# Pinned and Fixed Connections in "2 Storey X-Braced" Frames

Marina Stoian, Helmuth Köber[✉], and Ramona Marcu

Technical University of Civil Engineering Bucharest,
Bulevardul Lacul Tei nr. 124, 020396 Bucharest, Romania
koberhelmuth@yahoo.de

**Abstract.** The main objective of the present paper is to point out the influence of fixed and pinned connections, for diagonals and beams, on the seismic behaviour of concentrically braced frames. Three constructive configurations were considered for two concentrically "2 storey X-braced" frames with six and ten storey, having two spans of 6.0 m and a storey height of 3.5 m.
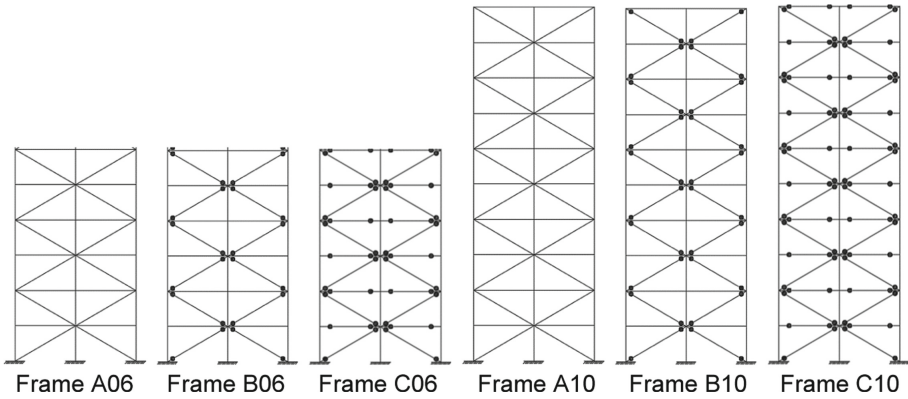
In the first constructive configuration, fixed connections were used for all kind of structural elements. In the second configuration, pinned connections were considered at the ends of the diagonals and fixed connections were used for the beams. In the third configuration, pinned connections were considered for both diagonals and beams.

Each distinct constructive configuration was subjected to dynamic nonlinear analyses using Vrancea acceleration records, calibrated to a peak ground acceleration value of about 0.3 times the acceleration of gravity. The behaviour of the different frame configurations during these analyses was compared (the extreme values for base shear forces and horizontal floor displacements, the amount of dissipated energy, the maximum inelastic deformations and member forces recorded during the analyses). The steel consumption was estimated for all analysed constructive variants.

**Keywords:** Dissipated energy · Plastic hinge rotations · Braces elongations · Plastic failure mechanism · Estimated steel consumption

## 1 Introduction

The present paper intends to illustrate the influence of hinged and fixed connections for diagonals and beams on the seismic behaviour of 2X-braced concentrically braced frames. Three constructive configurations (named A, B and C) were considered for two concentrically "2 storey X-braced" frames with six and ten storeys, having two spans of 6.0m and a storey height of 3.5 m (see Fig. 1). In case of configuration A, fixed connections were considered for all kind of structural elements. For configuration B, pinned connections were used at the ends of the diagonals and fixed connections were used for the beams. In case of configuration C, pinned connections were considered for both diagonals and beams.

Frame A06    Frame B06    Frame C06    Frame A10    Frame B10    Frame C10

**Fig. 1.** Analysed constructive configurations of the concentrically braced frames.

The seismic design load was evaluated according to the provisions of the in charge Romanian seismic design code P100-1/2013 [1] and EN1998-1:2004 [2]. All kind of structural elements (diagonals, beams, central and lateral columns) had built-up I-shaped cross-sections. A behaviour factor q = 4.0 was taken into consideration for the seismic design of all considered concentrically braced frames [1, 2]. The strength and general stability checks were performed according to EN1993-1-1:2005 [3].

The webs of the braces cross-sections were rotated normally to the plane of the frame, in order to avoid out-of-plane buckling of the diagonals.

The following alternative seismic design procedures were used to dimension the different structural elements of the concentrically braced frames [4, 5]:

In case of configurations A and B (frames A06, A10, respectively B06, B10):

– At first the diagonals were sized as primary dissipative elements to the forces generated by the non-amplified seismic design load ($S_{code}$).
– Then an amplification factor ($\Omega^N$) was evaluated for the designed cross-sections of the diagonals [1, 2]: $\Omega^N = min\{\Omega^N_i\}$; $\Omega^N_i = N_{pl,Rd,i}/N_{Ed,i}$.
– Secondary dissipative elements (potentially plastic zones with reduced cross-sections, located along the frame girders) were sized for the forces produced by an amplified seismic load [4, 5]: $1.1 \cdot \gamma_{ov} \cdot \Omega^N \cdot S_{code}$ (see Fig. 2).
– For the cross-sections of these potentially plastic zones an additional amplification factor ($\Omega^M$) was evaluated: $\Omega^M = min\{\Omega^M_i\}$; $\Omega^M_i = M_{pl,N,Rd,i}/M_{Ed,i}$.
– The cross-sections of the columns and beam segments outside the potentially plastic zones were dimensioned for the forces produced by an even more amplified seismic load [4, 5]: $1.1 \cdot \gamma_{ov} \cdot \Omega^N \cdot 1.1 \cdot \gamma_{ov} \cdot \Omega^M \cdot S_{code}$.

In case of configuration C (frames C06, C10):

– The braces of the concentrically braced frames were sized as primary dissipative elements for the forces generated by the non-amplified seismic design force ($S_{code}$).
– Then an amplification factor ($\Omega^N$) was evaluated for the designed cross-sections of the diagonals [1, 2]: $\Omega^N = min\{\Omega^N_i\}$; $\Omega^N_i = N_{pl,Rd,i}/N_{Ed,i}$.

- The cross-sections of the beams were dimensioned for the forces produced by an amplified seismic load: $1.1 \cdot \gamma_{ov} \cdot \Omega^N \cdot S_{code}$.
- The cross-sections of the columns were dimensioned for the forces produced by an even more amplified seismic load: $1.1 \cdot \gamma_{ov} \cdot \Omega^N \cdot 1.1 \cdot \gamma_{ov} \cdot 1.05 \cdot S_{code}$ (in order to ensure an overstrength of the columns compared to the beams when subjected to seismic actions).
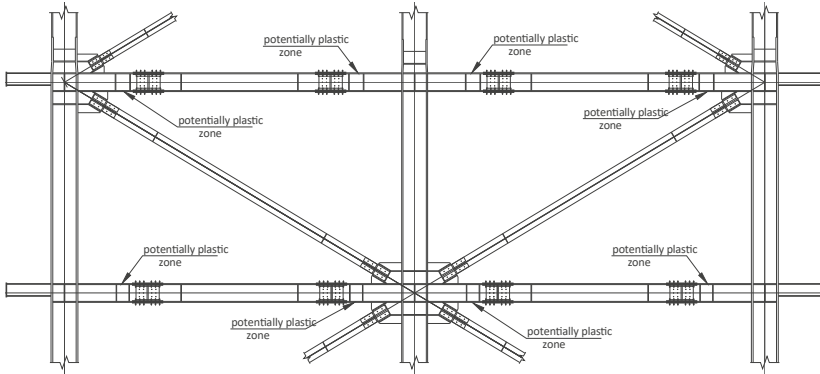


**Fig. 2.** Potentially plastic zones along the frame girders (configuration A).

All concentrically braced frames resulted after the design were subjected to dynamic nonlinear analyses [6], using the N-S components of Vrancea acceleration records from 04.03.1977, 31.08.1986 and 30.05.1990, recorded in or around Bucharest and calibrated to a peak ground acceleration value of about 0.3 times the acceleration of gravity. Rayleigh damping was taken into consideration [7]. Mass and stiffness proportional damping factors were considered for the first and third modal periods [6].

## 2 Extreme Values for Base Shear Forces and Horizontal Floor Displacements

In almost all cases during dynamic nonlinear analyses, larger base shear forces and smaller horizontal floor displacements could be noticed in case of configuration A, for both groups of frames (see Fig. 3).
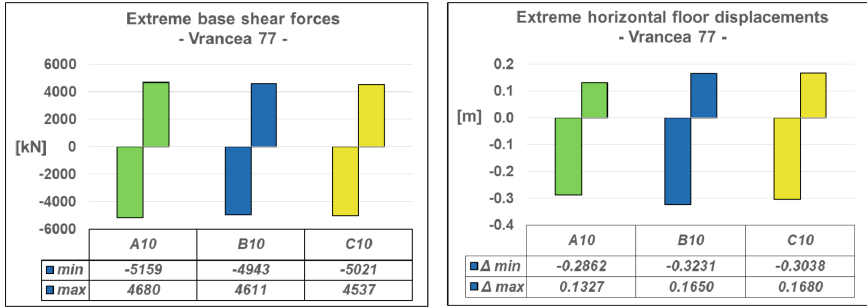
**Fig. 3.** Extreme values for base shear forces and horizontal floor displacements.

## 3    Maximum Plastic Deformations in Diagonals and Beams

In case of the six storey frames, the largest plastic brace elongations and the largest plastic hinge rotations in the girders could be noticed during the dynamic nonlinear analyses with the Vrancea 86 or Vrancea 90 acceleration record. For the ten storey frames, the largest plastic deformations in the diagonals and girders, were recorded during the dynamic nonlinear analyses with the Vrancea 77 acceleration record.
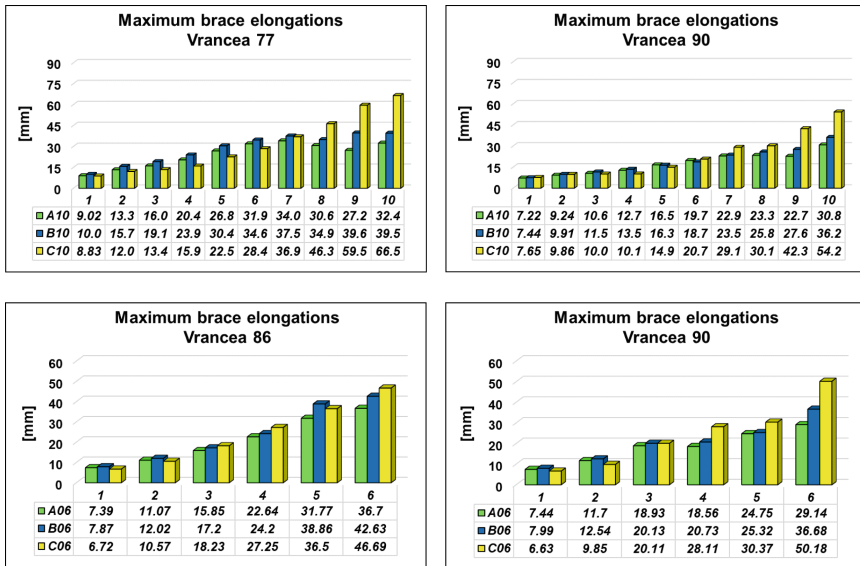


**Fig. 4.** Maximum brace elongations during dynamic nonlinear analyses.

In most cases, the highest brace elongation values could be noticed at the upper storey of the frames C06 and C10 (see Fig. 4), due to local dynamic amplifications at the last storey (due to the whip effect). At the lower storey, the largest plastic

deformations along braces were generally noticed in case of frame B06 and respectively frame B10.

On average, the differences of the maximum plastic deformations recorded in the diagonals of the frames C10 and A10 were about 20% for the Vrancea 77 acceleration record and about 22% for the Vrancea 90 acceleration record (see Fig. 4).

On average, the differences between the maximum plastic brace elongations of the C06 and A06 frames were about 13% for the Vrancea 86 acceleration record and over 9% for the Vrancea 90 acceleration record (see Fig. 4).

Larger inelastic rotations in the potentially plastic zones in the beams were usually observed for frame B06 and respectively for frame B10 (see Fig. 5), compared to those noticed for frame A06 and respectively for frame A10. On average, the differences of the maximum plastic deformations recorded in the girders were about 23% in case of the frames B10 and A10 (for the Vrancea 77 acceleration record) and about 22% in case of the frames B06 and A06 (for the Vrancea 90 acceleration record).
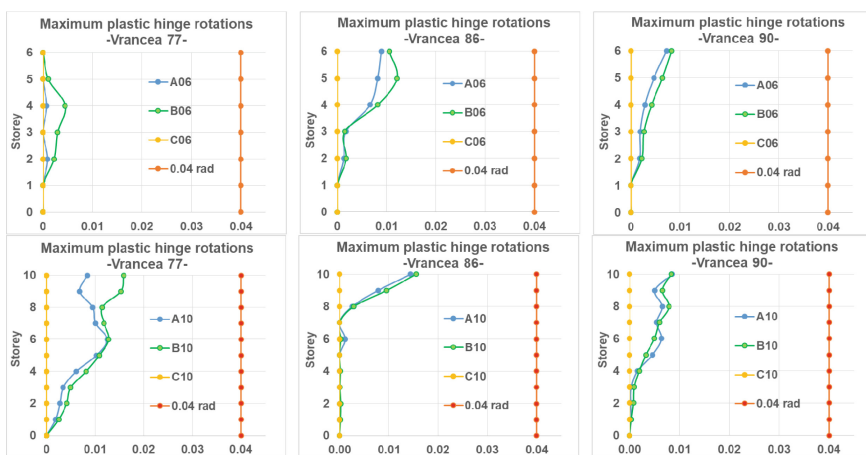


**Fig. 5.** Maximum plastic hinge rotations in the girders.

The components of the dissipated energy during dynamic nonlinear analyses with different acceleration records are indicated in Fig. 6 and Fig. 7. Zone 1 (yellow) represents the amount of energy consumed through damping. Zone 2 (black) indicates the kinetic energy. Zone 3 (green) represents the amount of energy consumed through plastic deformations in the diagonals. Zone 4 (red) represents the amount of energy dissipated through inelastic deformations in the potentially plastic zones placed in the girders. Zone 5 (blue) indicates the amount of energy consumed through plastic deformations in unwanted zones. Since no plastic deformations occurred outside the diagonals and the potentially plastic zones located in the girders and near the bottom of first storey columns, zone 5 is represented only as a blue line!
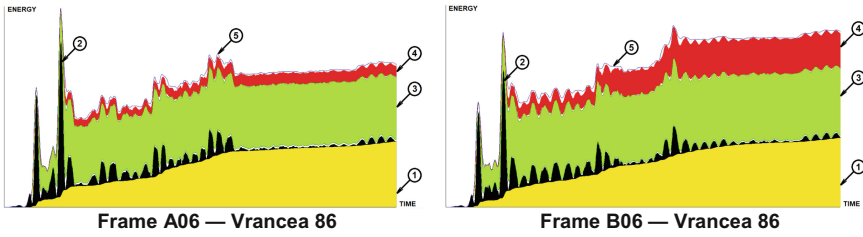
**Fig. 6.** Dissipated energy components for six storey frames.

By analysing the graphics in Fig. 6 and 7, it can be noticed that the amount of energy dissipated through plastic deformations in the braces is always larger than the one in the girders (the area of zone 3 is always larger than the one of zone 4). It can be also observed, that the amount of energy dissipated through plastic deformations in the girders of configuration B frames is larger than the one for configuration A frames (the area of zone 4 is always larger for configuration B frames).
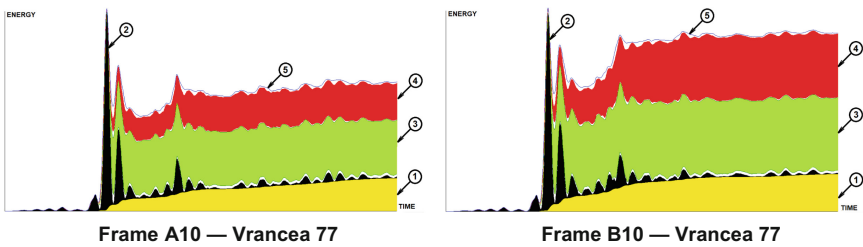


**Fig. 7.** Dissipated energy components for ten storey frames.

## 4   Maximum Member Force Values

The greatest axial force values were recorded during dynamic nonlinear analyses in the girders of frame C06 and respectively C10, while the smallest ones were noticed in the girders of frame A06 and respectively A10. On average, the differences were up to 15% between frame C06 and A06 for the Vrancea 86 acceleration record (see Fig. 8), respectively 25% between frame C10 and A10 for the Vrancea 77 acceleration record.

The pinned beam/column connections in case of configuration C lead to much smaller bending moments along the frame girders compared to configurations A and B. The maximum bending moment values along the girders were quite the same in case of configuration A and B, (on average, the differences were less than 1.6%).
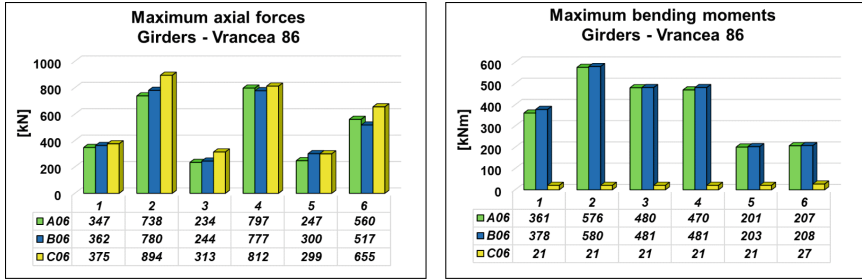
**Maximum axial forces**
**Girders - Vrancea 86**

| | 1 | 2 | 3 | 4 | 5 | 6 |
|---|---|---|---|---|---|---|
| A06 | 347 | 738 | 234 | 797 | 247 | 560 |
| B06 | 362 | 780 | 244 | 777 | 300 | 517 |
| C06 | 375 | 894 | 313 | 812 | 299 | 655 |

**Maximum bending moments**
**Girders - Vrancea 86**

| | 1 | 2 | 3 | 4 | 5 | 6 |
|---|---|---|---|---|---|---|
| A06 | 361 | 576 | 480 | 470 | 201 | 207 |
| B06 | 378 | 580 | 481 | 481 | 203 | 208 |
| C06 | 21 | 21 | 21 | 21 | 21 | 27 |

**Fig. 8.** Maximum member forces in the girders.

In most situations during dynamic nonlinear analyses, the largest axial forces in the central columns were noticed for frame C06 and respectively frame C10. On average for frame A10 and B10, the values were about 12.6% and respectively 10.7% lower, compared to frame C10, (see Fig. 9). On average for frame A06 and B06, the values were about 5.8% and respectively 19.6% smaller, compared to frame C06 (Vrancea 86). The greatest bending moments in the central columns were recorded in most cases for frame B10 and respectively for frame B06. On average for frame A10 and C10, the values were about 8.8% and respectively 44.9% lower, compared to the ones in frame B10, (see Fig. 9). On average for frame A06 and C06, the values were up to 7.5% and 51.5% smaller, compared to frame C06 (Vrancea 86).
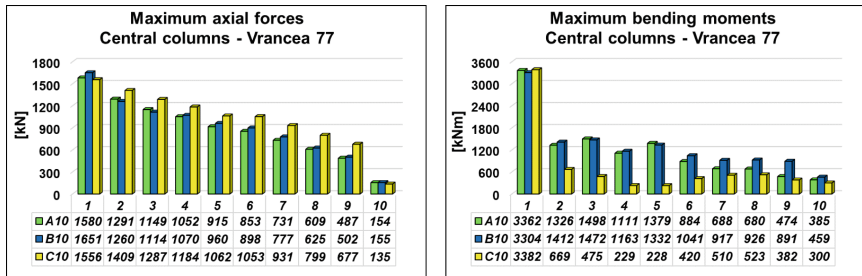


**Maximum axial forces**
**Central columns - Vrancea 77**

| | 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 | 10 |
|---|---|---|---|---|---|---|---|---|---|---|
| A10 | 1580 | 1291 | 1149 | 1052 | 915 | 853 | 731 | 609 | 487 | 154 |
| B10 | 1651 | 1260 | 1114 | 1070 | 960 | 898 | 777 | 625 | 502 | 155 |
| C10 | 1556 | 1409 | 1287 | 1184 | 1062 | 1053 | 931 | 799 | 677 | 135 |

**Maximum bending moments**
**Central columns - Vrancea 77**

| | 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 | 10 |
|---|---|---|---|---|---|---|---|---|---|---|
| A10 | 3362 | 1326 | 1498 | 1111 | 1379 | 884 | 688 | 680 | 474 | 385 |
| B10 | 3304 | 1412 | 1472 | 1163 | 1332 | 1041 | 917 | 926 | 891 | 459 |
| C10 | 3382 | 669 | 475 | 229 | 228 | 420 | 510 | 523 | 382 | 300 |

**Fig. 9.** Maximum member forces in the central columns.

During dynamic nonlinear analyses, larger axial forces were noticed in the lateral columns of frame A10 and respectively A06. For the Vrancea 77 acceleration record, on average, about 2.2% smaller values were recorded for frame B10, respectively up to 8.1% lower values were registered for frame C10, compared to the ones in frame A10. On average, for the analyses with the Vrancea 86 acceleration record, the axial forces in the lateral columns of frame A06 were about 4.1% and respectively 7.2% larger, compared to those in frame B06 and frame C06, (see Fig. 10).
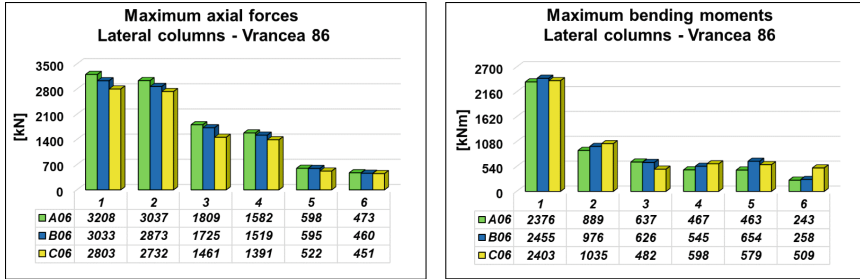
**Fig. 10.** Maximum member forces in the lateral columns.

In most cases, the smallest bending moments were observed during dynamic nonlinear analyses in the lateral columns of the frames with pinned braces girders connections. Compared to frame C10, for the analyses with the Vrancea 77 acceleration record, the maximum bending moment values in the lateral columns were on average higher with about 14.1% and respectively 17.8% in case of frame A10 and B10. On average for frame A06 and frame B06, the values were about 14.1% and respectively 17.8% larger, compared to the ones in frame B06, (see Fig. 9).

## 5   Estimated Steel Consumption

The smallest total estimated steel consumption values were obtained for configuration C, while the largest values were observed for configuration A (see Fig. 11). The largest differences between the overall estimated steel consumption values were about 6.5% between frame A06 and frame C06 and respectively up to 5.1% between frame A10 and frame C10.
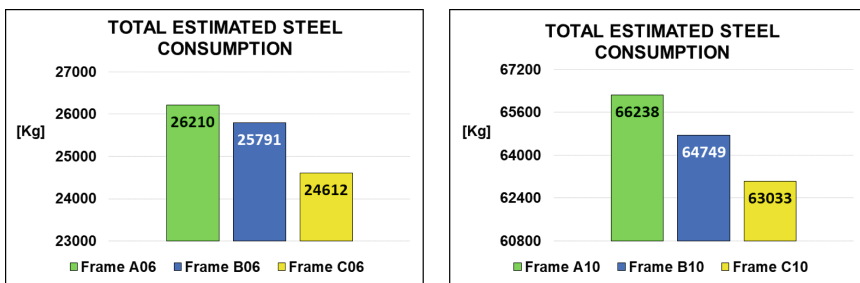


**Fig. 11.** Overall estimated steel consumption for the analysed frames.

## 6   Conclusions

In terms of estimated steel consumption, the configuration with pinned connections at the ends of the diagonals and beams (configuration C) proved to be the most advantageous. At the higher storey of these frames, however, local dynamic amplification effects (whip effects) were observed, leading to higher horizontal floor displacements and larger plastic deformations in the braces and girders.

The configuration with rigid connections for beams and diagonals (configuration A) leads to the lowest plastic deformations, during the performed dynamic nonlinear analyses, for both beams and diagonals.

The configuration with rigid connections for beams and pinned connections for braces (configuration B) leads to intermediate results between configuration A and C, in terms of maximum inelastic deformations and estimated steel consumption values.

## References

1. Ministry of Regional Development and Public Administration (2013) Seismic design code - Part 1 - design rules for buildings, indicative P100-1/2013, Bucharest, Romania
2. EN 1998-1:2004 (2004) Eurocode 8: design of structures for earthquake resistance, Part 1: general rules, seismic actions and rules for buildings
3. EN 1993-1-1: 2005 (2005) Eurocode 3: design of steel structures, Part 1-1: general rules and rules for buildings
4. Marcu R, Köber H (2021) Seismic design methods for concentrically braced frames. In: IOP conference series: earth and environmental science, vol 664, no 1. IOP Publishing, Bristol, p 012086
5. Köber H, Stefănescu B (2009) Seismic design procedures for centrically braced frames. In: Proceedings of 6th STESSA conference - behaviour of steel structures in seismic areas, CRC Press/Balkema, Taylor & Francis Group, Philadelphia, pp 665–670
6. Tsai KC, Li JW (1994) Drain2D+ a general purpose computer program for static and dynamic analyses of inelastic 2D structures. Taipei, Taiwan
7. Chopra AK (2014) Dinamic de estructuras, cuarta edicion. Pearson Education, Mexico