Proceedings of the 9th U.S. National and 10th Canadian Conference on Earthquake Engineering
Compte Rendu de la 9ième Conférence Nationale Américaine et
10ième Conférence Canadienne de Génie Parasismique
July 25-29, 2010, Toronto, Ontario, Canada • Paper No 217

INVESTIGATION OF THE SEISMIC VULNERABILITY OF EXISTING RC BUILDINGS WITH MASONRY INFILL WALLS

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ABSTRACT

In this paper the seismic vulnerability of old multi-storey reinforced concrete (RC) buildings reinforced with substandard details is assessed as a function of the interstorey drift. Diagnostic tools for direct and fast assessment such as the sway index of the building, the fundamental period, the drift magnitude at the onset of yielding, and the fundamental shape of vibration of the structure are adopted. It may be shown that any of these characteristics depends on a single design parameter, namely the ratio of the area of the vertical members to the plan floor area of the building; depending on the mode of construction, the participation of infills may be pertinently accounted for in this diagnostic ratio. A methodology is presented where demand, expressed in terms of lateral interstorey drift, is related through pertinent expressions of lateral stiffness to the column, wall, and infill area ratio of the storey. Using this procedure, Interstorey Drift Spectra (IDS) are derived for rapid assessment of existing RC buildings, whereby the anticipated drift demand is linked to the design characteristics of the structure. The methodology is tested through application to an old pre-code RC building that underwent severe damage during a recent strong ground motion.

Introduction

The large majority of the existing building stock in Greek urban centers was designed and constructed before 1985. Due to the practice of using an open first storey for parking space in residential buildings in the Greek urban centers, many of these structures are characterized by lack of stiffness. This problem is accentuated by light reinforcing details, and other compounded effects such as corrosion. Recent earthquakes in Greece revealed the typical deficiencies of this category of buildings and underlined the need for rapid and palatable tools for assessing the seismic vulnerability of the built environment.

In the postworld war era and for a long period, seismic coefficients were assigned low values. (In recent revisions of the Greek seismic design codes, seismic coefficients have been reset to 3÷4 fold their original values (EAK 2000)). The design philosophy in former years was based on allowable stress design, and therefore there was no control of the mode of failure and the corresponding deformation capacity of the individual members. The structural systems of that period presented deficiencies both in the global (structural system) and the local level (individual

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member design level). The non-uniform distribution of stiffness / mass along the height of the building as well as in the floor plan often cause severe damage leading to failure in buildings with torsional issues and localization of damage in a few floors. At the member level insufficient confinement (sparse and inadequately anchored stirrups) play a determining role in the damage pattern that is expected to occur under severe ground shaking where deformation demand is expected to be high (Fig. 1).

Figure 1. Damage patterns in old type RC members.

For the class of buildings described, a critical step towards moderating their seismic vulnerability is the reduction of seismic displacements through control of the lateral stiffness. Post earthquake field inspections underline the important role of masonry infill walls in controlling lateral displacement and therefore the occurrence of damage, despite the fact that they are not considered as load-carrying structural members.

Today, identifying seismically deficient structures is usually treated as an assessment issue, which requires the estimation of the pushover curve to lateral loads. For a large number of existing buildings, this procedure involves disproportionally great effort compared to the uncertainty related to the details and the actual morphology of the structural system. The number of floors, the floor area and the area of the vertical members are the only data readily available for most existing RC buildings. The challenge is with these few elements to single-out those buildings that present higher vulnerability.

The most meaningful diagnostic tools for direct and fast assessment are the index of sway of the building, its fundamental period and its fundamental shape of vibration. Deviation of these indices from the normal range of values suggests an abrupt change of mass or stiffness. It is interesting and easy to prove, that all these characteristics depend on a single parameter which is the ratio of the area of the vertical members to the plan floor area of the building. In the present paper, damage of reinforced concrete (RC) structural systems with masonry infill walls is investigated as a function of the area of the vertical members and the interstorey drift of the floor. For a specific scenario of seismic excitation in a high seismicity region, the minimum requirement in terms of the area of RC columns and masonry infill walls is established in order to secure selected performance levels for the interstorey drift (i.e. to control the level of expected damage). It is noted that the proposed methodology applies to regions where RC structural systems (frames or dual systems) with masonry infill walls constitute common construction practice (e.g. in Mediterranean countries). The proposed methodology is used to investigate the seismic vulnerability of a building typical of older construction practices severely damaged during the 2003 Bingöl earthquake (Ramirez et al. 2003).

Estimation of Building's Lateral Stiffness

The buildings that are at the center of the present research are of old type detailing characterized by rigid diaphragms. Thus, sway during earthquakes in such structures is secured by deformation of the vertical members. Furthermore, it is that the building has a typical layout of vertical RC members, which is identical in all floors. Storey height is h_s. For direct and relatively simple estimation of the lateral stiffness of the building, the multi-storey system is idealized to a generalized equivalent single-degree of freedom. The consequence of this assumption is that the building is considered to oscillate following always a unique shape of lateral displacement which approaches the fundamental translational mode, whereas the contribution of the higher modes in this case is neglected. The reliability of the results is influenced greatly by the selection of the adopted shape function for the displacement shape.

The lateral stiffness of a frame with rigid diaphragms is composed by the lateral stiffness of the individual storeys. Storey stiffness results from the summation of the stiffness of the storey's vertical members, i.e., columns, walls, as well as masonry infill walls. The work equivalent contribution of a storey with total stiffness K_i, to the stiffness K of the building is given by:

$$K= \sum_{i=1,N} K_i \cdot \Delta\Phi_i^2 = \left[\sum_{j=1,n_c} K_{i,j}^{c} + \sum_{k=1,n_{wc}} K_{i,k}^{wc} + \sum_{p=1,n_{wm}} K_{i,p}^{wm} \right] \Delta\Phi_i^2 \tag{1}$$

where $\Delta\Phi_i$ is the difference in the shape between successive floors. Expressions as derived after algebraic manipulation for estimating the stiffness for RC columns, $K^c{}_j$, RC walls, $K^{wc}{}_k$ and masonry infill walls K_i^{wp} are presented in Table 1. The stiffness of the masonry infill walls, $K^{wm}{}_p$, may be expressed by two alternative approaches depending on the type of seismic response of the structural system as shown in Table 1.

Depending on the type of the structural system, dual system or frame, two alternative expressions for the total storey stiffness, K_i, are extracted.

<u>*For Dual systems*</u>: Stiffness expressions for RC walls and masonry infill walls have the same form, but are differentiated with regards the modulus of elasticity and the member's length (Table 1). Hence, an equivalent dimensionless area for RC walls and masonry infill walls is defined, $\rho_{wm,i}{}^e$, in terms of masonry infill walls:

$$K_i \approx \frac{E_c A_f}{100 h_s} \rho_{c,i} + \frac{E_{wm} A_f}{h_s \left(4 \frac{h_s{}^2}{l_{m,ave}{}^2} + 2.50 \right)} \rho_{wm,i}^{e} = \frac{A_f}{h_s} \left(B_c \rho_{c,i} + B_{wm} \rho_{wm,i}^{e} \right) \tag{2}$$

$$\text{where } \rho_{wm,i}^{e} = \rho_{wm,i} + \frac{E_c}{E_{wm}} \frac{\left(4 \frac{h_s^2}{l_{m,ave}^2} + 2.50 \right)}{\left(4 \frac{h_s^2}{l_{w,ave}^2} + 2.50 \right)} \rho_{wc,i} \tag{3}$$

<u>*For Frames*</u>: Storey stiffness, K_i, comprises column and infill wall stiffness contributions. Infill wall participation is reduced with lateral drift, due to the higher inherent flexibility of the frame structure.

$$K_i \approx \frac{E_c A_f}{100 h_s}\rho_{c,i} + \frac{0.5\sqrt{f_{wk} f_{wtm}}\,A_f}{\Theta_i h_s}\rho_{wm,i} = \frac{A_f}{h_s}\left(B_c \rho_{c,i} + B'_{wm}\rho_{wm,i}\right) \quad (4)$$

Note that the storey stiffness expressions (Eq. 2 and 4) derived above are a sum of terms each being the product of a constant, B_c, B_{wm} or B'_{wm}, times the dimensionless area of columns ($\rho_{c,j}$) or masonry infill walls ($\rho^e_{wm,i}$ or $\rho_{wm,i}$), respectively. Since stiffness relates to period, and from there to spectral displacement demand and therefore to interstorey drift or damage, it is evident that to control damage, stiffness should exceed a minimum limit value. This expressed in terms of the area ratios of vertical members takes the form:

$$\rho_{c,i} + \frac{B_{wm}}{B_c}\rho^e_{wm,i} \geq \rho_{min} \rightarrow \text{dual system} \quad (5a)$$

$$\rho_{c,i} + \frac{B'_{wm}}{B_c}\rho_{wm,i} \geq \rho_{min} \rightarrow \text{frame} \quad (5b)$$

The ρ_{min} value depends on the number of storeys above the floor level in consideration, the degree of damage that may be tolerated (quantified by the magnitude of acceptable interstorey drift, Θ_i), and the quality of materials (a reduced member stiffness should be considered in case of steel corrosion). This relationship is applied at any storey level and may be adjusted to an intermediate level between successive storeys in case of short columns.

Table 1. Expressions for estimating the stiffness for RC columns, K^c_j, RC walls, K^{wc}_k and masonry infill walls K_i^{wp}.

Element	Lateral stiffness expressions
Columns	$K^c_i = \sum_{j=1,n_c} K^c_{i,j} \approx \frac{E_c A_f}{100 h_s}\rho_{c,j}$
RC walls	$K^{wc}_i = \sum_{k=1,n_{wc}} K^{wc}_{i,k} = \frac{E_c A_f}{h_s}\sum_{k=1,n_{wc}} \frac{\rho_{wc,k}}{\left(4h_s^2/l_{w,k}^2 + 2.50\right)} \approx \frac{E_c A_f}{h_s}\frac{1}{\left(4h_s^2/l_{w,ave}^2 + 2.50\right)}\rho_{wc,i}$
Masonry infill walls	<u>*For Frames*</u>: $K^{wm}_i = \sum_{p=1,n_{wm}} K^{wm}_{i,p} = \frac{0.5\sqrt{f_{wk} f_{wtm}}\,A_f}{\Theta_i h_s}\rho_{wm,i}$, where Θ_i the lateral drift in the i-th floor. <u>*For Dual systems*</u>: $K^{wm}_i = \sum_{p=1,n_{wm}} K^{wm}_{i,p} = \frac{E_m A_f}{h_s}\sum_{p=1,n_{wm}} \frac{\rho_{wm,p}}{\left(4h_s^2/l_{m,k}^2 + 2.50\right)} \approx \frac{E_{wm} A_f}{h_s}\frac{1}{\left(4h_s^2/l_{m,ave}^2 + 2.50\right)}\rho_{wm,i}$
E_c, E_m=elastic modulus of concrete and masonry, respectively; A_f=floor area; h_s=storey height; $\rho_{c,j}$, $\rho_{wc,k}$, $\rho_{wm,p}$=dimensionless area of columns, walls and masonry infills, respectively; f_{wk}=masonry compressive strength; f_{wtm}=masonry flexural strength; $l_{w,ave}$=average length of RC wall; $l_{m,ave}$=average length of masonry infill; Θ_i=storey drift	

Interstorey Drift Spectra for Existing RC Buildings

The Interstorey Drift Spectra (IDS) for retrofitted buildings have been developed as a new design tool by Thermou et al. (2009) for seismic upgrading of existing buildings. The IDS enable the designer to have direct inspection of the consequences, resulting from the selection of the retrofit scenario, on the response characteristics of the structure at the local as well as the global level. With the help of the IDS, it is possible to relate stiffness demand to the target response of the upgraded structure. Similar methods have been utilized in the past for the estimation of the seismic vulnerability of existing buildings (Gülkan and Sozen 1996).

In the approach studied in the present paper, predisposition for damage of a structural system to a given seismic excitation scenario and hence the vulnerability of the system is defined as a function of the expected interstorey drift.

Given the percentage of the area of columns, $\rho_{c,i}$, walls, $\rho_{wc,i}$, and masonry infill walls, $\rho_{wm,i}$, over the floor plan area, as well as the material characteristics (steel and concrete) the Interstorey Drift Spectra for existing RC buildings may be derived.

In the proposed methodology demand is defined by the Type 1 elastic spectrum (EC8 2004) for the design region 0.15 sec<T<2.00 sec. In the present study a high seismicity region is selected with peak ground acceleration α_g=0.36g for subsoil class B. According to Eurocode 8 (2004) the following values apply: soil parameter S=1.2, spectral acceleration amplification factor for 5% viscous damping β_o=2.5, period values that define the limits of the constant acceleration branch T_B=0.15 sec, T_C=0.5 sec, and behavior factor q=1. Demand in terms of spectral displacements, S_d, for period values up to 2.00 sec is defined by:

$$0.15 \leq T \leq 0.50: \; S_d(T) = 0.268T^2 \; ; \; 0.50 < T \leq 2.00: S_d(T) = 0.134T \tag{6}$$

Considering a constant floor plan geometry along the height of the building and constant storey stiffness, the expression for period T for an N-storey building is defined by:

$$T = \frac{2\pi}{\omega} \doteq 4N\sqrt{\frac{M_i}{K_i}} = 4N \cdot Q_i; \text{where } Q_i = \sqrt{\frac{M_i}{K_i}} \tag{7}$$

$M_i=\mu \cdot A_f$ is the total storey mass, where $\mu=0.8\gamma_c t_f$ that corresponds to the 80% of mass activated by the first mode in systems with rigid diaphragms (γ_c is the density of reinforced concrete and t_f the thickness of the horizontal diaphragm). By substituting the storey mass, M_i, and Eq. 2 or 4 in Eq. 7, the term Q_i is:

$$\text{Dual systems: } Q_i = \left[\frac{\mu h_s}{B_c \rho_{c,i} + B_{wm} \rho^e_{wm,i}}\right]^{0.5} \tag{8}$$

$$\text{Frames: } Q_i = \left[\frac{\mu h_s}{B_c \rho_{c,i} + B^{/}_{wm} \rho_{wm,i}}\right]^{0.5} \tag{9}$$

The shape function considered is the shear response shape ($\Phi(x)=\sin(\pi x/2L)$). The interstorey drift of the first floor, Θ_1, is equal to:

$$\Theta_1 = \frac{S_d}{h_s}\sin\frac{\pi}{2N} \tag{10}$$

The interstorey drift of the first storey, Θ_1, for a shear type building is defined by:

$$0.15 \le T \le 0.50 : \Theta_1 = 4.288\frac{N^2 Q_1^2}{h_s}\sin\frac{\pi}{2N} \tag{11a}$$

$$0.50 < T \le 2.00 : \Theta_1 = 0.536\frac{NQ_1}{h_s}\sin\frac{\pi}{2N} \tag{11b}$$

Eq. (11) may be utilized in order to describe the interstorey drift at yield of the first storey, Θ_1, for shear type buildings and simultaneously the interstorey drift at the last storey for flexural type buildings, Θ_N (that would correspond to a flexural response shape: $\Phi(x)=\cos(1-\pi x/2L)$).

Interstorey Drift Envelopes

The expressions derived within the framework of the proposed methodology may be used to construct the IDS for existing RC buildings. One class of interstorey drift envelopes is depicted in Fig. 2(a), where the interstorey drift, Θ_i, is related to the composite index of dimensionless area of vertical members. This index corresponds to the sum of the area ratio of columns ($\rho_{c,i}$) and τo an effective area ratio of masonry infill walls $[(B'_{wm}/B_c)\cdot\rho_{wm,i}]$ for frames, whereas to the sum of the area ratio of columns ($\rho_{c,i}$) and the effective area ratio of walls and masonry infill walls $[(B_{wm}/B_c)\cdot\rho^e_{wm,i}]$ for dual systems. The calculations were performed for: (1) buildings up to six storeys, (2) seven levels of peak interstorey drift, Θ_i, 0.50%, 0.75%, 1.00%, 1.25%, 1.50%, 1.75% και 2.00%, (3) target period values between 0.15 sec≤T≤0.50 sec that correspond to the plateau of the elastic design spectrum, hence to the maximum spectral acceleration values, and (4) a concrete quality C12/15 and masonry infill walls with f_{wk}=5 MPa and f_{wtm}=0.30 MPa. The results presented herein correspond to both shear and flexural type of buildings with the proviso that the critical storey in the case of shear type buildings is the first floor, whereas in flexural type buildings it is the last storey.

Diagrams similar to those of Fig. 2(a) may be used to estimate the peak interstorey drift at the critical storey for building with various numbers of floors. For an elastic system yielding corresponds to an interstorey drift of 0.5%, whereas for a system of ductility equal to 2, the associated drift is 1.0%. A critical issue that needs to be examined in case of existing buildings is whether the building may reach yielding or whether premature failure in shear may occur prior to yielding. If the latter, the drift values taken from the curves of Fig. 2(a) need to be corrected by multiplying them with the ratio of the shear strength, $V_{Rd,tot}$, over the capacity strength of the storey, $V_{y,tot}$. The shear strength of a given storey is given by the summation of the shear strengths of its vertical members. The shear strength of an existing member, $V_{shear,j}$, comprises the contributions of the web reinforcement (stirrups), $V_{w,j}^{st}$, and of the concrete web, V_c:

$$V_{shear,j} = k(V_{w,j}^{st} + V_{c,j}) \tag{12}$$

Based on recent research, shear resistance of reinforced concrete members subjected to cyclic shear reversals degrades with the number of cycles and the magnitude of the imposed displacement ductility, μ_Δ. This phenomenon is taken into account in the assessment of the resistance of the existing cross section through the reduction factor, k, which is given as a function of displacement ductility, μ_Δ (Moehle et al. 2002). The shear force required to develop the ideal flexural resistance of the member is equal to $V_{y,j}=M_{y,i}/L_s$, where L_s is the shear span length. If the building yields at an average displacement value, $\Delta_{y,ave}$, the shear strength and capacity are defined as $V_{Rd,tot}=\Sigma V_{shear,j}$ and $V_{y,tot}=\Sigma V_{y,j}$, respectively, whereas the yield point is defined by:

$$\Delta_{Rd} = \Delta_{y,ave} \frac{V_{Rd,tot}}{V_{y,tot}} \leq \Delta_{y,ave} \tag{13}$$

This means that the drift values as defined from the curves of Fig. 2(a) will have to be reduced by the ratio $V_{Rd,tot}/V_{y,tot}$ if <1.

Various forms of alternative design charts may be constructed. Such an example is depicted in Fig. 2(b) where for a given value of peak storey drift, Θ_i=1%, the relationship between the area ratio of the first storey columns ($\rho_{c,i}$) and the effective combined area ratio of masonry infill and RC walls (expressed by the term $\rho^e_{wm,i}$ for dual systems) for frames up to six storeys is presented. The figures are plotted for interstorey drift Θ_i=1% and concrete quality C20/25. It is observed that as the number of storeys increases, the required percentage of the two parameters increases for attainment of the same damage level. Another class of design chart could be constructed if it is of interest to assess buildings with a specific number of storeys and of a specific concrete quality for various drift levels.

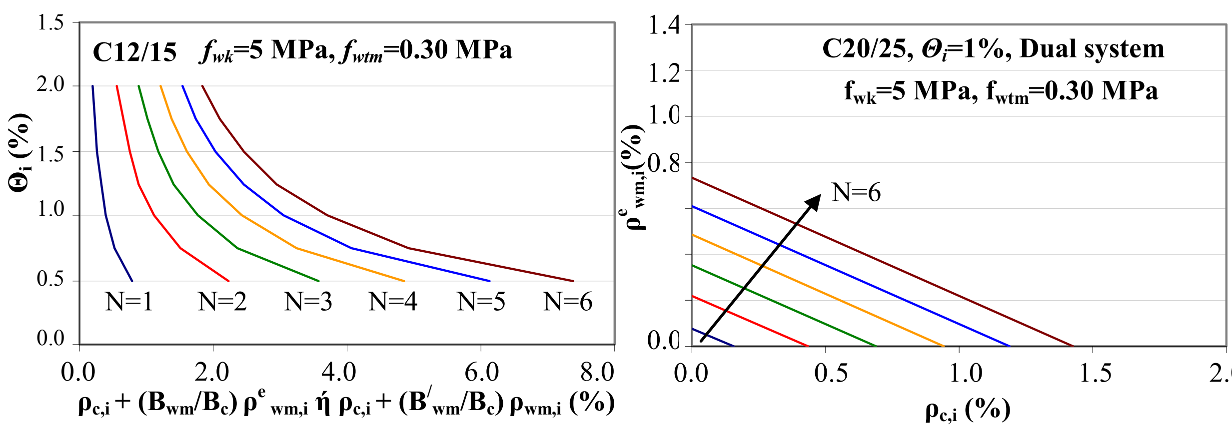

Figure 2. Vulnerability curves which relate (a) the composite dimensionless index of area of the vertical floor members for various levels of drift, Θ_i; (b) the percentage of columns, $\rho_{c,i}$, and the equivalent percentage of RC and masonry infill walls, $\rho^e_{wm,i}$, of the storey for a given quality of concrete for drift Θ_i=1.0% for dual system.

Illustrative Example

A three-storey RC frame severely damaged during the 2003 Bingöl earthquake (Ramirez et al. 2009) was selected to be assessed utilizing the interstorey drift envelopes derived according to the proposed methodology. The typical floor plan layout of the building is shown in Fig. 3(a). The dimensions of the 14 columns are presented in Table 2. The thickness of the masonry walls was 0.24 m. Based on the data provided by Ramirez et al. (2003) and necessary assumptions made from the descriptions for element detailing and material properties provided, the percentage of the longitudinal reinforcement of columns was 1% of the column area, steel stress at yield was f_y=300 MPa for the longitudinal reinforcement and f_y=220 MPa for stirrups, concrete had a nominal concrete compressive strength $f_c^{/}$=12 MPa (low quality), whereas masonry had a compressive strength f_{wk}=3 MPa. Transverse reinforcement comprised stirrups of 8 mm bar diameter at 250 mm (Fig. 3(b)). Both longitudinal and transverse reinforcement comprised smooth bars.

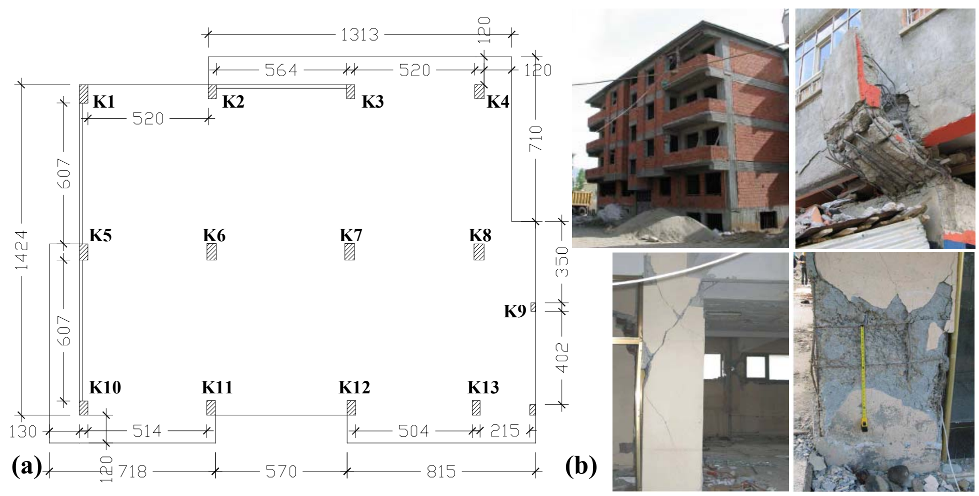

Figure 3. (a) Floor plan of the RC frame (dimensions in cm); (b) Typical RC frame building, column reinforcement details and column shear failure (Ramirez et al. 2003)

The results of the shear strength assessment of the ground floor columns are presented in Table 2. As it is shown in the last column of Table 2, where the shear strength is compared to the flexural strength, all the columns failed in shear before reaching yield. The average interstorey drift at yield was estimated equal to $\Theta_{y,ave}$=0.35% (Table 2). Thus, yield was expected to occur at a lower interstorey drift value due to premature failure in shear equal to $\Theta_{RD}=\Theta_{y,ave}V_{Rd,tot}/V_{y,tot}$ =0.35·0.36=0.13% ($V_{Rd,tot}/V_{y,tot}$=0.36).

Given the floor plan of Fig. 3(a), the percentage of columns, $\rho_{c,i}$, was estimated equal to $\rho_{c,i}$=1.0%, whereas the percentage of the masonry infill walls was equal to $\rho_{wm,i}$= 1.34%. The vulnerability curves that relate the composite dimensionless index of area of the vertical floor members, $[\rho_{c,i} + (B_{wm}/B_c)\, \rho^{e}_{\ wm,i}]$ or $[\rho_{c,i} + (B^{/}_{wm}/B_c)\, \rho_{wm,i}]$, for various levels of drift, Θ_i were constructed in Fig. 4(a) for $f_c^{/}$=12 MPa and f_{wk}=3 MPa. For the drift level Θ_{RD}=0.13% demand in terms of the composite index is equal to $[\rho_{c,i} + (B^{/}_{wm}/B_c)\, \rho_{wm,i}]$=6.9%. This value may be satisfied for various combinations of $\rho_{c,i}$ and $\rho_{wm,i}$ as shown in Fig. 4(b). Thus two alternative solutions were defined depending on whether $\rho_{c,i}$ or $\rho_{wm,i}$ would be kept constant. The first one described by the blue arrows in Fig. 4(b) required $\rho_{c,i}$=1% and $\rho_{wm,i}$=3.6%, whereas the second described by the green arrows in Fig. 4(b) required $\rho_{wm,i}$=1.34% and $\rho_{c,i}$=4.67%.

From the above, it is obvious that none of the requirements of the two alternative solutions is satisfied by the RC frame building, denoting the deficiency of the system to sustain a drift level in the first storey as low as 0.13%. The conclusion is that damage is expected to occur both in RC columns and masonry infill walls.

Table 2. Shear strength assessment of columns.

Columns	b	h	N_i (kN)	v_i	$ID_{y,i}$ (%)	$M_{y,i}$ (kNm)	$V_{y,i}$ (kN)	$L_{b,a}$ (mm)	$V_{w,i}$ (kN)	$V_{c,i}$ (kN)	$V_{shear,i}$ (kN)	$V_{shear,i} < V_{y,i}$
K1	370	790	237.8	0.07	0.27	369.4	188.5	376	22.1	31.5	53.6	✔
K2	350	600	428.6	0.17	0.35	267.6	136.5	281	22.1	33.8	55.9	✔
K3	340	600	474.1	0.19	0.35	275.5	140.6	281	22.1	35.3	57.4	✔
K4	400	600	364.2	0.13	0.35	272.1	138.8	281	22.1	33.3	55.4	✔
K5	370	700	427.4	0.14	0.30	357.3	182.3	331	22.1	37.1	59.2	✔
K6	400	720	639.5	0.19	0.29	463.4	236.4	341	22.1	48.4	70.5	✔
K7	420	720	646.1	0.18	0.29	478.2	244.0	341	22.1	49.7	71.8	✔
K8	430	700	395.6	0.11	0.30	382.9	195.4	331	22.1	38.8	60.9	✔
K9	200	370	267.9	0.30	0.57	73.2	37.4	166	13.1	17.5	30.6	✔
K10	370	610	357.0	0.13	0.34	264.4	134.9	286	22.1	31.8	53.9	✔
K11	370	620	413.3	0.15	0.34	287.6	146.7	291	22.1	34.3	56.4	✔
K12	370	620	415.4	0.15	0.34	288.2	147.0	291	22.1	34.4	56.5	✔
K13	340	630	259.3	0.10	0.33	237.2	121.0	296	22.1	26.5	48.6	✔
K14	250	450	98.5	0.07	0.47	79.2	40.4	206	16.3	13.5	29.7	✔
				$ID_{y,ave}$=	0.35	$V_{y,tot}$ =	2090.0			$V_{Rd,tot}$=	760.7	

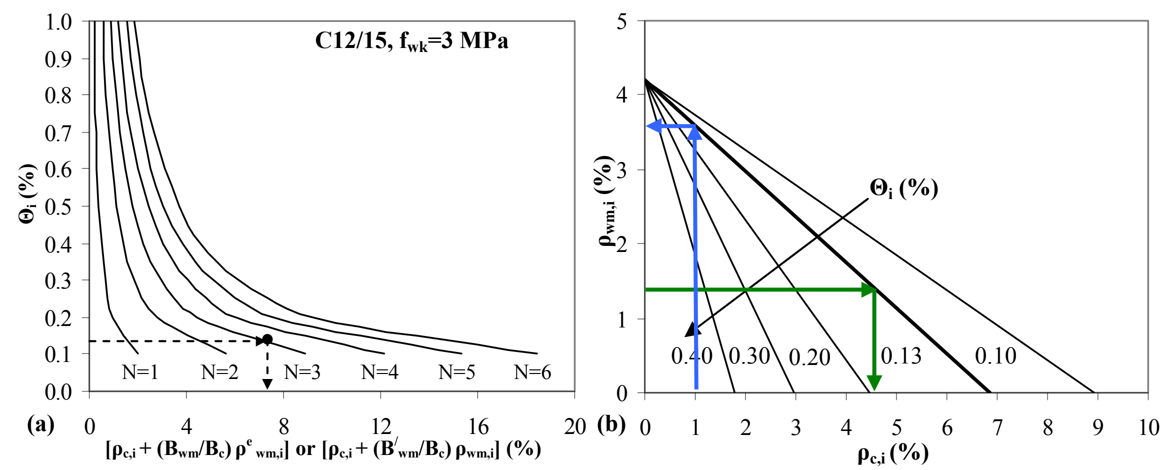

Figure 4. Vulnerability curves that relate (a) the composite dimensionless index of area of the vertical floor members for various levels of drift, Θ_i; (b) the percentage of columns, $\rho_{c,i}$, and (a) the percentage of masonry infill walls, $\rho_{wm,i}$ for frames.

Conclusions

In this paper a methodology for direct assessment of the seismic vulnerability of buildings of old type construction with masonry infill walls was presented. The methodology is based on the assumption that the lateral storey stiffness of a RC building is given by the summation of stiffness of all vertical members undergoing the same lateral translation. Building lateral stiffness is obtained as the work equivalent contributions of the individual storeys, the weighting function being the fundamental shape of vibration. Demand is expressed in terms of lateral interstorey drift and is related through pertinent expressions to the lateral stiffness of the column, wall, and infill area ratios within a storey. With the selection of the shear response shape, damage is estimated in terms of interstorey drift for a specific seismic hazard representing the Design Code scenario. Interstorey Drift Spectra (IDS) were derived for rapid assessment of existing RC buildings, whereby the anticipated drift demand is linked to the design characteristics of the structure. The proposed methodology is simple in its application and the extracted vulnerability curves facilitate substantially the designer in the assessment of existing RC buildings. Seismic vulnerability of existing buildings in terms of interstorey drift of the critical storey is assessed given the geometrical characteristics and the material properties of the system. The implementation of the proposed methodology to an existing building that was severely damaged in recent earthquake verifies the validity and the efficiency of the proposed methodology.

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