Comparative analysis of two seismic energy dissipation solutions in the case of a steel structure

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Abstract. The present paper aims at the comparative analysis of two seismic energy dissipation solutions for a steel structure in frames. We opted for non-braced frames and centrally braced frames. In the case of unbroken frames, the resistance structure consists of steel beams and columns. Potential plastic areas are formed mainly at the ends of the rulers and, to a lesser extent, in columns. Horizontal actions are mainly taken by bending the structural elements. To ensure the most efficient observance of the rigidity requirements for lateral actions, the solution of the rigid beam-column joint is required. The use of the concept of weak beam - strong column leads to the appearance of plastic joints in the beams, close to their joints with the columns. Centric braced frames can take over the horizontal actions, which are manifested in their plane, through axial forces. Energy dissipation occurs mainly by plasticizing the extended diagonals. The comparative analysis consists of highlighting the results obtained from the structural calculation with the help of a commercial automatic calculation program for the two solutions: non-braced frames and centrally braced frames. An office building with 2 levels was analyzed, located in Constanța, where the two types of the staff were provided in turn. The obtained results will be compared from the point of view of the amount of steel used, so from the economic point of view. Another comparison refers to the displacements to SLS and SLU and to the seismic energy dissipation capacity. Finally, another possible comparison refers to the over-resistance of the structural system.

1. Case study of steel frame structure

1.1. General data about the structure

The structure, which is to be analyzed, is made up of unbroken frames, with rigid nodes and is a steel frame with 2 stories, 3 openings and 4 beams (see figure 1). This structure is made of reinforced concrete floor, made of lost corrugated sheet steel formwork that supports a system of secondary beams.

The structure is located in Constanța and is to be designed according to the DCM ductility class. The gravitational loads on the floor are made up of the permanent load of 5 kN/m² and the payload of 3 kN/m².

The main features of the structure are summarized below: opening: L = 6 m; opening in the other direction: B = 5 m.

Floor height: h = 3.5 m; height: ground floor + 2 floors (3 levels); height: H = 10.5 m; plan size: 18 x 20 m; location: Constanța; permanent load on the floor, \( g_{k} = 5 \text{kN/m}^2 \).

The payload on the floor, \( q_{k} = 3 \text{kN/m}^2 \); the structure will be designed according to the DCM ductility class; class of importance and exposure class III, according to table 4.2 of P 100-1/2013; the factor of importance: \( \gamma_{I,e} = 1.0 \) [1].
The structural calculation is performed with an automatic calculation program. In this study, the second order calculation and imperfections, were considered. SR EN 1993-1-1 presents three methods for taking into account second-order effects and imperfections. The third method was adopted: for basic cases, by individual stability checks of equivalent bars, using buckling lengths corresponding to the overall mode of instability of the structure.

1.2. Calculation of strength and stability of the steel frame structure contains. Verification of the strength and general stability of the beams

SR EN 1993-1-1 [2] will be used for the verification and conformity of the beams to the general stability in the hypothesis that "only at one end a plastic joint has been formed" (1, 2, 3).

\[
\frac{M_{Ed}}{M_{pLRd}} \leq 1.0
\]  
\[
\frac{N_{Ed}}{N_{pLRd}} \leq 0.15
\]  
\[
\frac{V_{Ed}}{V_{pLRd}} \leq 0.5
\]

**Figure 1.** 3D spatial model of steel frame structure.
1.3. Calculation of the super resistance of the structural system

The over-resistance (table 1, table 2) of the structural system, used in the calculation of the stresses in the non-dissipative components, is determined by the relation:

\[
\Omega_T = 1.1 \cdot \gamma_{ov} \cdot \Omega^M
\]  

(4)

\[
\Omega^M = \min(\Omega^M_i)
\]  

(5)

\[
\Omega^M_i = \frac{M_{pl,Rd}}{M_{Ed}}
\]  

(6)

\(\Omega_T\) it is the value of the super-resistance of the structural system. 
\(\Omega^M\) minimum value of \(\Omega^M_i\) calculated for all beams with the potential plastic area; 
\(M_{pl,Rd,i}\) is the plastic resistance of the projectors in the beam \(i\); 
\(M_{Ed,i}\) this is the bending moment in the beam and in the load group (figure 2).

| Opening | Level | \(M_{Ed}\) | \(M_{pl,Rd}\) | \(M_{Ed}/M_{pl,Rd}\) | \(\Omega^M_{i,Rd}/M_{Ed}\) | \(\Omega^M\) | \(\Omega^M/\Omega^M\) (%) | \(\Omega^M_T\) |
|---------|-------|-----------|----------------|----------------------|------------------------|---------|-----------------|---------|
| A-B     | 1     | 194.9     | 696.7          | 0.28                 | 3.57                   | 0.18    | 4.91            |         |
|         | 2     | 204.7     | 696.7          | 0.29                 | 3.40                   | 0.00    | 0.12            |         |
| P       | 1     | 174.4     | 696.7          | 0.25                 | 3.99                   | 3.4     | 1.99            | 36.94   |
|         | 2     | 183.3     | 696.7          | 0.26                 | 3.80                   | 4.00    | 0.40            | 10.55   |
| B-C     | 1     | 145.3     | 696.7          | 0.20                 | 4.79                   | 1.39    | 29.06           |         |
|         | 2     | 195.3     | 696.7          | 0.28                 | 3.56                   | 0.17    | 4.67            |         |
| C-D     | 1     | 204.9     | 696.7          | 0.29                 | 3.39                   | 0.00    | 0.00            |         |
|         | 2     |           |                 |                      |                        |         |                 |         |
Table 2. Super resistance calculus $\Omega_T$, Y direction.

| Openinglevel | $M_{Ed}$ | $M_{Ed,Rd}$ | $M_{Ed}/M_{Ed,Rd}$ | $\Omega_M^{M}$ | $\Omega^M$ | $\Omega^M - \Omega^M$ (%) | $\Omega_T$ |
|--------------|----------|-------------|---------------------|----------------|---------|--------------------------|---------|
| 1-2          | 2        | 165.71      | 548.5               | 0.302          | 3.310   | 0.52                     | 15.84   |
| P            | 1        | 191.33      | 548.5               | 0.330          | 3.026   | 0.24                     | 7.94    |
| 2            | 2        | 165.88      | 548.5               | 0.349          | 2.867   | 0.08                     | 2.82    |
| 2-3          | 1        | 187.96      | 548.5               | 0.302          | 2.918   | 0.13                     | 4.54    |
| P            | 1        | 196.09      | 548.5               | 0.358          | 2.797   | 0.01                     | 0.41    |
| 2            | 2        | 166.82      | 548.5               | 0.304          | 3.288   | 0.50                     | 15.27   |
| 3-4          | 1        | 189.08      | 548.5               | 0.345          | 2.901   | 0.12                     | 3.97    |
| P            | 1        | 196.89      | 548.5               | 0.359          | 2.786   | 0.00                     | 0.00    |
| 2            | 2        | 164.31      | 548.5               | 0.300          | 3.338   | 0.55                     | 16.55   |
| 4-5          | 1        | 179.76      | 548.5               | 0.328          | 3.051   | 0.27                     | 8.70    |
| P            | 1        | 190.84      | 548.5               | 0.348          | 2.874   | 0.09                     | 3.07    |

In the X direction, the maximum and minimum values of $\Omega_M^M$ differ by $14.89\% < 25\%$. Exceptions are made to the values obtained for the beams from the last level, which results in being oversized.

In the Y direction, the maximum and minimum values of $\Omega_M^M$ differ by $16.55\% < 25\%$.

Figure 3. Bending moment diagram, $M_{Ed}$-Ax 5; Ax C.

1.4. Verification of the strength and general stability of the columns

The verification at ULS of the columns on the ground floor is simplified for the most requested element, located within axis 5.

Calculation forces for columns and determination according to the relations (7, 8, 9) from P 100-1/2013.

\[
N_{Ed} = N_{Ed,G} + \Omega_T N_{Ed,E}
\]

\[
M_{Ed} = M_{Ed,G} + \Omega_T M_{Ed,E}
\]

\[
V_{Ed} = V_{Ed,G} + \Omega_T V_{Ed,E}
\]

$\Omega_T$ is the value of the super-resistance of the structural system; $\Omega_T = 3.0$ (P100-1/2013, Annex F- Table F.1)
We had the following results:

\[ M_{y,Ed,1} = 357.31 \text{ kNm}; \quad M_{z,Ed,1} = 49.18 \text{ kNm}; \quad M_{y,Ed,2} = 3.09 + 331.33 = 334.42 \text{ kNm}; \]

\[ M_{z,Ed,2} = 0 + 107.17 = 107.17 \text{ kNm} ; \quad N_{Ed} = 1.13 + 727.84 = 728.97 \text{ kNm} ; \quad V_{z,Ed} = 484.76 + 399.64 = 884.4 \text{ kN} ; \quad V_{y,Ed} = 9.26 + 66.82 = 76.08 \text{ kN} \]

(figure 4).

The plastic resistances of the design of the section are:

\[ M_{pl,Rd} = \frac{W_{pl,y} \cdot f_y}{\gamma_M} = \frac{7933 \cdot 275}{1.1} = 1983.25 \text{ kNm} \] (10)

\[ N_{pl,Rd} = \frac{A \cdot f_y}{\gamma_M} = \frac{35440 \cdot 275}{1.1} = 8860 \text{ kN} \] (11)

\[ V_{pl,z,Rd} = \frac{A_{v,z} \cdot f_y}{\sqrt{3} \cdot \gamma_M} = \frac{13960 \cdot 275}{\sqrt{3} \cdot 1.1} = 2014.95 \text{ kN} \] (12)

Thus, the relations of specific provisions P100 could be verified.

Figure 4. Bending moment diagram, \( M_{y,Ed,1} \).

Other calculations made refer to: bending and compression checking; checking the buckling of the columns; checking the beam-column joints [3-6].

For M-N interaction: \( k_{xy} = 0.79; \quad k_{yx} = 0.53; \quad k_{xy} = 0.42; \quad k_{zx} = 0.78 \) and applying the verification relationship results: \( 0.05 + 0.13 + 0.01 = 0.19 < 1.00 \) (19%) and \( 0.06 + 0.07 + 0.02 = 0.14 < 1.00 \) (14%).

1.5. Checking the beam-columns joints

The joints of the beams with the columns, at a projected structure that dissipates the seismic energy in the beams, must be designed to work in the elastic domain during the earthquake.

The total rotational capacity of the beam-column node, denoted by, must be ensured at cyclic loads, without producing degradations of strength and rigidity greater than 20%.

1.6. Vibration modes

10 own modes of vibration were taken into account. Own vibration periods \( T \) and effective modal masses \( M \) in relation to the total mass of the structure are presented in table 3.
It can be observed that the sum of the effective modal masses from the first 4 eigenmodes of vibrations exceeds 90% of the total mass of the structure, being fulfilled the requirement in section 4.5.3.3 of P 100-1.

The masses of the structure were calculated automatically from the gravitational loads applied to the structure.

**Table 3. Mass on levels.**

| Level            | \( \varphi \) | \( \Psi_{2,i} \) | \( \Psi_{E,i} \) | \( G_{k,j} \) (kN/m²) | \( Q_{k,i} \) (kN/m²) | Surface (m²) | Mass (tons) |
|------------------|---------------|------------------|------------------|------------------------|------------------------|--------------|-------------|
| Ground floor     | 0.8           | 0.3              | 0.24             | 5                      | 3                      | 180          | 104.95      |
| Floor 1          | 0.8           | 0.3              | 0.24             | 5                      | 3                      | 180          | 104.95      |
| Floor 2          | 1             | 0.3              | 0.3              | 5                      | 3                      | 180          | 108.26      |
| Total            |               |                  |                  |                        |                        |              | 318.17      |

**Table 4. Center of mass and rigidity on the floor.**

| Number of level | Level name      | Center of mass | Center of rigidity |
|-----------------|-----------------|----------------|--------------------|
|                 |                 | X              | X                  |
|                 |                 | Y              | Y                  |
| 1               | Ground floor    | 9.00           | 9.00               |
|                 |                 | 10.00          | 10.00              |
| 2               | floor 1         | 9.24           | 9.00               |
|                 |                 | 10.29          | 10.00              |
| 3               | floor 2         | 9.21           | 9.00               |
|                 |                 | 10.26          | 10.00              |

The deformed structure in the first proper way of vibration is presented in figure 5.

Modal mass on X direction is 999.74 t from 1202 t meaning 83% (vibration mode 1). Modal mass on Y direction is 994.9 t from 1202 t meaning 82.7% (vibration mode 2).

**Figure 5.** Modal Characteristics Mode 1; \( T = 0.54 \text{s} \).
Table 5. Check displacements at the service limit state (SLS).

| level          | Direction X | Direction Y | ν  | q  | d_rе, SLS | d_rа, SLS | check |
|----------------|-------------|-------------|----|----|-----------|-----------|-------|
| floor 2        | 0.0125      | 0.0001      | 0.5| 4  | 0.0250    | 0.0002    | 0.0263 ok |
| floor 1        | 0.0131      | 0.0002      | 0.5| 4  | 0.0262    | 0.0004    | 0.0263 ok |
| Ground floor   | 0.0099      | 0.0004      | 0.5| 4  | 0.0198    | 0.0008    | 0.0263 ok |

\[ d_rе, SLS = 0.0262 < d_rа, SLS = 0.0263 \rightarrow \text{is verified} \]

Table 6. Check displacement at last limit state (SLU).

| level          | Direction X | Direction Y | c  | q  | d_rе, ULS | d_rа, ULS | check |
|----------------|-------------|-------------|----|----|-----------|-----------|-------|
| floor 2        | 0.0115      | 0.0001      | 1.074| 4  | 0.0494    | 0.0004    | 0.0875 ok |
| floor 1        | 0.0131      | 0.0002      | 1.074| 4  | 0.0563    | 0.0009    | 0.0875 ok |
| Ground floor   | 0.0099      | 0.0004      | 1.074| 4  | 0.0425    | 0.0017    | 0.0875 ok |

\[ d_rе, ULS = 0.0563 < d_rа, ULS = 0.0875 \rightarrow \text{is verified} \]

2. Calculation of strength and stability of the steel frame structure with diagonals

2.1. General data on structure
The structure, which is to be analyzed, is made up of a centrally braced X-shaped frame arranged in the transverse axis 1 and 5 (X direction) and in the longitudinal axis A and D (Y direction). The steel frame has 2 floors, 3 openings and 4 open beams (see figure 6). This structure is made of reinforced concrete floor, made of lost corrugated sheet steel formwork that supports a system of secondary beams. The structure is located in Constanta and is to be designed according to the DCM ductility class.

Figure 6. The spatial model of steel frame structure with diagonals in the calculation program.
The diagonals were dimensioned according to Eurocode [2].

2.2. Calculation of the super resistance of the structural system

Table 7. Calculation of the super resistance $\Omega_T$, on the X direction.

| Opening | level | $N_{Ed}$ | $N_{pl,Rd}$ | $N_{Ed}/N_{pl,Rd}$ | $\Omega_i^N = N_{pl,Rd}/N_{Ed}$ | $\Omega^N$ | $\Omega^N - \Omega_0^N$ (%) | $\Omega_T$ |
|---------|-------|-----------|--------------|---------------------|---------------------------------|-----------|-----------------------------|-----------|
| B-C     | 2     | 117.22    | 531          | 0.221               | 4.53                            | 1.53      | 33.87                       |           |
|         | 1     | 177.27    | 531          | 0.334               | 3.00                            | 0.00      | 0.00                        | 4.28      |
| P       | 143.21| 531       | 0.270        | 3.71                | 0.71                            |           | 19.21                       |           |

Table 8. Calculation of the super resistance $\Omega_T$, on the Y direction.

| Opening | level | $N_{Ed}$ | $N_{pl,Rd}$ | $N_{Ed}/N_{pl,Rd}$ | $\Omega_i^N = N_{pl,Rd}/N_{Ed}$ | $\Omega^N$ | $\Omega^N - \Omega_0^N$ (%) | $\Omega_T$ |
|---------|-------|-----------|--------------|---------------------|---------------------------------|-----------|-----------------------------|-----------|
| B-C     | 2     | 82.22     | 531          | 0.155               | 6.46                            | 2.25      | 34.90                       |           |
|         | 1     | 126.29    | 531          | 0.238               | 4.20                            | 0.00      | 0.00                        | 6.01      |
| P       | 109.61| 531       | 0.206        | 4.84                | 0.64                            |           | 13.21                       |           |

The difference between the maximum and minimum ratio $\Omega_i^N$ is less than 25% for each direction of the structure. In the X direction this difference is 19.21%, and in the Y direction it is 13.21%. Exceptions to the rule make diagonals to the 2nd floor, which results in being oversized.

Other calculations made refer to: the buckling of the column; compression and bending verification; calculation of joints of dissipative elements (bracing) [3, 4].

2.3. Joints of dissipative elements (braces)

A gusseted welded joint was adopted to allow buckling of the bracing in out of plan [5]. The buckling is realized by bending a portion of the gusseted with a width equal to 2 times its thickness, $t_g$. The joint must be designed to have a resistance of at least $1.1 \cdot \gamma_{ov}$ to the joined element (bracing), according to sections 6.7.3 and 6.5.5 of P 100-1.

The joint will be dimensioned according to SR EN 1993-1-8 to the axial force:

$$N_{Ed,imb} = 1.1 \cdot \gamma_{ov} \cdot N_{pl,Rd} = 1.1 \cdot 1.3 \cdot 531 = 759.33 \ kN$$

3. Conclusions

One of the principles of anti-seismic design is the constraint of a favorable structural mechanism for energy dissipation (plasticization mechanism) under high-intensity seismic actions, thus preventing the collapse.

In the first case of a steel frame structure, the dissipation of the seismic energy takes place through the formation of the plastic joints at the ends of the beams, representing the main dissipative elements. The buckling of the element, the suppleness of the sections, and the presence of important forces of compression and/or shearing influence the appearance of the plastic capable bending and providing the capability of rotation. In order that the capable bending moments of the section and its rotational capacity are not diminished, the axial force was limited to 15% of the plastic axial force of the section, and the shear force to 50% of the capable plastic cutting force.

The large bending moments at the ends of the beams led to substantial deformations of the core panels of the columns at the right of the joint (by cutting force). These deformations led to important
displacements, and to reduce lateral displacements and prevent structural damage, it was necessary to use elements (columns and beams) with larger dimensions.

The most unfavorable relative displacement, at SLS, is $d_{SLS}^{r} = 2.62 \text{ cm}$, and at ULS $d_{ULS}^{r} = 5.63 \text{ cm}$. In case two, steel frame structure with diagonals, the dissipative zones are located in diagonally, when they are stretched.

For steel structure with diagonals were used steel profiles HEM550 for columns, IPE 550 for main transverse beams, IPE500 for longitudinal main beams, and IPE 300 for secondary beams, resulting in a total weight of 124 tons. The period of the fundamental vibration mode is $T_1 = 0.54 \text{ s} < T_c = 0.7 \text{ s}$. The sum of the effective modal masses exceeds 90% of the total mass of the structure from the first 4 eigenmodes of vibrations.

The most unfavorable relative displacement, at SLS, is $d_{SLS}^{r} = 2.62 \text{ cm}$, and at ULS it is $d_{ULS}^{r} = 5.63 \text{ cm}$.

In the case of frames with diagonals, the dissipative zones are located in diagonal, when they are stretched. The bracing required for the stretching has been dimensioned in such a way that the capable plastic effort of the $N_{pl,kd}$ cross-section is greater than the computational effort of the $N_{Ed}$ seismic combination. The presence of bracing diagonals causes the system to be loaded mainly with axial forces, having smaller bending moments in the beams and implicitly smaller dimensions of these.

For the steel frame structure with diagonals, HEB 500 steel profiles were used for columns, IPE 400 for main beams, IPE330 for longitudinal beams, IPE 200 for total beam and for diagonal beams, and HEA100 for diagonals, resulting in a total weight of 81 tons.

The period of the fundamental vibration mode is $T_1 = 0.48s < T_c = 0.7s$. The sum of the effective modal masses exceeds 90% of the total mass of the structure in the first 5 eigenmodes of vibration.

The most unfavorable relative displacement, at SLS, $d_{SLS}^{r} = 1.96 \text{ cm}$, and at ULS it is $d_{ULS}^{r} = 4.82 \text{ cm}$. The steel frame structure (first case) will have a better behaviour to the seismic action, it presents a high ductility, ensuring the dissipation of a large amount of energy, but it does not require larger sections of the elements. It is preferred by architects as it does not condition the positioning of gaps for windows on the facade.

The centrally braced structure has a limited ability to dissipate seismic energy. The existence of diagonals makes the beam sections smaller, and therefore an advantage in terms of economic value. It is not preferred by architects as it conditions the positioning of the gaps for windows on the facade.

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