Influence of brace over-strength distributions on the seismic response of steel braced frames

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SUMMARY
A general methodology to investigate the relation between seismic performance and brace over-strength distributions in steel braced frames is presented. The influence of brace over-strength distributions on the seismic motion are studied through sensitivity analysis including the random nature of the seismic input. The actual nonlinear variations of engineering demand parameters (EDPs), as well as their linearly approximated variations, are derived. Afterwards, a range of variation of the brace over-strengths is defined and the brace over-strength patterns that are the most unfavourable for each EDP are identified and the relevant maximum increment of each EDP is determined. Finally, the relationship between the brace over-strength used in the seismic design and the maximum seismic performance reduction is obtained.

Keywords: sensitivity analysis, buckling-restrained braces, brace over-strength, soft storey formation.

1. INTRODUCTION

Concentrically braced frames (CBFs), eccentrically braced frames (EBFs), and more recently buckling-restrained braced frames (BRBFs), have proved being efficient seismic resistant systems with high elastic lateral stiffness as required for drift control. However, beside their advantages, CBFs, EBFs, and BRBFs suffer from low post-elastic stiffness, especially in the case of frames with non-moment resisting beam-to-column connections, due to the low post-yield stiffness of their braces. Thus, these structural systems might show significant tendency to soft storey formation during seismic events (Tremblay 2002, 2003, Elghazouli 2010, Rossi and Lombardo 2007, Bosco and Rossi 2008, Kiggins and Uang 2005, Fahnestok et al. 2007, Ariyaratana and Fahnestock 2011). This tendency is strongly related to the regularity of the brace over-strength distributions over the building height. Such issue has been observed even when limitations in the maximum difference in brace over-strength are satisfied. Overall, the studies available in the technical literature delineate the need to further examine the influence of the brace over-strength on the structural seismic performance as well as the effectiveness of current code recommendations.

In this paper a general methodology to investigate the relation between seismic performance and brace over-strength distributions is presented. The influence of brace over-strength distributions on the seismic motion are studied through response sensitivity analysis including the random nature of the seismic input. The actual nonlinear variations of specific demand measures of interest in seismic response, i.e., Engineering Demand Parameters (EDPs), as well as their linearly approximated variations, are derived. Afterwards, a range of variation of the brace over-strengths is defined and the brace over-strength patterns that are the most unfavourable for each EDP are identified and the relevant maximum increment of each EDP is determined. To this end, once that the response sensitivity analysis results are available, the proposed approach requires the solution of a constrained extreme value problem, as for example found in structural optimization analysis. Finally, the relationship between the brace over-strength used in the seismic design and the maximum seismic performance reduction is obtained. Results for a BRBF used as case study are presented and discussed.

2. SEISMIC DEMAND SENSITIVITY ANALYSIS OF NONLINEAR STRUCTURES

It is assumed that the motion of the structural system is described by the following differential equations and relevant initial conditions:

$$\dot{\boldsymbol{\chi}}(\boldsymbol{\theta};t)=\mathbf{a}(\boldsymbol{\chi}(\boldsymbol{\theta};t),\boldsymbol{\theta})+\mathbf{p}(t) \qquad \boldsymbol{\chi}(\boldsymbol{\theta};0)=0 \tag{1}$$

where $\boldsymbol{\theta}\in\Re^m$ is a vector collecting m parameters defining material and geometric properties of the structural system, $\boldsymbol{\chi}:\Re^m\times[0,T]\mapsto\Re^s$ is a vector-valued function describing the evolution of the s state variables, e.g., displacement, velocity and state variables of history-dependent constitutive material models, $t\in[0,T]$ is the time and T the duration of the dynamic analysis, a superposed dot represents one derivative with respect to time, $\mathbf{p}:[0,T]\mapsto\Re^s$ is a function describing the seismic ground motion, $\mathbf{a}:\Re^m\times\Re^s\mapsto\Re^s$ a vector-valued nonlinear function describing the response of the structure. It is assumed that **a** and **p** are continuous functions and that **a** is not an explicit function of time. Given a reference motion $\boldsymbol{\chi}_0:[0,T]\mapsto\Re^s$ corresponding to assigned values of the material and geometric parameters collected in $\boldsymbol{\theta}_0$, the variation of the motion in consequence of a variation $\hat{\boldsymbol{\theta}}$ of the parameters can be linearized in the neighbourhood of the reference motion, i.e., for $\|\hat{\boldsymbol{\theta}}\|\to 0$, by means of the following series expansion:

$$\boldsymbol{\chi}(\boldsymbol{\theta}_0+\hat{\boldsymbol{\theta}};t)=\boldsymbol{\chi}_0(t)+\mathbf{S}_0(t)\hat{\boldsymbol{\theta}}+\mathrm{o}(\|\hat{\boldsymbol{\theta}}\|) \qquad \forall t\in[0,T] \tag{2}$$

where $\mathbf{S}_0:[0,T]\mapsto\Re^s\times\Re^m$ is the sensitivity matrix at $\boldsymbol{\chi}_0$ defined by the condition

$$\mathbf{S}_0(t)\mathbf{e}=\lim_{\lambda\to 0}\frac{\boldsymbol{\chi}(\boldsymbol{\theta}_0+\lambda\mathbf{e};t)-\boldsymbol{\chi}_0(t)}{\lambda} \qquad \forall\mathbf{e}:\|\mathbf{e}\|=1 \tag{3}$$

and whose components $S_{0ij}(t)=\partial\chi_i(\boldsymbol{\theta};t)/\partial\theta_j\big|_{\boldsymbol{\theta}=\boldsymbol{\theta}_0}$ describe the ratio between the variation of the *i*-th component $\chi_i(t)$ of $\boldsymbol{\chi}$ due to the variation of the *j*-th component θ_j of $\boldsymbol{\theta}$ when $\boldsymbol{\theta}=\boldsymbol{\theta}_0$ and at the considered time instant t, o( ) is the "little-o" Landau symbol, i.e. given a function $f(x)$ and a positive function $\phi(x)$, then $f=\mathrm{o}(\phi)$ means that $f/\phi\to 0$. Similarly, the response of the perturbed structural system in the neighbourhood of the reference motion $\boldsymbol{\chi}_0$ may be written as (time dependence omitted for the sake of brevity):

$$\mathbf{a}(\boldsymbol{\chi}(\boldsymbol{\theta}_0+\hat{\boldsymbol{\theta}}),\boldsymbol{\theta}_0+\hat{\boldsymbol{\theta}})=\mathbf{a}\big|_0+\nabla_\chi\mathbf{a}\big|_0\mathbf{S}_0\hat{\boldsymbol{\theta}}+\nabla_\vartheta\mathbf{a}\big|_0\hat{\boldsymbol{\theta}}+\mathrm{o}(\|\hat{\boldsymbol{\theta}}\|) \tag{4}$$

where the symbol $\big|_0$ attached after a function means that that function is evaluated at the reference motion $\boldsymbol{\chi}_0$ corresponding to $\boldsymbol{\theta}=\boldsymbol{\theta}_0$. The sensitivity matrix can be deduced from the differential equation:

$$\dot{\mathbf{S}}_0=\nabla_\chi\mathbf{a}\big|_0\mathbf{S}_0+\nabla_\vartheta\mathbf{a}\big|_0 \tag{5}$$

obtained differentiating Equation (2) with respect to time, using Equation (1) and then comparing the result to Equation (4). Equation (5) is solved and $\mathbf{S}_0$ determined, the motion variation $\hat{\chi}$ directly and linearly related to the small variations $\hat{\boldsymbol{\theta}}$ of the structural parameters:

$$\hat{\boldsymbol{\chi}}\cong\mathbf{S}_0\hat{\boldsymbol{\theta}} \tag{6}$$

Commonly, in seismic analysis the structural performance is evaluated by one or more EDPs. An EDP is a positive scalar d giving a measure of the structural damage occurring during the motion produced by the earthquake and it is a derived quantity of $\boldsymbol{\chi}$:

$$d = \mathsf{D}(\chi) \tag{7}$$

where $\mathsf{D}: U_\chi \rightarrow \Re$ is a functional operator acting on the space of motions U_χ. Consequently, the previous sensitivity analysis must be extended to evaluate the EDP variations $\hat{d}$ due to the small variations $\hat{\boldsymbol{\theta}}$ of the model parameters through the difference:

$$\hat{d} = \mathsf{D}\left(\chi_0 + \mathbf{S}_0\hat{\boldsymbol{\theta}}\right) - \mathsf{D}(\chi_0) \tag{8}$$

The relationship between d and the motion $\boldsymbol{\chi}$ is usually nonlinear. Coherently with the sensitivity approach, oriented to investigate the neighbourhood of the reference motion, the relationship between d and $\boldsymbol{\chi}$ may be linearized:

$$\hat{d} = \mathsf{L}_0\mathbf{S}_0\hat{\boldsymbol{\theta}} + o\left(\left\|\hat{\boldsymbol{\theta}}\right\|\right) \tag{9}$$

introducing the operator L_0 defined as the derivative of D :

$$\mathsf{L}_0\hat{\chi} = \lim_{\lambda\rightarrow 0}\frac{\mathsf{D}(\chi_0 + \lambda\mathbf{c}) - \mathsf{D}(\chi_0)}{\lambda} \qquad \forall\mathbf{c}: \|\mathbf{c}\| = 1 \tag{10}$$

In this way a complete overview of the effects on the seismic demand due to any combination of the variations of the structural parameters $\boldsymbol{\theta}$ is obtained. This is a qualitative and quantitative information that is quite important in understanding the structural behaviour as well as in studying the propagation of uncertainties of $\boldsymbol{\theta}$ to the structural response. In addition, structural design requires precise information on the entity of seismic performance reduction, measured by the EDPs, due to possible deviations from the reference design solution. It is thus crucial for the sake of safety: (i) to establish the range of the potential deviations from the design solution, and (ii) to assess the largest performance reduction to be expected within the set of admissible deviations. To this end, it is essential to complete the sensitivity analysis with a constrained extreme problem where the EDP is the objective function:

$$\begin{aligned} &\mathsf{D}\left(\chi_0 + \mathbf{S}_0\hat{\boldsymbol{\theta}}^*\right) - \mathsf{D}(\chi_0) = \max \\ &\mathbf{g}\left(\hat{\boldsymbol{\theta}}^*\right) \geq \mathbf{0} \\ &\mathbf{n}\left(\hat{\boldsymbol{\theta}}^*\right) \leq \mathbf{0} \end{aligned} \tag{11}$$

and $\hat{\boldsymbol{\theta}}^*$ is the most dangerous variation of $\boldsymbol{\theta}$ which must be found within the set defined by the constraints **g** and **n**. Clearly, the problem solution becomes simpler when both the EDP expression and the constraint equations are linear.

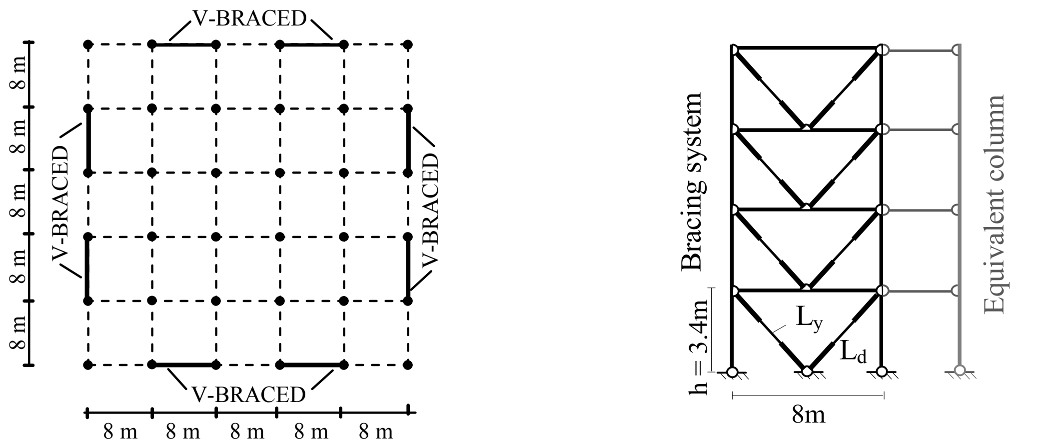
The formulation presented above assumes a deterministic seismic input **p** while it is well-known that the seismic input is affected by an high level of randomness that cannot be neglected in the design. Various code provisions, e.g., Eurocode 8, suggest the description of the seismic input uncertainty with the use of response results averaged over the response results obtained from a sufficiently large number of accelerograms. According to this approach, the EDPs variations is evaluated by a mean sensitivity function $\mathbf{S}_0$ obtained from the analyses performed using an adequately large set of accelerograms. Different and more advanced treatments of the seismic input randomness are beyond the objectives of this study.

The seismic response analysis of nonlinear structures is commonly performed through the finite element (FE) method where the structure motion is reduced to the time histories of a set of discrete variables, i.e., the generalized displacements and velocities of the nodes of the FE model and the required state variables of the inelastic material models. The computation of the response sensitivities within the FE framework has been investigated in the past and several methods are available, such as the Direct Differentiation Method (DDM) and the Finite Difference Method (FDM) (Kleiber et al. 1997).

3. CASE STUDY: STEEL FRAME WITH BUCKLING-RESTRAINED BRACES

3.1. Description of the case study

A realistic 4-storey steel frame is considered as case study. Interstorey height h = 3.4 m is constant between adjacent floors; columns are continuous with pinned beam-to-column connections and hinge restraints at the base. Four V-bracing systems (BRBs) for each direction are the only seismic resistant components (Fig. 1a). Masses from vertical live and superimposed dead loads are 1200 kNs^2/m for each floor. Seismic design was based on spectrum type 1 of Eurocode 8 for ground type B with design ground acceleration $a_g = 0.35g$. The yield length of BRBs (L_y) is one third of the overall length (L_d) of diagonal braces (Fig. 1b). The design yield stress of the BRBs is 275 MPa, columns and beams are made of steel with design yield stress equal to 355 MPa. Details on geometric and material data not given here can be found in (Zona et al. 2012).



(a) (b)

Figure 1. Testbed structure: (a) floor configuration; (b) simplified planar model.

3.2. Finite element modelling

In order to reduce its degrees of freedom, the FE models used in this study comprise only one single bracing system and an equivalent column representative of the section properties and loads of the pertinent columns not parts of the bracing system (Fig. 1b). A dedicated displacement-based finite element code developed by the first author was used for response and response sensitivity analyses. Beams and columns were modelled using geometric nonlinear (moderate rotations theory) Euler-Bernoulli frame FEs with linear elastic steel (elastic modulus E = 210000 MPa). The BRBs were modelled with truss elements having rigid links to represent the unrestrained non yielding segments in 2/3 of the brace total length (Fig. 2a). An elastoplastic constitutive model based on a simple rheological scheme (Fig. 2b), specifically developed for steel BRBs (Zona and Dall'Asta 2012), was used in this study for the yielding segments of trusses representing the BRBs. Such BRB model has only one internal variable, i.e., plastic strain, and was formulated in order to replicate experimental behavioural aspects (isotropic hardening and tension-compression asymmetry), as well as to include some highly desired requirements (explicit computation of the plastic component of the deformation as required in BRB capacity models, smoothness of the elastic-to-plastic transition to improve convergence rate, limited number of parameters to facilitate its implementation and use in response sensitivity analysis). A typical cyclic response of the adopted constitutive model under a sinusoidal strain history with linearly increasing amplitude is shown in Fig. 2c where ε is the BRB axial strain, σ the BRB axial stress in the yielding core, σ_y and ε_y are the initial yield stress and yield strain of the BRB core, respectively. Response results of the adopted BRB elastoplastic model were validated against different experimental tests available in the technical literature.

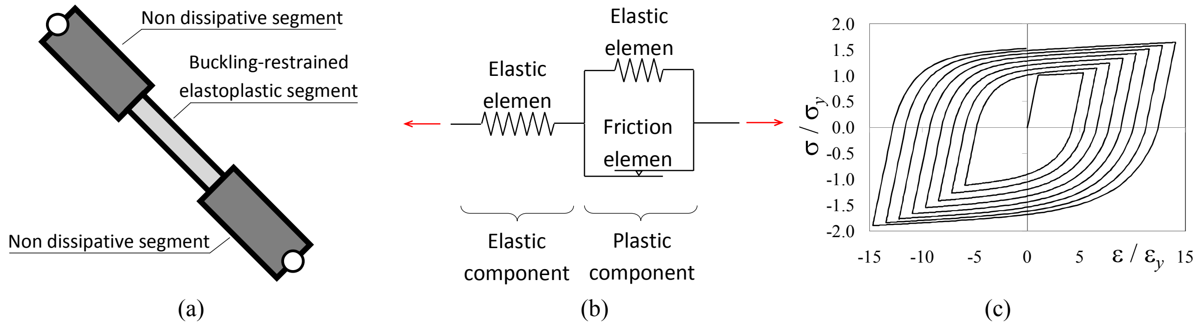


Figure 2. BRB model: (a) truss element with dissipative and non dissipative segments; (b) rheological model of the elastoplastic dissipative segment; (c) typical cyclic response.

In addition to the damping provided by the dissipative elastoplastic braces, a global damping for the structure was included using the Rayleigh model, with the damping matrix proportional to the mass matrix and updated stiffness matrix, and 5% of the critical damping assigned to the first and second vibration modes. The constant average acceleration method with constant time step Δt = 0.01 s, in conjunction to the Newton-Raphson iterative procedure, was used in all the dynamic analyses performed.

3.3. Seismic nonlinear response analysis

The behaviour of the testbed structures was studied through time history analyses having as seismic input 28 natural ground motions selected from the PEER strong motion database and scaled so that the elastic response spectra for the records matched the Eurocode 8 elastic spectrum at the first natural period T_1 = 0.746 s. The spectra of the selected ground motions together with their average spectrum as well as the adopted Eurocode 8 elastic spectrum are depicted in Fig. 3. More details on the selected ground motions can be found in (Zona et al. 2012).

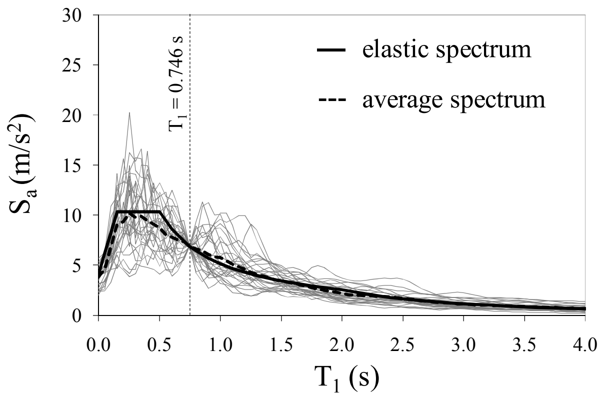


Figure 3. Pseudo-acceleration elastic spectra of the selected ground motions and relevant average spectrum compared to the Eurocode 8 elastic spectrum.

Incremental dynamic analyses (IDA) were performed to assess the performance of the bracing systems. The multiplier of the scaled selected ground motions leading to a maximum interstorey drift, averaged over the 28 ground motions, equal to the design value (1% of the storey height) is 0.94 corresponding to the elastic spectrum with a_g = 0.33g. This acceleration is very close to the design one, notwithstanding the record to record variability, the possible concentration of inelastic deformation at some storey levels, and the effects of the geometric nonlinearity included in the FE model. Response results, i.e., interstorey drifts, BRB cumulative ductility, and stress of bracing columns at each storey are reported in Table 1 in terms of mean values (averaged over the 28 ground motions) and coefficients of variations (COVs) of the absolute peak values. For each nonlinear time history analysis it was verified that beams and columns remained within their elastic range.

Table 1. Response results at a_g = 0.33g for the considered set of 28 ground motions.

Floor	Interstorey drift		BRB cumulative ductility		Bracing column stress (MPa)	
#	mean	COV	mean	COV	mean	COV
4	0.78%	22.18%	66.23	47.34%	143.85	14.03%
3	0.69%	23.25%	50.10	37.47%	182.30	16.39%
2	0.88%	35.18%	59.15	41.59%	186.67	8.43%
1	0.99%	35.22%	76.28	43.62%	183.51	7.42%

3.4. Seismic nonlinear response sensitivity analysis

Response sensitivities of the selected EDPs, i.e., ζ_i (interstorey drift at the i-th floor normalized with respect the interstorey height h) and μ_i^c (cumulative plastic strain at the i-th floor normalized with respect to the BRB yield strain ε_y) were computed with respect to the independent sensitivity parameters $\theta_k = A_k / A_{0k}$, being A_k the actual core area and A_{0k} the core area of the reference design solution of each of the two BRBs at the k-th floor. For ζ_i attention is limited to the sensitivities of each EDP when its peak value is attained, whereas for μ_i^c the value attained at the end of the ground motion is considered.

The local response sensitivity results could be questioned being derivatives of the peak response and thus representative of the local effect of small variations of the sensitivity parameters θ_k. In order to clarify this issue, the global sensitivities of the peak values of the considered EDPs, computed with respect to θ_k using the FDM for finite variations $\Delta\theta_k$ = 0.125 and 0.250, are also shown in this study. In addition, the FDM with $\Delta\theta_k$ = 0.1, 0.01, and 0.001 was used to approximate local sensitivities, showing that the convergence of the FDM approximation is achieved with $\Delta\theta_k$ = 0.01 without incurring in the step-size dilemma. When FDM is used, the sensitivities are computed from the maximum values of the EDPs in the reference and perturbed motion, even if the maximum values are attained in different time instants.

Despite the inevitable differences in structural response obtained from the various seismic inputs, similar qualitative trends were observed for the sensitivity results, regardless of the ground motion considered, as can be observed in Table 2 where the COVs of the maximum absolute values of the response sensitivity results for each floor, evaluated over the 28 ground motions and computed using $\Delta\theta_k$ = 0.01, 0.125, and 0.250, are given.

Table 2. COVs of the response sensitivity results for the 4-floor case over the considered set of 28 ground motions.

Floor	COV of interstorey drift sensitivities			COV of cumulative ductility sensitivities		
#	$\Delta\theta_k$ = 0.01	$\Delta\theta_k$ = 0.125	$\Delta\theta_k$ = 0.250	$\Delta\theta_k$ = 0.01	$\Delta\theta_k$ = 0.125	$\Delta\theta_k$ = 0.250
4	38.36%	30.37%	30.63%	42.16%	45.18%	46.69%
3	33.78%	28.56%	31.19%	38.89%	38.50%	38.75%
2	48.01%	27.46%	25.27%	34.41%	35.19%	36.95%
1	43.89%	39.31%	38.14%	39.11%	40.49%	41.85%

Response sensitivity results (averaged over the set of 28 ground motions) with the normalized interstorey drifts as EDPs of interest are summarized in Fig. 4. The local response sensitivities, i.e., $\partial\zeta_i/\partial\theta_k$, are depicted in Fig. 4a, the global response sensitivities, i.e., $\Delta\zeta_i/\Delta\theta_k$, in Fig. 4b,c.

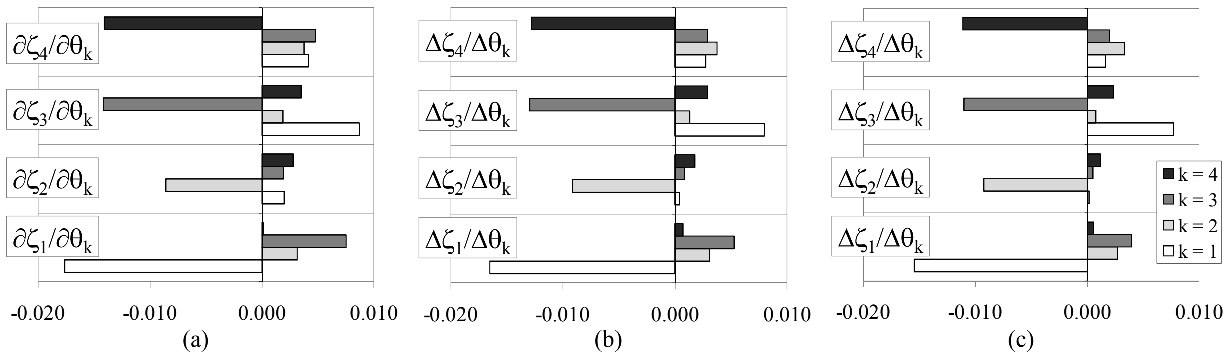


Figure 4. Normalized sensitivities of the maximum interstorey drifts:
(a) local; (b) global ΔA_k / A_k = 12.5%; (c) global with ΔA_k / A_k = 25%.

Comparisons between local and global sensitivities, i.e., between Figs. 4a and 4b,c, reveals some (typically minor) numerical differences. However, it is important to observe that the qualitative results are basically the same. Thus the response sensitivities (local derivatives) of the interstorey drifts with respect to the sensitivity parameters θ_k, are to some extent representative of the effects of finite variations of θ_k, and thus of finite variations of the BRB core areas.

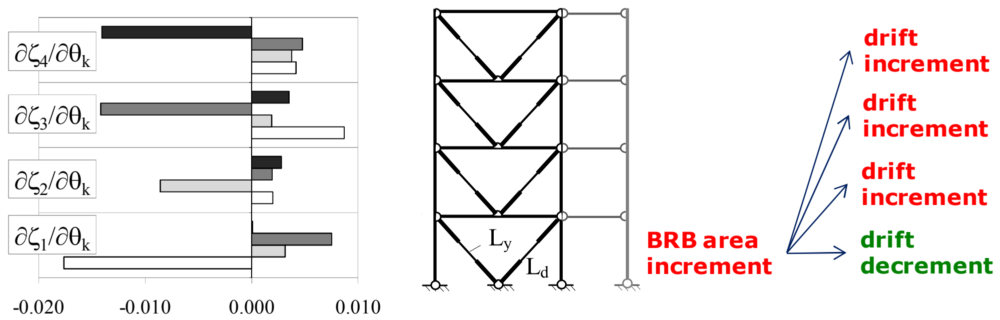


Figure 5. Considerations deduced from sensitivities: increment of BRB area at a given floor causes reduction of peak interstorey drift in that same floor and increment of peak drift values at the other floors.

A positive (negative) value of the sensitivity means an increment (decrement) of the relevant EDP due to the increment of the sensitivity parameter considered. Thus, the results shown in Fig. 4 allow the quantification of the increments and decrements of peak interstorey drifts due to the sensitivity parameters. In addition to such quantification, qualitative considerations can be deduced. For example, it is observed that the increment of the internal core area of the BRBs of a given floor causes a notable reduction of the peak interstorey drift of that same floor and a smaller increment of the peak drift values of all the other floors (Fig. 5).

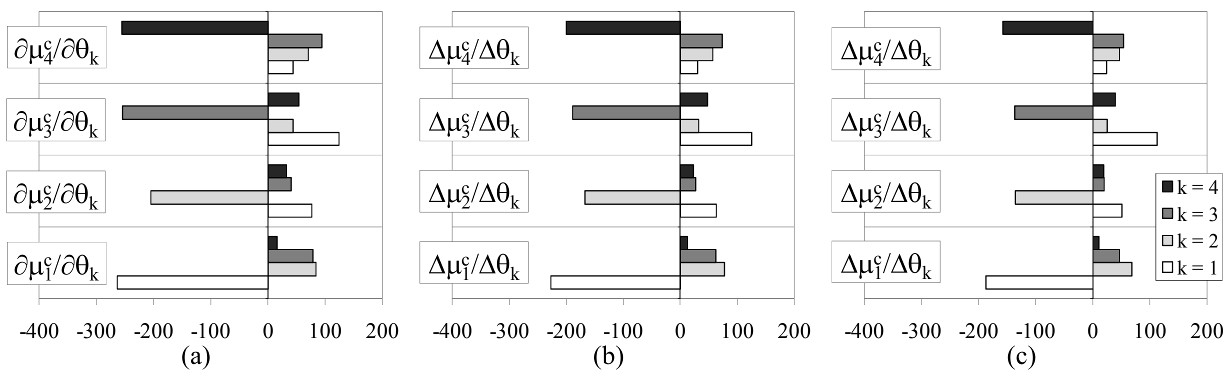


Figure 6. Normalized sensitivities of the BRB cumulative ductility:
(a) local; (b) global ΔA_k / A_k = 12.5%; (c) global with ΔA_k / A_k = 25%.

Similar trends of the local and global sensitivities (averaged over the set of 28 ground motions) are observed when the EDPs related to the seismic demand on the BRBs are considered, i.e., local ($\partial\mu_i^c/\partial\theta_k$) and global ($\Delta\mu_i^c/\Delta\theta_k$) response sensitivities of the cumulative ductility (Fig. 6). It is observed that the increment of the core area of the BRBs at a given floor decreases the seismic demand on the BRBs of that same floor and typically increases the seismic demand on the other BRBs, with increment amplitude usually smaller than the amplitude of the decrement.

3.5. Effects of over-strength distributions based on sensitivity analysis results

Response sensitivity diagrams shown in Figs. 4 and 6 can be used to identify the variation of the sensitivity parameters (BRB core areas) resulting in largest increment in the EDPs considered. The most unfavourable combination of sensitivity parameters is found according to the linearized formulation and set of variations described section 2. In this way, the response sensitivities assume a role similar to influence lines: whereas influence lines permit to locate moving loads in bridge analysis so to attain the largest effects, response sensitivity diagrams permit to estimate the worst combination of the variation of the BRB core areas to attain the largest EDP increment.

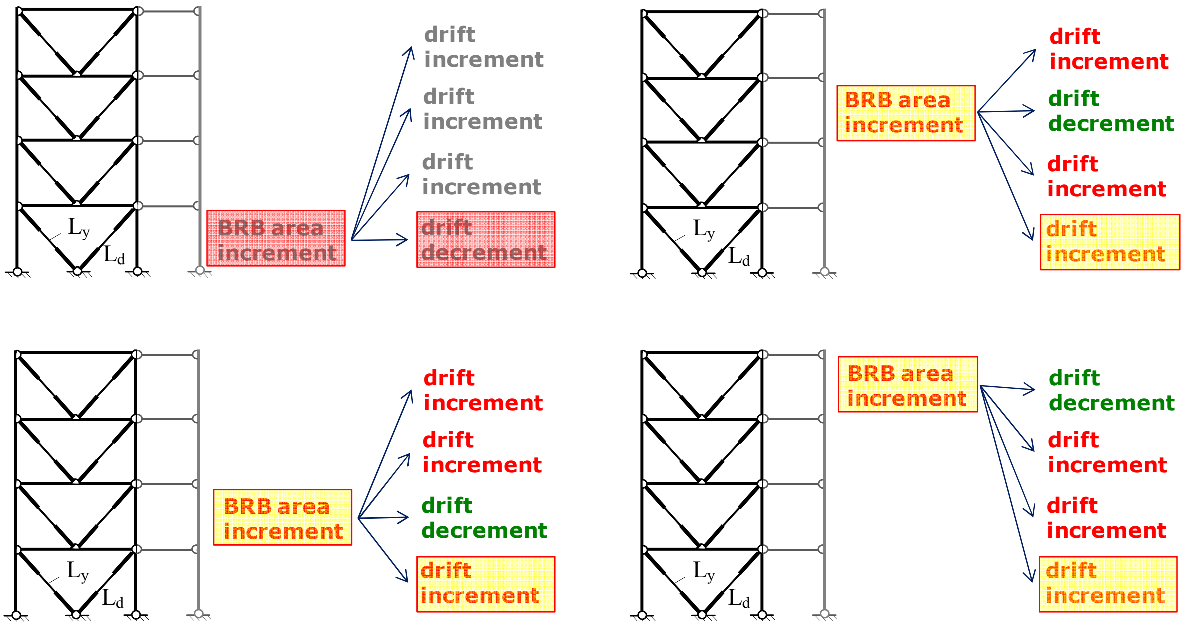


Figure 7. Evaluation of the effect of BRB area increments for the determination of the maximum interstorey drift increment at the first floor.

For example, if the goal is to evaluate the maximum interstorey drift at the first floor of the considered case study, then the brace areas of the second, third and fourth floors must be increased (positive sensitivity) and the brace areas at the first floor left unchanged (negative sensitivity), as depicted in Fig. 7.

Results derived from the set of variations with amplitudes spanning from 0 to 25%, are henceforth reported and commented. Results (averaged over the set of 28 ground motions) for the interstorey drift as EDP are given in Fig. 8a. For comparison purposes, results obtained with an homogeneous amplification of all sensitivity parameters θ_k are reported in Fig. 8b. Additionally, in order to verify the validity of the linear approximations based on the local response sensitivity calculations, the same Fig. 8 also show the actual EDP variations obtained from the nonlinear analysis with finite increments of BRB areas. It is observed that the linear approximations based on the local response sensitivity results give fairly accurate predictions of the actual variations, in particular when the amplifications in the BRB areas are below 12.5%. The results in Fig. 8 show significant differences between the maximum interstorey drifts derived from the most adverse combinations of BRB over-strengths and those derived from uniform BRB over-strengths. The BRB uniform over-strength reduces the interstorey drifts. On the other hand, the worst combination of BRB over-strength for the *i*-th interstorey drift significantly increases the *i*-th interstorey drift, e.g., the minimum interstorey increment is about 10% in floor #2 for 25% parameter increment. In addition, the worst combination of BRB over-strength for the *i*-th interstorey drift has the effect of reducing the other interstorey drifts (results not shown in the presented figures for the sake of brevity) with reductions in the range from 1% to 30%, always smaller than the increments in the *i*-th interstorey drift.

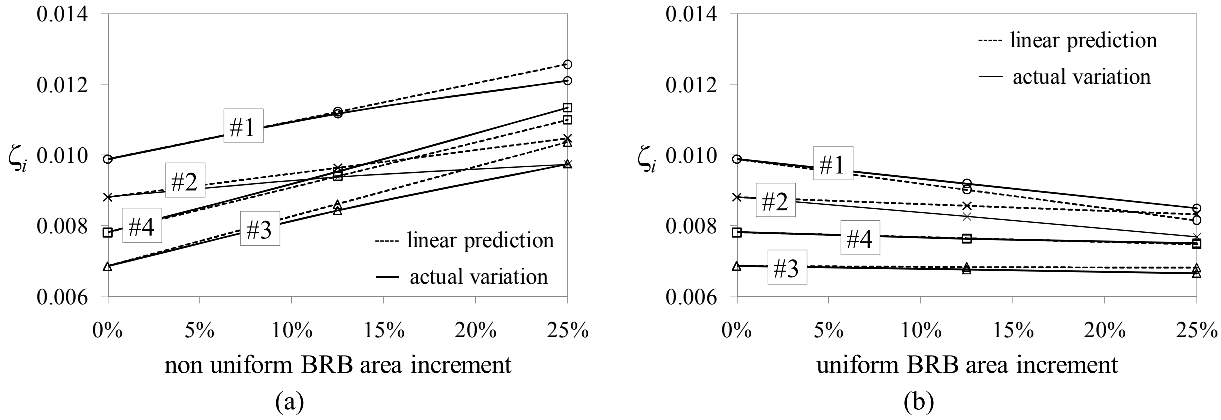


Figure 8. Variation of the maximum interstorey drift from:
(a) worst combination of increment of BRB areas based on sensitivity results;
(b) uniform increment of BRB areas.

Results similar to those of the interstorey drift are observed for the BRB cumulative ductility (Fig. 9). The increment of BRB cumulative ductility under its most adverse combinations of BRB over-strength is more important that those noted for the interstorey drift.

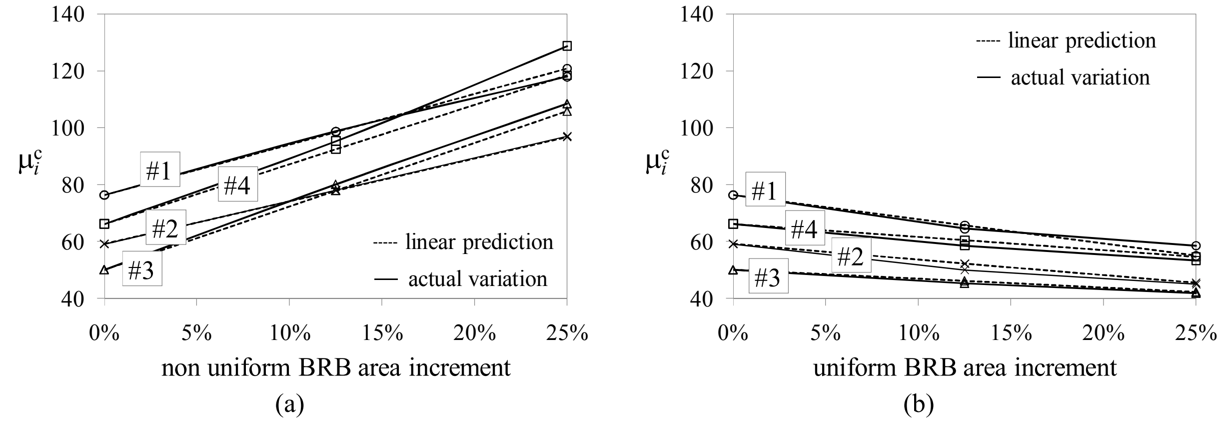


Figure 9. Variation of the BRB cumulative ductility from:
(a) worst combination of increment of BRB areas based on sensitivity results;
(b) uniform increment of BRB areas.

4. CONCLUSIONS

In this paper the influence of the brace over-strength on the seismic performance of steel braced frames is studied using response sensitivity analysis results. A realistic 4-storey steel frame with continuous columns, pinned beam-to-column connections, and braced with BRBs is considered as case study, taking as reference solution the design based on the modal response spectrum method. The damage of non structural elements is monitored by global EDPs given by the interstorey drift of each floor, the damage of the braces is monitored by the local EDPs given by the cumulative ductility of each BRBs, the internal core areas of the BRBs at each storey are chosen as set of independent sensitivity parameters. Seismic response and response sensitivity analyses are performed with a material and geometric nonlinear finite element code, using as input a set of 28 natural accelerograms. The results averaged over the considered set of accelerograms are used to illustrate the relationships between the brace over-strength irregularity and the seismic performance reduction. Specifically, the most unfavourable brace over-strength patters for each EDP are identified based on sensitivity analysis results, and the relevant worst possible increments of each EDP with respect to the reference design solution are predicted using a linear approximation based on local sensitivities and compared to the actual increments computed from nonlinear analysis. In this way it is highlighted how response sensitivity results are a powerful, adequately accurate and relatively simple strategy to indentify the worse effects of the selected sensitivity parameters on EDPs, effects that otherwise could not be correctly quantified. These effects can be important even when irregularities in brace over-strengths are less than 25%. Thus, response sensitivity analysis results can be used to better understand the structural seismic response, and as an efficient tool for safer and more effective seismic designs.

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