

Design of steel frames with buckling restrained braces

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SUMMARY:

Buckling Restrained Braces (BRBs) are very effective in dissipating energy through stable tension-compression hysteretic cycles. Their behaviour was studied extensively in the past decades and successfully experimented for the seismic protection of buildings. Despite the knowledge acquired from experimental and numerical studies, many seismic codes (i.e. Eurocode 8) do not yet stipulate provisions for the design of earthquake-resistant structures equipped with BRBs. This paper describes a procedure for the seismic design of steel frames equipped with BRBs. The results of the numerical investigation are intended for the validation of the design procedure and for the proposal of an appropriate value of the behaviour factor.

Keywords: Seismic design, behaviour factor, steel structures, buckling restrained braces

1. INTRODUCTION

The seismic design of structures is generally performed according to the force-based approach by means of a design spectrum which is reduced with respect to the elastic spectrum by means of the behaviour factor q . Seismic codes provide different values of q for the structural types. Some seismic codes, i.e. the Eurocode 8, do not consider any specific value of the behaviour factor for the design of frames equipped with buckling restrained braces. Some other seismic codes, i.e. the “NEHRP Recommended Provisions for Seismic Regulation for New Buildings and other Structures” (FEMA 450) and the AISC 2005 standard, stipulate values of the behaviour factor equal to 7 or 8 in case BRBs are inserted in frames with either pinned or moment resisting connections, respectively. As pointed out in other studies (Della Corte et al., 2011), these values of the behaviour factor are comparable to that suggested in the Eurocode 8 for the design of high-ductility moment resisting frames. The effectiveness of these high values of q has not been thoroughly investigated with reference to BRBs inserted within systems with low redundancy, i.e. within concentrically braced frames with pinned beam-column connections. In this paper, a set of steel frames equipped with BRBs is designed by means of different values of the behaviour factor. The seismic behaviour of these frames is evaluated by nonlinear dynamic analysis and a proper value of q is proposed. This value is expected to lead to the strict verification of the life safety performance level requirements.

2. CYCLIC BEHAVIOUR OF BRBs

Conventional buckling restrained braces (Fig. 1) consist of a ductile steel section restrained by a steel tube filled with concrete. An unbonding layer is interposed between the steel core and the surrounding concrete so that the central yielding core can deform longitudinally without interaction with the mechanism that restrains lateral and local buckling (Uang and Nakashima, 2004). As an alternative, “all-steel” buckling-restraint systems have been developed where the steel core is typically separated from the steel buckling-restraint unit by a small void (Della Corte et al., 2011). The brace is joined to the surrounding frame by means of the unrestrained non yielding segment which has to remain elastic

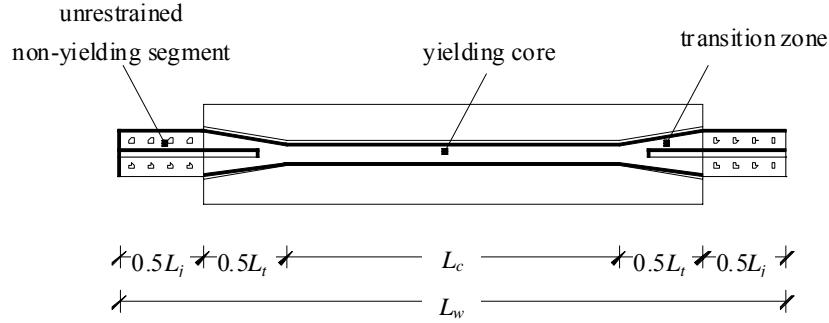


Figure 1. Schematic of a BRB

during cyclic loading. This performance is obtained by the adoption of a cross-sectional area A_j larger than the area A_c of the yielding core. The yielding core and the unrestrained non yielding segment are connected by an elastic transition zone with area A_t larger than A_c and generally lower than A_j .

The cyclic behaviour of the BRBs has been investigated extensively by means of laboratory tests. These tests show hysteretic loops with nearly ideal bilinear shapes and with moderate kinematic and isotropic hardening. The strength in compression is generally slightly greater than that in tension. The maximum ductility capacity, i.e. the ratio of the BRB maximum deformation Δ_{\max} to the BRB yield deformation Δ_y , is in the range 15 - 20 (Fahnestock et al., 2003).

2. PROPOSED DESIGN PROCEDURE

In this section a design procedure for concentrically braced frames equipped with buckling restrained braces in the chevron configuration is proposed. The braces are designed on the basis of the internal forces resulting from the modal response spectrum analysis of the frame. The design spectrum is obtained by reducing the elastic response spectrum by means of an assigned value of the behaviour factor q . According to the capacity design principles, the design internal forces of beams and columns are obtained by equilibrium assuming that all the braces are fully yielded and characterized by an axial deformation equal to that corresponding to the attainment of the maximum ductility capacity (*fully hardened braces*). For the sake of simplicity, the flexural stiffness of the columns is neglected. The yielding core, the transition zone and the unrestrained non yielding segment of the BRB are connected in series. Owing to this, the buckling restrained brace is modelled as a truss element characterized by an equivalent cross-section area A_{eq} given by the following relation

$$A_{eq,i} = \frac{A_{c,i}}{\frac{L_j}{L_w} \frac{A_c}{A_j} + \frac{L_t}{L_w} \frac{A_c}{A_t} + \frac{L_c}{L_w}} \quad (2.1)$$

where L_c , $0.5L_t$, $0.5L_j$ and L_w are the length of the yielding core, the length of the transition zone, the length of the unrestrained non yielding segment and the length of the whole brace, respectively (Fig. 1).

2.1. Design of the buckling restrained braces

The BRBs are designed in terms of required strength and ductility. The cross-sectional area of the core of the buckling restrained brace is designed by equating the design storey shear V_{Ed} to the lateral strength V_{Rd} provided by a pair of chevron BRBs, i.e. to the storey shear corresponding to the yielding of the braces

$$A_{c,i} = \frac{V_{Ed,i}}{2f_y \cos\theta} \quad (2.2)$$

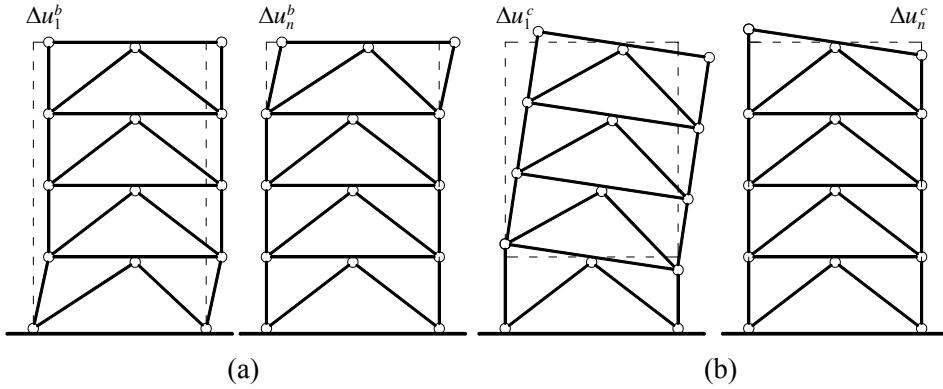


Figure 2. Deformative contribution of (a) brace and (b) columns in frames equipped with BRBs in the chevron configuration

In this relations f_y is the yielding stress of the core and θ is the angle of inclination of the brace with respect to the longitudinal beam axis.

The required ductility is evaluated on the basis of the qualifying cyclic tests described in AISC 2005. According to this seismic code, the buckling-restraining system must be able to sustain deformations corresponding to 2.0 times the design storey drift Δu^d . This latter parameter is the maximum accepted storey drift demand in the event of the design ground motion and is therefore a design choice. However, the selected value of Δu^d cannot be very high because the available ductilities of the BRBs on the market are limited. To evaluate the required ductility of the BRBs, the storey drifts Δu_i produced by the design seismic forces are first calculated by a computer program. In the present paper the cross-sectional area assigned to the BRBs is exactly that required by the design storey shear and therefore the storey drifts Δu_i are equal to the yielding storey drifts Δu^y . If the flexural stiffness of the columns is neglected, the storey drifts Δu_i are also equal to the sum of the contributions Δu^b and Δu^c provided by the axial deformations of braces and columns (Fig. 2). In particular, the contribution provided by the braces is

$$\Delta u_i^b = \frac{A_c f_y L_w}{E A_{eq} \cos \theta} \quad (2.3)$$

Consequently, the contribution provided by the columns can be expressed as

$$\Delta u_i^c = \Delta u_i^y - \Delta u_i^b \quad (2.4)$$

As the design storey drifts Δu^d are much larger than the storey drifts Δu^y , in the occurrence of design ground motions the BRBs undergo plastic deformations. When the yielding storey drift Δu^y is exceeded, the storey drift provided by the axial deformations of the columns becomes negligible and the storey drift is chiefly due to the plastic deformations of braces, i.e.

$$(\mu_i - 1) \cdot \Delta u_i^b = \Delta u^d - \Delta u_i^y \quad (2.5)$$

μ_i being the required ductility of the braces. Finally, the maximum design ductility of the braces $\mu_{max,i}$ is calculated in such a way that the buckling-restraining system sustains deformations corresponding to 2.0 times those caused by the design storey drifts, i.e.

$$\mu_{max,i} = 2\mu_i = 2 \frac{\Delta u^d - \Delta u_i^c}{\Delta u_i^b} \quad (2.6)$$

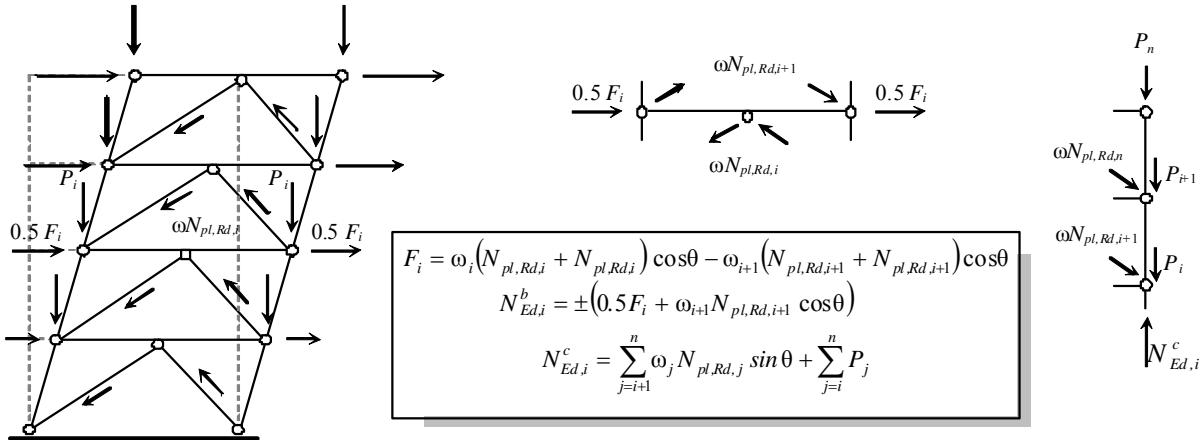


Figure 3. Evaluation of design internal forces in beams and columns

2.2. Design of beams and columns

According to capacity design principles, the design internal forces of beams and columns are obtained by equilibrium assuming that all the braces are fully yielded and hardened (Fig. 3). First, the horizontal forces F_i on both ends of the beam are evaluated based on the horizontal equilibrium of the beam; second, the axial force on each part of the beam is calculated supposing that F_i are equally shared between the two ends of the beam of the i -th floor. Finally, the axial load on the column is evaluated by translational equilibrium in the vertical direction.

The ultimate axial forces of braces in tension and in compression are equal to $\omega N_{pl,Rd}$ and $\beta\omega N_{pl,Rd}$, respectively. The tension strength adjustment factor ω is the ratio of the maximum tension force to the axial yielding force $N_{pl,Rd}$, while the compression strength adjustment factor β is the ratio of the maximum compression to the maximum tension force. In this study, on the basis of the results of many tests (Merrit et al., 2003, Newell et al., 2006), a bilinear backbone curve is adopted to represent the BRB behaviour. The post-yield stiffness ratio k_h is assumed equal to 5.36%. This value is calibrated so that a tension strength adjustment factor equal to 1.75 corresponds to an available ductility equal to 15. Thus, the tension strength adjustment factor is calculated at each storey as a function of the available ductility μ_{max} by means of the following relation

$$\omega_i = 1 + k_h \cdot (\mu_{max,i} - 1) \quad (2.7)$$

The compression strength adjustment factor is assumed to be unitary.

3. ANALYSED BUILDINGS

The design procedure described in Section 2 is applied to 4-, 8- and 12-storey buildings standing on soft soil (classified as Soil C according to Eurocode 8). The braces are located in the central span of the frames on the perimeter (Fig. 4). All beam-to-column connections are pinned and the whole seismic force is sustained by the chevron braces. Gravity columns are oriented as shown in Fig. 4 and subjected to gravitational loads equal to 9.16 kN/m^2 in the non-seismic design situation. Both gravity columns and columns belonging to the braced frames are continuous for two storeys at a time and pinned at the base.

The storey mass is estimated according to the Eurocode 8 based on an average value of dead and live loads equal to 5.0 kN/m^2 . The total seismic force is evaluated on the basis of the elastic spectrum proposed by the Eurocode 8 for soil type C and reduced by the behaviour factor q . To define a proper value of the behaviour factor, each frame is designed assuming a value of q ranging from 2.5 (value

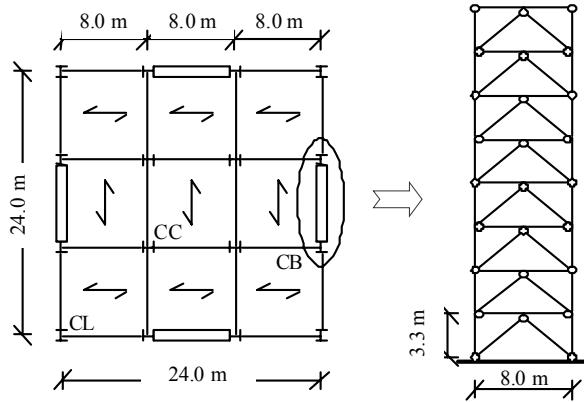


Figure 4. Plan layout of the building and arrangement of the chevron braced frames

stipulated by the Eurocode 8 for conventional chevron braced frames) to 6.5 in step of 1.0. The peak ground acceleration a_g is equal to 0.35 g. $P\Delta$ effects are neglected in the design phase. As suggested by FEMA 356 for the life safety performance level of braced steel frames, the maximum accepted storey drift is equal to 1.5% of the storey height. Consequently, a design storey drift Δu^d equal to 1.5% h is considered to design the ductility of the BRBs.

In accordance with common applications of BRBs, the length of the yielding core is supposed to be equal to $0.5 L_w$, while the transition zone and the unrestrained non yielding segment are characterized by $A_c/A_t = 0.5$ and $A_c/A_j = 0.3$, respectively. The length of the unrestrained non yielding segment and that of the transition zone are $L_j/2 = 0.65$ m and $L_t/2 = 0.5(L_w - L_c - L_j)$. Steel grade S 235 ($f_y = 235$ MPa) is used for braces and beams; steel grade S 235, S 275 ($f_y = 275$ MPa) or S 355 ($f_y = 345$ MPa) is used for columns. According to conventional design practice, the same column cross-section is adopted for two consecutive storeys.

The results of the design of the buckling restrained braces are summarized in Tables 3.1 and 3.2 with reference to the 8-storey frames. In particular, the tables report the area of the cross-section of the yielding core and the available ductility of the BRBs obtained assuming different values of the behaviour factor. The increase in the behaviour factor, and thus the reduction of the design seismic forces, leads to smaller values of the area of the cross-section adopted for the BRBs. Consequently, the higher the value of the behaviour factor, the greater the period of the designed frames. Instead, the adopted value of q has a negligible effect on the design of the available ductility of the BRBs. The required ductility is generally greater at the lower storeys but its distribution is fairly constant in elevation. Its mean value ranges from 16.7 (obtained for $q = 2.5$) to 17.6 (obtained for $q = 6.5$). The same consideration applies to the heightwise distribution of the tension strength adjustment factor because this parameter is calculated at each storey on the basis of the required ductility. The mean values of the tension strength adjustment factor, which are not reported in tables, range from 1.85 to 1.89.

4. NONLINEAR DYNAMIC ANALYSES

The seismic response of the designed frames is evaluated by nonlinear dynamic analyses carried out by means of the OpenSees computer program. The viscous damping forces are obtained through the Rayleigh formulation. In particular, a viscous damping ratio equal to 0.03 is fixed for periods equal to those of the first and second modes of vibration. The $P\Delta$ effects are included in the analysis. The results of the nonlinear dynamic analyses are used to evaluate the level of damage of the structures. The reference level of damage is characterized by assigned axial deformations of braces and by assigned internal forces of brittle members (beams and columns of the braced frame, gravity columns).

The set of twenty ground motions adopted in the FEMA/SAC project (Somerville et al., 1997) is used

Table 3.1. Cross-sectional area (cm^2) of the yielding core of the BRBs of the 8-storey frames

Storey	$q = 2.5$	$q = 3.5$	$q = 4.5$	$q = 5.5$	$q = 6.5$
8	18.45	11.49	8.07	6.29	5.75
7	30.24	18.29	12.47	9.62	9.04
6	38.99	22.97	15.20	11.61	11.23
5	46.06	26.73	17.30	13.09	12.95
4	52.17	30.19	19.47	14.68	14.60
3	57.41	33.50	21.79	16.47	16.25
2	61.55	36.32	23.95	18.21	17.73
1	63.92	38.04	25.35	19.36	18.67

Table 3.2. Available ductility of the BRBs of the 8-storey frames

Storey	$q = 2.5$	$q = 3.5$	$q = 4.5$	$q = 5.5$	$q = 6.5$
8	15.95	16.56	16.85	17.21	17.24
7	15.99	16.50	16.72	17.10	17.13
6	16.11	16.62	16.75	17.11	17.13
5	16.49	16.84	17.02	17.26	17.27
4	16.73	17.19	17.26	17.47	17.46
3	17.20	17.56	17.68	17.82	17.81
2	17.58	17.82	17.95	18.05	18.05
1	18.13	18.23	18.29	18.33	18.35

in this study. These ground motions represent seismic events having a probability of exceedance equal to 10 percent in 50 years in the U.S. Los Angeles area. All the accelerograms are scaled by a factor 0.88 so that the mean 5% damped spectrum of the twenty ground motions matches the elastic spectrum used in design.

4.1. Numerical model

The numerical analyses are carried out on a two-dimensional model which represents half the structure of the building. The ground motions are applied along the x -direction. The braced frame designed in the previous section is coupled with six gravity columns which are pinned at the base: two columns are oriented so as to provide the maximum lateral stiffness and four columns so as to provide the minimum lateral stiffness.

The single buckling restrained brace is modeled by means of three elements. The outermost elements are elastic and schematize the unrestrained non yielding segments and the transition zone connected in series. Thus, each of the outermost elements is characterized by a length and a cross-sectional area equal to

$$L^1 = L^3 = \frac{L_j + L_t}{2} \quad (4.1)$$

$$A^1 = A^3 = \frac{A_c(L_j + L_t)}{L_j A_c / A_j + L_t A_c / A_t} \quad (4.2)$$

The central element simulates the yielding core. It is modeled by a “nonlinear BeamColumnElement” with the cross section defined by the “Section Aggregator object”. This command aggregates groups of uniaxial material objects into a single section model. In particular, a uniaxial Giuffré-Menegotto-Pinto material object characterized by a resistance equal to $f_y A_c$, a stiffness equal to $E A_c$ and a strain-hardening ratio $k_{h,c}$ (ratio between post-yield tangent and initial elastic tangent) is defined to represent the section force-deformation response for the axial degree-of-freedom, while an elastic behaviour is considered with reference to flexure. The value of the strain-hardening ratio $k_{h,c}$ of the core is set equal to 3.88%. This value has been selected so that the post-yield stiffness ratio of the whole buckling

restrained brace is equal to 5.36%. Beams and columns of the braced frames are modeled by elastic beam column elements.

4.2. Acceptance criteria

For each nonlinear dynamic analysis, the maximum required ductility of braces is determined as the ratio of the maximum plastic axial deformation to the axial elongation at yielding

$$\mu_{req} = \frac{\max(\Delta L_b^+, -\Delta L_b^-) - \Delta L_{by}}{\Delta L_{by}} \quad (4.3)$$

The required axial elongation (ΔL_b^+) and shortening (ΔL_b^-) are due to the plastic deformation of the core and to the elastic deformation of both the transition zone and the unrestrained non yielding segment. Thus, ΔL_b^+ and ΔL_b^- are calculated as a function of the horizontal and vertical displacements of the ends of the brace. The axial elongation of the brace at yielding is calculated as

$$\Delta L_{by} = \frac{f_y}{E} \left(L_j \frac{A_c}{A_j} + L_t \frac{A_c}{A_t} + L_c \right) \quad (4.4)$$

The required ductility is compared to the maximum available ductility. 3/4 of the design available ductility is assumed as the reference value in analogy to what stipulated by EC8 - part 3 with reference to the plastic rotation capacity of members in flexure for the life safety performance level. Values of the maximum normalized ductility $\bar{\mu} = \mu_{req}/(3/4\mu_{max})$ lower or higher than 1 indicate that the ductility required by the earthquake is lower or higher than the available value, respectively.

Beams and columns of the braced frames and gravity columns are brittle elements. Therefore, inelastic deformations or buckling phenomena are not allowed in these members. The seismic performance of these members is examined in terms of normalized resistance demands: the normalized axial force resistance demand and the normalized bending moment resistance demand. The first parameter is calculated at each step of the analysis as the ratio of the axial force N_{Ed} to the buckling resistance $N_{b,Rd}$ of the member reduced due to the bending moment M_{Ed} ; the second is calculated as the ratio of M_{Ed} to the design plastic moment resistance $M_{N,Rd}$ reduced due to N_{Ed} . Both flexural and buckling resistances are determined according to EC3 assuming that the partial safety coefficients γ_{M0} and γ_{M1} are equal to 1. Beams and columns fulfill the life safety limit state requirements if the maximum values of both the abovementioned ratios are lower than 1.0.

4.3. Seismic performance of the designed structures

Figure 5 shows the heightwise distribution of the normalized required ductility of the braces of the 4-, 8- and 12-storey frames. Specifically, at each storey the value is the median obtained for the 20 ground motions. Different curves are represented in order to distinguish systems designed by means of different values of the behaviour factor. Frames designed by means of low values of the behaviour factor are characterized by a fairly uniform distribution of the normalized ductility; if higher values of q are considered, the maximum value of the normalized ductility increases at the lower two storeys and becomes larger than 1.0 for a value of q which depends on the number of storeys of the building.

The maximum value of the normalized ductility is calculated and plotted in Figure 6, as a function of the corresponding value of q for systems with 4, 8 and 12 storeys. Thus, it is possible to evaluate the value of q corresponding to $\bar{\mu}=1$ by linear interpolation. Specifically, the values 6.12, 5.08 and 5.47 are obtained with reference to the 4-, 8- and 12-storey frames, respectively.

All the non-dissipative members are characterized by maximum values of the normalized axial force and bending moment lower than 1.0 at all the storeys, independently of the adopted value of the

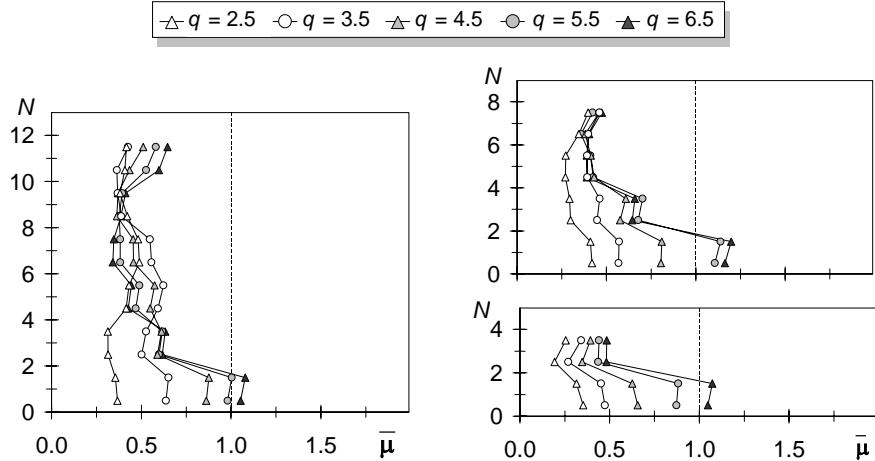


Figure 5. Normalized required ductility in the braces of frames designed assuming different q

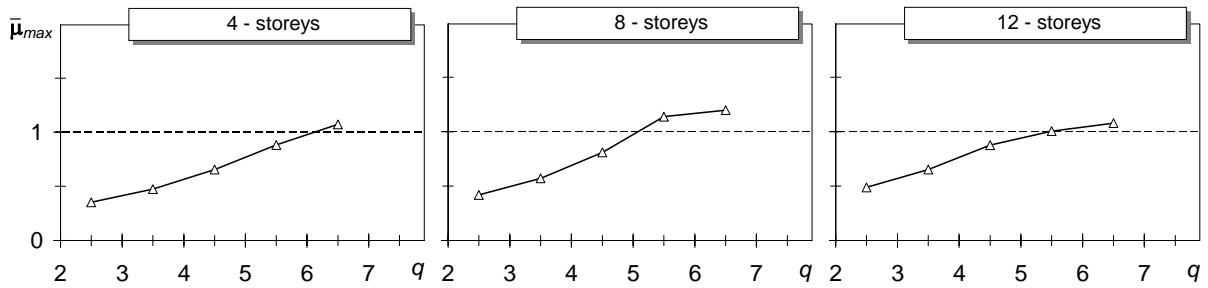


Figure 6. Evaluation of the behaviour factor corresponding to a maximum value of the maximum normalized ductility equal to one.

behaviour factor. Indeed, no yielding or buckling occurs in non dissipative members. As an example, the heightwise distribution of the normalized axial force and bending moment is plotted in Figures 7 and 8 for the columns belonging to the braced frame (CB-type columns) and for the gravity columns arranged so as to provide the maximum lateral stiffness (named as CC-type columns in Figure 3).

On the basis of these few results, a value of the behaviour factor equal to 5 seems to be suitable for designing frames equipped with buckling restrained braces in the chevron configuration. This value is lower than that suggested by FEMA and ASCE and is in line with the results obtained in Marino et al. (2007) and Castaldo et al. (2009).

5. CONCLUSIONS

The paper presents the results of a preliminary study aimed at defining a proper value of the behaviour factor q for the design of concentrically braced frames equipped with buckling restrained braces. Specifically, a set of steel frames equipped with BRBs is designed by means of many values of the behaviour factor. On the basis of the seismic response, expressed in term of axial deformations of braces and internal forces of brittle members, a value of q equal to 5 seems to be suitable for the design of this structural type. It should be noted that in this paper a design storey drift equal to $1.5\%h$ has been considered. Different choices for the value of this parameter lead to different available ductilities of the BRBs and therefore to different values of the behaviour factor.

It is worth noting that all the considered frames have been designed neglecting second order effects and assuming columns which are continuous for two storeys at a time. In order to extend the validation of the proposed value of the behaviour factor to more general structural configurations, further

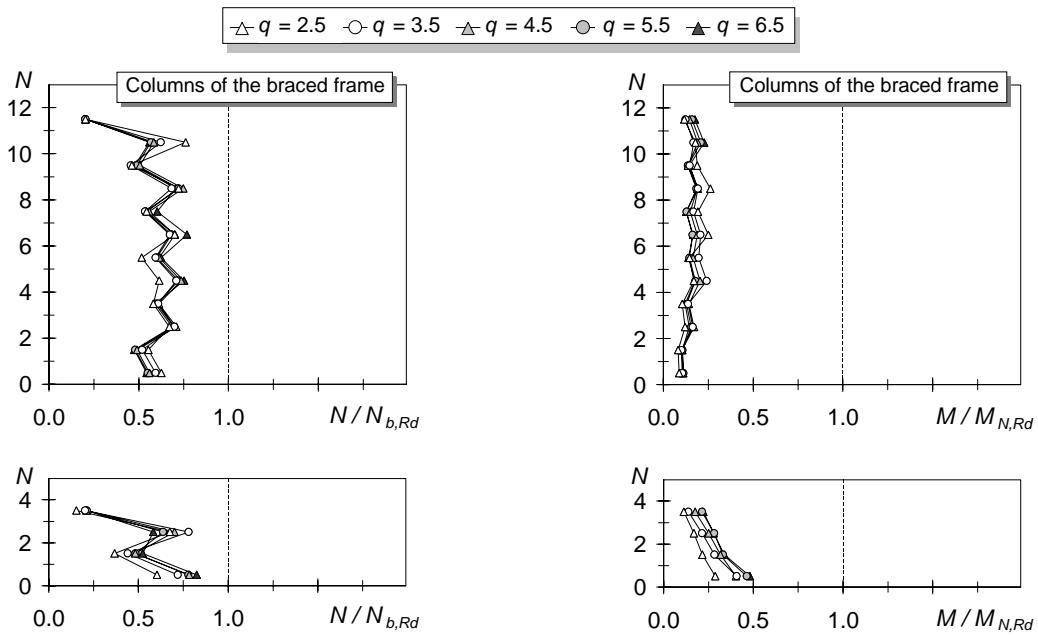


Figure 7. Normalized axial forces and bending moments in columns of the braced frames (CB)

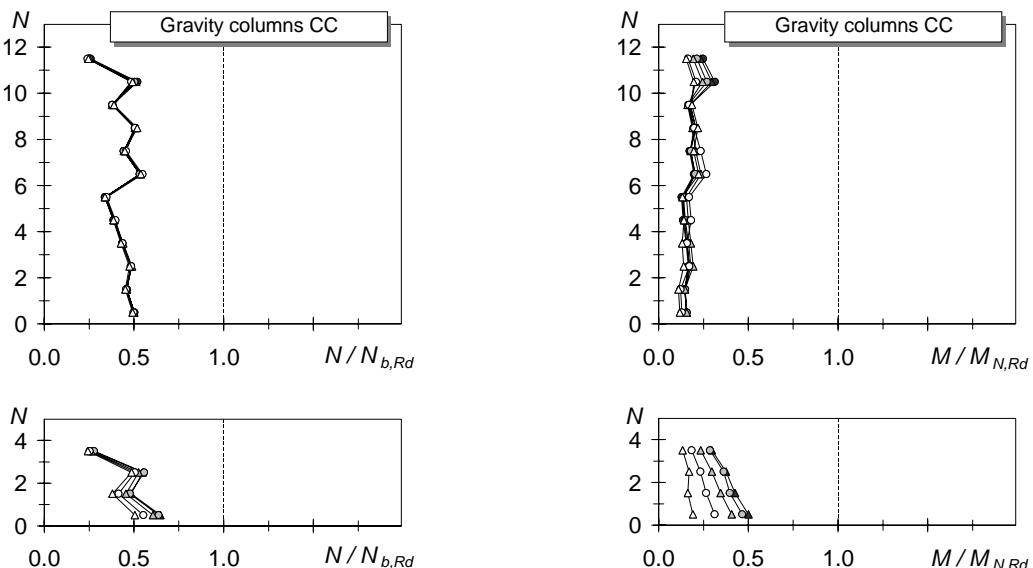


Figure 8. Normalized axial forces and bending moments in CC-type gravity columns

investigations are required. In particular, these analyses should regard frames characterised by continuous columns and designed on the basis of provisions which include $P\Delta$ effects.

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