13[th] World Conference on Earthquake Engineering
Vancouver, B.C., Canada
August 1-6, 2004
Paper No. 424

INFLUENCE OF FLOOR STRUCTURES ON SEISMIC PERFORMANCE OF RC FRAMES

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SUMMARY

Modern codes for earthquake resistant design allow, for particular situations characterized by structural regularity, use of simplified procedures based on plane models which, however, are unable to take into account the spatial interaction with the other components of the structure. The present work is aimed at evaluating the influence of the beam-floor interaction on the absorption of seismic actions in RC framed structures and, in particular, at estimating the reduction in terms of internal forces experienced by beams as a consequence of the contribution of elements belonging to floor structures. For this purpose a parametric investigation has been developed; it identified the direction of floor joists parallel to the beam as the most important factor with regard to the above cited interaction (reduction in bending moment in beams up to 40%). Inelastic analyses, executed according to a performance based approach, have shown a tendency towards an increase of damage levels due to the influence of floor structures.

INTRODUCTION

Among various strategies for earthquake-resistant design, the most used one is based on procedures which aim to provide the structure with energy dissipation capacity on the occasion of severe seismic events. This target is pursued, in general, by the application of a hierarchical criterion (CEN [1]) which, through suitable strength and ductility levels conferred to structural elements, leads to a structural response characterized by spread plasticity and stable hysteretic mechanisms. On the other hand, recent seismic events have demonstrated that structural performance may be influenced, for instance, by local failures having a brittle nature or by the onset of dissipation mechanisms which are different with respect to the expected ones due to the interaction with other components, structural or not, of the building. Despite the importance of this problem for earthquake–resistant structures, researches about the floor-beam interaction are limited. Most of them deals with the validity of rigid floor hypothesis and with consequences of the assumption of flexible diaphragm on the inelastic response of the structure (Button [2], Roper [3], De Matteo [4], Dolce [5], Masi [6]).

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Based on these remarks, an investigation has been carried out in order to evaluate effects of interaction between floor structures and adjacent beams in RC buildings, interaction which may induce an alteration of "weak beam - strong column" mechanism, fundamental for the achievement of a satisfactory inelastic response. In particular, the research was aimed at evaluating variation of solicitation levels in beams delimiting floor zones with respect to results obtained by conventional procedures which in general disregard the spatial nature of the problem. Besides, a calculation of the spatial structure in the elastic-plastic range, accounting for the complicated hysteretic mechanisms characterizing behaviour of RC elements, is rather onerous. For this reason the investigation has been initially based on a simple elastic model (Barbetti [7]), representing a portion of a spatial frame having a usual geometry if referred to the Italian building practice. The elastic modelling allowed a parametric analysis which identified the direction of floor joists parallel to the beam as the most important factor with regard to the floor-beam interaction. Results of parametric analysis have been then used in order to evaluate the effects of floor collaboration on the inelastic performance of a sample frame. It has been observed that such a collaboration determines, in general, an increase in damage indexes with a consequent abatement of safety levels of the structure.

THE INVESTIGATION

The investigation has been divided into two phases. The first one was addressed to define alteration of internal forces in the beams due to interaction with floor structures. For this purpose a parametric investigation has been developed, based on a sub-structure representative of a portion of a spatial frame. The second phase was aimed at evaluating effects due to the above interaction on the inelastic response of a sample frame.

Elastic modelling of the selected sub-structure

Investigation was developed in a parametric way, based on a finite element model representing a portion of a spatial frame characterized by regularity in plan and along the height (Figures 1). In particular the model, including adjacent floor fields and relating beams, has been defined by assuming contra-flexure points at mid-span of beams and mid-height of columns, according to a structural deformation due to only lateral loads. Use of an elastic model can be justified by the fact that the objective of the parametric analysis was to define the point of the onset of the inelastic phase as determined by the floor-beam interaction; therefore this assumption corresponds to a linearization of the elastic-cracking phase, usual in the analyses of RC structures.

It was assumed a floor type frequently adopted in the Italian practice, consisting of RC joists and upper slab, with interposed hollow tiles. Contribution of tile elements was disregarded in the analysis.

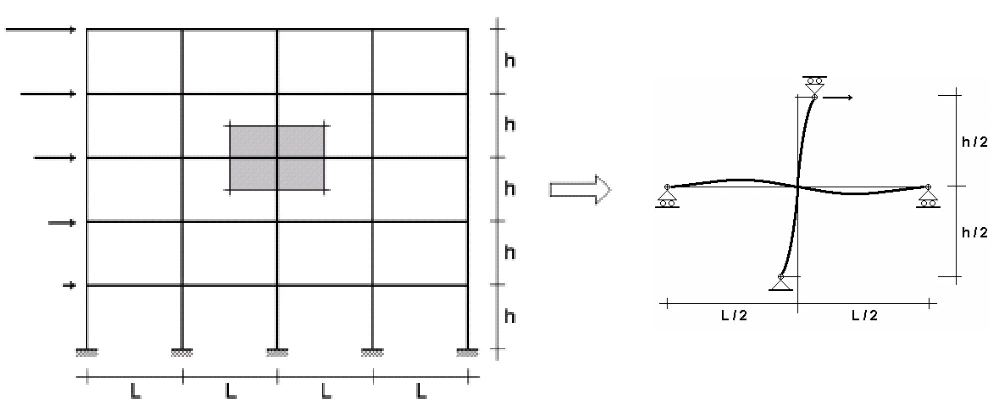


Figure 1. Definition of the sub-system for the parametric analysis.

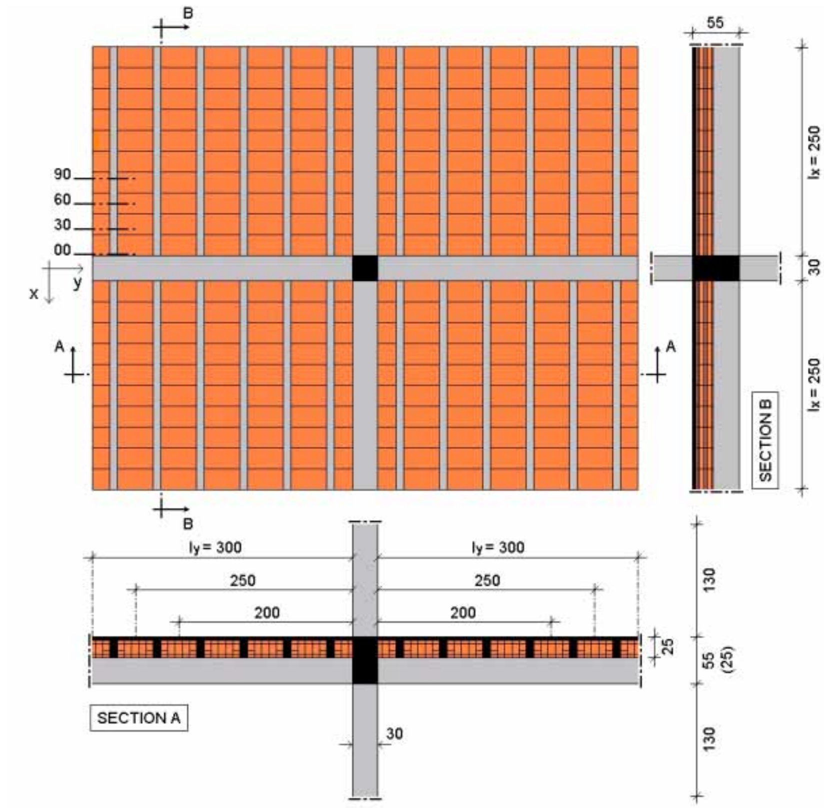


Figure 2. Basic configuration for the analyzed models (dimensions in cm).

Variables of study were dimensions of floor fields, stiffness of beams bordering floor fields, presence or not of beam flange, in the two possible configurations of floor joists parallel or perpendicular to the beam of the considered frame (Nudo [8]). Basic configuration for the assumed models is illustrated in Figure 2. Analyzed models were divided into two groups, each consisting of twelve models, and referred to a hypothetical frame having direction parallel to the *x* axis. Characteristics of the first group are reported in Table 1, where joist direction is assumed to be parallel to the reference beam (*x* direction).

Table 1. Geometrical characteristics of models 1÷12 (dimensions in cm).

model n.	beam cross-section (b×H)		floor dimensions		flange (beam y)
	beam x	beam y	lx	ly	
1	30×55	30×55	250	300	no
2	30×55	30×55	250	300	yes
3	30×55	30×55	250	250	no
4	30×55	30×55	250	250	yes
5	30×55	30×55	250	200	no
6	30×55	30×55	250	200	yes
7	30×55	30×25	250	300	no
8	30×55	30×25	250	300	yes
9	30×55	30×25	250	250	no
10	30×55	30×25	250	250	yes
11	30×55	30×25	250	200	no
12	30×55	30×25	250	200	yes

* Column dimensions: cross-section 30×30 cm, mid-height 130 cm.

Models of the second group had the same characteristics as the first one, except for joist direction which is assumed to be perpendicular to the reference beam (*y* direction).

Elastic analyses have been developed by a refined mesh (Figure 3) consisting of 8-nodes "brick" elements. Internal forces have been monitored at four different locations, respectively positioned at 0 cm (section 00), 30 cm (section 30), 60 cm (section 60), 90 cm (section 90) from the reference beam.

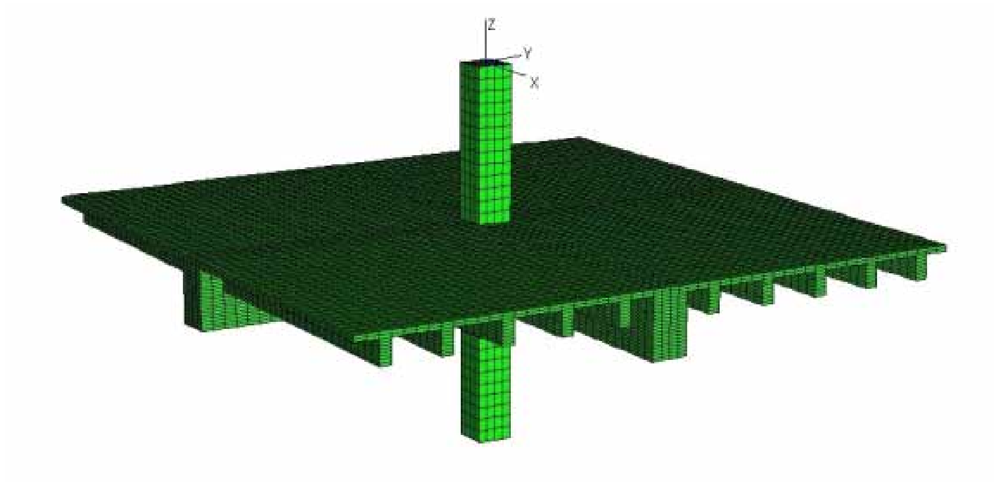
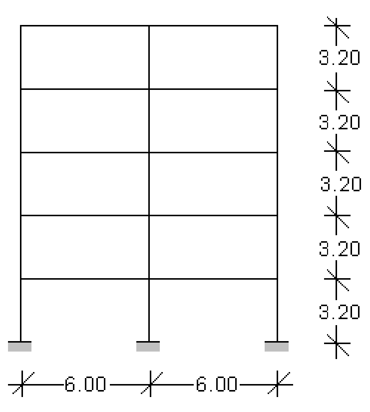


Figure 3. The finite element model.

Inelastic analysis of the sample frame

Results from the parametric investigation have been subsequently used to evaluate effects due to floor-beam interaction on the inelastic response of a 2-bays, 5-stories RC sample frame (Figure 4). It has been dimensioned according to Eurocode 8 (EC8) specifications for high ductility class, assuming soil type B (medium-stiffness soil) and a peak ground acceleration (PGA) equal to 0.35 *g*.

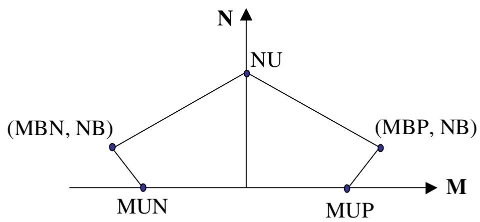
Inelastic investigation has been developed by using the code IDARC-2D (Valles [9]). In particular, two different procedures have been used for evaluation of inelastic response: static (push-over) analysis and dynamic analysis. Mechanical characteristics of structural elements at critical regions have been defined by means of a tri-linear moment-curvature relationship and by simplified *M-N* interaction domains (Figure 5a). In the case of dynamic analyses, refined hysteretic laws (Sivalsen [10]) accounting for the main phenomena of mechanical degradation typical of RC elements have been adopted (Figure 5b).



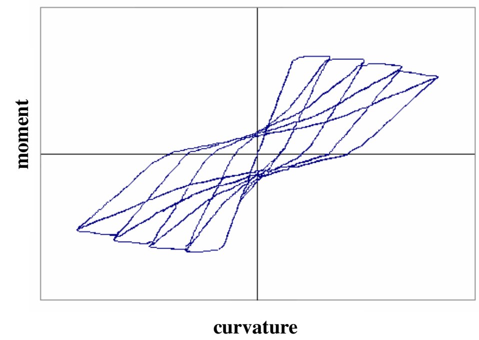
	beams			central column		lateral columns	
story	b×H (cm)	ρ (%)	ρ' (%)	b×H (cm)	ρ_{tot} (%)	b×H (cm)	ρ_{tot} (%)
1	30×60	0.9	0.6	40×50	1.3	30×45	1.4
2	30×60	0.9	0.6	40×45	1.7	30×40	1.6
3	30×55	1.0	0.7	40×45	1.4	30×35	2.4
4	30×50	1.1	0.8	40×40	2.0	30×30	2.8
5	30×45	1.2	0.9	40×40	2.0	30×30	2.8

concrete: f_{ck} = 25 MPa
steel: f_{yk} = 440 MPa

Figure 4. Sample frame: geometrical characteristics, reinforcements and materials.



a)



b)

Figure 5. *M-N* interaction domain and evolutive-degrading hysteretic model (Valles [9]).

In order to reproduce effects of floor-beam interaction on sample frame, as determined by the parametric analysis, suitable strength characteristics have been assigned to beams so as to delay attainment of the plastic phase.

Dynamic analyses have been performed assuming a seismic input consisting of five accelerograms characterized by an average spectrum which is very close to the elastic response spectrum of EC8 for the specified soil class and PGA. Elastic spectra of the five records, their average spectrum and the elastic spectrum of EC8 are shown in Figure 6. Evaluation of inelastic response of the sample structure has been carried out according to a performance based approach; for this reason evolution of plastic response of the sample structure has been studied by adopting increasing values of PGA, obtained by scaling in a suitable way the five accelerograms.

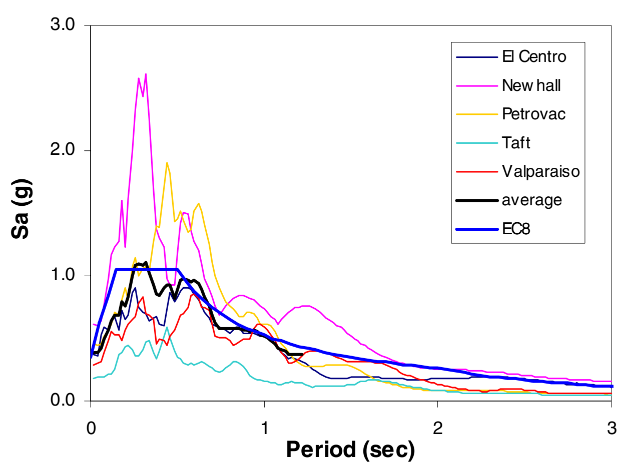


Figure 6. Elastic spectra of records used for the dynamic analysis.

RESULTS AND DISCUSSION

Results of the elastic analysis

Results of the preliminary elastic analysis are presented in terms of absorption of bending moment by the reference beam. In particular, Figures 7 and 8 illustrate, with reference to the two groups of analyzed models, variations between bending moments deriving from the finite element model and values obtained by conventional calculation procedures based on a plane frame composed by one-dimensional elements.

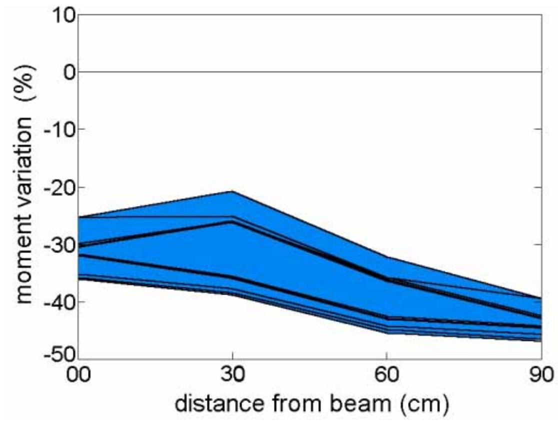


Figure 7. Parallel joist models: variation of bending moment in the beam due to floor collaboration.

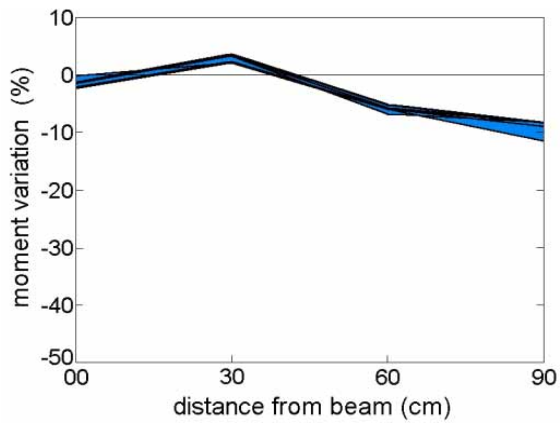


Figure 8. Perpendicular joist models: variation of bending moment in the beam due to floor collaboration.

Evaluation of stress distribution at section "00" pointed out a "joint effect", that is an alteration in distribution of normal and shear stresses near joint regions, as already found in previous works of the authors (Nudo [11]). Obtained results show that the factor "floor joist direction" is the most important one with respect to the other analyzed factors; in particular, in the case of floor joists parallel to the reference beam a reduction in bending moment over 40% has been observed, whereas the same reduction was more limited in the case of floor joists perpendicular to the reference beam (about 10%).

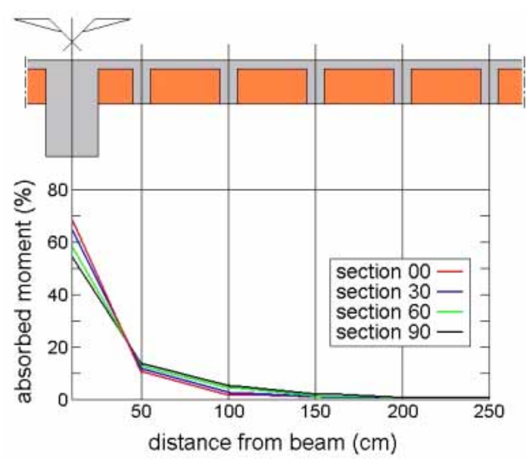


Figure 9. Contribution of floor joists in absorbing bending moment.

Contribution of each floor joist in absorbing bending moment is illustrated in Figure 9 with reference to the situation of floor joists parallel to the beam. From the diagram it is possible to notice that the floor joist adjacent to the beam gives a contribution of about 12% in absorbing bending moment, the subsequent joist of about 4%, while contribution of remaining elements is negligible. By this way it is then possible to identify a collaborating floor zone which includes, in substance, the two joists near to the beam. In this respect, it has to be underlined that the main amount of bending moment absorbed by joists is due to the couples generated by eccentric axial forces that develop inside them and into the beams.

Inelastic response of the sample frame

Non-linear static analysis
Evaluation of effects induced by floor collaboration on the inelastic response of the sample frame has been carried out with reference to the situation that proved to exert the greater influence on structural response, that is floor joists parallel to the beam. To this end a preliminary study, consisting in performing a push-over analysis of the twelve models of the first group, has been developed. The relative normalized "base shear (*V/W*) – top displacement (*s/H*)" diagrams are illustrated in Figure 10. In the same figure curves corresponding to the average response of models and to capacity of sample frame without floor collaboration (SAMPLE) have been also reported.

From Figure 10 it can be observed that floor collaboration induces a remarkable increase in strength of the structure and, therefore, a considerable delay in attainment of plastic phase; on the other hand no substantial variations in responses of different models can be appreciated, therefore subsequent analyses have been performed with reference to a sample structure having mechanical properties averaging characteristics of the twelve models (MEAN).

From curves of Figure 10 it is also possible to observe a reduction in global ductility for models accounting for floor collaboration. This evidence is also confirmed by plastic patterns shown in Figure 11. In particular it illustrates distributions of plastic hinges at different steps of loading history with reference to frames accounting for floor collaboration (MEAN) or not (SAMPLE). Due to the different strength capacity of the two models, definition of conventional point representing the change from elastic to plastic phase has been based on the intersection between extensions of elastic and plastic branches of the respective response curves (Park [12]). From the same figure it is also possible to notice that floor collaboration determines a plastic burdening in columns, so reducing possibility of activating a more effective collapse mechanism characterized by a greater involvement of beams.

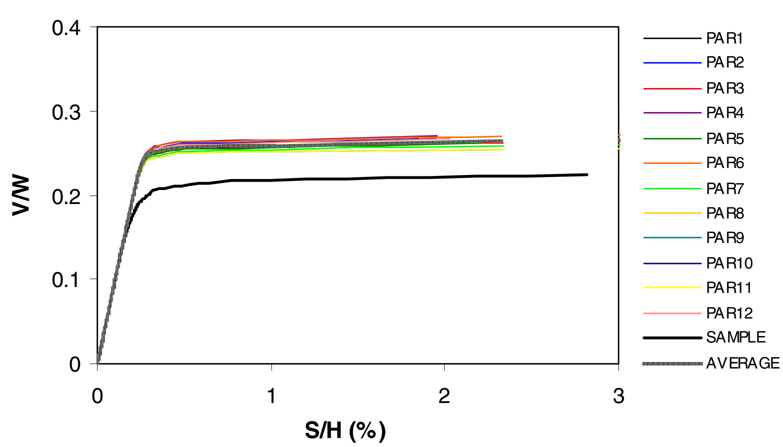


Figure 10. Push-over curves of the parallel joist models.

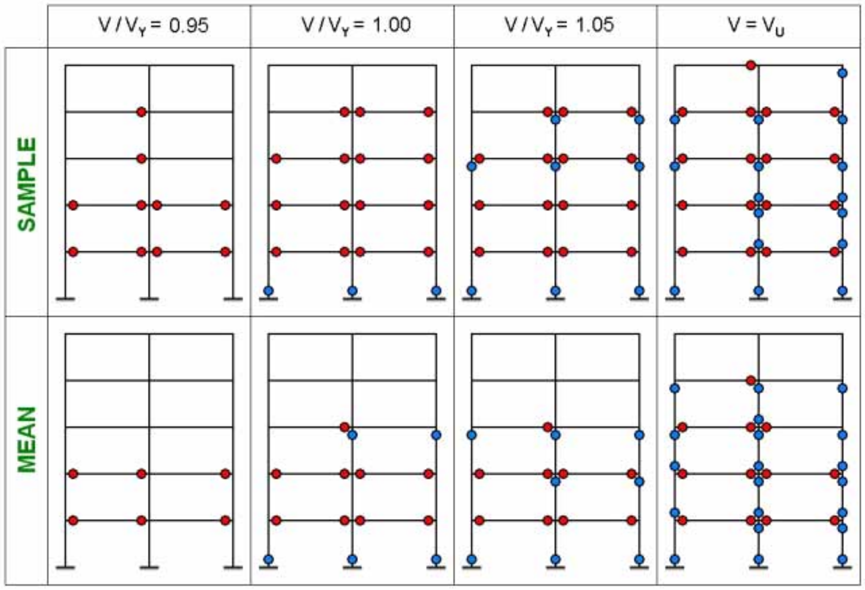


Figure 11. Distribution of plastic hinges at different loading steps (V_Y, V_U: yielding, ultimate shear).

Non-linear dynamic analysis

Tables 2 and 3 contain results, in terms of maximum displacements at top story of the analyzed sample frames, including or not floor collaboration, obtained for each of above specified accelerograms. Figures 12 and 13 illustrate, respectively, results of the dynamic analysis in terms of maximum displacements, measured at the top of the frame, and of interstory drifts for increasing values of the PGA. From these diagrams it is possible to observe that floor collaboration determines a reduction in absolute displacements, whereas interstory drifts increase. Since interstory drifts are generally accepted as damage indicators for frame structures, it follows that floor collaboration determines an early attainment of damage thresholds then reducing safety levels of the structure. In other words floor collaboration causes an evolution of crisis mechanisms characterized by a reduced dissipation capacity, therefore contrary to objectives pursued by the modern design codes.

Table 2. Maximum displacements (mm) at top story of the frame taking into account floor collaboration.

Record	0.2 *g*	0.25 *g*	0.3 *g*	0.35 *g*	0.4 *g*	0.45 *g*	0.5 *g*	0.55 *g*	0.6 *g*
El Centro	60.0	69.9	73.5	75.0	79.9	87.9	100.5	116.9	121.0
Newhall	72.5	91.0	112.9	126.6	135.4	164.0	183.7	210.3	245.6
Valparaiso	51.0	61.8	68.7	89.5	103.3	115.9	124.3	135.2	161.7
Taft	39.3	42.8	46.7	49.9	51.1	59.3	71.3	88.5	98.7
Petrovac	61.7	71.8	81.5	90.4	101.4	106.9	117.6	126.4	127.1
average	56.9	67.5	76.7	86.3	94.2	106.8	119.5	135.5	150.8

Table 3. Maximum displacements (mm) at top story of the frame not taking into account floor collaboration.

Record	0.2 *g*	0.25 *g*	0.3 *g*	0.35 *g*	0.4 *g*	0.45 *g*	0.5 *g*	0.55 *g*	0.6 *g*
El Centro	52.9	58.7	62.7	66.5	78.8	88.4	96.3	103.7	115.8
Newhall	73.9	91.5	108.6	117.8	151.9	177.0	210.5	254.1	285.6
Valparaiso	55.3	65.4	73.8	92.7	93.7	120.3	141.5	173.0	197.9
Taft	33.5	35.7	38.0	46.2	62.4	75.0	87.5	100.9	124.7
Petrovac	61.0	70.6	81.4	88.3	97.7	101.3	107.9	137.5	152.5
average	55.3	64.4	72.9	82.3	96.9	112.4	128.7	153.8	175.3

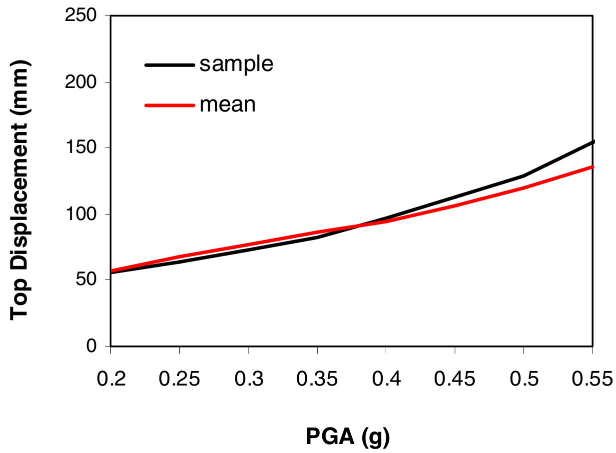


Figure 12. Average of maximum displacements for increasing PGA.

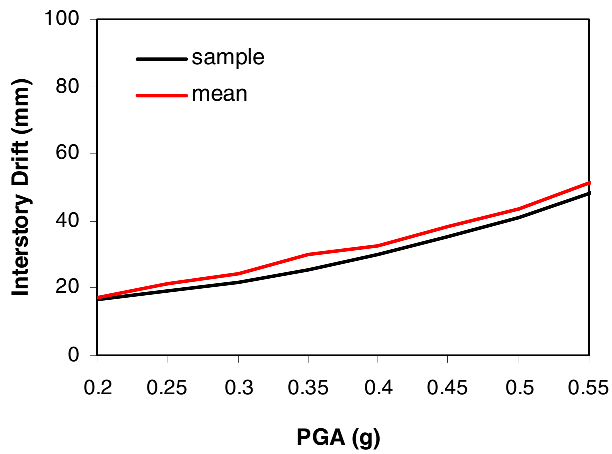


Figure 13. Average of maximum interstory drifts for increasing PGA.

Figure 14 illustrates profiles along the height of maximum interstory drifts for sample frame taking into account or not floor collaboration, due to the El Centro record for a PGA equal to 0.5 g. It is observed a local magnification of drifts due to floor collaboration. Figure 15 shows the corresponding plastic patterns where a greater involvement of columns with respect to beams is pointed out.

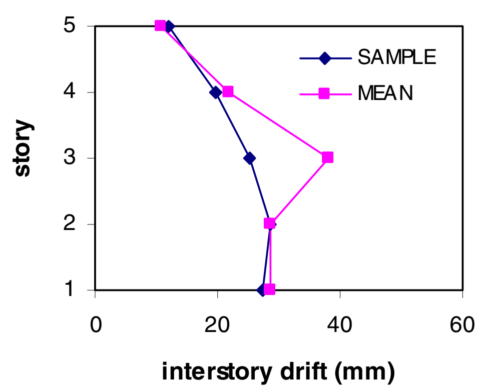


Figure 14. Maximum interstory drifts obtained for El Centro record (PGA = 0.5 g).

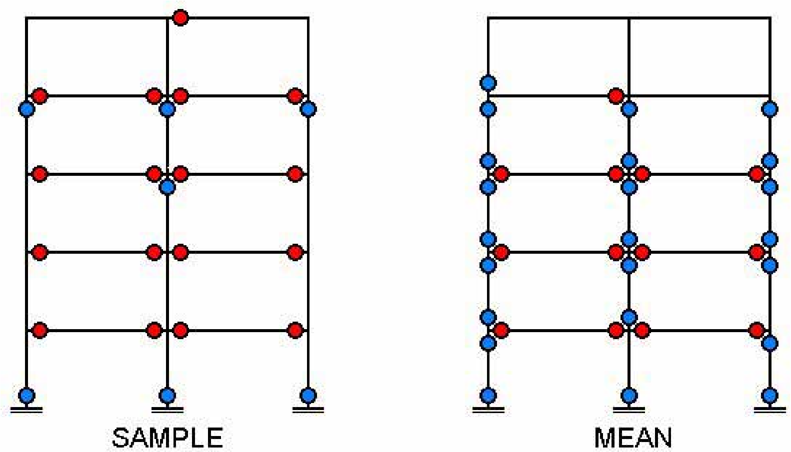


Figure 15. Distribution of plastic hinges for El Centro record (PGA = 0.5 *g*).

Fragility analysis

Results given by non-linear dynamic analysis have been also used to assess structural response according to a performance based approach; in particular, the influence of floor collaboration on seismic response has been determined through a fragility analysis, that is by evaluating probability of exceeding suitable limit states or damage thresholds.

As is known, points of fragility curves represent probability that the response parameter of the structure exceeds the assumed limit state for a given earthquake intensity. In the specific case evaluation of fragility curves has been carried out by assuming for the response domain a conventional probability density (Gaussian), characterized by the mean (μ) and standard deviation (σ) obtained from the sample of results provided by the dynamic analysis. Each response domain was represented by structural response in terms of interstory drifts obtained by considering as seismic input the ensemble of accelerograms previously defined. Due to the limited dimension of the assumed sample, corresponding statistical parameters (μ, σ, *cov*) have to be considered as indicative and randomness assigned to samples might be strongly approximated in lack. The function adopted to represent fragility curves was the two parameter log-normal distribution, which is completely defined if at least three points of it are known (Barron Corvera [13]). In this work fragility curves have been defined by nine points obtained assuming, as seismic input, increasing values of PGA from a minimum of 0.2 *g* to a maximum of 0.6 *g*, with steps of 0.05 *g*.

After definition of response domains relating to different values of PGA, each of them has been compared to limit values corresponding to suitable performance classes. For this purpose, damage thresholds proposed by the SEAOC [14] have been adopted (see Table 4); such a classification assumes interstory drifts as response parameters for framed structures.

Table 4. Limit values assumed for performance analysis [14].

Limit States	Interstory drift	
	(% h)	*(mm)*
Fully Operational	0.5	16
Operational	1.5	48
Life Safe	2.5	80
Collapse Prevention	5	160

By this way it was possible to evaluate probability of exceeding performance thresholds corresponding to the limit states "Fully Operational" (*FO*), "Operational" (*OP*) and "Life Safe" (*LS*). Efficacy in representing fragility curves depends on position of points obtained as a function of seismic input. In fact, it is clear that definition of fragility curves is better if points are positioned near the intermediate part of curves, that is in zones where probability values are far from the lower limit (P=0) and the upper limit (P=1).

Diagrams in Figure 16 show probability of threshold exceeding (PTE) as a function of PGA, for the two different models taking into account or not floor collaboration. It can be noticed that fragility curves corresponding to the considered limit states (*FO*, *OP*, *LS*) have been fitted in a satisfactory way.

Figure 17 illustrates fragility domains obtained by superimposing fragility curves due to the analyzed models with reference to the assumed limit states. Therefore they express the influence of beam-floor interaction on safety levels defined on the basis of the same limit states. It can be noticed that curves representing probability of exceeding the "Fully Operational" and "Operational" performance levels are quite distinct, so defining fragility regions having a not negligible extension; on the contrary, fragility curves corresponding to the "Life Safe" limit state are almost coincident. The amplitude of *FO* and *OP* regions reveals an appreciable incidence of the parameter "floor collaboration" on probability of exceeding the considered limit states. Concerning the "Operational" limit state, for instance, it can be observed that for a value of PGA equal to 0.35 *g* probability of exceeding the assumed damage threshold is of about 8%, for the model without floor collaboration, and of 16% in the case accounting for floor contribution, so reducing in a considerable measure the corresponding safety level.

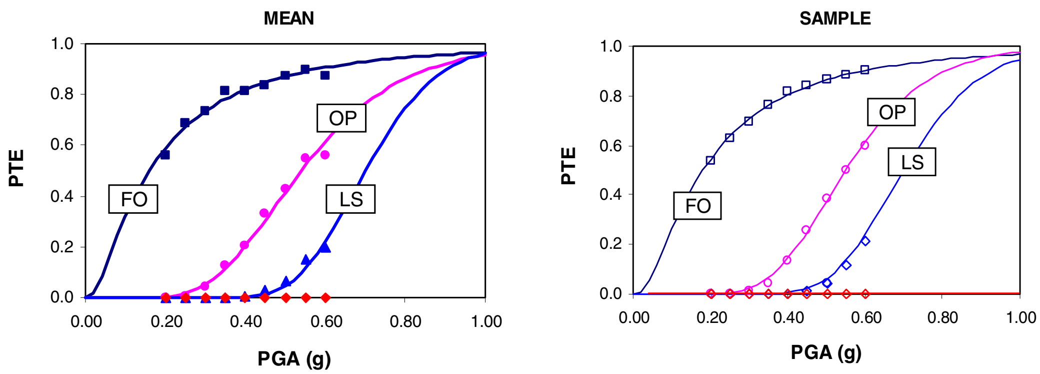


Figure 16. Fragility curves for sample frame taking into account or not floor collaboration.

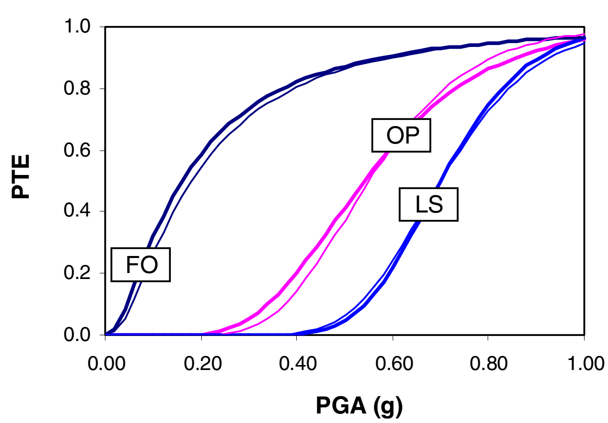


Figure 17. Comparison between fragility curves for floor collaboration.

CONCLUSIONS

The main objective of the work was to evaluate, through a parametric analysis, variation in internal forces of beams belonging to spatial frames due to interaction with floor structures. Obtained results showed that, in the case of floor joists parallel to the frame plane, reduction in bending moment absorbed by beams was of about 40%, while the same reduction was well lesser (about 11%) in the case of floor joists perpendicular to the frame plane. In order to evaluate the influence exerted by floor-beam interaction on seismic response, with particular reference to possibility to alter rules of capacity design, an inelastic investigation has been also performed. Such investigation was limited to a sample structure designed according to EC8 specifications for high ductility class. Obtained results pointed out a negative influence exerted by floor-beam interaction. In particular, developed analyses revealed an increase in plastic demand in columns and larger interstory drifts under the adopted accelerograms. A fragility analysis showed also a greater probability of exceeding conventional damage thresholds in the case of frame model including floor collaboration.

REFERENCES

1. CEN. "Eurocode 8: Design of structures for earthquake resistance". Part 1: General rules, seismic actions and rules for buildings. Draft N. 6, Brussels, November 2002.
2. Button, M.R., Kelly, T.E., Jones, L.R. "Influence of diaphragm flexibility on the seismic response of buildings". Proc. 8th World Conference on Earthquake Engineering, San Francisco, Vol. 4, 1984: 759-766.
3. Roper, S.C., Iding, R.H. "Appropriateness of the rigid floor assumption for buildings with irregular features". Proc. 8th World Conference on Earthquake Engineering, San Francisco, Vol. 4, 1984: 751-758.
4. De Matteo, F., Lorusso, F., Mancusi, A., Masi, A. "Sulla deformabilità dell'impalcato negli edifici in c.a. in zona sismica". L'Industria Italiana del Cemento, 1988, 12: 776-788.
5. Dolce, M., Lorusso, V.D., Masi, A. "Inelastic seismic response of building structures with flexible diaphragm". Proc. 10th World Conference on Earthquake Engineering, Madrid, Vol. 7, 1992: 3967-3972.
6. Masi, A. "Il ruolo degli orizzontamenti nella risposta sismica non lineare di strutture multipiano in c.a.". Proc. 7th Italian Conference on Earthquake Engineering, Siena, Vol. 2, 1995: 983-992.
7. Barbetti, G., Nudo, R., Sarà, G. "Inelastic seismic response of reinforced concrete columns affected by non-uniformities in joint zones". Proc. 11th European Conference on Earthquake Engineering, Paris, 1998 (CD-Rom).
8. Nudo, R., Sarà, G., Viti, S. "Influenza dell'interazione trave-solaio sull'assorbimento delle azioni sismiche nelle strutture intelaiate in c.a.". Proc. 11th Italian Conference on Earthquake Engineering, Genova, 2004 (Cd-Rom).
9. Valles, R.E., Reinhorn, A.M., Kunnath, S.K., Li, C., Madan, A. "IDARC-2D: A computer program for the inelastic analysis of buildings". Technical Report NCEER-96-0010, State University of New York at Buffalo, Buffalo, NY, 1996.
10. Sivalsen, M.V., Reinhorn, A.M. "Hysteretic models for deteriorating inelastic structures". Journal of Engineering Mechanics, 2000, 126(6): 633-640.
11. Nudo, R., Sarà, G. "Problemi di modellazione delle strutture intelaiate in c.a. in zona sismica". Proc. 5th Italian Conference on Earthquake Engineering, Palermo, Vol. 1, 1991: 527-536.
12. Park, R. "Ductility evaluation from laboratory and analytical testing". Proc. 9th World Conference on Earthquake Engineering, Tokyo, Vol. 8, 1988: 605-616.
13. Structural Engineers Association of California (SEAOC). "Blue Book: Preliminary guidelines for performance based seismic engineering". Part 2. Sacramento, December 1998.
14. Barron Corvera, R. "Spectral evaluation of seismic fragility of structures". PhD Dissertation, Department of Civil, Structural & Environmental Engineering, State University of New York at Buffalo, Buffalo, NY, 2000.