STESSA 2006 – Mazzolani & Wada (eds)
© 2006 Taylor & Francis Group, London, ISBN 0-415-40824-5

Probabilistic analysis of a PR steel-concrete composite frame

C. Amadio, D. Moschino
Department of Civil Engineering, University of Trieste, Italy

M. Fragiacomo
Department of Civil Engineering, University of Canterbury, New Zealand

ABSTRACT: The paper investigates the seismic performance of a partially-restrained steel-concrete composite frame using the probabilistic approach. The analyzed frame has been tested at the ELSA laboratory of the Joint Research Centre of Ispra (Italy). The component method has been used to model both interior and exterior composite joints. The Latin Hypercube method has then been employed to draw the probabilistic distribution curves of the joints. Hence, the fragility and performance curves of the whole PR composite frame have been determined for four limit states.

1 INTRODUCTION

Modern codes for design in earthquake-prone regions require the structure to satisfy some performance objectives during the service life (CEN 1996, SEAOC 1996, Cornell & Krawinkler 2000). This design philosophy, known as Performance Based Seismic Design (PBSD), combines some structural performance levels with pre-fixed intensities of the seismic action. In this context, it has recently pointed out that the use of the probabilistic approach should be preferred when evaluating the structural performance. The traditional deterministic approach, in fact, may lead to inconsistencies between predicted and noticed structural damages because of the uncertainties in the models used to evaluate both the demand and capacity. Conversely, the probabilistic approach can fully consider all of the uncertainties affecting the prediction of the seismic behaviour and, therefore, the actual seismic performance.

The seismic reliability of a given structural typology can be evaluated by means of the fragility curves. Such curves provide the probability of occurrence $F_r(x)$ of a given Limit State (LS), conditioned on the parameter IM, representing the seismic hazard (which is usually the Peak Ground Acceleration PGA, the spectral acceleration S_a, or the spectral displacement S_d):

$$F_r(x) = P[LS|IM = x] \tag{1}$$

where the limit state LS is considered to be reached when a control variable assumes a pre-defined value. For a frame, usually the InterStorey Drift Angle (*ISDA*) or a damage parameter, such as the Park and Ang index D_{PA} (Park & Ang 1985), are assumed as control variables. Once the fragility curves are known, the probability of failure P_f, or limit state probability, can be evaluated with the equation:

$$P_f = P[LS|IM = x] \cdot P[IM \geq x] = \int_0^{+\infty} H(x) \cdot \frac{dF_r(x)}{dx} dx \tag{2}$$

where $H(x)$ represents the seismic Hazard function, generally expressed in terms of $IM = S_a$ (Song & Ellingwood 1999), according to the equation:

$$H(x) = P\left[S_a > x\right] = 1 - \exp\left[-\left(x/\mu\right)^{-k}\right] \tag{3}$$

μ and k being, parameters determined according to the characteristics of the site.

In the paper, following the PBSD approach, it is investigated the seismic performance of a partially-restrained steel-concrete composite frame constituted by partially encased composite columns connected to composite beams with steel profiles and concrete slab cast on corrugated steel sheathing.

2 THE ANALYSED FRAME

The analysed frame, tested at the Joint Research Centre of Ispra (Bursi et al. 2004), represents a full-scale two-storey steel-concrete composite building. The building is made of three parallel two-bay main frames with different span lengths of 5 m and 7 m spaced 3 m one to another, the interstorey height being 3.5 m. The frames are connected in the perpendicular direction by secondary beams pinned at the ends and braced with only-tension members (Fig. 1a).

The frame was designed according to the Eurocode 4 (CEN 1992) and 8 (CEN 1996) for a *PGA* of 0.4 *g*. The composite columns, partially encased, are made of steel profiles HEB 260/280 for the exterior/interior columns, respectively. The main beams are made of an IPE 300 steel profile connected by means of Nelson shear studs to the upper 15 cm thick concrete slab poured on a type Brollo EGB 210 corrugated steel sheathing. The analyses have been carried out for the intermediate longitudinal frame (Fig. 1b).

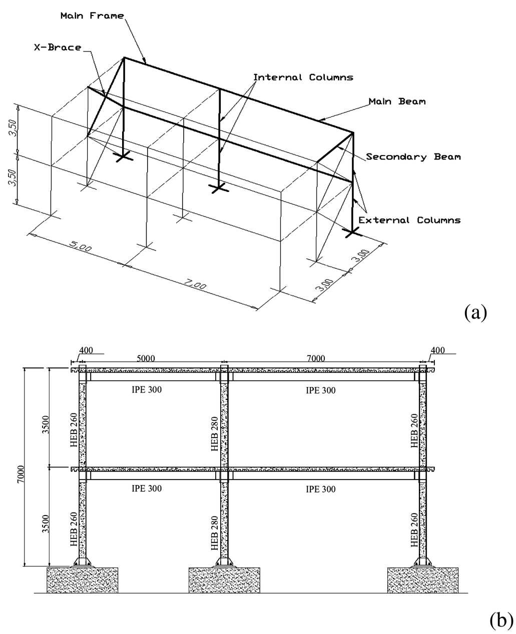

Figure 1. The analysed Structure, (a) spatial view, (b) the analysed frame.

3 THE PROBABILISTIC ANALYSIS

The probabilistic approach is based on the following steps:

1. Definition of the most critical source of uncertainty: the seismic event;
2. Modelling of the structure and beam-to-column connection, including the definition of the sources of uncertainty;
3. Choice of the model used for the damage evaluation, and definition of the performance levels for the structure;
4. Execution of the incremental dynamic analyses with statistical interpretation of the outcomes and determination of the fragility and collapse curves.

3.1 *Choice of the seismic inputs*

Since this study is aimed to the determination of the fragility curves for a structure in a generic earthquake-prone area, according to Altug & ElnaShai (2004), a number of 10 recorded earthquake ground motions has been considered. Such ground motions have been selected so as to represent a wide range of possible seismic events of relevant intensity (magnitude ≥5.8). The indicators of seismic intensity considered when selecting the ground motions have been: peak ground acceleration, *PGA*; magnitude *Ms*; modified Mercalli scale *MM*. Table 1 summarizes the characteristics of the earthquake ground motions selected for the analyses.

3.2 *Sources of uncertainty*

The possible sources of uncertainty for the frame are: the geometry of structural parts, the intensity and type of loads, the mechanical properties of materials, the type of hysteresis loop of the structural joints and composite beams. In this paper, the attention has been focused on the mechanical properties of materials and their influence on the strength of the joints, the composite beams and the whole frame. Permanent and variable loads, as well as the geometry of the structure,

Table 1. Characterization of earthquake ground motions.

	Earthquake	Date	Country	Station	Component	Ms	MM	PGA [g]	Sd_{max} [cm]	Sa_{max} [g]
GM1	Imperial -Valley	15/05/1940	USA	El Centro	S00E	7.1	X/XI	0.348	28.0	0.935
GM2	Friuli	15/09/1976	Italy	Buia	N-S	6.1	IX	0.109	9.4	0.327
GM3	Alkoin	24/02/1981	Greece	Xilikastro	N-S	6.7	IX	0.290	20.1	1.018
GM4	Friuli	06/05/1976	Italy	Tolmezzo	E-W	6.3	IX	0.315	11.2	1.030
GM5	Tabas	16/09/1978	Iran	Boshroych	N79E	7.3		1.004	10.3	0.339
GM6	Campano Lucano	23/11/1980	Italy	Irpinia,Calitri	E-W	6.7	VIII	0.175	18.6	0.595
GM7	Lazio- Abruzzo	07/05/1984	Italy	Cassino-Sant'Elia	N-S	5.8	VII	0.110	3.8	0.393
GM8	Kocaeli	17/08/1999	Turkey	Yesilkoy	N-S	7.8		0.089	16.5	0.366
GM9	Gazli	17/05/1976	Uzbekistan	Gazli	E-W	7.0		0.720	50.4	2.008
GM10	Montenegro	15/04/1979	Montenegro	Bar-S.O.	E-W	7.0		0.363	40.6	1.305

have been regarded as deterministic quantities. The yield stresses of steel and rebars, and the ultimate stresses of concrete, bolts and stud connectors have then been assumed as random variables, characterized by a statistical distribution.

3.2.1 *Construction steel*

According to what suggested in Piluso et al. (2003), the statistical distribution of the yield stress f_y has been assumed as dependent on the thickness of the plates which make up the profile. It was pointed out that the lognormal distribution best represents the experimental distribution. Furthermore, the mean of the logarithm of the yield stress can be assumed as linearly dependent on the thickness t, with decreasing trend:

$$E(\ln f_y) = c_1 - c_2 t = 5.766 - 0.007t \tag{4}$$

where c_1 and c_2 are material parameters dependent on the type of steel, t is the thickness in mm, and f_y is the yield stress in N/mm^2. Table 2 summarizes the statistical parameters of the random variables assumed for the beams and columns of the frame tested at Ispra, where $f_{y,m}$ is the mean value of the yield stress, s is the standard deviation of the yield stress, λ and ξ are, respectively, the mean value and the standard deviation for the lognormal distribution:

$$\lambda = \ln f_{y,m} - \xi^2/2 \quad , \quad \xi = \sqrt{\ln(COV^2 + 1)} \tag{5}$$

and COV is the coefficient of variation defined as:

$$COV = s / f_{y,m} \tag{6}$$

3.2.2 *Rebars*

For rebars, a lognormal distribution with a COV equal to 6% has been assumed, according to what suggested by Erberik & Elnashai (2004).

3.2.3 *Concrete*

It is generally accepted in literature that the compression strength of concrete (f_c) may be represented by a normal distribution. Dymitiotis et al. (1999) suggest a coefficient of variation of 15% for such a quantity. The mean value can be obtained from the characteristic strength of the material using the equation:

$$f_{c,m} = f_{c,k} / (1 - k \cdot COV) \quad , \qquad k = 1.64. \tag{7}$$

Table 4 summarises the statistical properties assumed for concrete.

3.2.4 *Bolts*

The ultimate tensile strength of bolts has been considered as normal distributed with mean value $E(f_u) = 1.2 \times f_{u,k}$, and coefficient of variation of 2% (Piluso et al. 2003).

3.2.5 *Stud connectors between steel beam and r.c. slab*

A normal distribution with coefficient of variation of 4% has been assumed. The mean value of the ultimate strength has been obtained from the corresponding characteristic value.

Based on the aforementioned mechanical properties of the materials, the strengths of joints, regarded

Table 3. Statistical parameters for rebars.

Rebars	λ	ξ	COV	$f_{y,m}$ [N/mm^2]	S [N/mm^2]
Fe b 44k (B450 C)	6.17	0.06	0.06	477.09	28.63

Table 4. Statistical parameters for concrete.

Concrete	$f_{c,k}$ [N/mm^2]	COV	$f_{c,m}$ [N/mm^2]	S [N/mm^2]
Class C25/30	30.36	0.15	38.43	5.76

Table 5. Statistical parameters for bolts.

Bolts	f_{ub} [N/mm^2]	COV	$f_{ub,m}$ [N/mm^2]	S [N/mm^2]
Class 10.9	1000.00	0.02	1070.0	21.4

Table 2. Statistical parameters for steel components.

Steel components (Fe 360)		t [mm]	λ	ξ	COV	$f_{y,m}$ [N/mm^2]	S [N/mm^2]
Column flange	HEB260	17.50	5.64	0.07	0.07	283.1	19.8
Column web	HEB260	10.00	5.70	0.07	0.07	298.4	20.9
Column flange	HEB280	18.00	5.64	0.07	0.07	282.2	19.8
Column web	HEB 280	10.50	5.69	0.07	0.07	297.4	20.8
Beam flange	IPE 300	10.70	5.69	0.07	0.07	296.9	20.8
Beam web	IPE 300	7.10	5.72	0.07	0.07	304.5	21.3
End plate		15.00	5.66	0.07	0.07	288.1	20.2

Table 6. Statistical parameters for connector studs.

Studs	$f_{us,k}$ [N/mm^2]	*COV*	$f_{us,m}$ [N/mm^2]	*S* [N/mm^2]
Stud Nelson 3/4"	517.00	0.04	553.4	22.1

Table 7. Statistical parameters of the composite beam under sagging bending moment.

$M_{pl,m}$ [KN/m]	*S* [KN/m]	λ	ξ	*COV*
426.62	20.38	6.055	0.048	0.05

Table 8. Statistical parameters of the composite beam under hogging bending moment.

$M_{pl,m}$ [KN/m]	*S* [KN/m]	λ	ξ	*COV*
252.79	10.94	5.532	0.043	0.04

as stochastic variables, have been computed through a Monte Carlo simulation. Since the plastic hinges may be formed also inside the beams, the same procedure has been applied for the strength of steel-concrete composite beams.

3.3 *Statistical simulation of beams and joints*

The Monte Carlo simulation has been used to calculate the plastic resistant moment of the composite beam according to the Eurocode 4. 10,000 pseudo-random values in accordance with the statistical distributions previously defined have been generated for each random variable using the Box and Muller method. The mean values $M_{pl,m}$, standard deviations *s*, and coefficients of variation *COV* of the plastic moments have then be computed. Some statistical tests, like the χ^2 and the K-S (Kolmogorov-Smirnov) tests, have pointed out that the best probability distribution function fitting with the obtained statistics is the lognormal one. Obtained results are reported in Tables 7 and 8 for the 5 m bay composite beam.

The beam-to-column exterior and interior joints have been schematised using the component models depicted in Figures (2a) and (2b), respectively.

In this model all the axial springs are characterised by three-linear relationships with no degrade of stiffness and strength under cyclic loading. The relevant points of the relationships for the steel and concrete slab components have been evaluated according to the Annex J of Eurocode 3 and 4, and according to what suggested by Faella et al. (2000). The composite joints have then been analysed using the Abaqus Finite Element code (Hibbit et al. 1997). The stratified sampling

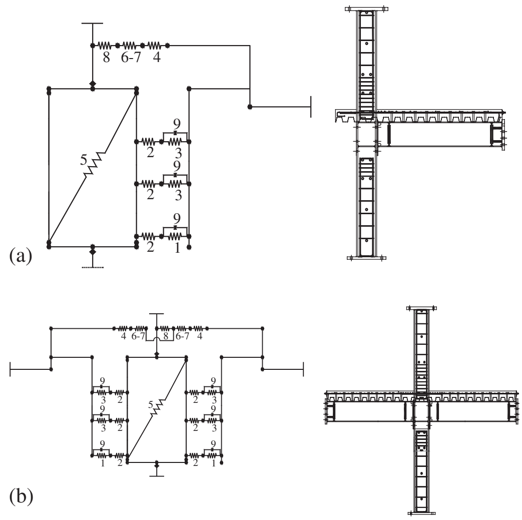

Figure 2. Component model for exterior (a) and interior (b) joint.

Table 9. Statistical parameters for the composite joints.

Type of joint	Bending	Macro-model		
		M_r[kNm]	σ[kNm]	*COV*
External	M > 0	263.86	16.87	0.06
	M < 0	200.14	9.74	0.05
Internal	M > 0	178.07	10.09	0.06
	M < 0	107.34	8.61	0.08

technique, also known as the Latin Hypercube Sampling, has been employed (Olsson et al. 2003). A number of 200 exterior and 200 interior joints subjected to positive and negative moments has been analysed. The statistical tests of χ^2 and Kolmogorov-Smirnov have proved that the lognormal probability distribution function *PDF* better represents the obtained statistics of the joint strengths. The statistical values of the resistant moment in terms of mean value M_r, variance σ and coefficient of variation *COV* are displayed in Table 9.

3.4 *Frame analysis*

In order to draw the fragility curves of the frame, the damage parameters must be identified, along with the values assumed by such quantities when the different performance levels have been achieved. The damage indexes considered in this paper are the interstorey drift angle *ISDA*, which is given for a frame by:

$$ISDA = \max_{i=1}^{n}(\delta_i / h_i) \tag{8}$$

where *n* is the number of stories, δ_i is the interstorey drift, and h_i the interstorey height, and the global Park

Table 10. Damageability limit states expressed through the global Park and Ang and the ISDA index.

Level of damage	D_{PA}	ISDA %	Consequence
LS0: Reduced	0.1	0.5	Usable building
LS1: Limited	0.4	1.0	Repairable building
LS2: Significant	0.8	2.5	Irrecoverable building
LS3: Near Collapse	1.5	5.0	Loss of building

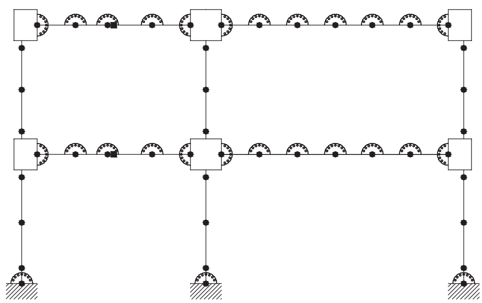

Figure 3. FE model adopted for the frame.

and Ang parameter D_{PA}, evaluated for the whole frame as the weighed average of the local Park and Ang indexes D_i of each plastic hinge formed in the frame, as given by the equation:

$$D_{PA} = \sum_i \lambda_i \cdot D_i \qquad D_i = \frac{\vartheta_m}{\vartheta_u} + \frac{\beta}{M_y \cdot \vartheta_u} \int dE \ ; \qquad (9)$$

with $\sum_i \lambda_i = 1; \quad \lambda_i = E_{p,i} / (\sum_i E_{p,i})$,

with $\sum_i \lambda_i = 1$; $\lambda_i = E_{p,i}/(\sum_i E_{p,i})$, where ϑ_m is the largest rotation achieved during the seismic event in the *ith* plastic hinge or beam-to-column joint, ϑ_u is the ultimate rotation of the *ith* plastic hinge or beam-to-column joint under monotonic loading, β is a parameter assumed as equal to 0.15, M_y is the yield moment, dE_p is the increment of dissipated plastic energy.

With reference to the assumed four levels of damage: "Reduced" LS0, "Limited" LS1, "Significant" LS2, and "Near Collapse"LS3, the damage index values shown in Table 10, have been determined.

The analysis has been carried out through the nonlinear FE software SAP2000 version 9 (2004), by using the FE model depicted in Figure 3.

The beam-to-column joint has been modelled using a link element with bilinear non-symmetric moment-rotation relationship, the elastic stiffness being obtained through the component method. The strength of the joint has been considered as a random variable and computed using the statistical simulation described in the previous paragraph on the basis of the statistical distribution of the material mechanical properties. The composite beams have been modelled by means of elastic beam elements linked one to another with rigido-plastic rotational springs. The strength of such springs has been considered as a random variable and computed using the statistical simulation previously described. The columns have been modelled with linear beam elements and connected to the foundation with an elasto-plastic rotational spring.

The FE model of the frame has been validated against the results of the pseudo-dynamic tests performed at ISPRA (Bursi et al. 2004). A sample of 15 pseudo-random frames has been generated using the stratified Latin Hypercube Sampling method, based on 9 random variables: strength of the column-to-foundation joint, positive and negative ultimate moments for the 5 m and 7 m bay beams, positive and negative ultimate moments for the interior and exterior beam-to-column joint.

The earthquake, as discussed previously, is the main source of uncertainty, since the recorded ground motion is random in terms of peak ground acceleration *PGA*, duration, and frequency content. Such uncertainties have been taken into account by subjecting the 15 frames generated above to the 10 recorded ground motions summarised in Table 1. Each ground motion, described by the history of ground acceleration, has been scaled on 10 values of seismic intensity, represented by the spectral displacement S_d evaluated for the natural period of the structure $T = 0.506$ s. Such a quantity is commonly regarded as the most stable and representative parameter of the seismic intensity (Altug & ElnaShai 2004). The fragility curves have been obtained as a result of the 1500, ($15 \times 10 \times 10$), nonlinear time-history analyses and of the statistical treatment of obtained results.

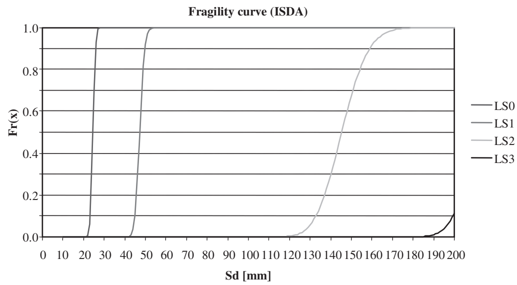

Figure 4. Fragility curves (*ISDA*) in terms of S_d.

The fragility curves obtained by assuming the *ISDA*, and the global Park and Ang indexes as damage parameters, are displayed in Figures 4 and 5, respectively. The trends of the fragility curves in terms of *ISDA* demonstrate that the frame is subjected to significant interstorey drifts even for low seismic intensities. The collapse strength of the frame (LS3 damage limit state) predicted using the *ISDA* parameter is larger than that obtained using the global Park and Ang index, as can be noted from Figures 4 and 5.

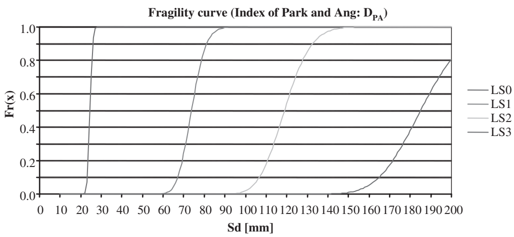

Figure 5. Fragility curves (D_{PA}) in terms of S_d.

Table 11. Probability F_r of reach of a given damage limit state for the frame tested at ISPRA.

		LS0	LS1	LS2	LS3
ISDA	**Sd**	23 mm	42 mm	113 mm	175 mm
	PGA	~0.1g	~0.25g	~0.6g	~1.0g
	Fr	6.02E-02	1.44E-03	1.61E-04	1.35E-05
DPA	**Sd**	23 mm	59 mm	87 mm	126 mm
	PGA	~0.1g	~0.35g	~0.45g	~0.75g
	Fr	6.05E-02	1.37E-03	1.15E-04	1.04E-05

Table 11 reports the probability F_r of reach a given damage limit state once a seismic event with a pre-fixed spectral displacement S_d is given. The values corresponding to the medium *PGA* for the set of ground motions used in the analyses are also reported.

The probability F_r does not represent the actual probability of failure for the structure since such probability will also depend on the seismic hazard of the region where the structure is located. In order to calculate the failure probability, the curve of seismic hazard must be introduced. Such a curve provides the annual probability of exceeding of a given seismic intensity (see eq. 3), which is generally measured by the spectral acceleration S_a. In order to calculate the probability of failure for the frame under study, the seismic hazard curve proposed by Song & Ellingwood (1999) for the state of California has been considered. Such a curve, represented by eq. (3) with parameters $k = 2.38$ and $\mu = 0.045$, provides values of seismic hazard compatible with those of the Irpinia earthquake-prone region (Italy). The obtained performance curves are displayed in Figures 6 and 7. They can be considered as the final outcome of a reliability analysis carried out using a full probabilistic approach.

4 FINAL REMARKS

In order to evaluate the performance of the composite frame on the basis of a probabilistic analysis, an acceptable value of the annual probability of failure P_f must be defined for each of the four damage limit states as: LS0 $\rightarrow 10^{-2}$; LS1 $\rightarrow 10^{-3}$; LS2 $\rightarrow 10^{-4}$; LS3 $\rightarrow 10^{-5}$.

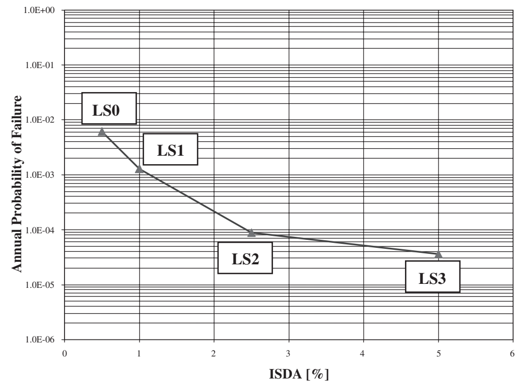

Figure 6. Performance curve of the structure (SDA).

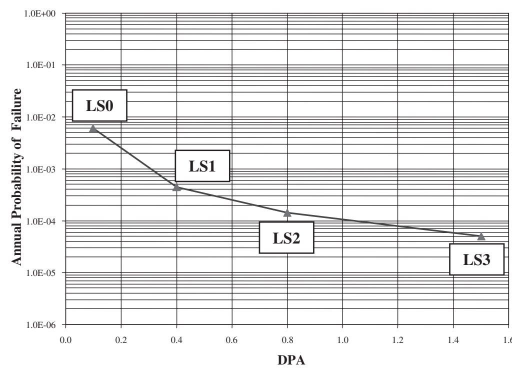

Figure 7. Performance curve of the structure (DPA).

From figure 7, it can be noted that the frame exhibits a P_f of about $6 \cdot 10^{-3}$, $4 \cdot 10^{-4}$, $1.4 \cdot 10^{-4}$, and $5 \cdot 10^{-5}$ for the reduced, limited, significant, and near collapse limit state, respectively. It can be concluded that the frame is slightly under-designed for the LS3, while it is slightly over-designed for the reduced and limited damage limit states. This result is in contrast with the design of the frame obtained using the static analysis according to the Eurocode 8 deterministic approach. In this case, in fact the serviceability limit states are the most critical conditions.

Another important point revealed by the probabilistic analysis is that the acceptable values for the *PGA* at the collapse are around $0.75g$ (Table 11). Conversely, the deterministic analysis based on the static pushover overestimates the resistance of the frame, leading to *PGA* larger than $1.3g$. Based on the analyses carried out, it is also possible to assert that the Park and Ang index represents the structural damage better than the InterStorey Drift Angle for steel-concrete composite frames. Such a type of structure is, in fact, characterised by and extensive plasticization of the joints and composite beams, therefore the Park and Ang index which accounts also for the plastic dissipated energy leads to more homogenous results.

REFERENCES

Altug Erberik, M. & ElnaShai Amr, S. 2004. Fragility analysis of flat-slab structures. *Engineering Structures*, Elsevier, pp. 937–947.

Bursi, O.S. Caramelli, S. Fabbrocino, G. Molina, J. Salvatore, W. Taucer, F. Tremblay, R. Zandonini, R. 2004. 3D Full Scale Seismic Testing of a Steel-Concrete Composite Building at ELSA. *Report, Institute for the Protection and the Security of the Citizen, European Laboratory for Structural Assessment (ELSA)*, Ispra (VA), Italy.

CEN, 1992. European Committee for Standardisation. Eurocode 4. Common Unified Rules for Composite Steel and Concrete Structures. ENV 1994-1-1.

CEN, 1996. European Committee for Standardisation. Eurocode 8. Design provisions for earthquake resistance structure. ENV 1998-1-1.

Cornell, C.A. & Krawinkler, H. 2000. Progress and challenges in Seismic Performance Assessment. http://peer. berkeley. edu/news/2000spring/performance/html. febwetg4wt4.

Dymitiotis, C. Kappos, A.J. Chryssanthopoulos, M.K. 1999. Seismic reliability of RC frames with uncertain drift and member capacity. *Journal of structural Engineering*, ASCE 1999; 125(9): 1038–47.

Faella, C. Piluso, V. Rizzano, G. 2000. Structural steel semirigid connections. CRC PRESS.

12.Hibbit, Karlsson & Sorensen Inc. 1997. Abaqus, Version 5.8, U.S.A.

Olsson, A. Sandberg, G. Dahlblom, O. 2003. On Latin hypercube sampling for structural reliability analysis. *Structural Safety*, Elsevier, 25 (2003) 47–68.

Park, Y.J. Ang, AH-S. Wenn, Y.K. 1985. Mechanistic seismic damage model for reinforced concrete. *Journal of Structural Engineering*, ASCE, 111(4), 722–739.

Piluso, V. Rizzano, G. Totoli, G. 2003. Progettazione a completo ripristino di resistenza di collegamenti flangiati trave colonna: approccio probabilistico. *Costruzioni Metalliche*, LV(2003), n. 1.

Sap2000 version 9.1 2004. Computer and Structures, Inc, Berkeley, CA.

SEAOC. 1996. Recommended Lateral Force Requirements and Commentar. Appendix B-Vision 2000, Seismology Committee, Structural Engineers Association of California.

Song, J. & Ellingwood, B.R. 1999. Seismic reliability of special moment steel frames with welded connections. *Journal of Structural Engineering*, ASCE, 125(4), 372–384.