The effect of different modelling approach on seismic analysis of steel concentric braced frames

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SUMMARY:

In this paper it is presented a numerical study devoted to investigate the accuracy of physical-theoretic brace models using force-based finite elements with fibre discretization of the cross section in predicting the non linear response under monotonic and cyclic loads of steel concentric braced frames. In the first part of the study, it is investigated the influence of geometric nonlinearity to account for buckling of single brace. The second part of the paper is concerned with the modelling issues of whole braced structures. The effectiveness of the modelling approach is verified on the nonlinear static and dynamic behaviour of different type of bracing configurations. The model sensitivity to brace-to-brace interaction and the capability of the model to mimic the response of complex bracing systems is analyzed.

Keywords: concentric braced frames; numerical modelling, seismic analysis

1. INTRODUCTION

The seismic performance of steel conventional concentric braced frames (CBFs) is mostly influenced by the cyclic behaviour of steel braces, which are the primary members devoted to dissipate the input energy in the modern philosophy of capacity design. As it is well known, the hysteretic response of concentric braces is characterized by the buckling in compression, the yielding in tension and significant pinching when the deformation reverses. As a matter of fact this non-linear performance is very complex to be simulated. On the other hand, an accurate model for braces is essential for improving the accuracy of the computed inelastic seismic response of braced frames.

In general, the hysteretic models used to mimic the brace response introduce significant simplifications if compared to the experimental behaviour. These differences in brace hysteretic characteristics could lead to uncorrect prediction of the peak responses or even behaviour modes (Khatib *et al.* 1988; Uriz and Mahin 2008). In literature, three different modelling approaches may be recognized (Uriz and Mahin 2008): (i) phenomenological models (PM); (ii) continuum finite element models (FEM); (iii) physical-theory models (PTM).

In the present study, the PTM approach was used. According to this method, the brace hysteretic behaviour is modelled at least with two elements connected by a generalized plastic hinge for braces simply pinned. Inelastic hinges concentrated at the element ends and midspan are used in the case of fixed-end braces (Giberson 1967). In this type of models a geometric nonlinearity (namely an initial camber) is directly introduced to account for buckling of braces (Nonaka 1973, Zayas *et al.* 1981, Ikeda and Mahin 1984, Soroushian and Alawa 1990, Remennikov and Walpole 1997, Jin and El-Tawil 2003, Dicleli and Mehta 2007, Dicleli and Calik 2008, Uriz and Mahin 2008, Nip *et al.* 2010). Anyway, this camber was not uniformly defined. Indeed, although there is a great number of research studies on the application of PTMs, non-uniform modelling approaches have been adopted by different authors. Hence, it is necessary to univocally calculate the amplitude of initial camber in a rational manner in order to reproduce the buckling response as close as possible to the experimental behaviour. This consideration motivated the present study.

The paper is organized into two main parts. After a brief introduction on the basic features of the generated models, it is presented and discussed a parametric analysis devoted to examine the influence of input data on hysteretic behaviour of the individual brace. In the second part the modelling aspects of braced frames are investigated. The effectiveness of modelling assumptions are validated by means of an extensive set of correlation studies with experimental results available from literature on single braces (Black *et al.* 1980) and on building prototypes in pseudo-static (Wakawayashi *et al.* 1970, Yang *et al.* 2008) and dynamic (Uang and Bertero 1986) conditions.

2. GENERAL MODELLING ASSUMPTIONS

The numerical models implemented in this study were generated using the nonlinear finite element based software "Seismostruct" (Seismostruct Ltd. http://www.seismosoft.com/en/HomePage.aspx). The models were developed using the force-based (FB) distributed inelasticity elements (Filippou and Fenves 2004, Fragiadakis and Papadrakakis 2008).

The cross-section behaviour is reproduced by means of the fibre approach, assigning a uniaxial stressstrain relationship at each fibre. A number of 100 fibers to mesh the cross section of braces was used.

The Menegotto-Pinto (MP) hysteretic model (Filippou *et al.* 1983) was used to simulate the steel behaviour. The parameters characterizing this model have been calibrated on the basis of the average stress-strain relationship derived from cyclic coupon tests performed by Black *et al.* (1980).

The calibrated material parameters are reported in Table 1, while the comparison of the numerical response and the experimental average envelope is plot in Figure 1 (Landolfo *et al.* 2010).



Figure 1. Calibrated vs. Experimental steel response.

Table 1. Calibrated parameters of steel hysteretic models.

		2				
Steel model	$E_{ m h}$	R_0	A_1	A_2	A_3	A_4
MP	0.025	20.00	18.50	0.15	0.00	1.00

The numerical integration method used is based on the optimized Gauss-Lobatto distribution (Bathe 1995). A number of 5 integration points has been considered.

The braces were modelled with two elements only, arranged to have a bilinear shape with an initial camber (Δ_0). Figure 2 schematically shows the type of model adopted in this study, where integration points (IP) and the end joints (J_i) are clearly highlighted.



Figure 2. The implemented model to mimic the brace behaviour.

It is important to highlight that some phenomena as the plastic local buckling and the low-cycle fatigue effects are kept beyond the scope of this study.

3. INFLUENCE OF CAMBER AMPLITUDE

In order to investigate the influence of different amplitude of initial camber, the numerical curves were compared to the experimental tests on strut specimens carried out by Black *et al.* (1980). In the following, for brevity sake, the results are shown for strut 1 only.

The amplitude of camber Δ_0 was varied using the following theoretical formulations :

1. *ECCS-78*. The buckling curves presented in ECCS 1978 have been obtained on the basis of Ayrton-Perry theory (1886). The initial deflection is obtained from the condition corresponding to the achievement of the yield stress in the outermost fibre under the combined presence of the buckling load N_b and the related bending moment $M(N_b)$, obtained having assumed an initial sinusoidal shape, thus leading to the following Equation:

$$\Delta_0 = \frac{W}{A} \cdot \alpha \cdot \sqrt{\bar{\lambda}^2 - 0.04} \tag{1}$$

Being W the section modulus in the buckling plane, A the cross section area, $\overline{\lambda}$ the dimensionless slenderness and α takes into account the element imperfections and characterized the buckling curves adopted in ECCS 1978 Rondal and Maquoi (1978, 1979).

2. Georgescu (1996). Starting from the same hypotheses, the camber is given by the following equation:

$$\Delta_0 = \left(\frac{1}{\chi} - 1\right) \cdot \left(1 - \frac{\chi f_y}{\sigma_E}\right) \cdot \frac{W}{A}$$
(2)

where χ is the buckling reduction factor and σ_E is the critical Eulerian stress. The buckling reduction factor can be obtained according to EN 1993:1-1(2005) as function of $\overline{\lambda}$ and α .

- 3. *EN 1993:1-1 (2005)*. For structural analysis EN 1993:1-1(2005) recommends to introduce initial local bow imperfections of members in frames sensitive to buckling in a sway mode. The code provides the values of such imperfections in terms of Δ_0/L , where L is the member length.
- 4. Dicleli & Mehta (2007). According to this theory the initial camber Δ_0 is derived assuming along the length of the brace a linear variation of the second-order bending moment generated by the axial force in the deflected bi-linear configuration of the strut, by imposing the equilibrium state at the mid-brace the second-order transverse displacement Δ_b of the brace at buckling load N_b . Hence, the initial eccentricity is obtained as:

$$\Delta_0 = \frac{M_{pb}}{N_b} \cdot \left(1 - \frac{N_b L^2}{12EI}\right) \tag{3}$$

5. *Dicleli & Calik (2008)*. The initial camber Δ_0 is derived assuming that the sinusoidal deformed shape of the brace prior to buckling and the imposing the second order flexural equilibrium in the section located at the mid-length of the buckling semi-wave, Δ_0 is obtained as follows:

$$\Delta_{0} = \frac{M_{pb}}{N_{b} \left(1 + \frac{N_{b}L^{2}}{8EI \left(1 - \frac{N_{b}L^{2}}{\pi^{2}EI} \right)} \right)}$$
(4)

As it can be easily observed, for each theoretical formulation the value of initial camber varies if the brace section and the brace slenderness change. As it is expected, the results obtained with ECCS-78

and Georgescu formulations are very similar, owing to the similar theoretical assumptions. The camber amplitude calculated according to EN 1993:1-1 results is three times larger. The camber amplitude given by Dicleli and Mehta formulation is almost twice the value calculated with Dicleli and Calik.

The strut was initially analyzed under monotonically increasing axial compression displacements. The monotonic response curves in terms of axial force–axial displacement and axial force-lateral deflections for strut 1 are shown in Figure 3. This figure clearly illustrates the sensitivity of the initial buckling load to the assumed initial camber, where differences in load-carrying capacity diminish as axial displacements increase.



Figure 3. Influence of camber under monotonic loading.

The influence of camber under cyclic conditions has been also investigated. As it can be observed in Figure 4, a striking resemblance between the test and analysis result is recognized for the model having the amplitude of camber set according to Dicleli and Calik (2008). The results obtained using the other camber formulations lead to a non-negligible misestimate of buckling strength. For all examined formulations the calculated residual post-buckling strength was larger than the experimental value. This outcome highlights one of the limit of PTMs, which is the impossibility to take into account the deterioration phenomena due to the accumulation of plastic deformation in locally buckled parts of plastic hinge zones.

At the light of the considerations shown in this Section, it can be noted that the better modelling approximations were obtained using the Dicleli and Calik (2008) formulation. Therefore, in the following the parametric analysis are shown under these hypotheses.

4. VALIDATION AGAINST NON-LINEAR STATIC ANALYSIS

4.1.Generality

In order to assess the accuracy of the adopted modelling approach to predict the non-linear static response of different braced frame configurations, a X-CBF and an inverted V zipper CBF were analyzed. These bracing configurations were chosen because they show the most complex behaviour among the typical brace schemes used in building.

In monotonic and cyclic static analysis, it was applied an incremental horizontal displacement history equal to that experimentally applied during each test. In particular, the geometric nonlinearity formulation (i.e., "large displacements and small strains") was adopted and the Skyline solver was used for each displacement-step to ensure the equilibrium of the internal member forces and overall frame base shear at each iteration.

4.2. X-CBFs

The experimental data from tests conducted on four nominally identical one-storey dual X-braced portal frames by Wakawayashi *et al.* (1970) has been used to assess the validity of the investigated numerical modelling in case of non-linear static pushover and static time history conditions.



Figure 4. Influence of camber on cyclic response.

In the numerical model, full strength and full rigid beam-to-column joints have been considered. Both beam and columns were modelled as distributed plasticity elements with 5 IPs and 100 fibres per section. The braces have been modelled as perfectly pinned. The restraint effect of the diagonal in

tension has been taken into account in the calculation of the geometrical slenderness λ of X-diagonal braces. This effect halves the brace in-plane buckling length, while it is taken as inefficient for out-of-plane buckling. Hence, the geometrical in-plane slenderness has been calculated considering the half brace length, while the out-of-plane ones considering the entire brace length. In this case the in-plane slenderness is maximum and the corresponding camber is calculated with Dicleli and Calik formulation (2008) considering the half brace length, thus resulting equal to 0.30%Lo, being Lo the brace buckling length, assumed equal to the length between the brace intersection point and the end working point.

In monotonic and cyclic analysis, it is applied an incremental horizontal displacement history equal to that experimentally applied during each test.

As shown in Figures 5a,b, both the monotonic and cyclic performances of the X-CBF specimens have been satisfactorily simulated. The deformed shapes in monotonic and cyclic conditions, shown in Figure 6, are very close to that exhibited during the tests.

Some differences can be recognized at high displacement demands where some damages in connections was experimentally observed and, as expected, the models cannot reproduce this phenomena.



Figure 5. X-CBFs: numerical vs. experimental response: monotonic (a) and cyclic (b) condition.



Figure 6. Deformed shapes at peak displacement in monotonic (a) and at the final stage of cyclic pushover (b).

4.3. Inverted V-CBFs with zipper struts

Yang *et al.* (2008) performed a pushover test on a 1/3 scaled model of a 2D zipper frame at the Structural Engineering Laboratory at Georgia Tech. The frame consisted in three storey one zipper braced bay.

In the numerical model, full strength and full rigid beam-to-column joints have been considered. All

elements were modelled as distributed plasticity elements with 5 IPs and 100 fibres per section. The braces and the zipper struts have been modelled as perfectly pinned. The amplitude of brace cambers calculated according to Dicleli and Calik (2008) resulted equal to equal to 0.40%, 0.46% and 0.55% of the brace buckling length assumed equal to the length between the working points, as in the previous case. Analogously to the case of X-CBFs, the MP steel has been used for all members.

As depicted in Figure 7a, the obtained numerical response strictly matches the experimental curve, reproducing a collapse mode close to that observed in the test. Indeed, the model catch the sequence of brace buckling at first and second storey, which corresponds to the first singularity in the numerical curve.



Figure 7. Inverted V-CBFs with zipper struts: numerical vs. experimental curve (a), collapse mode (b,c).

5. VALIDATION AGAINST NON-LINEAR DYNAMIC ANALYSIS

The modelling of CBFs in dynamic conditions needs to take into account more aspects than those examined in static loading. In particular, the inertia effects, the equivalent viscous damping and the hysteretic response of non-linear elements should be carefully addressed, because all of them could significantly affect the dynamic behaviour of the structure. As a consequence, the incorrect modelling of one of them leads to unrealistic and inaccurate numerical outcomes.

Hereinafter, the influence of these aspects on the numerically predicted dynamic response of CBFs has been verified and validated on the basis of a shaking table test carried out by Uang and Bertero (1986, 1989) at the University of California (Berkeley). They tested a 3D prototype steel concentrically braced building, having six – storeys and a square plan with three frames in the both directions and a composite floor system. The tested specimen was a reduced-scale prototype from a full-scale building designed according to U.S. Uniform Building Code (UBC) 1979 and Japanese design codes. The size of the prototype was obtained using a scale factor of 0.3048, which complied with the weight, height, and plan limitations of the shaking-table equipments. The model complied with all the material requirements except that of mass density.

The beams and columns are modelled as distributed plasticity elements with 5 IPs and 100 fibres per section. All beam-to-column joints are simulated as full strength and full rigid. The braces are modelled as fixed at both ends and the initial camber resulted equal to 0.11%L according to Dicleli and Calik formulation (2008), being *L* the length between the brace working points.

At each floor all nodes are constrained by a rigid in-plane diaphragm allowing to have only three dynamic degrees of freedom at each floor, i.e. two translations and one torsional rotation.

In dynamic time history analysis the numerical response was calculated using the Newmark numerical integration scheme with a time-step of 0.005 sec and internal iterations within each time-step. tangential stiffness damping has been used assuming a damping ratio of 1% at first and second mode. The acceleration input is applied to all nodes at the basis of the model, which correspond to those physically attached to the shake table platform.

Figure 8 shows the comparison between numerical and experimental response in terms of interstorey drift ratios. As it can be noted the model satisfactorily match the experimental response, also in terms of collapse mode (Figure 9b).



Figure 8. Experimental vs. numerical interstorey drift (model with tangential stiffness damping).



Figure 9. Numerical model (a) and deformed shape at 9.414 sec (b).

5.1 Influence of brace camber

The correct definition of brace camber is even more important in dynamic than in static conditions, because the amplitude of camber affects the numerical hysteretic behaviour and consequently the

hysteretic damping. In order to show the sensitivity of CBF inelastic dynamic response on brace camber in Figure 10 it is depicted the comparison between the numerical response in terms of interstorey drift ratios at the levels where the braces buckle (namely the second and the fifth storey). As it can be easily recognized the model with camber by EN 1993:1-1widely missed the prediction of displacements. This is due to the larger value of brace camber (= 0.20L) thus leading to anticipate the brace buckling. Hence, the larger is the brace camber and the larger is stiffness decrease, resulting in larger damage concentration and residual drifts.



Figure 10. Influence of initial camber on the dynamic response.

6. CONCLUSION

The main aspects related to the numerical modelling for seismic analyses of steel concentric braced frames were highlighted and discussed.

The problem of the accurate model of hysteretic behaviour of the individual brace was first addressed. To this end, physical-theoretic brace models are implemented using force-based elements with distributed or concentrated inelasticity and fibre discretization of the cross section. The features of the nonlinear finite element based software Seismostruct were adopted.

A parametric study has been presented to examine the influence of the initial camber to trigger brace buckling.

In order to validate the modelling tools, the numerical results have been compared to the outcomes of available experimental testing (Black *et al.* 1980), which demonstrated the ability of the Dicleli and Calik (2008) model to represent realistically the buckling strength, the post-buckling behaviour, the tensile strength, out of-plane deformations, and overall hysteretic behaviour.

Once examined the response of single brace, the modelling aspects of whole braced structures were investigated, as well. The effectiveness of the proposed modelling recommendations is verified in nonlinear static field for different type of bracing systems tested in literature, namely X-CBFs (Wakawayashi *et al.* 1970) and inverted V zipper CBFs (Yang *et al.* 2008). These configurations have been selected to test the model sensitivity to brace-to-brace interaction and to verify the versatility of the model also on more complex configuration as the case of inverted V zipper CBF. The comparison between numerical and experimental response curve showed an excellent agreement.

Finally, the aspects related to the numerical modelling of CBFs in nonlinear dynamic conditions were investigated. The effectiveness of numerical results has been verified on the basis of shaking table tests carried out by Uang and Bertero (1986, 1989). Comparing the numerical curves to those experimentally obtained, it has been observed that initial stiffness elastic damping, which has been widely used in most of existing studies, is inappropriate. Because it leads underestimating the displacement demand owing to the presence of large artificial damping forces when the structure enters in the inelastic range. On the contrary, the use of tangential stiffness damping is the most appropriate, because it gave an excellent prediction of displacement response.

The influence of brace camber on the inelastic dynamic response has been also investigated. The amplitude of camber affect the numerical hysteretic behaviour and consequently the hysteretic damping, thus leading to miss the displacement demand and the residual drifts.

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