Seismic design methods for concentrically braced frames

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Abstract. The present paper is intended to point out the main differences existing among the seismic design procedures for concentrically braced frames contained in different seismic design codes. Two ten storeyed concentrically braced frames were designed taking into consideration the provisions of three seismic design codes: the Romanian norm P100-1/2013 [1], the European standard EN 1998-1:2004 [2] (respectively SR EN 1998-1:2005 [3]) and the American code AISC 341-16 [4]. All structural members of the concentrically braced frames (braces, beams and columns) had built up I-shaped cross-sections, sized according to the provisions of SR EN 1993-1-1:2006 [5]. Each designed frame was subjected to static nonlinear analyses. The seismic design loading state in different kind of structural members and the estimated steel consumption, the maximum values of base shear forces and horizontal floor displacements were compared. The successive formation of plastic hinges and the plastic failure mechanism in the different designed frames was analysed. A configuration with additional potentially plastic zones along the frame girders was proposed in order to improve the behaviour of the concentrically braced frames during static and dynamic nonlinear analyses.

1. Introduction
Two ten storeyed concentrically braced frames were analysed. The storey height was 3.5m. The considered frames, had two spans of 6.0m and were equipped with ascendant and descendant diagonal bracings, as indicated in Figure 1. The two frames were designed taking into consideration the provisions of three seismic design codes: the Romanian norm P100-1/2013 [1], the European standard EN 1998-1:2004 [2] (respectively SR EN 1998-1:2005 [3]) and the American code AISC 341-16 [4].

![Frame DC](image1)

![Frame DM](image2)

Figure 1. Analysed concentrically braced frames.
The analysed frames were designed for the same base shear force value, evaluated according to the in charge Romanian seismic design code P100-1/2013 [1] for the location of Bucharest, with a peak ground acceleration value of 0.3 times the acceleration of gravity. The behaviour of the designed frames during static nonlinear analyses [7] was compared.

All structural members (diagonals, girders and columns) had built up I-shaped cross-sections sized according to the prescriptions of SR EN 1993-1-1 [2]. The webs of the braces cross-sections were placed normally to the plane of the frames (in order to ensure the in-plane buckling of the diagonals), while the cross-sections of the frames girders and columns are orientated with the webs in the bracing plane.

2. Short description of the considered seismic design provisions

In the considered seismic design codes P100-1/2013 [1], EN 1998-1:2004 [2] 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.

2.1. Design according to P100-1/2013 and EN 1998-1:2004

The braces of diagonal concentrically braced frames should fulfil two types of requirements according to the provisions of P100-1/2013 [1] and EN 1998-1:2004 [2]:

- a slenderness check: \( \lambda \leq 2 \cdot \lambda_e; (\lambda_e \leq 2) \)
- a strength check of performed for all tensioned diagonal members: \( N_{pl,Rd} \geq N_{Ed} \)

Where:

\( \lambda \) = slenderness of diagonal member;

\( \bar{\lambda} \) = relative slenderness of diagonal member; \( \lambda_e = \frac{\lambda}{\lambda_e}; \) \( \lambda_e = \lambda_1 = \frac{\pi}{\sqrt{0.235}} \)

\( N_{Ed} \) = design value of the axial effort in the considered tensioned diagonal member (produced by the seismic combination of actions which contains the unamplified seismic action);

\( N_{pl,Rd} \) = design value of the plastic tension resistance in the gross cross-sectional area (no holes) according to SR EN 1993-1-1:2006 [5]; \( N_{pl,Rd} = A \cdot f_y / \gamma_M0 \);

\( A \) = gross cross-sectional area of the diagonal member;

\( f_y \) = characteristic value of the steel yielding stress;

\( \gamma_M0 \) = partial safety factor for strength checks taking into consideration the yielding stress of steel.

According to P100-1/2013 [1], EN 1998-1:2004 [2] the beams and columns will be checked for the capacity and stability ultimate limit states according to the provisions from SR EN 1993-1-1:2006 [5], considering the most unfavourable state of efforts determined from a load combination that includes the amplified seismic load. The design efforts for beams and columns in the seismic load combination are determined with the following relations in case of P100-1/2013 [1]:

\[
N_{Ed} = N_{Ed,G} + \Omega_t \cdot N_{Ed,E}; M_{Ed} = M_{Ed,G} + \Omega_t \cdot M_{Ed,E}
\]

Where:

\( N_{Ed,G} \); \( M_{Ed,G} \) = axial force and bending moment in the checked members generated by gravitational loads considered on the structure during the seismic action;

\( N_{Ed,E} \); \( M_{Ed,E} \) = axial force and bending moment in the checked members generated by the design seismic load (unamplified seismic load);

\( \Omega_t \) = overstrength factor of the structural system;

\( \Omega_t = 1,1 \cdot \gamma_{ov} \cdot \Omega_t^{(N)}; \) \( \Omega_t^{(N)} \) = \( \Omega_t^{(N)} = \frac{N_{pl,Rd,i}}{N_{Ed,i}} \);

\( \gamma_{ov} \) = material overstrength factor used in the design;

\( \gamma_{ov} = 1.40 \) for S235 steel grade [1];

\( N_{pl,Rd,i} \) = design value for the plastic tension resistance of diagonal “i”;

\( N_{Ed,i} \) = design value of the axial effort in the same considered diagonal “i” from the combination of actions with the unamplified seismic load.
In case of EN 1998-1:2004 [2] beams and columns with axial forces should meet the following minimum resistance requirement:

\[ N_{pl,Rd}(M_{Ed}) \geq N_{Ed,G} + 1.1 \cdot \gamma_{ov} \cdot \Omega \cdot N_{Ed,E} \]

Where:
- \( N_{pl,Rd}(M_{Ed}) \) is the design buckling resistance of the beam or the column in accordance with EN 1993, taking into account the interaction of the buckling resistance with the bending moment \( M_{Ed} \), defined as its design value in the seismic design situation;
- \( \gamma_{ov} \) = material overstrength factor used in the design; \( \gamma_{ov} = 1.25 \) for all steel grades according to [2];
- \( \Omega \) = is the minimum value of \( \Omega_i = N_{pl,Rd,i}/N_{Ed,i} \) over all the diagonals of the braced frame system.

In order to ensure an homogeneous dissipative behaviour of the diagonals the maximum and minimum \( \Omega \) values should not differ from each other with more than 25%.

2.2. Design according to AISC 341-16

The braces of diagonal concentrically braced frames should fulfill a slenderness check and a strength check according to the provisions of AISC 341-16 [4].

Braces in diagonal braced configurations shall have the maximum slenderness limited at 200, according to chapter F2.5b of AISC 341-16 [4]: \( \lambda = \frac{L_c}{r} \leq 200 \).

The strength check for tensioned braces is according to AISC 360-10 [6]:

\[ \frac{P_u}{\varnothing_t \cdot P_n} \leq 1.0 \]

Where:
- \( L_c \) = effective length of brace; \( r \) = governing radius of gyration;
- \( P_u \) = design value of the axial effort in the considered tensioned diagonal member (produced by the seismic design situation); \( \varnothing_t = 0.9 \);
- \( P_n \) = design value of the tensile yielding strength in the gross section; \( P_n = F_y \cdot A_g \);
- \( F_y \) = characteristic value of the yielding stress of steel (minimum specified yield stress);
- \( A_g \) = gross cross-sectional area of the member.

According to chapter F.2.3 of AISC 341-16 [4], the expected brace strength in tension is \( R_y \cdot F_y \cdot A_g \), while the expected brace strength in compression is permitted to be taken as the lesser of \( R_y \cdot F_y \cdot A_g \) and \( \left( \frac{1}{0.877} \right) \cdot F_{cre} \cdot A_g \), where \( F_{cre} \) is determined from Specification Chapter E of AISC 360-10 [5] using the equations for \( F_{cre} \), except that the expected yield stress, \( R_y \cdot F_y \), is used in lieu of \( R_y \).

Where:
- \( R_y \) = ratio between the expected yield stress (average) to the specified minimum yield stress (nominal); \( R_y = 1.3 \) for steel plates, according to table A.3.1 of AISC 341-16 [4];
- \( F_{cre} \) = critical elastic buckling stress [6];

The expected post-buckling brace strength shall be taken as a maximum of 0.3 times the expected brace strength in compression. Braces with a slenderness ratio of 200 (the maximum permitted by Section F2.5b of AISC 341-16 [4]) buckle elastically for permissible materials; the value of 0.3 \( \cdot F_{cre} \) for such braces is 14 MPa (14 N/mm²).

According to chapter F2.3 AISC 341-16 [4], the required strength of columns, beams, struts and connections in special concentrically braced frames shall be determined using the capacity-limited seismic load effect. The capacity-limited horizontal seismic load effect shall be taken as the larger force determined from the following analyses:
- an analysis in which all braces are assumed to resist forces corresponding to their expected strength in compression or in tension;
- an analysis in which all braces in tension are assumed to resist forces corresponding to their expected strength and all braces in compression are assumed to resist their expected post-buckling strength.

3. Maximum design member forces

In most cases, the largest values for axial forces and bending moments recorded along the girders during static linear analyses, were observed in case of the DC and DM frames sized according to AISC 341-16.
Compared to these values, the maximum axial forces in the girders of the frames sized according to EN 1998-1:2004 [2] were on average smaller with about 21% for both considered bracing configurations. In case of the design according to P100-1/2013 [1] the values were smaller with up to 16.7% for frame DC and 17.1% for frame DM (as shown in Figure 2).

On average the values of the bending moments in the girders of frame DC were about 69.7% smaller in case of the design according to EC 8 and about 63.1% smaller in case of the design according to P100 than the ones obtained in the design according to AISC 341-16 [4]. The bending moments along the girders of frame DM, were on average smaller with less than 62.3% for the design according to P100 and smaller with up to 39.6% compared to the values recorded in case of the design according to the American code [4], (see Figure 3).

For both analysed frames the greatest axial forces and bending moments along the central columns were recorded for the design according to AISC 341-16 [4]. Comparative to the axial forces recorded in the central columns of frame DC sized according to the American code, about 6 times smaller values were noticed on average in case of the design according to EC 8 and about 42.3% smaller values in case of the design according to P100, (see Figure 4). On
average for frame DM, up to 2.7 and respectively 3.3 times smaller axial forces were recorded along the central columns in the design according to P100 and respectively EC 8, compared to those noticed in the design according to AISC 341-16 [4].

For frame DC, the maximum bending moments that could be noticed in the central columns were on average about 58.6% smaller in case of EC 8, respectively 18.5% smaller in case of P100, than the ones observed for the frame sized according to AISC. In case of frame DM, the values of the bending moments in the central columns were on average about 31.1% smaller for EC 8 and up to 22.3% smaller for P100, compared to the ones observed in case of the design according to the American code AISC 341-16 [4], as indicated in Figure 5.

**Figure 5.** Maximum bending moments along central columns.

In most situations during static linear analyses the largest values for bending moments and axial forces could be noticed in the lateral columns of frame DC sized according to the Romanian standard P100-1/2013 [1], respectively in the lateral columns of frame DM designed according to the American code AISC 341-16 [4].

**Figure 6.** Maximum axial forces along lateral columns.

**Figure 7.** Maximum bending moments along lateral columns.

The values of the maximum axial forces noticed in the lateral columns for frame DC, in case of the design according to P100, were on average up to 7.8% larger, compared to those observed after the
design according to AISC and about 1% greater than the ones obtained after the design according to EC8, (see Figure 6). The values of the maximum axial forces recorded along the lateral columns of frame DM sized according to AISC 341-16 [4], were about 10.7% smaller for the design according to P100-1/2013 [1] and up to 19.3% smaller for the sizing according to EN 1998-1:2004 [2].

The values of the maximum bending moments recorded along the lateral columns of frame DC sized according to AISC were about 18.7% greater, compared to the values recorded for the design according to P100 and up to 54.8% larger than the ones noticed for the design of frame DC according to EC 8, as shown in Figure 7. In case of frame DM, the maximum bending moments values noticed along the lateral columns of the frame, sized according to AISC 341-16 [4], were on average about 13.9% greater, compared to the ones recorded for the design according to P100-1/2013 [1] and up to 23% larger than those obtained after the design according to EN 1998-1:2004 [2].

4. Estimated steel consumption

For both analysed bracing configurations, the smallest total estimated steel consumption values were obtained for frames designed according to EN 1998-1:2004 [2], while the largest values were observed for frames sized according to AISC 341-16 [4].

![Estimated steel consumption](image)

**Figure 8.** Overall estimated steel consumption.

For frame DC, a 12% and respectively 27% larger estimated steel consumption value was noticed for the design according to P100 and respectively AISC, compared to the one according to EC 8. In case of frame DM these values were about 9% greater for P100-1/2013 [1] and up to 23.4% larger for AISC 341-16 [4], compared to the frame designed according to EN 1998-1:2004 [2], as shown in Figure 8.

5. Behaviour during static nonlinear analyses

Static nonlinear analyses were performed with each of the three designed variants of the frame. The gravitational loads were maintained constant (100% self-weight, 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 some static nonlinear analyses stage „V” appeared before stage „IV” (as shown in Figure 9 and Figure 16)! During the static nonlinear analyses unfavourable distributions of plastic hinges led in case of all three considered design variants to the development of local plastic failure mechanisms: plastic hinges formed in all the frame members connected to the same joint (see Figure 9 for frame DC sized according to AISC 341-16 [3]), respectively the development of a soft storey mechanism (see Figure 10 for frame DC designed according to EN 1998-1 [2]).
No favourable global plastic failure mechanism could be noticed 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].

For both frames, the largest base shear force values recorded during static nonlinear analyses could be noticed for the frames designed according to AISC and the smallest base shear force values could be observed for the frames designed according to EC 8.

For frame DC, about 41% smaller base shear forces were noticed for the frame sized according to EC 8 and up to 28% smaller base shear forces for the frame dimensioned according to P100, compared to the values corresponding to the sizing according to AISC, (see Figure 11). In case of frame DM, the
maximum base shear force values were about 20% lower in case of the design according to EC 8 and respectively 11% in case of the dimensioning according to P100, comparative to the frame sized according to the American code.

The largest horizontal floor displacements during static nonlinear analyses were observed for the frames sized according to the Romanian code P100-1/2013 [1], while the smallest values for the frames designed according to the American code AISC 341-16 [4] (see Figure 12). For frame DC, up to 10% smaller horizontal displacements were recorded in case of the design according to EC8 and about 25% smaller horizontal displacements were noticed in case of the dimensioning according to AISC, compared to the values recorded in case of the frame sized according to P100. The values of horizontal floor displacements noticed in case of frame DM were about 15% smaller for the sizing according to EC 8 and up to 34% smaller in case of the frame designed according to AISC, compared to the frame dimensioned according to P100.

![Figure 12. Maximum horizontal floor displacements.](image)

Analysing the graphics in Figure 13, it can be observed, that during static nonlinear analyses the largest amount of mechanical work was consumed through inelastic deformations in case of the frames sized according to P100-1/2013 [1], for both considered centrically bracing configurations. The same remark can be made by analysing the area of the P-Δ diagrams in Figure 17.

![Figure 13. Maximum horizontal floor displacements.](image)

In case of frame DC, sized according to P100, the amount of energy consumed through plastic deformations (proportional to the graphic area of the P-Δ diagram in Figure 17) was over 27% larger, compared to the value obtained for the frame designed according to EC8 and about 3% greater than the one noticed for the frame sized according to AISC. For frame DM designed according to P100, the amount of energy dissipated through inelastic deformations was about 24% smaller in case of the frame dimensioned according to EC 8 and about 30% smaller in case of the frame sized according to AISC.

In order to improve the behaviour of the frames during static nonlinear analyses [10], additional potentially plastic zones, with reduced beam cross-sections resembling the „dog-bone” detail, were provided along all frame girders at distance of about 1,0m from all column axes, as shown in Figure 14.
The reduced flanges width of the girder cross-section in the potentially plastic zones was in accordance to the provisions of Annex F of P100-1/2013 [1], (see Figure 15).

\[ 0.1 \cdot b_{\text{girder}} \leq c \leq 0.25 \cdot b_{\text{girder}} \]

Figure 15. Consumed energy through inelastic deformations [1].

By providing these additional potentially plastic zones along all frame girders, the behaviour of the frames during static nonlinear analyses was improved. No plastic deformations could be noticed outside the potentially plastic zones along the girders and columns. A global plastic failure mechanism was imposed, as indicated in Figure 16 for frame DC sized according to AISC and provided with additional potentially plastic zones. As shown in Figure 17, more favourable P-Δ diagrams were obtained for both frames sized according to AISC 341-16 [4] and provided with additional potentially zones.
Although a small reduction of the maximum base shear force values could be noticed during static nonlinear analyses for both frames provided with additional potentially plastic zones (about 3,5% for frame DC and about 5,6% in case of frame DM), much larger horizontal floor displacements could be recorded and the amount of energy dissipated through the inelastic deformation of the structures increased with over 30% for frame DC and up to 41% for frame DM (see Figure 17).

6. The necessity of providing additional potentially plastic zones

Dynamic nonlinear analyses were performed with the DC and DM frames sized according to the American code AISC 341-16 [4], provided or not with additional potentially plastic zones along the frame girders. The Vrancea 1977 earthquake acceleration record, calibrated to a peak ground acceleration value of about 0,3 times the acceleration of gravity, was used in the dynamic nonlinear analyses [11]. Rayleigh damping was taken into consideration [10, 11].

In all considered seismic design procedures, indicated in the analysed seismic design codes [1, 2, 4], no clear strength hierarchy was provided between the girders and columns of the concentrically braced frames, when subjected to strong seismic actions. The analysed concentrically braced frames had a favourable, predictable behaviour during static and dynamic nonlinear analyses, as long as inelastic deformations were concentrated only in the diagonals (see Figure 18). If the load level during static and dynamic nonlinear analyses increases, uncontrolled plastic deformations will develop along the girders and columns and the behaviour of the concentrically braced frames become difficult to predict. Unfavourable plastic hinges distributions along the frame girders may lead to unfavourable plastic hinge distributions and to the development of local plastic hinge mechanisms along the girders (see Figure 19). Uncontrolled distributions of plastic deformations along the columns of the frame may lead to unfavourable plastic hinge distributions and to the collapse of the frame (see Figure 19).
Figure 19. Uncontrolled plastic hinges along girders and columns of frame DC (Vrancea 77 acceleration record).

Providing potentially plastic zones along the girders as zones with reduced cross-sections (resembling the dog-bone detail) ensure:
- Better control of the location of plastic hinges along the girders and so the development of local plastic hinge mechanisms along the frame girders are avoided;
- Better control of the plastic rotation demand along the girders (see Figure 20);
- Higher strength of the columns compared to the girders when subjected to seismic loads and so the development of local plastic hinge mechanisms along the columns are avoided (see Figure 16);
- A favourable global plastic failure mechanism for the concentrically braced frame can be sized clearly by design (see Figure 16).

Figure 20. Better control of the plastic rotation demand along the frame girders [8, 10].

It can be observed from Figure 20 that the values of the inelastic deformations along a frame girder depend strictly on the location of the plastic hinges along the beam (respectively on the distance „L” between the two plastic hinges along the frame girder).

In case of the static and dynamic nonlinear analyses of the frames provided with additional potentially plastic zones along the frames girders, no plastic deformations could be noticed in unwanted zones, outside the considered potentially plastic zones located in the diagonals, along the frame girders ends and near the bottom of all first storey columns (see Figure 16).

7. Conclusions
For both analysed bracing configurations, the largest base shear forces and the smallest horizontal floor displacements during static nonlinear analyses were observed for the frames sized according to the American code AISC 341-16 [4]. The smallest base shear force values could be noticed in all cases for the frames designed with the European standard EN 1998-1:2004 [2]. The largest horizontal floor displacements were recorded for the frames sized according P100-1/2013 [1].

Taking into consideration the amount of energy (mechanical work) consumed through inelastic deformations in all kind of structural elements during the static nonlinear analyses, a more favourable
behaviour was encountered for the frames designed in concordance to the prescriptions of the in charge Romanian seismic design code P100-1/2013 [1].

Smaller estimated steel consumption values were noticed for the frames sized according to the European standard EN 1998-1:2004 [2]. The largest estimated material consumption was obtained for the frames designed according to the American code AISC 341-16. The differences were up to 27% for the DM bracing system and about 23% in case of the DC bracing configuration.

No favourable global plastic failure mechanism was clearly sized by design for the considered concentrically braced frames, for all the three considered seismic design codes [1, 2, and 4]. For all designed frames, at the end of the static nonlinear analyses, failure occurred through the development of a weak storey failure mechanism, or through the development of plastic hinges in all the member ends located around a node.

In about the half of the analysed frames, the out of work of all diagonals of a concentrically braced frame is preceded by the development of plastic hinges in other kind of structural members (girders and columns). When subjected to strong seismic actions, in many cases on one hand it could be noticed, that the overstrength ensured for the columns and girders relative to the capacity of the diagonals was not sufficient, stage „V” appeared before stage „IV” during some of the performed static nonlinear analyses. On the other hand, when most of the diagonal members are out of work, no hierarchy was clearly imposed by design among columns and girders under seismic loads.

Providing additional potentially plastic zones along all frame girders had favourable effects on the behaviour of the concentrically braced frames during the performed static nonlinear analyses:
- the uncontrolled development of plastic hinges along the frame girders are avoided;
- the failure of the analysed frames occurred through the development of a favourable global plastic failure mechanism;
- a greater amount of energy is consumed through inelastic deformations of the frame.

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