, or around 10Hz after their cracking during the in-plane loading. Monotonic static and low frequency (0.5Hz and 1Hz) dynamic out-of-plane tests were performed under uniform pneumatic pressure, using the Brerwulf apparatus (Building Research Establishment, 1995). Two previously damaged panels with an aspect ratio L/H=2.2 were carried to failure at a uniform pressure of about 7kPa, i.e. at a pressure-to-masonry-strength ratio, p/f_{cv}, of about 0.65×10^{-3}, which is less than half the failure value predicted by the Bashandy et al (1995) model for such slenderness ratio (H/t=22.8), aspect ratio and horizontal-to-vertical moduli ratio ($E_h/E_v=0.3$) of the masonry panel. Three more previously damaged panels with aspect ratio L/H=1.4 did not fail at lateral pressures of 6.1, 8.5 and 9.4kPa, i.e. at p/f_{cv} ratios between 0.76×10^{-3} and 0.89×10^{-3}, which are about half the predicted collapse values of undamaged panels. Although the pre-ultimate response was nearly linear-elastic, the measured stiffness was about one-tenth to one-twentieth of that computed from the Timoshenko solution for elastic plates with two-way action. Four more previously damaged panels with L/H = 1.4 were tested to failure under 4-point concentrated loads applied at the thirds of the panel dimensions. If a conversion factor of 4/3 is applied on the sum of the concentrated loads to convert them to a work-equivalent uniform pressure, the failure pressure ranges from $0.8 \times 10^{-3} f_{cv}$ to $1.17 \times 10^{-3} f_{cv}$ ($10^{-3} f_{cv}$ on the average), still less than the failure pressure predicted according to Bashandy et al (1995). The difference is attributed to the previous damage and to the uncertaint in the conversion factor (a value of 2.5 is needed for equal displacement under the uniform pressure and the 4-point cases). Pre-ultimate unloading-reloading cycles suggest a hysteretic damping ratio of the panels in the out-of-plane direction of the order of 10%.

Modeling of out-of-plane panel dynamics and ultimate strength

In its out-of-plane direction an infill panel may be approximated as a SDOF system, subjected to a support excitation with acceleration time-history equal to the average of those of the two floors bordering the panel, in the direction normal to the panel surface. If the effective natural frequency of the panel in the out-of-plane direction is much higher than the predominant frequency of the support motion, as most available test results suggest, the panel follows the floor motion in a rigid-body fashion and is subjected to a transverse inertia force history equal to its total mass times the average acceleration time-history of the floors above and below. If its effective natural frequency is of the same order as the predominant excitation frequency, the panel may develop dynamic amplification of the floor motion. In approximation the out-of-plane dynamic response of a panel may be studied on the basis of the equation of motion of the "equivalent" SDOF system:

$$m_e\ddot{u} + c\dot{u} + k_e u = -m_e\ddot{u}_s \qquad (1)$$

In (1) m_e and k_e are the effective mass and the equivalent spring constant of the panel in the out-of-plane direction, u the transverse displacement of a representative point of the panel (e.g. of the center), c the damping coefficient and $\ddot{u}_s$ the support acceleration. To determine m_e and k_e, the dynamic out-of-plane deflections $u(\mathbf{x})$ at any point $\mathbf{x}$ of the panel should be expressed as: $u(\mathbf{x})=\phi(\mathbf{x})u_o$, i.e., in terms of the deflection u_o at the representative point, and of a supposedly known time-independent function $\phi(\mathbf{x})$, which describes the deflected shape up to a constant. u_o and $u(\mathbf{x})$ are functions of time t, but this dependence is not explicitly shown here. Accelerations $\ddot{u}(\mathbf{x})$ are obtained by double differentiation of $u(\mathbf{x})$ with respect to t and are related to $\ddot{u}_o$ as $\ddot{u}(\mathbf{x})=\phi(\mathbf{x})\ddot{u}_o$. The actual total lateral inertia force on the panel at time t is:

$$F = \int \ddot{u}(\mathbf{x})dm = \ddot{u}_o \int \phi(\mathbf{x})dm \qquad (2)$$

There are two ways to define the equivalent SDOF system of the panel. In the first one it is required that the total lateral force on this system is equal to the actual given by (2) but the resulting equivalent displacement, u_e, is less than that of the representative point, u_o. In the second one the displacement of the equivalent system is taken equal to u_o but the equivalent total lateral force is less than the actual. In both cases, it is required that the work done by the (equivalent) lateral force of the equivalent system on its (equivalent) deflection, is equal to the work done by the distributed inertia forces on the actual deflections of the panel, i.e. to:

$$W=\int df(\mathbf{x})u(\mathbf{x}) = u_o\int df(\mathbf{x})\phi(\mathbf{x}) \tag{3}$$

In (3) $df(\mathbf{x})$ is the inertia force on the infinitesimal mass dm at point **x** and equals $df(\mathbf{x})= \ddot{u}(\mathbf{x})dm=\ddot{u}_o\phi(\mathbf{x})dm$.

For the first, i.e. the equal total force hypothesis, $df(\mathbf{x})$ is expressed in terms of the actual total lateral force F from (2):

$$df(x)=\frac{\phi(x)\,dm}{\int\phi(x)\,dm}F \tag{4}$$

Substitution for df(x) from (4) into (3), yields:

$$W=Fu_o\frac{\int\phi^2(x)\,dm}{\int\phi(x)\,dm} \tag{5}$$

If u_e denotes the equivalent deflection of the SDOF system in the first hypothesis, the work of F on u_e is Fu_e and as it is required to equal W in (3), the equivalent deflection of the SDOF system is given by:

$$u_e=\frac{\int\phi^2(x)\,dm}{\int\phi(x)\,dm}u_o \tag{6}$$

As the equivalent acceleration of the SDOF, $\ddot{u}_e$, is obtained from (6) by double differentiation with respect to t, the equivalent mass of the SDOF system, $m_e=F_e/\ddot{u}_e$, is given by:

$$m_e=\frac{(\int\phi(x)\,dm)^2}{\int\phi^2(x)\,dm} \tag{7}$$

The equivalent stiffness of the SDOF system in the first hypothesis is given by:

$$k_e=\frac{F}{u_e}=K\frac{\int\phi(x)\,dm}{\int\phi^2(x)\,dm} \tag{8}$$

in which K is the lateral stiffness of the panel, as determined from the relation between total lateral force and the deflection at the representative point. The circular frequency of the equivalent system is, according to the first hypothesis:

$$\omega_e=\sqrt{\frac{k_e}{m_e}}=\sqrt{\frac{K}{\int\phi(x)\,dm}} \tag{9}$$

The second hypothesis, i.e. that of equal displacements of the SDOF system and of the representative point of the panel, corresponds to a work of the equivalent system equal to F_eu_o, with F_e denoting the equivalent total lateral force on the SDOF system. This work is equated to the right-hand-side of (3), to obtain $F_e=\int df(\mathbf{x})\phi(\mathbf{x})$, or, taking into account that $df(\mathbf{x})= \ddot{u}(\mathbf{x})dm=\ddot{u}_o\phi(\mathbf{x})dm$:

$$F_e=\ddot{u}_o\int\phi^2(\mathbf{x})dm \tag{10}$$

As the acceleration of the equivalent SDOF system is in this case $\ddot{u}_o$, (10) gives for the equivalent mass of this system according to the second hypothesis:

$$m_e=F_e/\ddot{u}_o=\int\phi^2(\mathbf{x})dm \tag{11}$$

(2) and (10), combined give the total lateral force on the equivalent according to the second hypothesis:

$$F_e=F\frac{\int\phi^2(x)\,dm}{\int\phi(x)\,dm} \tag{12}$$

Then the stiffness of the equivalent SDOF system is:

$$k_e=\frac{F_e}{u_o}=K\frac{\int\phi^2(x)\,dm}{\int\phi(x)\,dm} \tag{13}$$

and its circular frequency is again given by (9) but with m_e and k_e given by (11) and (13) instead of (7), (8).

To apply the above to an infill panel of (height) H and (length) L, a deflected shape $\phi(x)$ has to be assumed. In the elastic range $\phi(x)$ can be taken according to the first mode shape of the panel out-of-plane vibration, $\phi(x)=\sin(\pi x/L)\sin(\pi y/L)$, in which x is the length coordinate and y the vertical coordinate. Then $\int\phi^2(x)dm = M/4$ and $\int\phi(x)dm=4M/\pi^2$, where M denotes the total mass of the panel. Therefore:

For the 1st (equal total force) hypothesis: $m_e=0.657M,\ k_e=1.621K$ (14)

For the 2nd (equal displacement) hypothesis: $m_e=0.25M,\ k_e=0.617K$ (15)

In both cases: $\omega_e=\frac{\pi}{2}\sqrt{\frac{K}{M}}$ (16)

In the strongly nonlinear range, a complete plastic mechanism forms over the panel surface, consisting of a yield line originating at each one of the 4 corners of the panel at an angle θ to the horizontal and meeting at panel midheight, where a horizontal yield line forms. This yield line pattern partly concides with areas of in-plane cracking and damage of the wall. Angle θ is typically such that tanθ equals h/2l, where h and l denote the height and length dimensions of a masonry unit, as in-plane cracks often form in a step-wise pattern along bed and head joints of the masonry. Assuming that after formation of such a yield line pattern only relative rotations along the yield lines take place, the basic integrals of $\phi(x)$ are equal to $\int\phi(x)dm=0.5M(1-H/3L\tan\theta)$ and $\int\phi^2(x)dm=M(1-H/2L\tan\theta)/3$. Then, for the first (equal total force) hypothesis:

$$m_e=\frac{0.25M}{L\tan\vartheta}\frac{(3L\tan\vartheta-H)^2}{2L\tan\vartheta-H},\qquad k_e=K\frac{3L\tan\vartheta-H}{2L\tan\vartheta-H} \tag{17}$$

and for the second (equal displacement) hypothesis:

$$m_e=\frac{M}{3}\left(1-\frac{H}{2L\tan\vartheta}\right),\qquad k_e=K\frac{2L\tan\vartheta-H}{3L\tan\vartheta-H} \tag{18}$$

In both cases the effective circular frequency is:

$$\omega_e=\sqrt{\frac{K}{M}\frac{6L\tan\vartheta}{3L\tan\vartheta-H}} \tag{19}$$

Using the stiffness of the in-plane damaged panels of the ELSA test structure, as measured in the uniform lateral pressure tests using the Brerwulf apparatus, both (16) and (19) give a natural frequency, $\nu=\omega/2\pi$, of the cracked panel of 13Hz, which is indeed close to the measured values of about 10Hz.

Modeling of an infill panel as an equivalent SDOF system, requires ability to predict analytically the relationship between the total lateral force and the deflection at the center of the panel. From such a relationship the secant stiffness of the panel, K, can be obtained, at given actual (or equivalent) deflection at the center.

In his original one-way arching action model of panel ultimate strength, McDowell et al (1956) derived the following relationship for the ultimate lateral pressure on the panel, w_u:

$$\frac{w_u}{f_{cv}}=1.7\left(\frac{t}{H}\right)^2\left(1-\frac{f_{cv}}{E_v\varepsilon_v}\right)^2 \tag{20}$$

in which ε_v is compressive strain required to shorten the diagonal of a vertical section of the upper half of the panel to half the panel height H, causing, therefore, out-of-plane collapse:

$$\varepsilon_v=\frac{\sqrt{\left(\frac{H}{2}\right)^2+t^2}-\frac{H}{2}}{\sqrt{\left(\frac{H}{2}\right)^2+t^2}} \tag{21}$$

The lateral deflection at which ε_v and w_u take place is computed considering that the two halves of the panel rotate about the top, the bottom and the horizontal at midheight, while remaining plane:

$$\delta_u=\frac{tf_{cv}}{E_v\varepsilon_v} \tag{22}$$

The ratio LHw_u/δ_u gives, then, the lateral stiffness K of the panel.

Bashandy et al (1995) generalised the McDowell (1956) approach for two-way action in a panel of length L. This more general result for the ultimate lateral pressure is the following:

$$\frac{w_u}{f_{cv}} = 1.7\frac{t^2}{LH}\left[\left(\frac{L}{H}+ln2-1\right)\left(1-\frac{f_{cv}}{E_v}x_v\right)^2-\frac{E_h}{E_v}\frac{L}{H}\left(\frac{x_h-1}{x_v-1}\right)\ln\left(1-0.5\frac{H}{L}\right)\left(1-\frac{f_{ch}}{E_h}x_h\right)^2\right] \tag{23}$$

where f_{cv} and f_{ch} denote the compressive strength of masonry in the vertical and the horizontal direction and E_v, E_h the corresponding Elastic moduli. x_v equals $1/\varepsilon_v$ with ε_v from (21) and x_h is $1/\varepsilon_h$, with ε_h obtained from a relation similar to (21) in which t/H is replaced by t/L. Using w_u from (20) or (23) and δ_u from (22) and assuming a modulus-to-compressive-strength ratio $E_h/f_{ch} = E_v/f_{cv}$ of 1000, values of 0.57kPa/mm and 0.63kPa/mm are obtained for the stiffness of the ELSA test structure panels, which are in very good agreement with the measured stiffness and natural frequencies of the cracked panels.

A macro-Element model has been developed for the nonlinear dynamic response of an infill panel in the out-of-plane direction and for its in-plane mechanical response. The in-plane mechanical behavior is modeled through a pair of diagonal struts connecting the frame joints at opposite corners of the panel and effective only in compression. As far as the infills are concerned, these 4 nodes have only 12 DOFs, i.e. 3 translational DOFs each. Only 8 of these DOFs, i.e. the ones associated to the two in-plane translations of the nodes, correspond to non-zero terms of the Element stiffness matrix. The mass matrix of the Element has its diagonal terms associated to the above 8 in-plane translational DOFs full and equal to one-quarter of the total mass of the infill panel. The 4 translational DOFs of the Element representing the nodal displacements in the out-of-plane direction of the panel, have neither stiffness nor mass associated to them. They are present only for transmitting the average of their accelerations to an internal, 13th, DOF of the Element as a support excitation in the fashion of the right-hand-side of (1). The terms of the stiffness matrix relating the in-plane translational DOFs of opposite nodes of the infill panel Element are developed on the basis of a cyclic-nonlinear macromodel of the in-plane behavior of infill panels, which has been previously developed and calibrated by Panagiotakos and Fardis (1994) (see also Fardis and Calvi, 1994, Panagiotakos and Fardis, 1995).

The internal 13th DOF of the Element corresponds to the out-of-plane deflection (i.e. relative translation) of the centre of the panel, according to (10) to (13), or of its equivalent, according to (6) to (9). This displacement DOF satisfies the equation of motion, (1), or a more general form of it, in which $F_e(u)$ is used as the restoring force instead of $k_e u$ and is a nonlinear function of the out-of-plane deflection, u. The mass, m_e, associated to this DOF is computed according to (7) or (11), depending on whether in the equivalent SDOF system the actual total lateral force or the deflection at the center of the panel is preserved. If the second alternative is chosen, then (12) should be applied at each time step, with the fraction on the right-hand-side taken equal to $(2L\tan\theta-H)/(3L\tan\theta-H)$ (see (18)), to convert the equivalent force F_e into the actual out-of-plane force F transmitted from the infill panel to the frame. Moreover, for this second alternative (12) and (13) should be used to translate the actual nonlinear or linearized actual force (F) - actual deflection (u_o) relation into an equivalent lateral force (F_e) - actual deflection one, to be used in the SDOF out-of-plane model of the panel. Alternatively, if it is chosen to preserve forces in the SDOF model ((6)-(9)), panel out-of-plane forces F can be transmitted to the frame without any conversion, and (8) needs to be utilised to go from the actual force (F) - actual deflection at the center (u_o) relation, to the actual force (F) - equivalent deflection (u_e) one required by the SDOF model.

At each time step the nonlinear dynamic analysis of the global response provides the out-of-plane accelerations of the 4 nodes of the frame surrounding the infill panel, to be averaged for the calculation of the right-hand-side of (1). Then, for given values of the out-of-plane deflection u of the panel and of the corresponding velocity and acceleration at the previous time-step, (1) is solved for u. Results are used only to compute out-of-plane inertia, damping and reacting loads, transmitted as appropriate from the panel to the supporting frame at this time step. The stiffness, the damping and the mass terms of (1) are not assembled into the corresponding global matrices of the structure. (1) is solved, instead, separately for each panel, using as input from the frame just the information required for the calculation of its right-hand-side and providing as output to the structure in the back-substitution phase, contributions to its inertia, damping and Element internal force load vectors, to be used for the solution of the global equations of motion in the next time-step. The feedback of the panel out-of-plane dynamic response to the global dynamic response has a minor, yet, in general, beneficial effect on the global response of the frame in the horizontal direction normal to the infill panel, relative to the case in which the infills are considered as rigid blocks, riding along the vibration of the structure, without vibration and energy dissipation of their own.

As far as the response of each panel is concerned, the most important result is the estimation at each time-step of the total lateral force F on the panel, and the use of its value, in conjunction with information on the current state of in-plane damage of the panel (expressed, e.g., by its maximum previous in-plane shear distortion), to assess collapse or not of the panel due to the combination of in-plane and out-of-plane damage and demands. If failure is reached, the panel is removed, both in the in-plane and in the out-of-plane direction.

The hysteretic law to be used for the lateral force-deflection relation of the panel, can be of the simplified Takeda-type with 9 rules. However, if the simplified McDowell (1956) or Bashandy et al (1995) models of (20) to (23), are used, with brittle and abrupt failure considered to take place once the ultimate value of (20) or (23) is attained, then a linear-elastic approximation of the out-of-plane behavior of the panel is appropriate, with a viscous damping ratio of about 10%, reflecting the energy absorption in unloading-reloading cycles prior to panel ultimate failure.

PARAMETRIC NONLINEAR DYNAMIC RESPONSE ANALYSES OF INFILLED STRUCTURES

Systematic parametric studies were performed on infilled RC frame structures, in the form of nonlinear dynamic response analyses (Panagiotakos and Fardis, 1996). The purpose of these analyses was a) to identify those infill characteristics which control the magnitude of the effect of the infills on the global seismic response; b) to quantify the effect of infills on the response of a large variety of RC frame designs; and c) to access the effectiveness of the EC8 provisions regarding infilled RC structures.

Parametric studies were first performed at the simplest possible level, on a SDOF 5%-damped elastic frame of varying stiffness and elastic period, with infills of different ultimate strengths and variations in their force-deformation law. In this way response spectra were constructed, first for 4 artificial ground motions compatible with the EC8 5%-damped elastic spectrum for soil B, and then for a historical record with just two large acceleration peaks in each direction. Computed spectra give the peak displacement and the peak response acceleration of the system (which is related to the maximum force demand on the frame), for various infill strengths, up to twice the product of the system mass times the effective peak acceleration of the input. Different shapes of the infill force-deformation curves under monotonic loading were considered, ranging from rigid loading up to ultimate strength and constant force for further loading thereafter, to realistic loading and hardening up to ultimate strength but rather abrupt load-shedding thereafter. The general conclusion of these analyses was that infilling is almost always beneficial for the frame, in the sense of reducing the peak force and deformation demands on it, despite the apparent stiffening effect of the infill which may shift the effective period of vibration from the falling branch of the acceleration spectrum to the constant spectral acceleration range. The initial elastic stiffness of the infills is almost completely unimportant. On the contrary, the amount of post-ultimate softening is very important, in the sense that if the response is such that the infills shed completely their force, the force and deformation demands on the infilled frame are higher than on the bare one. A very important feature of the infills is their energy dissipation capacity, which, being much higher in the first post-cracking excursion of the infills in each direction of loading than in subsequent unloading-reloading cycles, is best exploited for impulsive ground motions with very few large acceleration peaks.

The parametric analyses on several fully infilled multistory RC buildings, designed according to EC8 as bare frames (Carvalho, !995, Fardis, 1995), confirm and reinforce the conclusions above: First, the precracking stiffness of infills is unimportant and the stiffening effect of infills does not increase force demands over those on the bare frame, because infills crack and separate early from the frame. As a result the nonlinear response of the infilled structure is controlled by the period of the frame. Second, the presence of infills delays first yielding of the frame and formation of a mechanism. The larger is the ultimate strength of the infills, the higher is the ground motion intensity required for yielding of the frame. As a result, the design acceleration and the Ductility Class (DC) for which the bare frame was designed is immaterial for its global response and structural damage, even for ground motion intensities many times higher than the design value. In other words a frame designed for 0.15g and DCL (low) behaves almost the same as one designed for 0.3g and DCH (high), when subjected, as infilled, to a spectrum-compatible ground motion with effective peak acceleration of 0.6g.

In heavily infilled structures the absence of infills from the ground story leads to formation of a soft-story there, at ground motion intensities which are not much higher than that of the design motion and for which the bare frame remains almost elastic. Horizontal drifts are localised in the soft-story and nearly all the energy dissipation takes place in its columns. As a result, these columns may develop high levels of structural damage, which, for heavy infilling of the upper stories and at high ground motion intensities (of the order of 2 or 3 times the design intensity) may lead to collapse of the structure. Dimensioning the first story beams and columns for seismic forces, which are increased over the design values for the bare frame in proportion to the magnitude of the infill ultimate strength relative to the design base shear of the bare frame, as required by the relevant provisions of EC8 for irregularly infilled structures, reduces, in general, structural damage in the soft story. In some cases of relatively heavy infilling and high excitation intensities, such an upgrading of soft-story columns against the effects of infill irregularity may increase their damage instead of reducing it, because, after such upgrading, seismic demands are increased and the heavier reinforced columns may have reduced ductility and deformation capacity. Damage to soft-story columns is reduced, though, if the beams of that story, which are not particularly hurt anyway by the infill irregularity, are not upgraded for this irregularity and are allowed to develop more inelasticity and absorb more energy, relieving, therefore, the soft-story columns.

CONCLUSIONS

Results todate of European research on the seismic response of infilled RC frames, confirm some of the current relevant provisions of EC8 but defy others. In the absence of irregularities, especially in elevation, infills have always beneficial global effects on the response of the frame. In view of this and of the finding that, due to the early cracking and separation of the infills from the frame, the nonlinear response is controlled by the frame period regardless of the elastic stiffness of the uncracked infills, the EC8 requirement to determine design seismic forces on the basis of the average of the elastic periods of the bare and of the infilled structure seems unnecessarily penalizing the infilled frames. The effects of infill irregularities in elevation on the safety of the frame are quite disconcerting and the EC8 provisions for designing against such effects, although in the right direction, need to be relaxed for the beams and revisited for the columns of the so-created soft-story. Out-of-plane collapse of infills due to their own inertia forces is important, in that it may lead to creation of irregularities in elevation. Fortunately, though, such an occurrence is not likely, even under very strong ground motions, except for very slender and weak in vertical compression infills.

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