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Development of Plastic Hinges in Steel and Composite Beams of Eccentrically Braced Frames

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Summary

The frame structures with eccentrically braced frames (EBF) are used world-wide and represent the alternative to the concentrically braced frames (CBF). The dissipative elements of eccentrically braced frames are characterized by the forming of plastic hinges, situated at the extremities of frame elements, preferably in the beams, and only at limit states in columns. The strength and ductility of EBF is directly related to the strength and ductility of the links. The seismic energy is dissipated by means of elasto-plastic shear cycles (for the short link), bending cycles (for the long link) and shear and bending cycles (for the intermediate link).

This paper presents numerical studies with the objective to investigate the behaviour of 2D steel and composite eccentrically braced frames under seismic loading with active links which yield in bending,

KEYWORDS: plastic hinges, dissipative zones, composite sections, numerical analysis, eccentrically braced frames

1. INTRODUCTION

Steel and composite eccentrically braced frames (EBFs) are a very efficient type of structures for structural seismic response. The EBFs combine the advantages of both MRF (moment resistant frames) and CBF (concentrically braced frames), the ductility as the characteristic of moment resisting frames with the lateral stiffness associated with concentrically braced frames. [1]

In case of EBF's with composite beams, the current practice recommends a complete detachment between steel beam and concrete slab in the dissipative zones. The lack of connection between the concrete slab and the steel beam in dissipative areas does not lead to a behaviour similar with a pure steel beam. The presence of the reinforced concrete slab has an important influence on development of plastic hinges in beams. [2] For a comparative analysis between the behaviour of eccentrically braced frames with steel sections and the ones with composite sections (without connection in dissipative zones), when are subjected to seismic loads, a series of nonlinear static analyses (pushover analyses) were carried out for both types of structures. The design has been performed according to N2 method

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provided by SR EN 1998. The structures were modelled as 2D frames with 6 stories. The software SAP2000 was used to create the numerical model.

2. DESCRIPTION OF PUSHOVER AND N2 METHOD

The nonlinear static analysis (pushover) is a method used to evaluate the post-elastic capacity of structures. This type of analysis assumes incrementally imposed displacements up to the development of plastic hinges. As the displacements continue to increase, plastic hinges will be progressively developed until a failure mechanism will be reached (global or local). The recording of horizontal forces and top displacement of structure generate a graphic called capacity curve of structure or pushover curve. This curve is not associated with any earthquake, being a characteristic of structure. The capacity curve highlights different characteristic bending moments from post-elastic behaviour of structure and provides information about the resistance and ductility of structure. [3]

The N2 method implies determination of target displacement using a nonlinear static analysis. The procedure involves obtaining the base shear force – displacement capacity curve, with the characteristic points marked representing the requirements for displacement according to limit states. The requirements are determined by displacement spectra for inelastic seismic response. General safety condition is: demands $\leq$ capacity. [4]

3. MODELING PARAMETERS FOR THE STRUCTURES

The analyzed structure was located in Bucharest, characterized by the following features:

- Ground acceleration a_g=0,30g;
- Control period of design spectrum: Tc= 1,6s;
- Design snow load: $S_{0,k}$=2,00kN/m^2
- Design live load: Q=3,0kN/m^2.
- Design dead load: G=4,5kN/m^2.

The structural behaviour factor q = 6,0, which represents the energy dissipation capacity of structure is taken from Tab. 6.3 (P100-2013) for a high seismicity area (H): q = 5αu/α1, where αu/α1 =1,2 for EBF.

The reinforced concrete slab (C25/30, Ø12/15 - PC52) was modelled with steel multi-layered non-linear type of elements, which can be modelled the concrete in the slab, but also the reinforcement. For both materials a nonlinear behaviour has been defined.

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Finer mesh was used in the potential plastic zones, where development of plastic hinges is expected. The type of mesh chosen for slab influences the result regarding the needed forces for development of plastic hinges. Different ways of meshing were studied until the optimum one, both in terms of results and analysis time.

The connection between the concrete slab and steel beam was modeled using link elements – available in software SAP2000. It has been chosen a version with fixed links, where the connection of concrete slab and steel beam is in centroid.

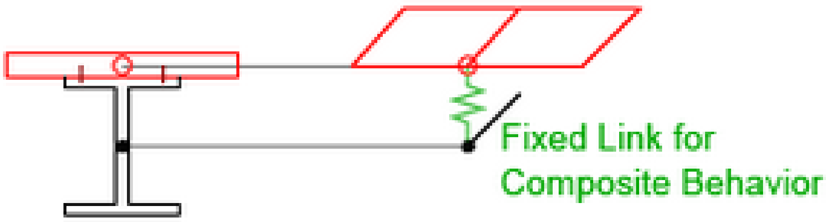

Fig. 1 Modelling of the composite beam – SAP2000 [3]

A distance of 30 cm between links was chosen taking into consideration different ways for layout of the links. A lower distance would increase the time for analysis, without difference in results.

Initial design was made with modal spectrum analysis (P100-2013), after which nonlinear static analyses – pushovers were performed, in order to evaluate the development of plastic hinges in dissipative zones and the capacity curve.

Two type EBF structures were analyzed: steel structure (S_4) and composite structure (C_4).

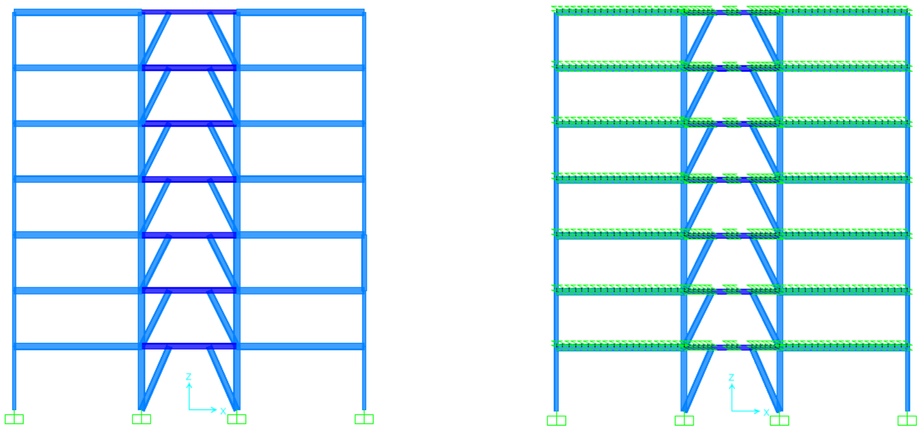

Fig. 2 EBF_S_6 (left) and EBF_C_6 (right)

Table 1 Description of the structures P+6E

Type	Central columns	Marginal columns	Braces	Beams	EBF beam	Type of EBF beam
EBF_S_6	HEB400	HEB240	HEB340 HEB280	IPE400	HEB360 HEB340 HEB280	Steel
EBF_C_4	HEB400	HEB240	HEB340 HEB280	IPE400	HEB320 HEB280	composite

For the steel structure – EBF_S_6, the beams of the frame are as follows: HEB360 for the first three levels, HEB340 for levels 4, 5, 6, and HEB280 respectively for the last level.

For EBF_C_6, after performing the push-over analysis using the same sections as for steel structure EBF_S_6, in which was added the contribution of concrete slab, yield mechanism is formed in columns and braces at first level of the central frame. For this reason, the assure the formation of plastic hinges in the dissipative zones (links) it was necessary a reduction of composite section to HEB320 (exception - beam from the last level, which remain HEB280).

After performing the push-over analysis, the base shear force – displacement capacity curve (global force – top displacement) was obtained. From capacity curves the target displacements have been determined for both structures.

Next the results obtained in pushover analysis, the results for N2 method and the local and global checking for this method are presented.

The target displacements for both types of structures (with steel beam and composite beam) were determined for the following limit states:

- Serviceability limit state (SLS);
- Ultimate limit state (ULS);
- Collapse prevention limit state (CPLS).

4. RESULTS OF THE PUSHOVER ANALYSIS

The obtained target displacement values for structures with steel beam are: 12,9 cm for SLS, 28,5 cm for ULS, respectively 46,0 cm for CPLS.

For the frame with composite beams are: 7,99 cm for SLS, 18,3 cm for ULS, respectively 32,1 cm for CPLS.

The obtained values are presented in the following graphics:

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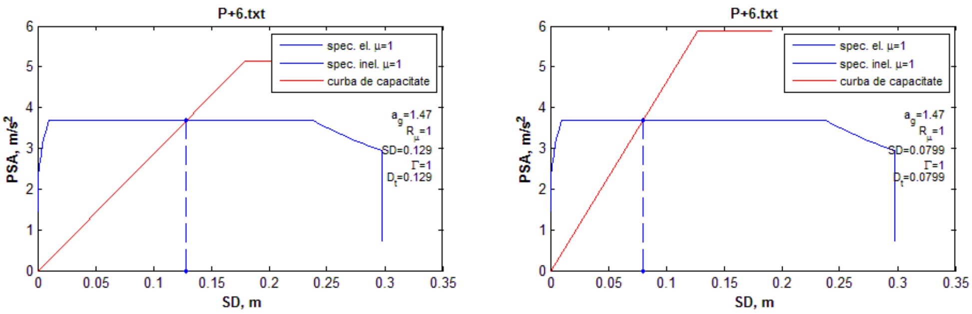

Fig. 3Target displacements SLS – EBF_S_6 left, EBF_C_6 right

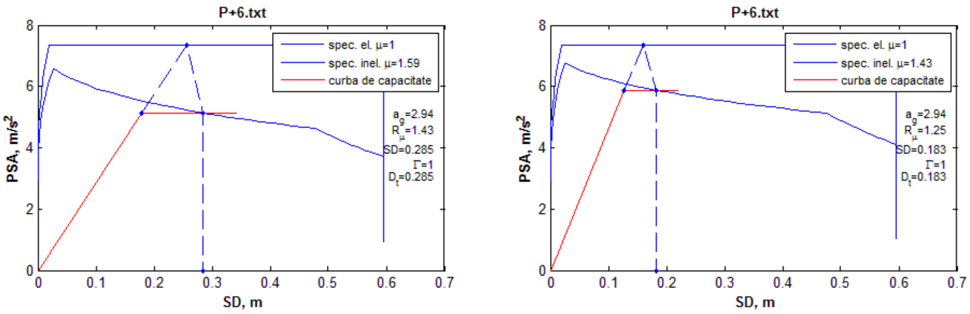

Fig. 4 Target displacements ULS – EBF_S_6 left, EBF_C_6 right

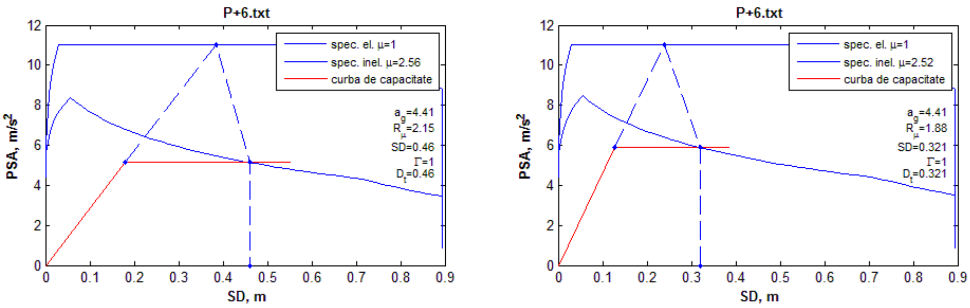

Fig. 5 Target displacements CPLS – EBF_S_6 left, EBF_C_6 right

From the graphics presented above it can be observed that the requirements for target displacements are higher for steel frames than for composite frames (12.9 cm vs, 7.99 cm for SLS, 28.5 cm vs. 18.3 cm for ULS, respectively 46 cm vs. 32.1 cm for CPLS)

The curves obtained from numerical analysis are shown in the next figure.

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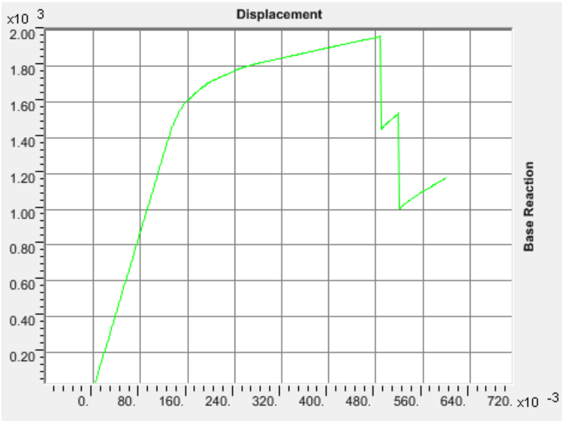
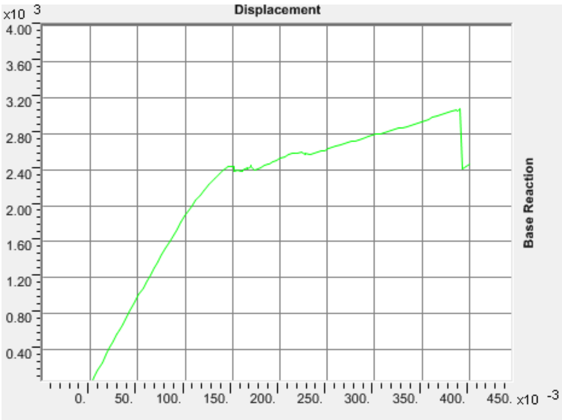


(a)



(b)

Fig. 6 Capacity curves for EBF_S_6 (a) and EBF_C_6 (b)

From these two force-displacement curves it can be observed that the contribution of concrete slab in case of EBF_C_6 is significant even if the dissipative zones are not provided with shear connectors. The maximum value of the force exceeds the value of 2900 kN for a 38 cm displacement compared with the steel frames, where the maximum value for the force is 2000 kN and the ultimate displacement is approx. 48 cm.

The following table presents the development of plastic hinges in the link .It was taken into consideration the following characteristic moments: development of first plastic hinge, the displacement for ultimate limit state (ULS) and the formation of failure mechanism.

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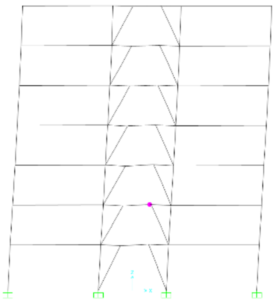
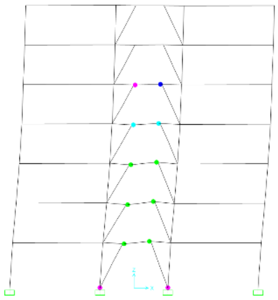
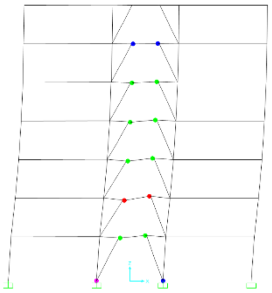
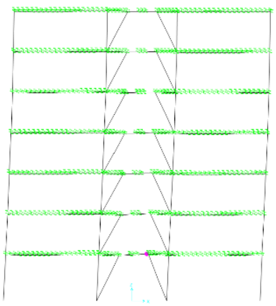
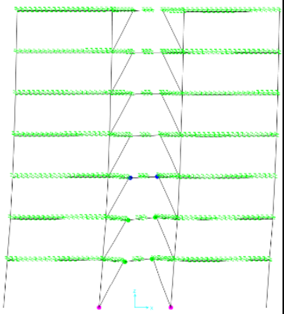
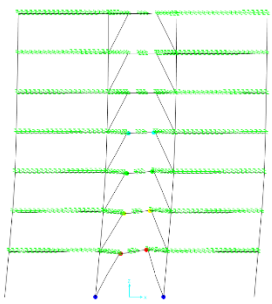
Table 2 Development of plastic hinges

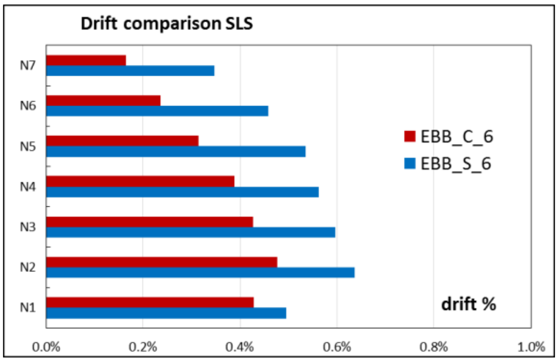
	Development of first PH	ULS	Collapse
EBF_S_6	 D_E=12.9 cm= D_{TSLS}	 D_{ULS}=28.5 cm	 D_U=48.9 cm> D_{TCPLS}
EBF_C_6	 D_E=9.1 cm > D_{TSLS}	 D_{ULS}=18.3 cm	 D_U=39.2 cm> D_{TCPLS}

The requirements for target displacement, calculated by using N2 method for all 3 limit states are reached for both types of structures (steel and composite).

In case of EBF_S_6 the first plastic hinge is developed approximately in the same time as achieving the requirements for serviceability limit state and plastic hinges are developed in links from the first five levels. For the requirements specific to ultimate state it can be observed that failure mechanism is formed in the link from the second level, by failure of both plastic hinges. The collapse mechanism occurs by failure of dissipative zone from beam of second level.

In case of composite frame EBF_C_6, the first plastic hinge is formed after reaching the requirements of SLS. For the requirements of ultimate limit state, the plastic hinges are developed in links of the first 3 levels. The collapse mechanism occurs by failure of dissipative zone from beam of the first level.

The obtained values for inter-story drift are presented in the following graphics:



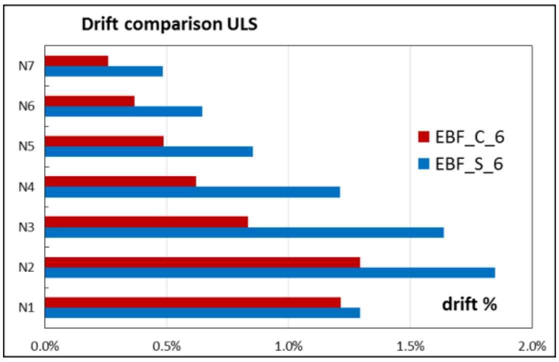
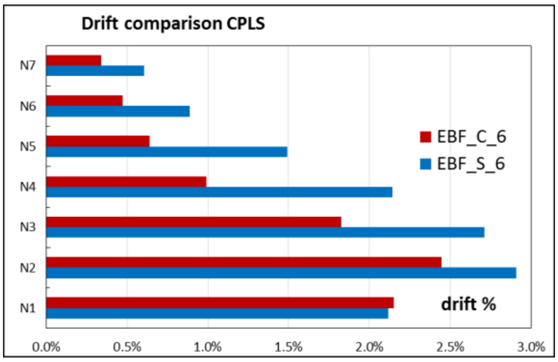


Fig. 7 Inter-story drift for EBF_S_6 and EBF_C_6 at SLS and ULS



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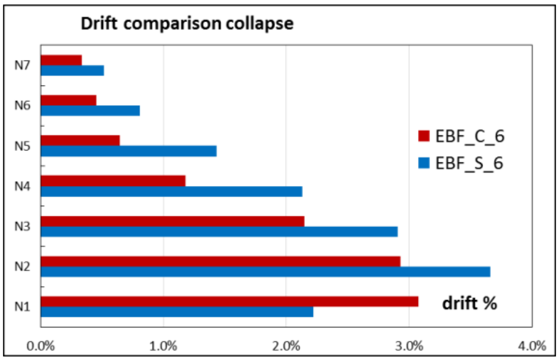

Fig. 8 Inter-story drift for EBF_S_6 and EBF_C_6 at CPLS and collapse

The values for the drift are within the values specified in the design norms in all the three limit states. There were taken into account the values from P100-2013, which provides a maximum value for the drift of 0.75% for SLS, 2.5% for ULS and 5% for CPLS provided by the FEMA356 (in P100-2013 does not exist requirements for CPLS).

The maximum value for the inter-story drift is reached to the second level for all the three limit states. (SLS, ULS, CPLS)

5. CONCLUSIONS.

The difference in behaviour between two types of eccentrically braced frames, steel and composite, has been studied. The contribution of the concrete slab is significant in regards to the global behaviour of structure, even in the situation when shear connectors are not present in the dissipative zones.

From the push-over analyses it can be observed that the requirements for target displacements are higher for steel frames than for composite frames. The energy dissipation is done through the development of plastic hinges for both types of structures.

The push-over analyses showed that in case of composite beams to lower values of the inter-story drift, plastic hinges are formed later, which leads to a stiffer structure. The values for inter-story drift are within the limit values provided by norms for both types of structures in all the three limit states.

In case of eccentrically braced frames with links that yield in bending, it will be recommend to use the composite beam solution because leads to a better seismic response than the pure steel beams.

Bibliography

1. [1] S.-H. Chao and S. C. Goel, "*Performance-based seismic design of EBF using target drift and yield mechanism as performance criteria*," Department of Civil and Environmental Engineering, The University of Michigan, 2005
2. [2] A. Ciutina, D. Dubina and G. Danku, "*Influence of steel-concrete interaction in dissipative zones of frames: I - Experimental study*" Steel and Composite Structures, Vol. 15, No. 3, 2013.
3. [3] S. v. CSI Berkeley, FAQ.
4. [4] P. Fajfar, ""*A nonlinear analysis method for performance-based seismic design*"," Earthquake Spectra, 16(3):573-92, 2000.
5. [5] SR EN1998-1:2004, *Design of structures for earthquake resistance - Part 1: General rules, seismic actions and rules for buildings.*
6. [6] SR EN1994-1-1, "*Design of Composite Steel and Concrete Structures. Part 1.1. General Rules and Rules for Buildings.* Brussels: CEN, European Committee for Standardisation".
7. [7] FEMA 356, "*Prestandard And Commentary For The Seismic Rehabilitation Of Buildings*" November, 2000.
8. [8] P100-1-2013, "*Cod de proiectare seismica*".