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About the seismic design of a "two storey-X" concentrically braced frame

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Abstract

The present paper analyses several seismic design procedures for concentrically braced frames equipped with a „2 storey-X" bracing system. The procedures were applied on a ten level structure situated in Bucharest, considering that the lateral loads are taken entirely by the braced frames. The concentrically braced frames were designed using four procedures (the one indicated in the in charge seismic design code and three other alternative procedures). The main objective was the strength hierarchisation of the structural elements in order to obtain by design a global plastic failure mechanism for the concentrically braced frames. Dynamic nonlinear analyses were performed with each designed frame configuration in order the study the sequence of the plastic joins formation, as well as the values of forces and deformations in the different structural members of the frame. Another objective was the comparison of the estimated steel consumption.

Rezumat

În prezenta lucrare au fost analizate comparativ diferite proceduri de dimensionare folosite pentru proiectarea seismică a unui cadru contravântuit centric în sistem „2X". S-a avut în vedere o structură cu zece niveluri amplasată în București, la care încărcările seismice orizontale sunt preluate integral de cadre contravântuite centric. Cadrele contravântuite centric au fost dimensionate pe rând prin patru proceduri diferite (cea indicată în normele de proiectare în vigoare și alte trei proceduri alternative de dimesnionare). Principalul obiectiv urmărit, a constat în ierarhizarea capacității de rezistență a diferitelor categorii de elemente structurale în vederea impunerii prin proiectare a unui mechanism global de cedare favorabil pentru cadrele contravântuite centric. Cadrele rezultate în urma dimensionării au fost supuse unor analize dinamic neliniare, urmărindu-se succesiunea formării articulațiilor plastice, comparându-se mărimea eforturilor și deformațiilor postelastice în diferitele tipuri de elemente structurale. Alt obiectiv comparat a constat în consumul estimat de oțel.

Keywords: global plastic failure mechanism, concentrically bracing, steel consumption, dissipated energy, dynamic nonlinear analyses

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1. Introduction

A ten storey structure located in Bucharest was considered, having two spans and six bays all of 6.0m, as illustrated in Fig. 1. The storey height was 3.5m. The concentrically "2-X" bracing system that was used for all braced frames is indicated in Fig. 1.

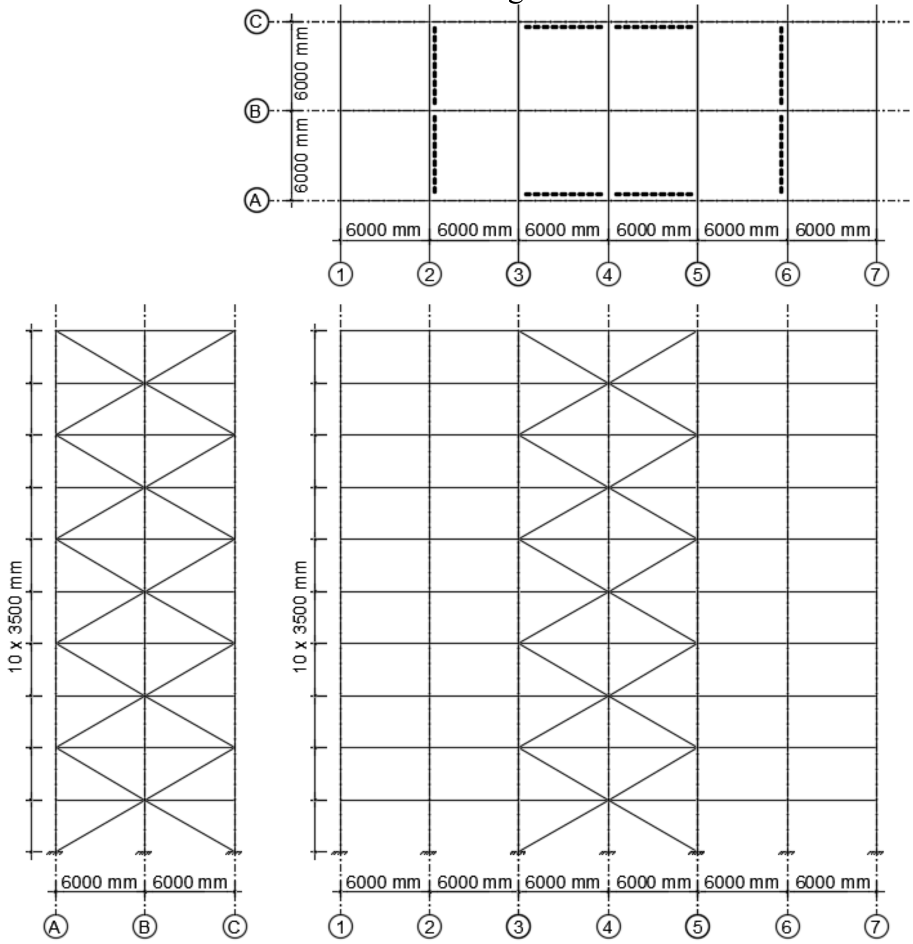


Figure 1. Plan view and elevation of the considered structure

2. Description of the considered seismic design methods

In all analyzed procedures the braces were sized for the force generated by the code seismic action S_{CODE}, evaluated according to P100-1/2013 [1]. It was considered that the whole horizontal seismic force was carried only by the braced frames. All the structural elements of the concentrically braced frames and moment resisting frames (columns, girders, braces) had built-up "I"-shaped cross-sections. In order to ensure the in-plane buckling of the braces, the cross-sections of the diagonals were rotated with the web normally to the plane of the braced frame [2]. Rigid connections were considered among all elements of the braced frame.

2.1 Procedure 1

The first considered procedure was the one indicated in the in charge seismic design codes EN 1998-1:2004 [3] and P100-1/2013 [1] for concentrically braced frames. The girders and columns were designed considering the forces generated by an increased seismic force $1.1 \cdot \gamma_{ov} \cdot \Omega_N \cdot S_{CODE} = 1.599 \cdot S_{CODE}$ compared to the one used for the design of the diagonals. The frame sized according to this procedure was named "Frame 1".

Where:
S_{CODE} = seismic base shear force used for the design;
γ_{ov} = over strength factor; γ_{ov} = 1.25 (according to EN 1988-1:2004 [3] and P100-1/2013 [1]);
Ω^N = minimum value of $\Omega_i^N = N_{pl,Rd,i} / N_{Ed,i}$, calculated for all the diagonals of the frame;
$N_{pl,Rd,i}$ = the design resistance of diagonal "i";
$N_{Ed,i}$ = the design of the axial force in the same diagonal "i" in the seismic design situation.

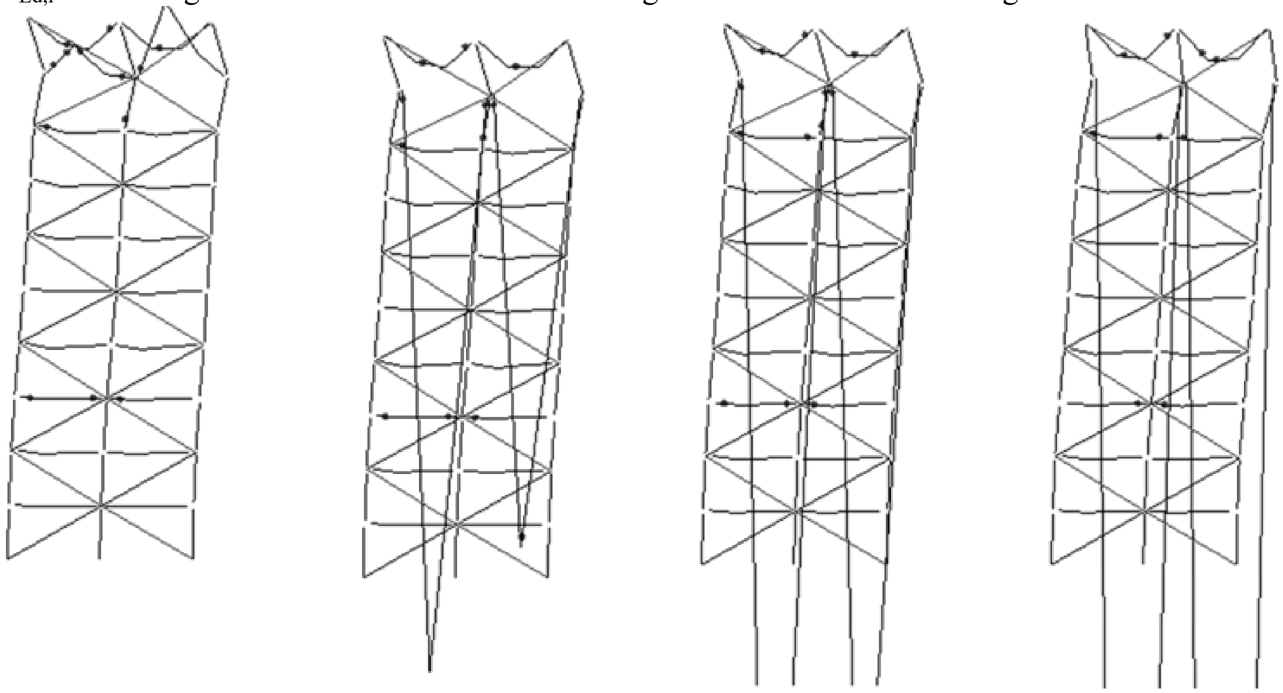


Figure 2. Failure of Frame 1 during the dynamic nonlinear analysis

"Frame 1", obtained after the design with procedure one, failed during the dynamic nonlinear analysis using Vrancea N-S 1977 acceleration record, plastic hinges appearing in an uncontrolled manner and leading to a local plastic collapse mechanism of the frame as shown in Fig. 2 [4].
To improve the behavior of the structure during the dynamic nonlinear analyses three alternative seismic design procedures were established based on the strength hierarchisation of the different structural elements of the concentrically braced frame.

2.2 Procedure 2

In the second considered procedure the tensioned diagonals were considered as primary dissipative elements and dimensioned for the forces produced by S_{CODE} according to the seismic design code EN 1998-1:2004 [2] and P100-1/2013 [1] and resulted $\Omega^N = 1.038$.
Potentially plastic zones placed in the girders were considered as secondary dissipative elements and their cross-sections were dimensioned for an increased seismic action $1.1 \cdot \gamma_{ov} \cdot \Omega_N \cdot S_{CODE} = 1.599 \cdot S_{CODE}$ and $\Omega^M = 1.026$ resulted [2].
The columns and the beam segments placed outside the potentially plastic zones were sized for an even more amplified seismic load of about $(1.1 \cdot \gamma_{ov} \cdot \Omega^N) \cdot (1.1 \cdot \gamma_{ov} \cdot \Omega^M) \cdot S_{CODE} = 2.526 \cdot S_{CODE}$. The frame sized according to this procedure was named "Frame 2" [2, 5].

2.3 Procedure 3

In case of design procedure three, the potentially plastic zones along the beams were considered as primary dissipative elements and dimensioned for S_{CODE} and for these elements $\Omega^M = 1.023$ resulted. After this the diagonals were designed for the increased seismic force $1.1 \cdot \gamma_{ov} \cdot \Omega^M \cdot S_{CODE}$

$= 1.575 \cdot S_{CODE}$ and considering that the load is taken only by the tensioned ones. In this procedure the diagonals are the secondary dissipative elements. Having the diagonals dimensioned resulted a $\Omega^N = 1.056$.

The columns and the beams placed outside the potentially plastic zones were dimensioned for the forced produces by an additional amplified seismic force equal to $(1.1 \cdot \gamma_{ov} \cdot \Omega^N) \cdot (1.1 \cdot \gamma_{ov} \cdot \Omega^M) \cdot S_{CODE} = 2.523 \cdot S_{CODE}$. This frame was named "Frame 3".

2.4 Procedure 4

In the fourth considered procedure both the diagonals and the potentially plastic zone along the girders were considered as primary dissipative elements and have been dimensioned for the seismic forces generated by S_{CODE}. In the case of diagonals resulted $\Omega^N = 1.038$ and in the case of the potentially plastic zone from the beams resulted $\Omega_M = 1.041$. The columns and the beams placed outside the potentially plastic zones were dimensioned for the forced produces by an amplified seismic load $1.1 \cdot \gamma_{ov} \cdot \min(\Omega_N, \Omega_M) \cdot S_{CODE} = 1.599 \cdot S_{CODE}$.

2.5 Modeling of the braces

The member forces used in the design for the beams and columns from procedures 2, 3 and 4 are obtained from static liner analyses for a frame having the diagonals considered with reduced axial stiffness [6]. The materials considered in the braces had reduced values for the modulus of elasticity. Compared to the value of Young modulus for steel ($E_{STEEL} = 2{,}1 \cdot 10^5$ N/mm^2), the following reduced values were considered for the brace materials [5]:

- for tensioned diagonals: $E_1 = E/1.1 \cdot \gamma_{ov} \cdot \Omega^N \approx 1.311 \div 1.313 \cdot 10^5$ N/mm$^2 \approx 0.62\ E_{STEEL}$;
- for compressed diagonals: $E_2 = \chi_{average} \cdot E_1 \approx 0.604 \div 0.737 \cdot 10^5$ N/mm$^2 \approx 0.29 \div 0.35\ E_{STEEL}$; where $\chi_{average}$ is the average value of the in-plane buckling factors evaluated for all the braces of the frames.

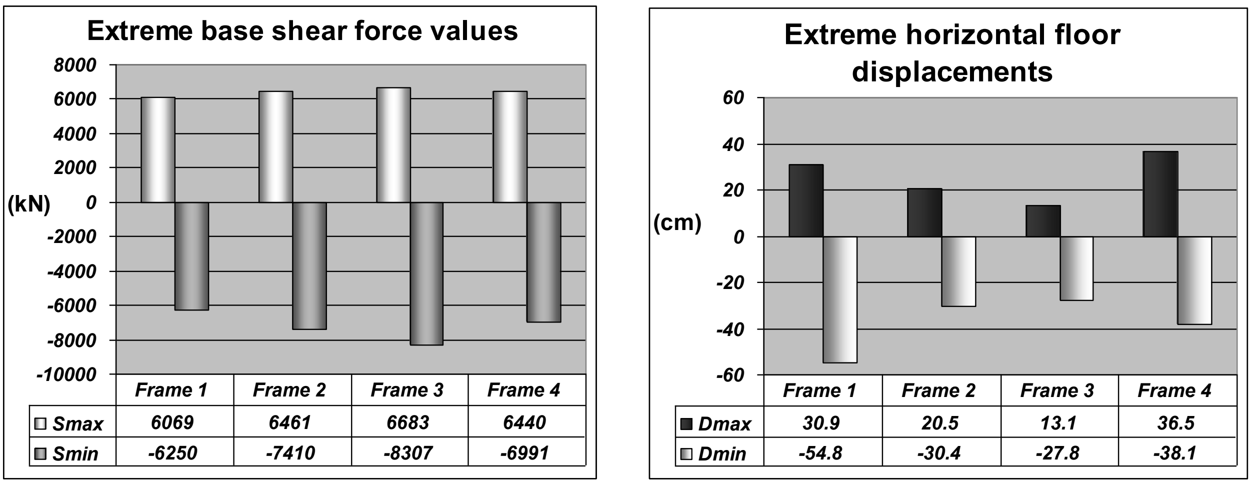


Figure 3. Extreme values for base shear forces and horizontal floor displacements

3. Results and comments

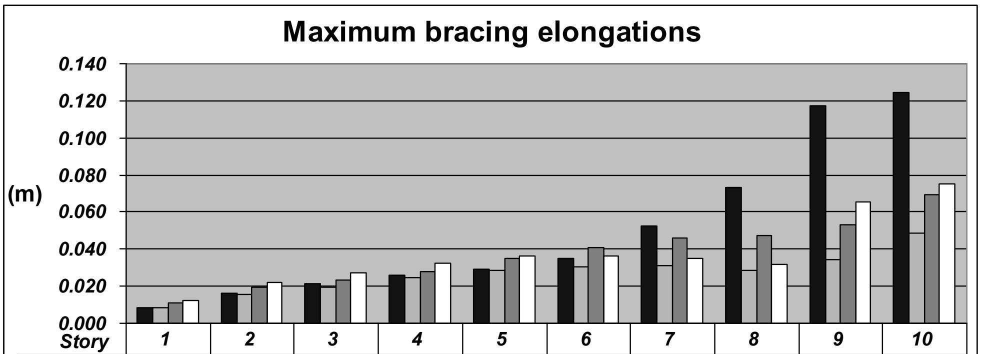
Dynamic nonlinear analyses using the Vrancea 1977 acceleration record were performed with each frame sized according to the four considered seismic design procedures. The peak ground acceleration was calibrated to a value of about 30% of the acceleration of gravity[4].

3.1 Horizontal displacements and base shear forces during dynamic nonlinear analysis

The extreme values recorded during the dynamic nonlinear analyses for base shear forces and horizontal floor displacements are shown in Fig. 3. The positive values S_{max} and D_{max} were recorded for one sense of motion, while the negative ones S_{min} and D_{min} were noticed for the opposite sense. The largest horizontal floor displacements were recorder in the case of frame 1 and the smallest for frame 3. The differences were about 97 % for one sense of motion and over 2.3 times for the other. In case of the base sheare forces, the greatest values were observed for frame 3 and the smallest for frame 1. The maximum differences were for up to 33% for one sense of motion and about 10% for the other sense (see Fig. 3).

3.2 Inelastic deformations during dynamic nonlinear analyses

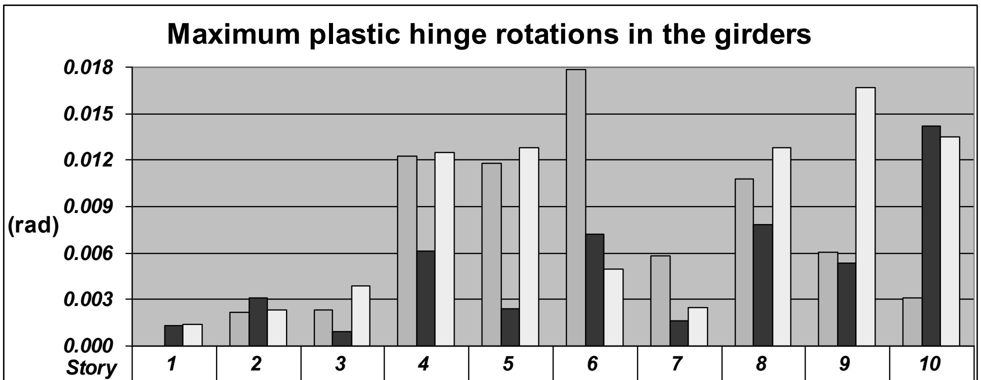
The three frames sized according to the alternative seismic design procedures had a favorable behavior during the dynamic nonlinear analyses. No inelastic deformations were noticed in the columns and beam segments outside the potentially plastic zones.



Story	1	2	3	4	5	6	7	8	9	10
Frame 1	0.008	0.016	0.021	0.026	0.029	0.035	0.053	0.073	0.117	0.124
Frame 2	0.009	0.016	0.020	0.025	0.029	0.031	0.031	0.028	0.034	0.048
Frame 3	0.011	0.019	0.023	0.028	0.035	0.041	0.046	0.047	0.053	0.070
Frame 4	0.012	0.022	0.028	0.032	0.036	0.037	0.035	0.032	0.065	0.075

Figure 4. Maximum bracing elongations

Fig. 4 illustrates the maximum bracing elongations recorded during the dynamic nonlinear analyses. The smallest elongations could be observed for frame 2 and the largest for frame 1. On average the values of the brace elongation for frame 2 were up to 36% smaller than for frame 3, about 32 % smaller compared to frame 4 and over 59 % smaller than the one recorded for frame 1.



Story	1	2	3	4	5	6	7	8	9	10
Frame 2	0.0000	0.0022	0.0024	0.0123	0.0118	0.0178	0.0058	0.0108	0.0061	0.0031
Frame 3	0.0013	0.0031	0.0009	0.0062	0.0024	0.0073	0.0016	0.0078	0.0053	0.0142
Frame 4	0.0014	0.0023	0.0039	0.0125	0.0128	0.0049	0.0025	0.0128	0.0167	0.0135

Figure 5. Maximum rotations in the potentially plastic zones for girders

The maximum inelastic rotations recorded during the dynamic nonlinear analyses in the potentially plastic zones located along the frame girders are shown in Fig. 5. The highest values are distributed differently for each frame. In case of frame 2 the largest deformations were noticed for the beams at the 6th story, for frame 3 at the 10th story and in case of frame 4 at the 9th story. On average the smallest values were recorded for frame 3 and the largest for frame 4. The values of the plastic hinge rotations along the girders of frame 4 were on average over 56% larger than the ones noticed in case of frame 3 and about 26% larger compared to frame 2.

3.3 Amount of dissipated energy

The amount of the different types of dissipated energy is indicated in the graphics in Fig. 3, 4 and 5. In all these graphics zone (1) represents the energy dissipated through damping, zone (2) represents the kinetic energy, zone (3) is the energy dissipated through elastic deformations in different kind of structural elements (braces, beams, respectively columns), zone (4) represents the energy dissipated through plastic deformations in the diagonals, zone (5) is the energy dissipated through plastic deformations in the potentially plastic zones of the beams, zone (6) represents the energy dissipated through plastic deformations in columns and beams outside the potentially plastic zones.

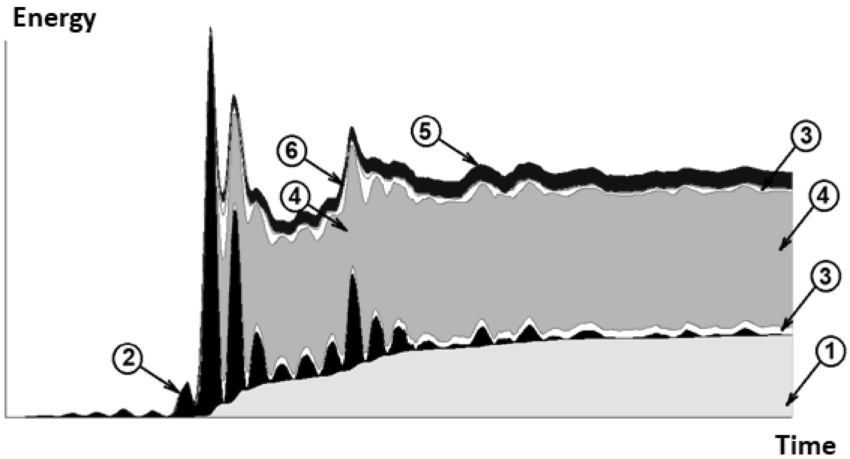


Figure 6. Dissipated energy for frame 2

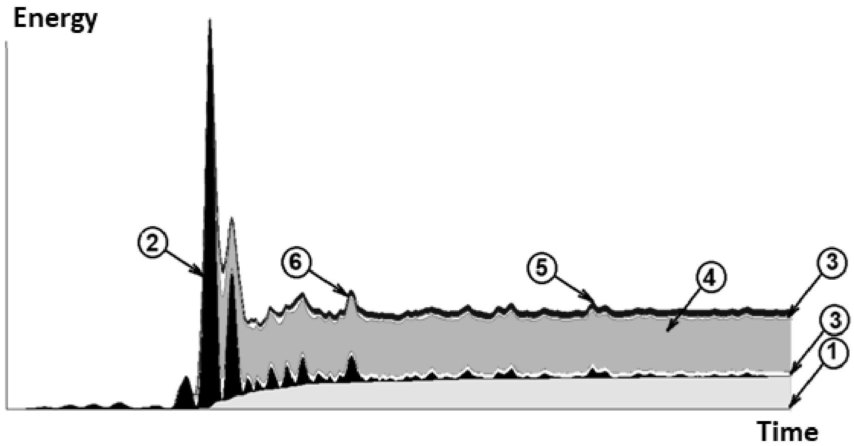


Figure 7. Dissipated energy for frame 3

Following remarks can be made by analysing the graphics in Fig. 6, 7 and 8:
- the largest amount of dissipated energy during dynamic nonlinear analysis could be observed in case of frame 4, while the smallest was noticed in case of frame 3 (the area of both zones (4) and (5) are the largest in Fig. 8 and the smallest in Fig. 7);

- for all frames most of the energy was consumed through inelastic deformations along the diagonals (zone (4) has the largest area in Fig. 6,7 and 8);
- no inelastic deformations could be noticed outside the diagonals and the potentially plastic zones located along the girders for frame 2, 3 and 4 (the area of zone (6) is equal to zero in Fig. 6, Fig. 7 and Fig. 8, respectively zone (6) is represented as a single curved line for all three frames sized with the alternative seismic design methods).

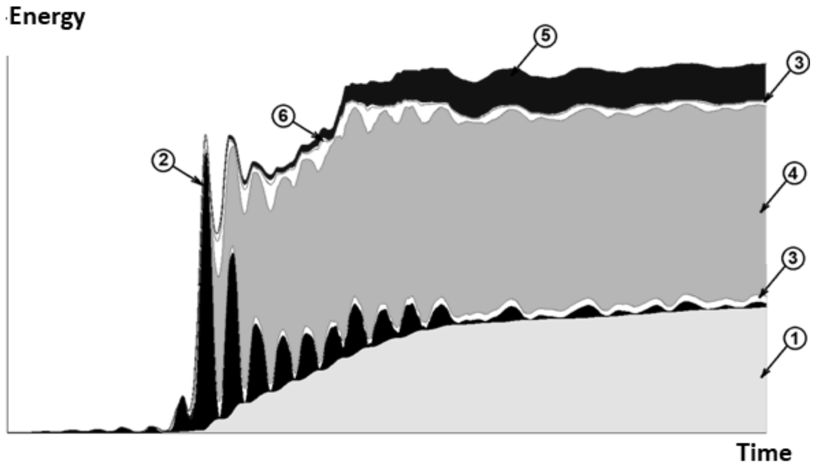
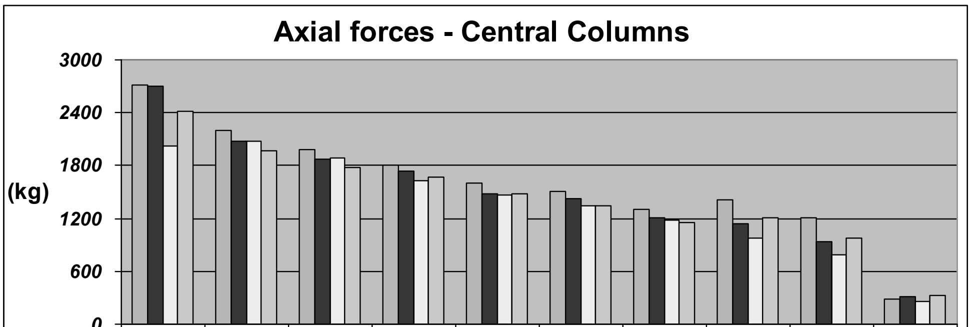


Figure 8. Dissipated energy for frame 4

3.4 Maximum values for bending moments and axial forces in different structural elements

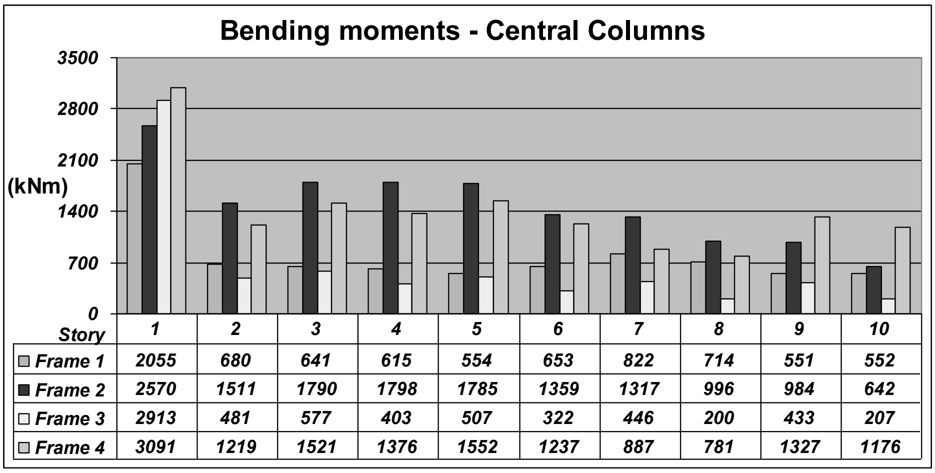
The largest axial forces during dynamic nonlinear analyses were observed in the central columns of frame 1, along the girders of frame 4 and respectively in the marginal columns of frame 3.
In most situations during dynamic nonlinear analyses the largest values for bending moments could be noticed in the different members of frame 2, while the smallest vending moments were recorded in the members of frame 1 (girders, central and marginal columns).
During dynamic nonlinear analyses, the maximum axial forces observed along the central column of frame 1 were up to 8% larger compared to frame 2, up to 12% bigger than for frame 4 and over 17% greater than in case of frame 3 (see Fig. 9).
The values of the maximum bending moments recorded during dynamic nonlinear analyses along the central column of frame 2 were about 4% greater compared to frame 4, over 27% bigger than for frame 3 and about 88% greater than for frame 1 (see Fig. 10).



Story	1	2	3	4	5	6	7	8	9	10
Frame 1	2709	2199	1980	1809	1600	1513	1298	1405	1202	279
Frame 2	2705	2073	1874	1732	1482	1429	1204	1139	937	313
Frame 3	2024	2083	1892	1635	1463	1344	1176	980	793	260
Frame 4	2416	1972	1777	1663	1481	1345	1153	1210	979	319

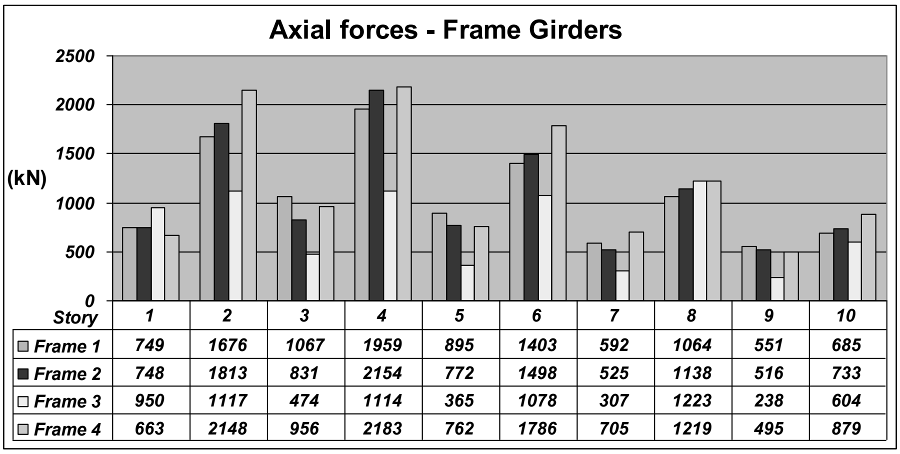
Figure 9. Maximum axial force values in the central columns during dynamic nonlinear analyses

In case of the marginal columns, the maximum axial forces noticed in case of frame 3 during dynamic nonlinear analyses were on average about 18% larger than the ones in frame 2, about 23% greater than the values in frame 4 and up to 50% bigger than the values recorded for frame 1.
The values of the maximum bending moments recorded along the marginal columns of frame 2 were about 4% greater, compared to frame 1, up to 8% larger compared to frame 4 and about 95% larger than the values noticed for frame 1.



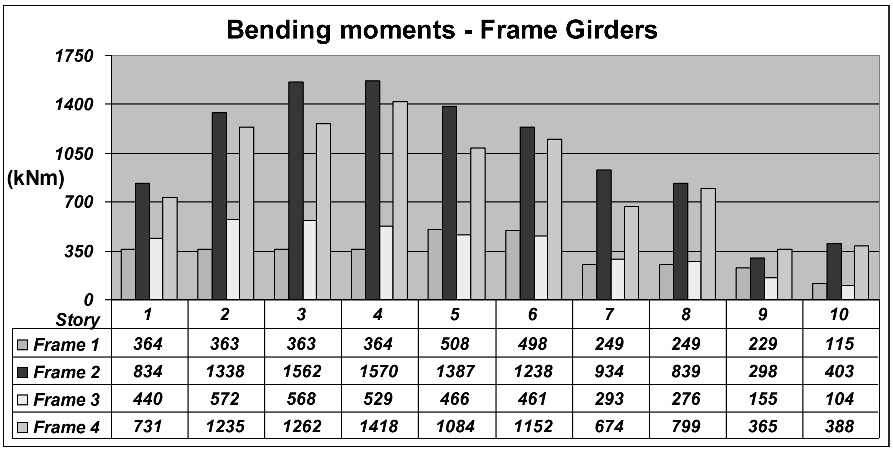
Story	1	2	3	4	5	6	7	8	9	10
Frame 1	2055	680	641	615	554	653	822	714	551	552
Frame 2	2570	1511	1790	1798	1785	1359	1317	996	984	642
Frame 3	2913	481	577	403	507	322	446	200	433	207
Frame 4	3091	1219	1521	1376	1552	1237	887	781	1327	1176

Figure 10. Maximum bending moments in the central columns during dynamic nonlinear analyses



Story	1	2	3	4	5	6	7	8	9	10
Frame 1	749	1676	1067	1959	895	1403	592	1064	551	685
Frame 2	748	1813	831	2154	772	1498	525	1138	516	733
Frame 3	950	1117	474	1114	365	1078	307	1223	238	604
Frame 4	663	2148	956	2183	762	1786	705	1219	495	879

Figure 11. Maximum axial force values in the frame girders during dynamic nonlinear analyses



Story	1	2	3	4	5	6	7	8	9	10
Frame 1	364	363	363	364	508	498	249	249	229	115
Frame 2	834	1338	1562	1570	1387	1238	934	839	298	403
Frame 3	440	572	568	529	466	461	293	276	155	104
Frame 4	731	1235	1262	1418	1084	1152	674	799	365	388

Figure 12. Maximum bending moments in the frame girders during dynamic nonlinear analyses

In most cases during dynamic nonlinear analyses, the largest axial force values recorded along the girders, were observed in case of frame 4 and the smallest ones in case of frame 3. These differences were on average up to 58% (as shown in Fig. 11).
The greatest bending moments along the frame girders were noticed for frame 2 and the smallest for frame 1 (see Fig. 12). On average the values of the bending moments in the girders of frame 2 were over 3 times larger than the ones in frame 1, up to 2.7 times greater than the values in frame 3 and about 14% larger than the values in frame 4.

3.4 Estimated steel consumption

The largest estimated steel consumption values were obtained in case of procedure 2 for the girders, the central and marginal columns, while procedure 3 lead to the highest estimated material consumption for the braces. The smallest steel consumption values were noticed in case of the girders and central columns sized according to procedure 1, respectively for the marginal columns designed with procedure 4 (see Fig.11).

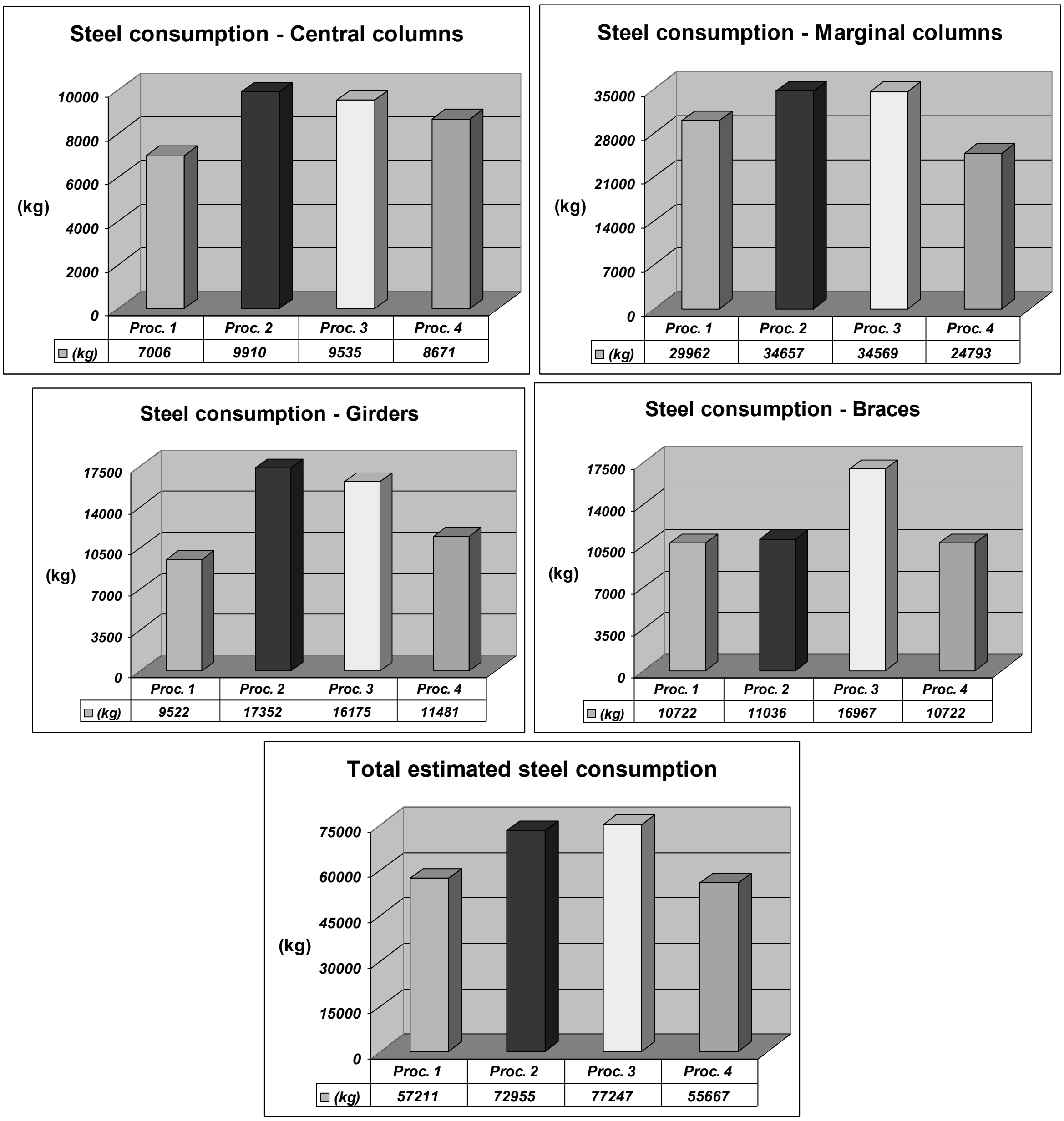


Figure 11. Estimated material consumption

The largest total estimated steel consumption was obtained for procedure 3 and the smallest for procedure 4. The estimated material consumption in case of frame 3 was up to 6% larger than the one for frame 2, about 35% higher compared to frame 1 and over 38% greater compared to frame 4.

4. Conclusions

The seismic design procedure indicated in the in charge European [3] and Romanian [1] seismic deign codes, lead to a concentrically braced frame that failed during the dynamic nonlinear analyses with Vrancea 1977 acceleration record. It is efficient for the seismic design of concentrically braced frames to consider for the dimensioning the forces that are obtained from an elastic analysis of the structure having the diagonals modeled with reduced axial stiffness. All the concentrically braced frames sized according to the alternative seismic design procedures 2, 3 and 4 (that used this braces modeling), had a favorable behavior during the dynamic nonlinear analyses, having all inelastic deformations concentrated along the braces and in the potentially plastic zones placed along the frame girders. Procedure two (that considers the braces as primary dissipative elements and the girders as secondary dissipative elements) provides the best approximation of the loading states that appear in the girders and columns of the frame when most of the adjacent diagonals are out of work, but leads on the other hand in most situations to the largest bending moments along the girders and columns. The smallest values for lateral floor displacements and for inelastic deformations along the girders and diagonals could be noticed for the frame sized according to procedure three (the one that considers potentially plastic zones along the frame girders as primary dissipative elements). This procedure leads on the other hand to the largest cross-sections for the braces and to the highest estimated overall steel consumption value.

The frame designed according to procedure four (the one where the diagonals and the potentially plastic zones along the girders are sized for the forces generated by the same seismic force) had the smallest estimated steel consumption value. Compared to the frames sized according to procedures two and four, larger inelastic deformations could be noticed during the dynamic nonlinear analysis along the braces and girders of the frame designed according to procedure four. Taking into consideration the behavior during dynamic nonlinear analyses and the estimated material consumption we recommend procedures four and two for the seismic design of concentrically braced frames.

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