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Comments Concerning the Seismic Design of Diagonal Braced Frames According to P100-1/2013

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Abstract

The present paper analyses some possibilities to improve the provisions concerning the seismic design of diagonal braced frames according to the Romanian code P100-1/2013. Three design variants were used for the dimensioning under seismic actions of a ten story concentrically braced frame, taking into consideration different contributions of the tensioned and compressed braces in carrying horizontal seismic forces. Static nonlinear analyses were performed with all the designed variants of the frame. The loading and deformation states at different steps during the analyses were compared. The distribution of plastic hinges in the structure was analyzed. A more favorable behavior of the frame during static nonlinear analysis was ensured by providing reduced cross-sections along all frame girders ends (resembling the dog-bone detail).

Rezumat

Prezenta lucrare analizează unele posibilități de îmbunătățire ale prevederilor de proiectare la acțiuni seismice pentru cadre contravântuite centric conform normei românești P100-1/2013. Au fost avute în vedere trei variante de dimensionare alternative la acțiuni seismice pentru un cadru cu zece niveluri și contravântuiri centrice dispuse alternant, considerându-se contribuții diferite de preluare a încărcărilor seismice orizontale de către diagonalele întinse și comprimate. Cu fiecare variantă obținută a cadrului s-au efectuat analize static neliniare. S-au comparat stările de eforturi și deformații în diferite stadii ale analizelor, urmărindu-se și distribuțiile articulațiilor plastice în structură. Prin prevederea de secțiuni reduse în lungul tuturor extremităților riglelor de cadru (detalii de tip dog-bone) s-a asigurat o comportare mult mai bună a cadrului în timpul analizei static neliniare.

Keywords: concentrically braced frame, dog-bone detail, static nonlinear analyses, dissipated energy, reduced Young modulus value, global plastic failure mechanism

1. Introduction

A ten storey structure located in Bucharest, Romania was considered having two spans and four bays of 6.0m. The storey height was 3.5m. A pair of diagonal concentrically braced frames was

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provided on each main direction of the structure, as shown in Fig. 1a). On both directions the braced frames had two spans of 6.0m (see Fig. 1b). It was considered that the whole horizontal seismic force was carried only by the diagonal braced frames.

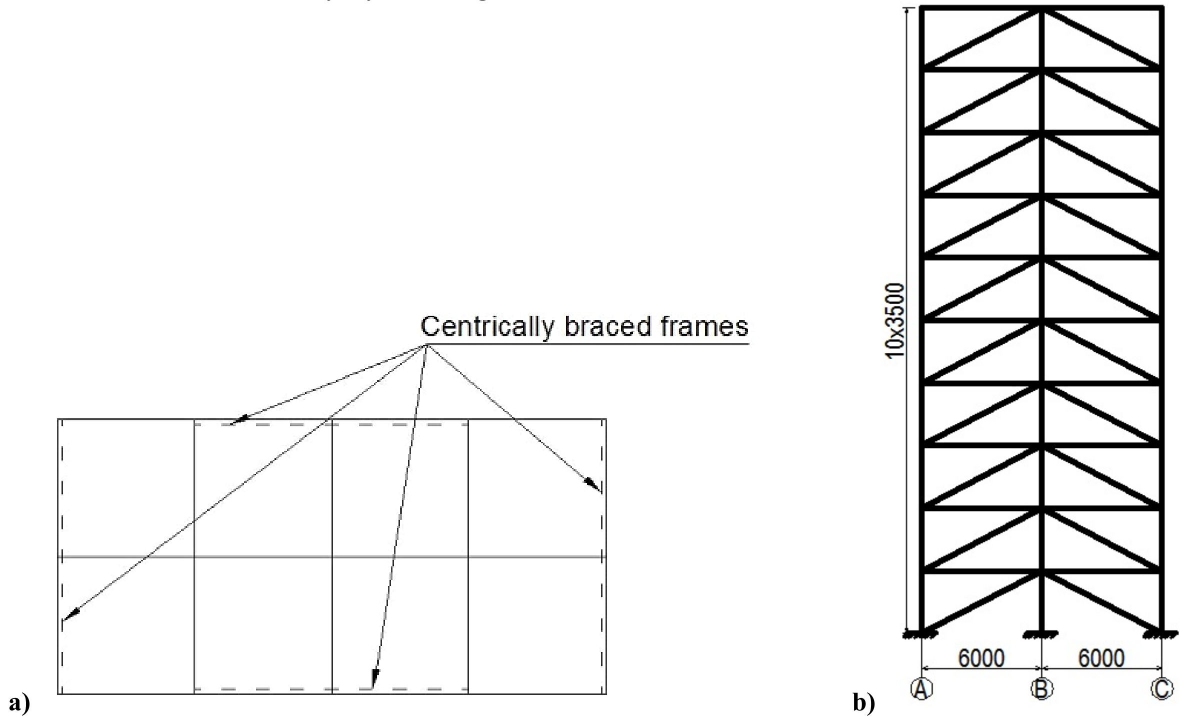


Figure 1. a) Plan view of the structure with the location of the braced frames; b) Elevation of a braced frame.

All kind of structural elements had "I"-shaped cross-sections made of welded steel plates. The cross-sections of the diagonals were rotated with the web normally to the plane of the braced frame, in order to ensure the in-plane buckling of the braces [1].
In the in charge seismic design codes P100-1/2013 [2], EN 1998-1:2004 [3] and ANSI/AISC 341-16 [4] it is specified that concentrically braced frames shall be design so that yielding of the diagonals in tension will take place before failure (buckling or yielding) of the beams or columns. According to the provisions of these codes, the diagonals of the concentrically braced frame were dimensioned for the forces produced by an unamplified code seismic action, while the girders and columns were designed to withstand the forces generated by an increased seismic action (compared to one used for the design of the diagonals).

2. Considered Seismic Design Methods

The analyzed concentrically braced frame was sized for the seismic forces evaluated according to the in charge Romanian seismic design code P100-1/2013 [2], taking into consideration three different ways of modelling the axial stiffness of the braces:
- Variant A: both tensioned and compressed diagonals were taken into consideration for the design of girders and columns of the frame; (only the tensioned diagonals were considered for the design of the braces); this is the current design procedure according to the in charge Romanian seismic code P100-1/2013 [2];
- Variant B: the compressed diagonal members were neglected completely in the design process for all kind of structural elements (diagonals, girders and columns);
- Variant C: the compressed diagonals were modelled considering a material with a reduced value of the Young modulus $E_{reduced} = 0.3 \cdot E_{steel}$ [3], while the tensioned diagonals were modelled with the unreduced value of the modulus of elasticity of steel $E_{steel} = 2.1 \cdot 10^5 N/mm^2$.

By designing the compressed diagonals with a low value for the modulus of elasticity equal to $0.3 \cdot E_{steel} \approx \chi_{medium} \cdot E_{steel}$ we tried to obtain a better approximation of the loading states that appear in the structure when most of the compressed diagonals reach their buckling capacity [1, 5, 6].
„Frame A" was dimensioned according to variant A, „Frame B" was the one sized according to variant B and „Frame C" was the frame designed according to variant C.
The following steps were considered in the seismic design:
- Concentrically braced frames diagonals were designed to the forces generated by the non-amplified seismic load design S_{code}. For the designed sections of the diagonals an amplification factor was evaluated (Ω^N).
- The other categories of structural elements (beams and columns) were sized for the forces produced by an amplified seismic load ($1.1 \cdot \gamma_{ov} \cdot \Omega^N \cdot S_{code}$) [2, 3].

Where:
- Ω^N is the minimum value of Ω^N_i; $\Omega^N = \min\{\Omega^N_i\}$;
- $\Omega^N_i = N_{pl,Rd,i}/N_{Ed,i}$; $N_{pl,Rd,i}$ is the plastic tensile design resistance of diagonal „i" and $N_{Ed,i}$ is the design value of the axial force in the same diagonal „i" in the seismic design situation [2, 3];
- γ_{ov} is the overstrength factor of the material (the ratio between the actual yield strength value and the nominal yield strength value); according to P100-1/2013 [2] for S235 steel $\gamma_{ov} = 1.40$.

In the following figure the first three modal periods for the analyzed frames are indicated:

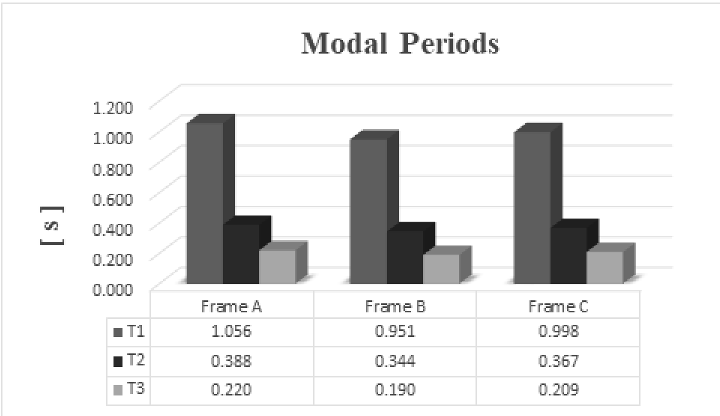


Figure 2. Modal Periods.

Frame A (dimensioned in accordance with the P100-1/2013 [2]) resulted to be the most flexible and Frame B (in which the compressed diagonals were neglected completely in the design process) resulted as the most rigid one.

3. Member forces used for the seismic design

In all three considered design procedure the diagonals cross-sections were sized for the forces generated in the structure by the same combination of loads that contained the effects of the unamplified seismic action, evaluated according to the in charge Romanian seismic design code P100-1/2013 [2].
The beams and columns were dimensioned for the forces generated by the combinations of loads that included the effects of an amplified seismic action: $1.1 \cdot \gamma_{ov} \cdot \Omega^N \cdot S_{code} \approx 1.623 \cdot S_{code}$.
The axial forces in the diagonals were higher in case of Frame A and Frame B (with about 31%, see Fig. 3), because in case of Frame C both the tensile and compressive capacity of the diagonals was taken into consideration.
In case of Frame A and Frame B only the tensioned diagonals were considered for the design of the braces cross-sections. For Frame C the horizontal seismic forces were carried by tensioned and compressed diagonals and the forces in the tensioned diagonals were smaller than the ones recorded

in case of Frame A and B.

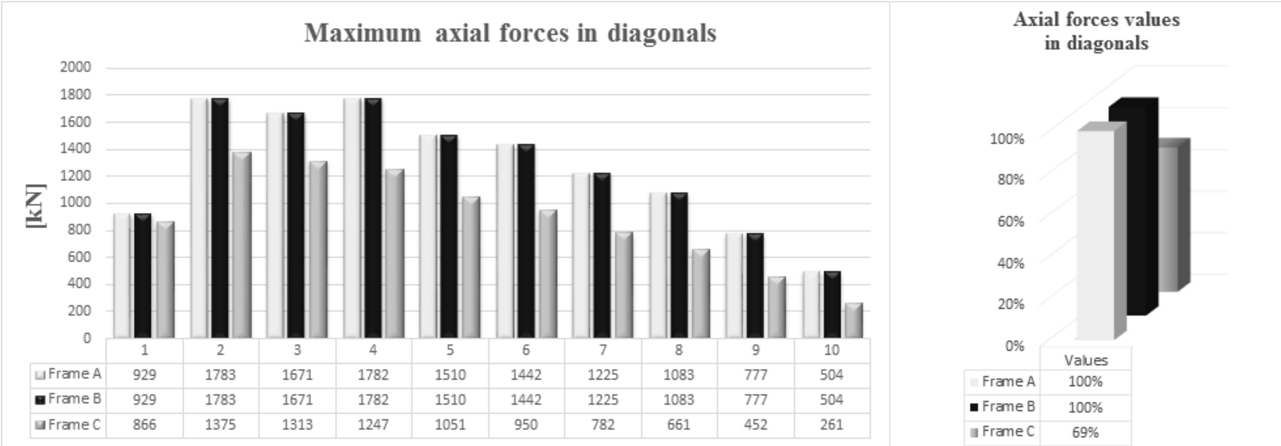


Figure 3. Maximum axial forces in diagonals.

The largest axial force values in the girders were noticed in case of Frame B (see Fig. 4). These values were up to 33% greater than the values obtained for Frame A and about 37% larger than the ones recorded for Frame C. The axial forces in the girders must balance the horizontal projections of the axial forces in the diagonals. In case of Frame B the compressed diagonals were neglected completely and for this reason the highest values of the axial forces were recorded along the girders placed in the same span with the tensioned diagonals.

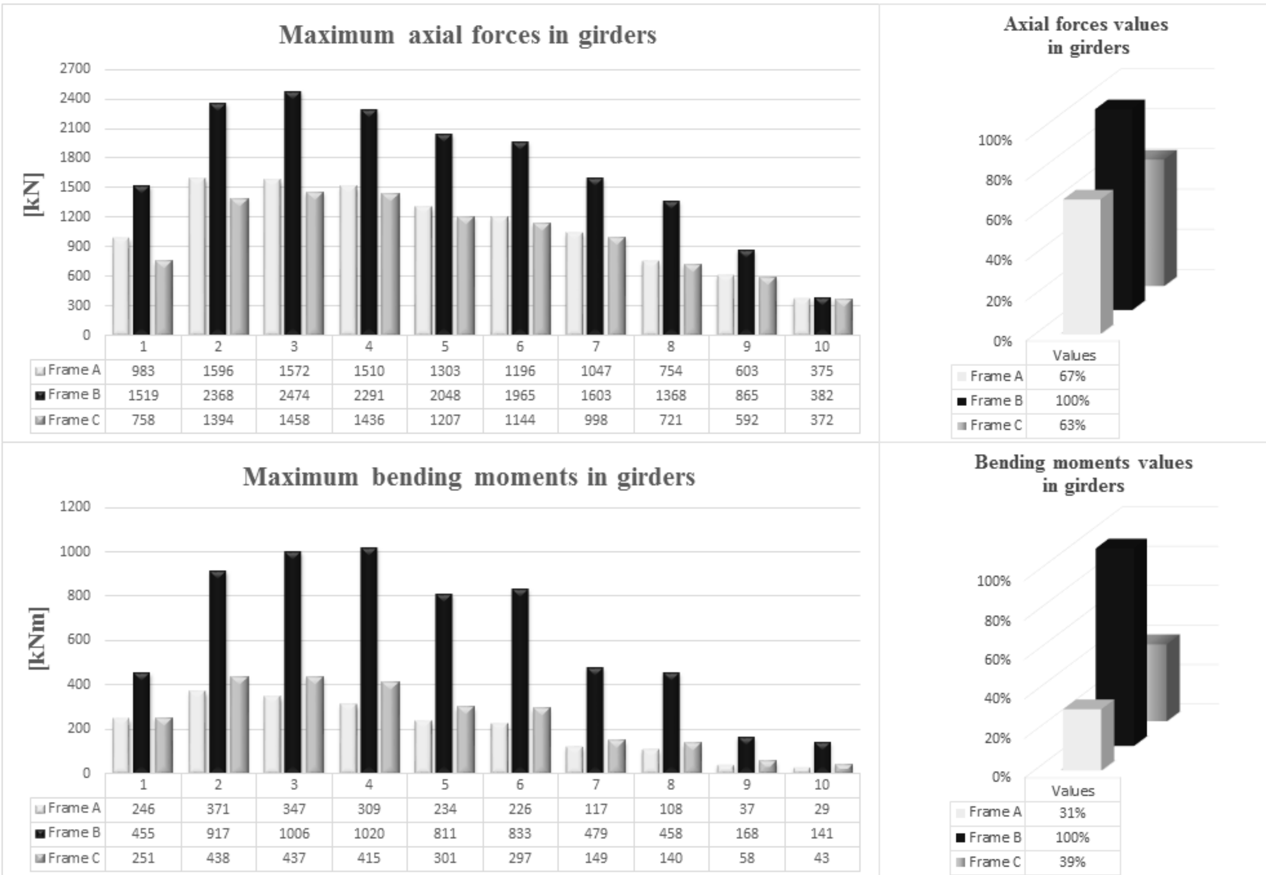


Figure 4. Maximum axial forces and bending moments in the girders.

The highest bending moment values could be noticed in the girders of Frame B. These values were about 2.5 times greater compared to the bending moments recorded in the girders of Frame C and about 3.2 times larger than those recorded in case of Frame A, as shown in Fig. 4. In case of Frame B the compressed diagonals were neglected completely and for this reason the highest values of the bending moments were recorded along the girders placed in the same span with the compressed diagonals.

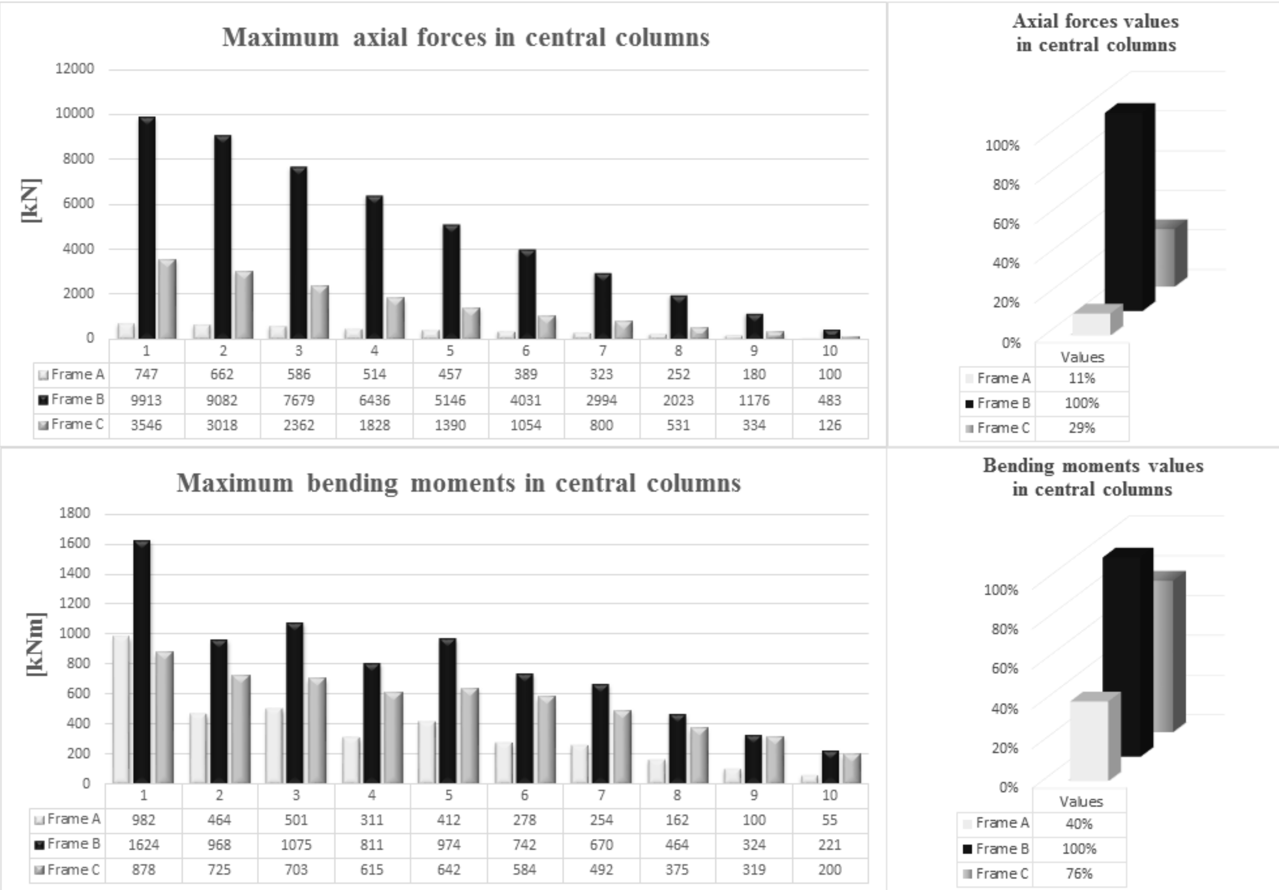


Figure 5. Maximum axial forces and bending moments in the central columns.

By neglecting completely the effect of compressed diagonals in case of Frame B, this frame worked mainly as a vertical single-span truss girder subjected to horizontal seismic forces. In the case of the other two analyzed frames, the diagonals from both spans of the frames were taken into consideration.

This explains the much higher axial force values observed in case of the central columns of Frame B. These axial forces were up to 3.4 times greater than the ones recorded along the central column of Frame C and about 9.0 times larger than those noticed in case of Frame A (see Fig. 5).

In the span of Frame B, where the compressed diagonals are neglected completely, the structure works as a moment resisting frame with rigid beam/column connections and for this reason the highest bending moment values could be noticed along the girders and central columns of Frame B, as shown in Fig. 4 (girders) and Fig. 5 (central columns).

The maximum bending moment values in the central columns, observed in case of Frame B, were up to 32% larger compared to the ones in Frame C and about 2.5 times greater compared to those recorded for Frame A.

The maximum axial forces values in the lateral columns could be observed in case of Frame A. The values were smaller for Frame B with about 69%. The values for Frame A and Frame C were in the same range (differences smaller than 3%, see Fig. 6).

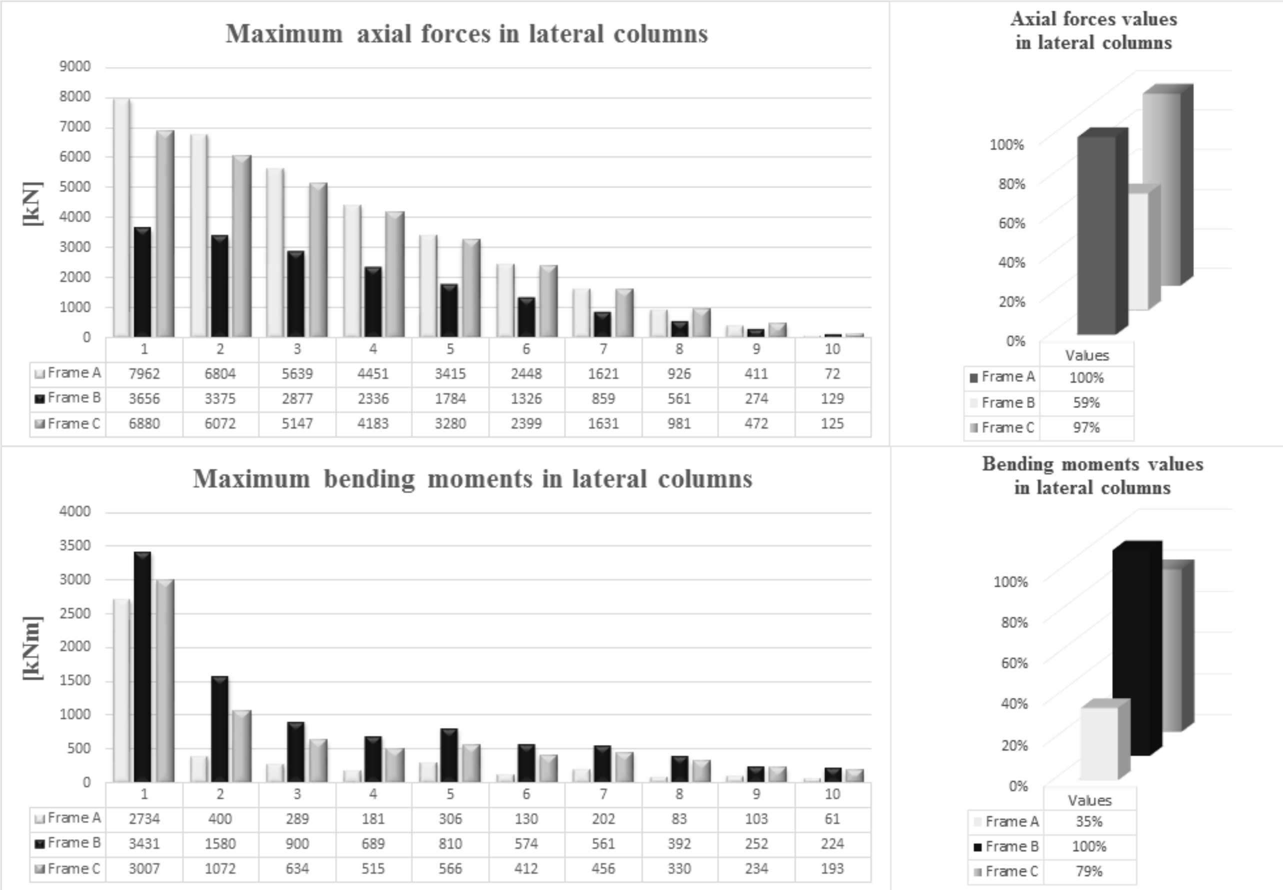


Figure 6. Maximum axial forces and bending moments in lateral columns.

The maximum bending moments values in the lateral columns were obtained in case of Frame B. Compared to these values, the ones in Frame A were 2.86 times smaller and those in Frame C were 1.27 times smaller, as shown in Fig. 6.

4. Static nonlinear analyses

Static nonlinear analyses were performed with each of the three designed variants of the frame. The gravitational loads were maintained constant (selfweight, 30% of live load, respectively 40% of snow load), while the effects of the horizontal seismic forces were increased progressively [7].
The following characteristic steps were compared during the analyses:
Step „I" – the step when the first compressed diagonal is taken out of work (through buckling);
Step „II" – the step of analysis when all compressed diagonals are out of work;
Step „III" – the step when the first tensioned brace is taken out of work (plastic hinges are developed in the brace under the combined action of bending moments and tensile axial forces);
Step „IV" – the step of analysis when all tensioned diagonals are out of work (through yielding);
Step „V" – the step when the first plastic hinge appears outside the braces (in a beam or column);
Step „VI" – the step when a local or global plastic hinge mechanism is developed in the frame.
During the static nonlinear analyses unfavorable distributions of plastic hinges led in case of all three considered design variants to the development of local plastic failure mechanisms (plastic hinges in all the frame members connected to the same joint for Frame B, respectively the development of soft storey mechanisms for Frame A and C). No favorable global plastic failure mechanism could be observed for all of the three considered design variants, because no clear strength hierarchy was provided by design between the capacity of girders and columns [8].

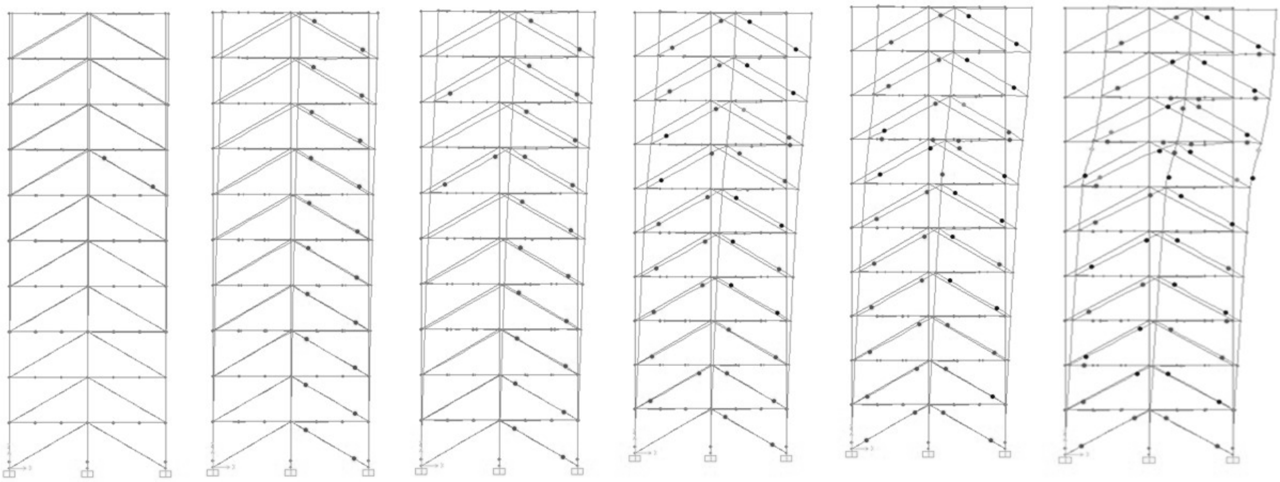


Figure 7. Step I, II, V, III, IV, VI during the static nonlinear analysis of Frame A.

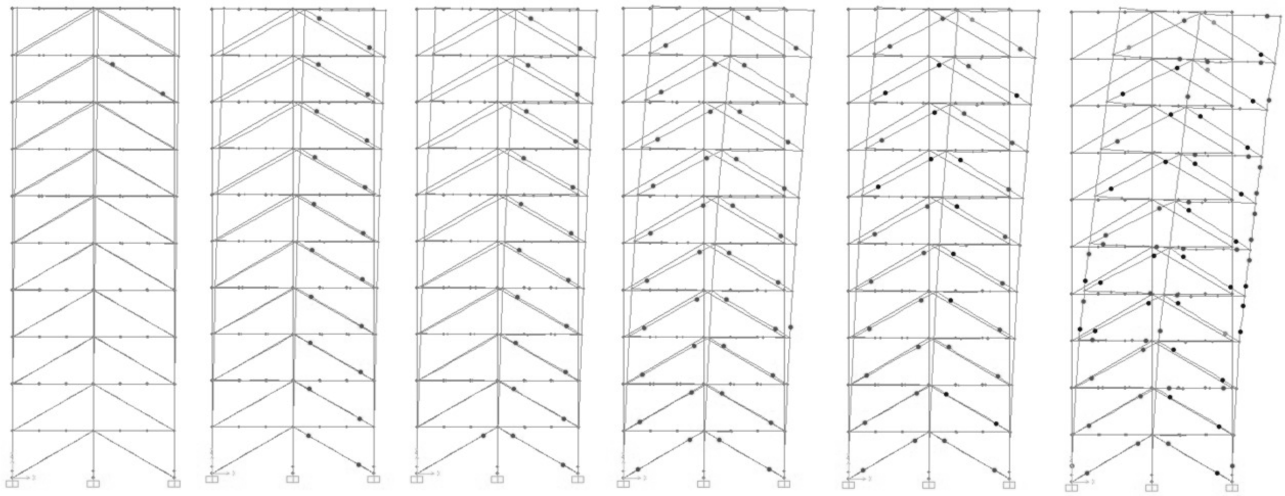


Figure 8. Step I, II, III, IV, V, VI during the static nonlinear analysis of Frame B.

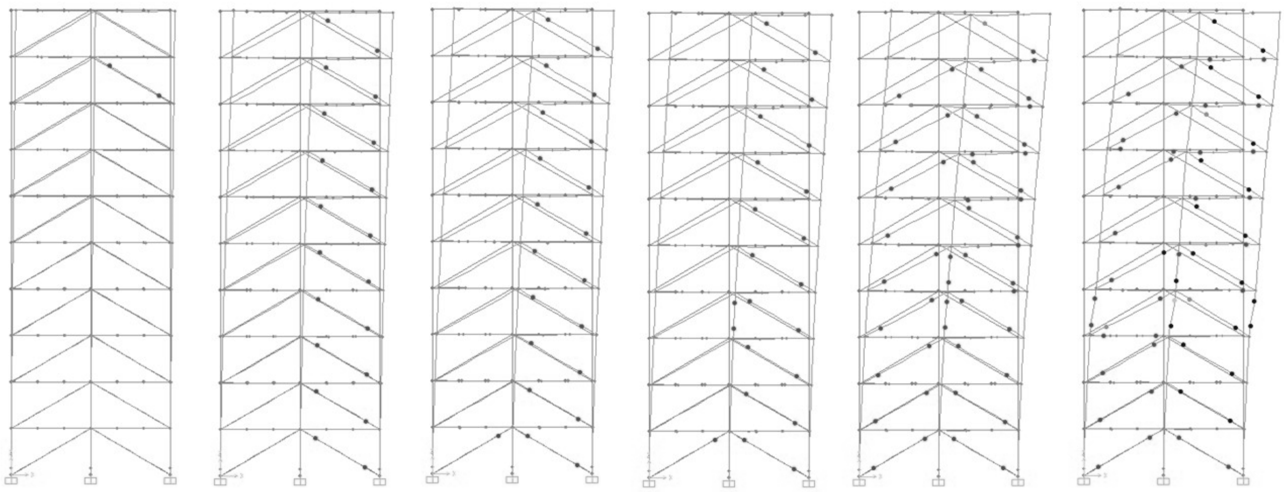


Figure 9. Step I, II, III, V, IV, VI during the static nonlinear analysis of Frame C.

During the static nonlinear analysis of Frame C step „V" (the step when the first plastic hinge appears outside the diagonals in the central column), appeared before step „IV" (the step of analysis when all tensioned diagonals are out of work), see Fig. 9.

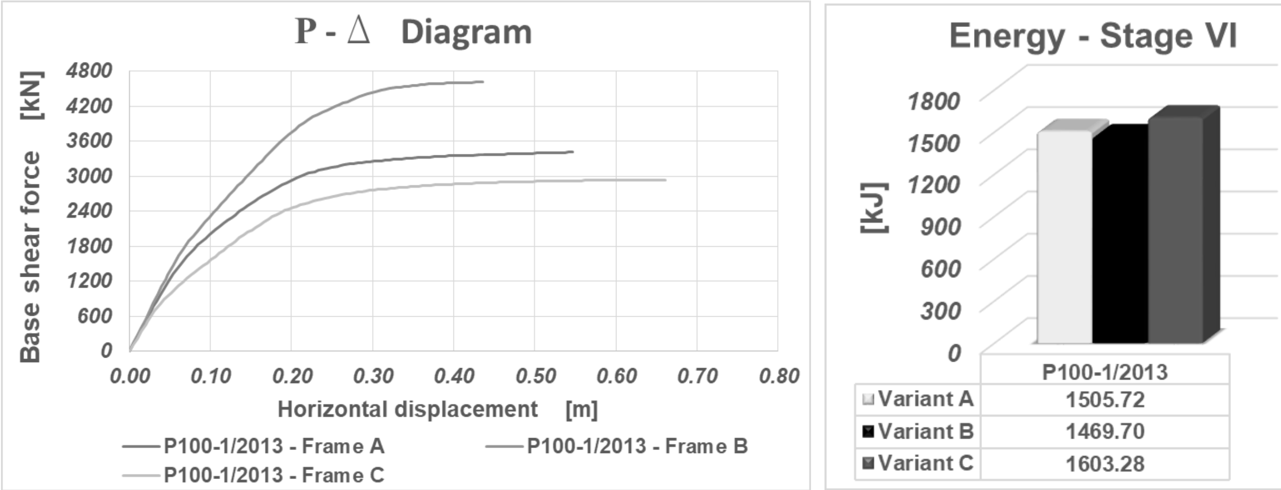
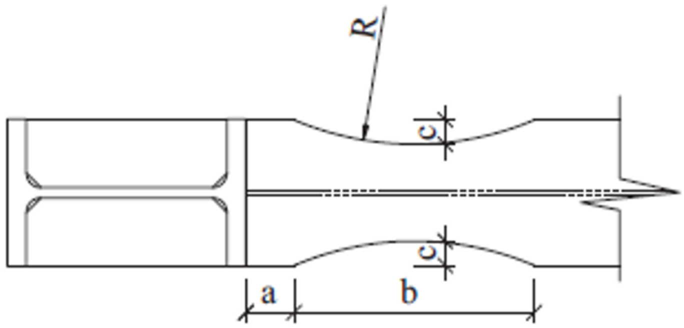


Figure 10. P - Δ Diagrams and estimated consumed energy after static nonlinear analyses.

The largest amount of energy (mechanical work) was consumed during the static nonlinear analysis of Frame C (see Fig. 10). The difference was about 6.5% compared to Frame A and over 9.0% compared to Frame B. In order to improve the behavior of Frame C during static nonlinear analysis additional potentially plastic zones with reduced cross-sections [6, 8] were provided (placed at about 1.0m from the axes of the columns along all frame girders), as indicated in Fig. 11.



$$0.1 \cdot b_{girder} \le c \le 0.25 \cdot b_{girder}$$

Figure 11. Potentially plastic zones with reduced cross-sections according with P100-1/2013 [2].

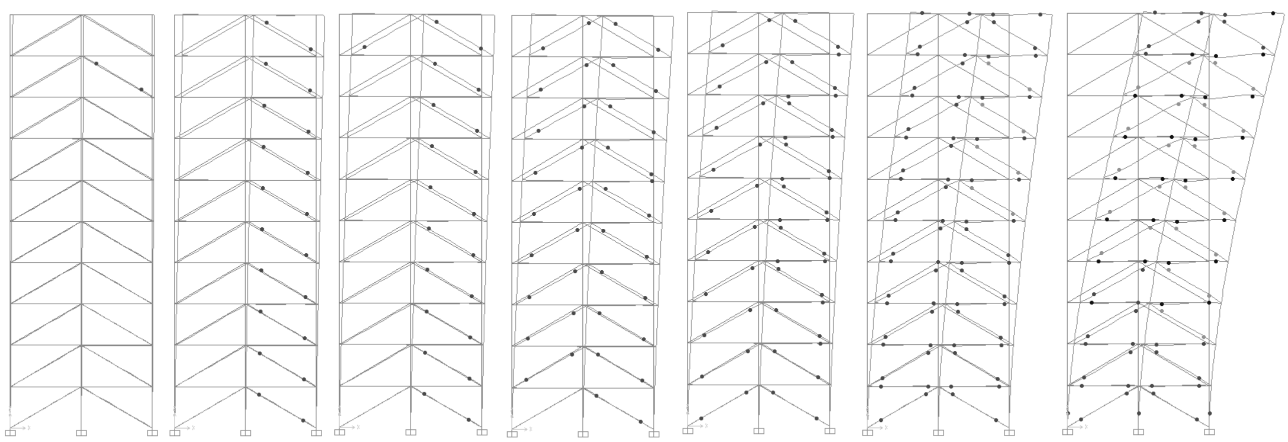


Figure 12. Stages of static nonlinear analyses for the frame provided with reduced cross-sections.

A global favorable plastic failure mechanism could be noticed in case of Frame C equipped with additional potentially plastic zones along the girders (see Fig.12) and a much larger amount of energy (about 55%) was dissipated during the static nonlinear analysis of this frame (see Fig. 13).

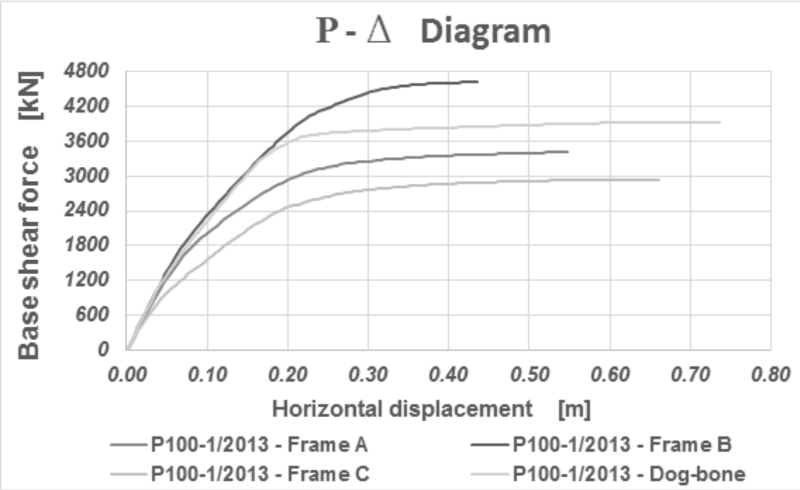


Figure 14. P - ∆ Diagrams after static nonlinear analyses.

5. Estimated steel consumption

The smallest cross-sections were obtained for the diagonals of Frame C, for the lateral columns of Frame B and for the girders and central columns of Frame A.
The largest estimated steel consumption value for the diagonals was obtained in case of Frame A and Frame B. For Frame C the value was smaller with about 25%.
For girders and central columns the largest estimated steel consumption values were obtained for Frame B. Compared to Frame A, these values were about 58% greater for the girders and over 2.1 times larger in case of the central columns (see Figure 6).
The largest estimated steel consumption value for lateral columns was recorded in case of Frame C. The differences compared to the values obtained for Frame A and B were smaller than 10%.
The smallest total estimated steel consumption value was obtained for Frame A, while the largest value was observed for frame B. The overall difference was about 29% (see Figure 15).

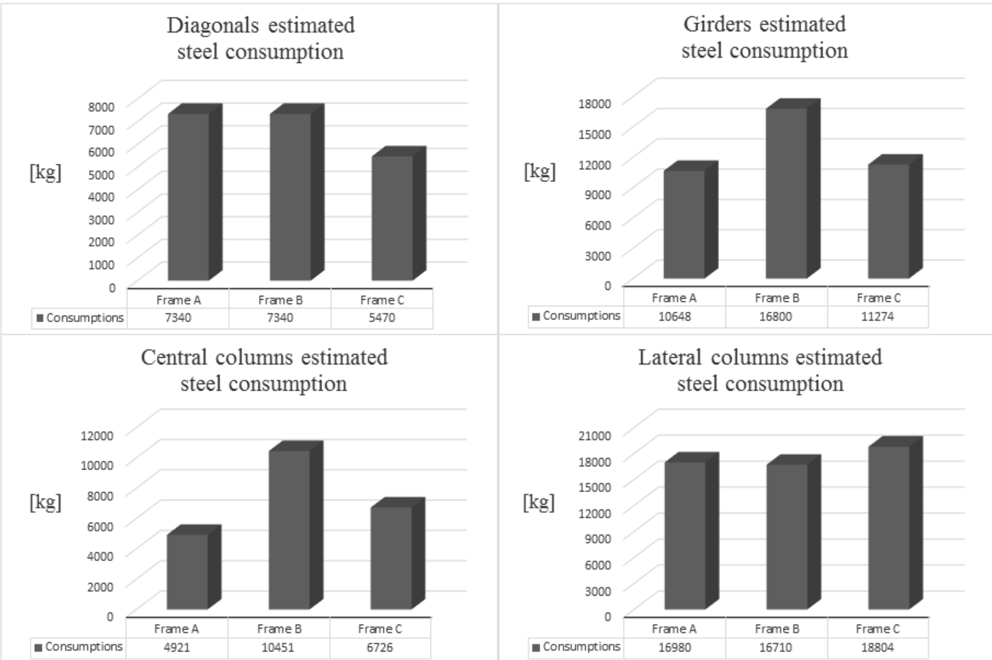


Figure 8. Estimated steel consumption for diagonals, girders, central and lateral columns.

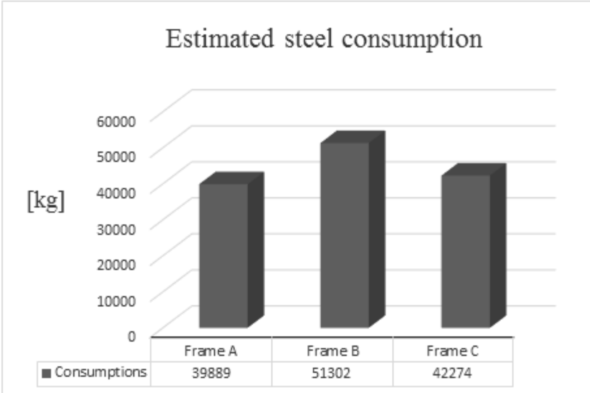


Figure 7. Overall estimated steel consumption.

6. Conclusions

All three frames sized according to the considered seismic design variants had a quite unfavorable behavior during the static nonlinear analyses. Inelastic deformations could be noticed during the analyses mainly in the braces but also in unwanted locations along girders and columns. A more favorable behavior of the frame during static nonlinear analysis was ensured by providing reduced cross-sections along all frame girders ends (resembling the dog-bone detail).
The design according to variant C approximates the best the loading states that appear along the girders and columns of the concentrically braced frame in the situation when most of the diagonals are out of work (through buckling respectively development of plastic hinges). Variant C with additional potentially plastic zones along the girders leads to the best behavior during static nonlinear analyses.

7. References

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