CONSIDERATIONS ON THE DESIGN, ANALYSIS AND PERFORMANCES OF ECCENTRICALLY BRACED COMPOSITE FRAMES UNDER SEISMIC ACTION

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Abstract. *Eccentrically braced frame (EBF) is a typical configuration for steel structures in seismic areas. In these structures, plastic deformations are localized in specific portions (so-called "seismic links") of the beams or of the bracing system. Cyclic deformations in these links are due to bending or shear force according to the length of the link. Eccentric configurations using seismic links can also be used for composite structures in which beams are realized with a steel profile connected to a collaborating connected concrete slab. This paper investigates different aspects of the design and performance of eccentrically braced composite frames.*

1 INTRODUCTION

Eccentrically braced frame (EBF) is a typical configuration for steel structures in seismic areas where it has proved its high efficiency (Ref. 3). EBFs combine the advantages of braced structure (i.e. significant lateral stiffness reducing therefore the sensitivity to second order effects and the damages to non structural elements) with high energy dissipation capability. A typical EBF consists of a beam, one or two braces and columns. Its configuration is similar to traditional braced frames with the exception that at least one end of each brace must be eccentrically connected to the frame. The eccentric connection introduces bending and shear forces in beam elements adjacent to the brace. The short segment of the frame where these forces are concentrated is the seismic link. Figure 1 shows four possible eccentric configurations suggested by Eurocode 8 (Ref. 1). Eccentric configurations using seismic links can also be used for composite structures in which beams are realized with a steel profile connected to a collaborating connected concrete slab.

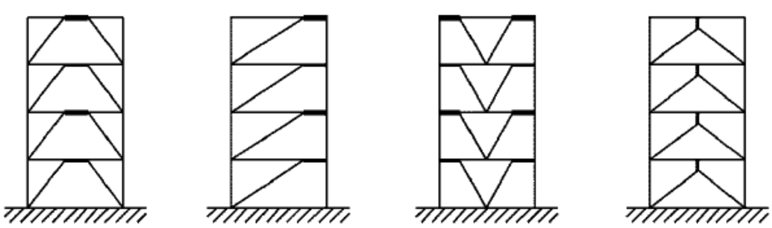


Figure 1: Typical EBF configurations.

The considerations and results summarized in the present paper are elaborated in the context of the European Research program OPUS funded by RFCS (Ref. 2) having the full title "Optimizing the seismic performance of steel and steel-concrete structures by standardizing material quality control". To make it short, the final objective of this research program is to assess the consequences of the variability of material properties on the efficiency of the capacity design of steel and steel-concrete composite structures. In this context, each partner of the research focuses on one specific structural type. University of Liège is dealing with composite braced frames (concentric and eccentric).

2 DESIGN OF CASE-STUDIES

2.1 Global geometry

In such a way to be able to assess influence of material variability, a set of case-studies has been prepared. More precisely, 2 buildings have been designed assuming that the seismic resistance is ensured in one direction by classical concentric bracings (X-bracings) and in the other direction by eccentric bracings. In the following, the focus of the paper is put exclusively on eccentric bracings. For more information, it is possible to refer to Ref. 2. Figure 2 shows the global geometry of the building (similar for both case-studies). Beams are designed as composite assuming a perfect collaboration of the concrete slab with the steel beams. Table 1 summarizes the main design assumptions considered for the two case studies: the first one corresponds to a situation of rather high seismicity, while the second corresponds to a situation of moderate seismicity. Two specific design choices are briefly commented in the following sections.

Table 1: Design conditions.

	Case-study 1	Case-study 2
Steel (profiles)	S355	S275
Concrete (slabs)	C30/37	C25/30
PGA	0.25g	0.1g
Spectrum type (Eurocode 8)	Type I	Type II
Soil type	B	C
q-factor	4	2

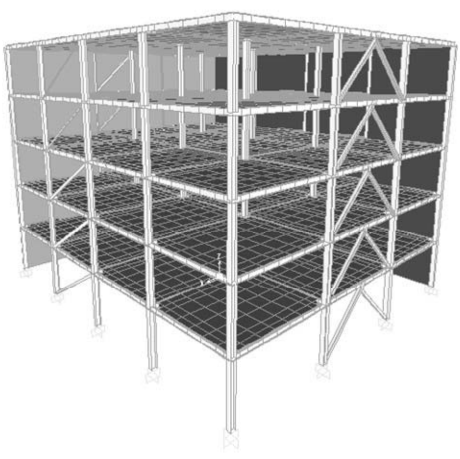


Figure 2: Global structural model.

2.2 Location of the seismic link

The first choice to be made when designing eccentrically braced frames is the location of seismic links, or in other words the appropriate choice between the four configurations proposed by Figure 1.

Classical solutions are configurations 1 to 3 with seismic links in the beam. However it must be noticed that in these situations, braces must be capacity-designed, which means that normal forces in the braces are conditioned by the resistance of the beam in order to guarantee that plastic hinges develop in the expected seismic links. In the case of composite structures using a configuration with seismic links in the beam, composite resistance of the beam is in general such that capacity design rules can lead to very important cross-sections of braces and that more economical solutions can be developed by using the fourth configuration, namely with vertical seismic link. This one has the advantages to reduce overdesign of braces and to guarantee without difficulties that beams (composite) are capacity-designed with respect to vertical link (pure steel). In case-studies developed in this paper, it has been chosen to use short vertical links in which energy is dissipated by shear yielding of the link's web. This solution is very flexible in that sense that vertical links are exclusively designed to withstand seismic forces and can thus be easily optimized, which is not so obvious in the case of seismic links in beams that are also conditioned by gravity load and where optimization capacities are reduced.

2.3 Beam-column connections

Another important aspect of the design is the identification of the primary seismic structure, i.e. the part of the structure that withstands seismic forces. When using eccentric bracings (and more generally when using whatever kind of bracings), it is expected that the primary structure is the bracing system. In the case of composite structures, this is only possible if concrete slabs are disconnected around the columns. If not so, stiffness of the composite frame is such that the structure is actually behaving as a combined moment resisting frame/braced frame. This results in the fact that, in the present case-studies, composite effect is exclusively considered in beams under gravity loads.

2.4 Final design

Final design is summarized in table 2, where profiles used for the bracing system are listed. Additional information is also given in table 3:

- θ factor (see definition in Eurocode 8) measuring the sensitivity to second order effects.

- overstrength of the seismic links (i.e. ratio between the actual resistance of the link and internal forces obtained from spectral analysis).

Table 2: Cross-sections of the bracing system

		Case-study 1	Case-study 2
Seismic link	1st storey	HE 450 B	HE 300 B
	2nd storey	HE 450 B	HE 300 B
	3rd storey	HE 400 B	HE 240 B
	4th storey	HE 340 B	HE 200 B
	5th storey	HE 280 B	HE 140 B
Diagonal braces	1st storey	HE 240 B	UPE 220
	2nd storey	HE 240 B	UPE 220
	3rd storey	HE 240 B	UPE180
	4th storey	HE 240 B	UPE 140
	5th storey	HE 240 B	UPE 80

Some comments can be made:

- Values of the θ factor are very low, evidencing the high lateral stiffness of the frames;

- Over-strengths are reasonably moderate, even in the case of moderate seismic action (case-study 2);

- Homogeneity requirement of Eurocode 8 is easily fulfilled (variation of the over-strength all over the building lower than 25%). This requirement is much easier to fulfill in eccentric bracings than in concentric bracings where additional requirements on upper and lower limit in brace's slenderness can often be in contradiction with homogeneity condition.

Table 3: θ-factors – Overstrength factors.

		Case-study 1	Case-study 2
θ factor	1st storey	0.05	0.06
	2nd storey	0.06	0.07
	3rd storey	0.05	0.06
	4th storey	0.05	0.05
	5th storey	0.04	0.04
Over-strength	1st storey	1.87	2.06
	2nd storey	1.80	2.01
	3rd storey	1.84	1.97
	4th storey	1.75	2.06
	5th storey	2.03	2.43

3 MODELLING

The second part of the study is the assessment of the seismic performances of the designed structures considering nominal values of the material properties. This is made using appropriate numerical simulations.

However, the choice of using short seismic links in which yielding is mainly associated with shear behavior implies that classical beam elements cannot be used directly. Indeed these elements are able to model correctly the bending behavior but they need to be adapted to account properly for shear yielding. The solution used here is presented in figure 3. It consists in adding a horizontal non linear spring at the junction of the beam and of the link. Properties of this spring are calibrated versus a non linear simulation of the link using shell finite elements.

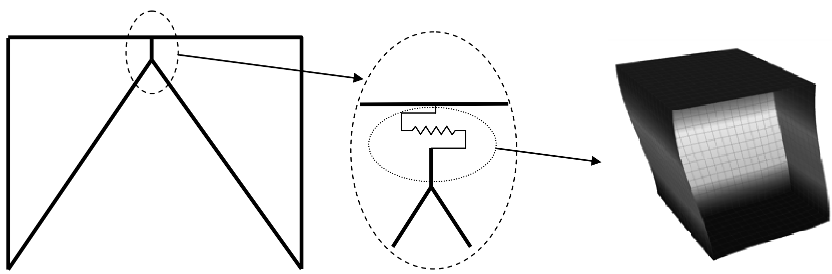


Figure 3: Numerical model.

4 NUMERICAL ASSESSMENT

4.1 Pushover analysis

The first assessment is made by using classical pushover analyses. A typical curve obtained for case-study 1 is given in figure 4. The slight difference between the load level corresponding to first yielding and the maximum of the pushover curve translates the good homogeneity of the overstrength ratio in the seismic links of the 5 storeys leading to a quasi-simultaneous yielding of the 5 links.

4.2 Incremental dynamic analysis

A more refined assessment is then made using incremental dynamic analysis. To this purpose, 7 accelerograms compatible with the reference elastic spectrum are generated. The structures are then analyzed using dynamic time-history analysis with increasing multiplier applied to the accelerograms (from 50% of the design PGA to 15 times the design PGA). Maximum values of specific characteristics are recorded for each acceleration level. Figure 5-a shows for example the evolution of the maximum top displacement for increasing value of the PGA for the first case-study. Figure 5-b compares then the couples "maximum top displacement/maximum base shear" obtained from dynamic analyses (plain lines) with similar curve obtained from static pushover (dash line).

Some comments can be drawn from there:

- Pushover curve can be seen as a reasonable lower bound of dynamic incremental curves;
- Curves from figure 5-a are typical of what can be used for Ballio-Setti assessment of q-factors. However due to the very low sensitivity of eccentrically braced structures, it can be seen that dynamic instability still don't occur even for a PGA equal to 15 times the design PGA.
- Significant yielding characterized by a strong bend in "base shear/top displacement" curves occurs at a PGA level of about 2 times the PGA. This is in good agreement with the overstrength values computed on the design examples and listed in table 3.

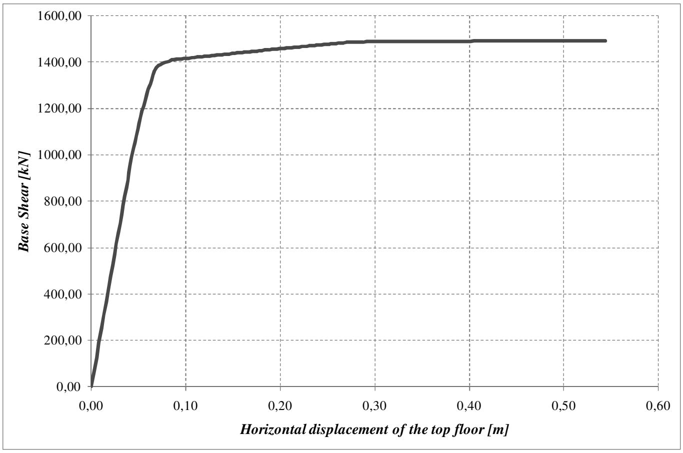
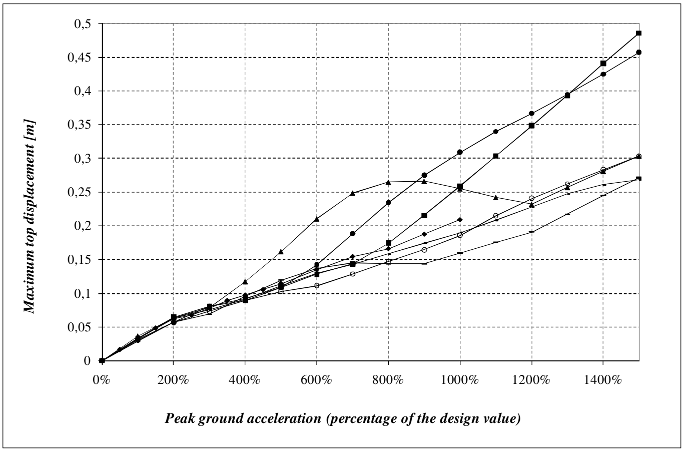


Figure 4: Pushover curve.



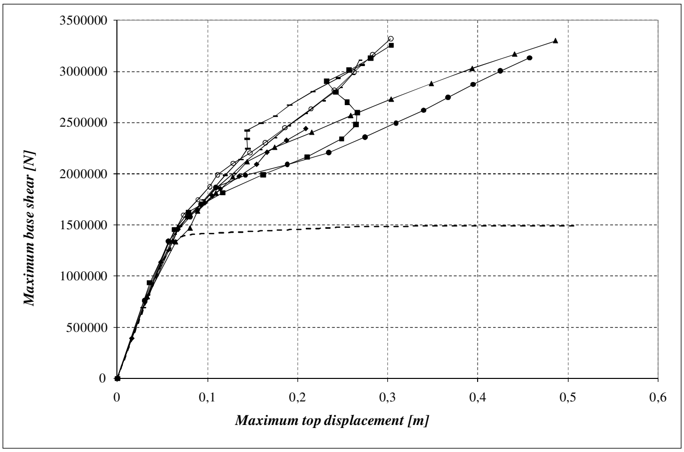


Figure 5: Incremental Dynamic Analyses (IDA) curves.

Additional interesting information can also be drawn from these results regarding actual values of the behavior factor q. To this purpose, it is assumed that collapse of the structure is obtained when rotation θ of the seismic link exceeds a given value. In the present study, values recommended by FEMA 356 (Ref. 4) are considered. For short links, governed by shear, 2 limit states are defined:

- Life safety: $\theta > 0.11$ rad
- Collapse prevention: $\theta > 0.14$ rad

On this base, it is possible for each dynamic incremental curve to define the multiplier of the reference PGA that leads to overcoming the rotation limit. An estimate of the behavior factor is then given by this particular value of the load multiplier. Values obtained for case-study 1 are given in table 4. In this table, no value corresponds to situation in which the limit value of the rotation is still not obtained even with a load multiplier of 15.

It can be seen that, in average, the estimated actual behavior factor is higher than the one considered for the design (q = 4, see table 1) whatever the safety level chosen.

Table 4: Estimated q-factor (case-study 1).

	Life Safety	Collapse prevention
Acc. 1	5.4	6.9
Acc. 2	3.5	3.8
Acc. 3	6.2	6.9
Acc. 4	4.6	4.6
Acc. 5	6.2	-
Acc. 6	-	-
Acc. 7	5.4	6.9
Average	**5.2**	**5.8**

5 STATISTICAL ASSESSMENT

The final aim of the research work summarized in this paper is to assess the influence of the variability of the material properties. To this purpose, the following methodology is defined:

- On the base of statistical data provided by steel suppliers, generation of sets of material properties that follow these statistical curves;
- Execution of dynamic analysis for each material sample. For each of the 7 reference accelerograms, the load multiplier corresponds to the FEMA 'collapse prevention' limit state obtained in section 4 using nominal material properties;
- For each analysis, record of the interesting parameters defining the structural behavior (link rotation, normal forces in braces, interstorey drifts, top displacement…).
- Statistical treatment of these results.

Examples of statistical distributions obtained for case-study 1 and for 100 material samples are given in figure 6 for the link rotation and for the normal force in braces. Even if these results are not fully statistically reliable, some preliminary conclusions can be drawn:

- Rotation demand obtained with actual material properties is always smaller than estimated on the base of nominal properties.
- Demand in terms of internal forces in braces is much lower than obtained by application of the capacity design rules of Eurocode (in this particular case, the design normal force is equal to 1612 kN).

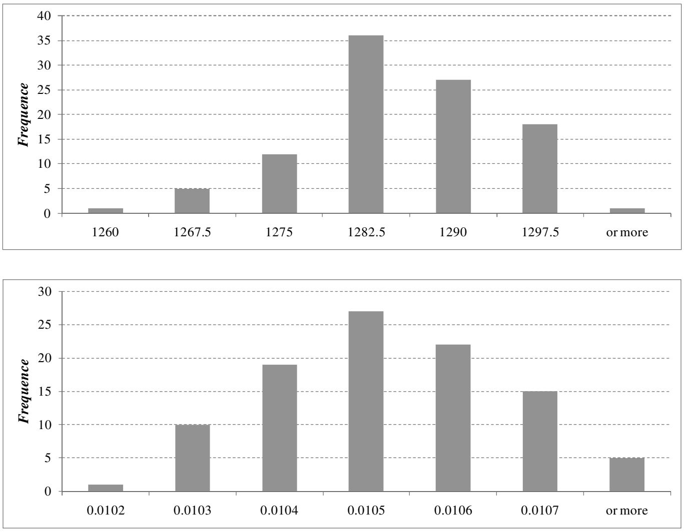


Figure 6: Example of statistical results – Normal forces in braces at 1st storey [kN] (top) – Rotation demand in seismic link at 1st storey [rad] (bottom).

6 PERSPECTIVES

Further developments of this research are still in progress. In particular, the number of material samples is enlarged in order to make the results more reliable. Similar work is also performed on concentric bracings.

The main outcomes will deal with assessment and possible recalibration of overstrength coefficients proposed by the codes in the capacity design procedures, in order to make them fit better with actual statistical distribution of steel properties.

7 ACKNOWLEDGEMENTS

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