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ANALYSIS OF LATERAL DEFLECTION OF STEEL MOMENT RESISTING FRAME

Steel Moment Frames, to meet certain lateral stability criteria, can often be considered overdesigned [1]. In this paper, an assessment of the behavior of steel frames under the influence of dynamic loads is made. By applying the criteria for capacitive design, proposed by Eurocode 8, the dimensioning of a 2D regular Moment Frame was carried out.

Special emphasis is placed on the implementation and interpretation of the rationality of the interstory drift sensitivity coefficient θ. To assess the rationality of the coefficient θ, a non-linear static analysis of the 2D Moment Frame was performed. In the analysis, the geometric and material nonlinearities of the elements are included using the concept of distributed plasticity. The stiffness of the beam-column connection is modeled as ideally rigid and its behavior is taken into account in the analysis. The results obtained in this way are compared with the criteria proposed by Eurocode 8.

Keywords: Steel Moment Frames, interstory drift sensitivity coefficient θ, nonlinear static analysis

1. INTRODUCTION AND MAIN CONCEPTS

Steel frames are sensitive to lateral deflections and these effects have quite an impact in the distribution of horizontal displacements and internal forces. The stresses resulting from these influences further increase the influences obtained from linear elastic analysis. The second order effects, which are obtained from these additional lateral displacements, during linear seismic analysis are taken through the sensitivity coefficient of relative storey displacements i.e θ–coefficient. This coefficient, according to the current regulation of Eurocode 8, is determined based on the expression [3]:

$$\theta_i = \frac{P_{tot,i} \cdot d_r}{V_{tot,i} \cdot h_i} \tag{1}$$

According to Eurocode 8 [3], second-order effects should not be taken into account for frames where θ ≤ 0.1, while for θ ∈ [0.1-0.2], P-

Δ effects should be taken into account by amplifying the effects according to the expression [2]:

$$\alpha = \frac{1}{1-\theta} \tag{2}$$

2. INTERPRETATION AND THE RATIONALITY OF THE CRITERION FOR THE COEFFICIENT Θ

If a linear elastic behavior of the structure is assumed until failure, then the coefficient θ can be represented in the following form:

$$\theta_i = \frac{P_{tot,i} \cdot d_e}{V_{tot,i} \cdot h_i} \tag{3}$$

The criterion from equation (1) applied to expression (3) means that the moment generated by gravity loads ($P_{tot,i}$) in the i-th floor is less than 10% of the elastic moment load generated by the lateral stiffness of the structure [7]. Lateral stiffness is determined by the following expression:

$$k_j = \frac{V_{tot,i}}{d_e} \tag{4}$$

The above statement, n algebraic form, can be represented by the following equation:

$$\begin{aligned} \theta_i^{el} &= \frac{P_{tot,i} \cdot d_e}{V_{tot,i} \cdot h_i} \\ &= \frac{P_{tot,i}}{h_i} \cdot \left(\frac{d_e}{V_{tot,i}}\right) \\ &= \left(\frac{P_{tot,i}}{h_i} \cdot \frac{1}{k_j}\right) \leq 0.1 \end{aligned} \tag{5}$$

The criterion proposed by Eurocode 8 uses the secant lateral stiffness, i.e. expression (4) should be reduced by an appropriate q-factor, i.e.:

$$k_j = \frac{V_{tot,i}}{\boldsymbol{q} d_e} \tag{6}$$

However, the limit for the sensitivity coefficient (θ) remains at 0.1. Parallel with the coefficient Θ_i, to determine the elastic instability of the structure from vertical loads, Eurocode 3 proposes the following delimitation [2]:

$$\alpha'_{cr} = \frac{1}{\alpha_{cr}} \leq 0.1 \ \left(\alpha_{cr} = \frac{F_{cr}}{F_{Ed}} \geq 1\right) \tag{7}$$

The coefficient α'_{cr} for linear static analysis (Eurocode 3), corresponds to the coefficient θ_i proposed by Eurocode 8. For example, if a construction with a behavior factor q = 5 is considered, the same condition considered in an elastic area (under the influence of seismic loads) would implied θ_i^{el} = 0.1/5 = 0.02. From the last one, one can clearly notice the conservatism of this approach.

3. LINEAR STATIC ANALYSIS ACCORDING TO EUROCODE

The frame presented in Figure 1 is analyzed. The span between the columns is equal to 8m. The height of each level is 3m. For beams and columns, steel of grade S235 with $\gamma_{ov} = 1.25$ is assumed, as suggested by Eurocode 8. The cross-section of all beams is assumed to be IPE400, while for intermediate columns, HEA340 cross section is assumed. The peripheral columns are selected with a cross-section of HEA280. The intensity of constant loads and variable loads for each level is given in Figure 1.

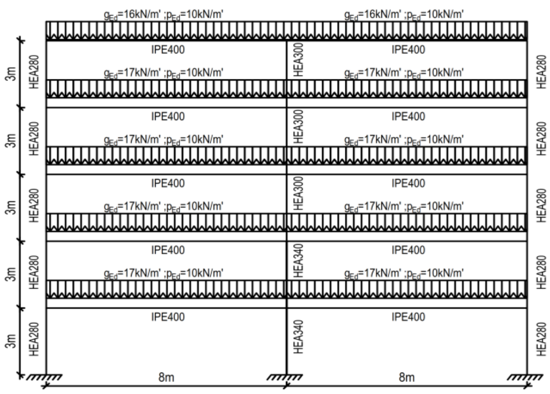

Figure 1. Analysed steel moment frame.

Due to the regularity of the frame in height, in this paper the method of equivalent lateral horizontal force was used for the linear elastic analysis.

3.1 STRONG COULMNS/WEAK BEAMS CRITERION

In order to avoid the formation of a mechanism at the local level in the columns (the so-called soft story/flexible floor), that is, to achieve a global ultimate mechanism of the structure, according to the recommendations of Eurcode 8, it is necessary to fulfill the following criteria [3]:

$$\sum W_{pl,c} > 1.3 \sum W_{pl,b} \tag{8}$$

$$2\sum W_{pl,c} > 1.3\sum W_{pl,b} \quad (9)$$

Where $W_{pl,c}$ and $W_{pl,b}$ denote the plastic bending modulus of the columns and beams, respectively. Expression (8) is used for the middle joints while expression (9) is used for the peripheral joints. Results for criterion: "Strong columns/weak beams".

Table 1. Results for criterion: "Strong columns/weak beams"

k	$W_{pl,HEA340}$ =[1826cm^3]
$W_{pl,IPE400}$	$W_{pl,HEA340}/W_{pl,IPE400}$= 1.42>1.3
k	$W_{pl,HEA280}$ =[1088cm^3]
$W_{pl,IPE400}$	$2W_{pl,HEA280}/W_{pl,IPE400}$= 1.68>1.3

3.2 SECOND ORDER EFFECTS- (ULS)

In the following table (Table 2) are shown the values for the θ coefficient. According to the calculated values for the coefficient θ, it is noted that in storey 2, the value of $\theta = 0.13 > 0.1$. In other levels, the θ-coefficient for this example has values less than 0.1. Based on the current Eurocode 8, second-order effects are taken into account by amplifying the seismic actions.

Table 2. Values for θ coefficient according to linear static analysis

Story	$d_r = qd_e$	θ
1	37.2	0.09
2	55.2	**0.13**
3	49.7	0.10
4	37.8	0.07
5	20.8	0.037

3.3 CODE REQUIREMENTS FOR COLUMNS AND BEAMS

Depending on the cross-sectional class limitations, which is correlated with the q-factor, in order to avoid exceeding the plastic load-bearing and rotation capacity in a location where plastic hinges are expected to form, as a result of the mutual action of moments, the transverse and axial forces, Eurocode 8 also proposes some limits [3].

For a seismic combination, the applied moments, transverse forces, and axial forces in the beams are determined according to Equations in [3], where, to account for the second-order effects, $M_{Ed,E}$, $V_{Ed,E}$ and $N_{Ed,E}$ are amplified by 1/(1-θ). To achieve the "Strong Columns/Weak Beams" criterion, the calculated internal forces in the columns, from the elastic model, should be amplified by means of the coefficient Ω, which for Moment Frames (MRF) is given by the expression:

$$\Omega = \min\left(\frac{M_{pl,Rd,j}}{M_{Ed,j}}\right) \quad (10)$$

3.4 DAMAGE LIMITATION- SLS

The damage limitation criterion is checked for an earthquake with a probability of occurrence higher than the design earthquake. In this paper, in order to better understand the boundary behavior of the system, the limit of α = 1% is assumed, that is, the non-constructive elements are separated from the moment frame. The criterion is checked according to the equation [5]:

$$\alpha = \frac{d_e \cdot q \cdot v}{h} \quad (11)$$

Where, d_e, q and h are linear inter-storey drift, behavior factor and the inter-storey height, respectively. In the following, a tabular presentation of the relative floor displacements is given in accordance with the provisions of Eurocode 8. The factor v, which reflects the return periods of seismic actions, is assumed with a value of v = 0.5. Consequently, the structural damages from an earthquake from the SLS condition are "v" times smaller than the damage from an earthquake from the ULS condition.

Table 3. Damage limitation check for the analyzed frame

Level	h[mm]	$\frac{v \cdot q \cdot d_e}{h}$ (%)	Performance limit (%)
1	3000	0.62	α =1
2	3000	0.92	
3	3000	0.82	
4	3000	0.63	
5	3000	0.34	

According to the results shown, the closest to the damage limit are the relative storey displacements of level 2, which are 0.92<1 = α. Given that the θ coefficient in level 2 has the highest value (θ = 0.13), such values were expected.

4. NON LINEAR STATIC ANALYSIS

In this paragraph, the example analyzed in point 3 is reanalyzed using non-linear static analysis. The non-linear static analysis was carried out with the software package SeismoStruct 2018. To simulate S235, the monoaxial model initially programmed by Yassin [1994] [10], which is based on the dependence proposed by Menegotto [7], [9], was used. The non-linear behavior of the steel is simulated with a reinforcement factor of 0.005.

The post-elastic behavior of beams and columns is simulated using the DP-Distributed plasticity formulation [4]. This is achieved by modeling the elements as non-elastic elements along their entire length and their non-linear properties are incorporated at the cross-sectional level. The global plastic behavior of the structure is obtained by integrating all cross-sections using shape functions built into the software package itself.

4.1 BEHAVIOUR OF GLOBAL SYSTEM

The global behavior of the system is represented by the Force-Displacement curve where a series of parameters are incorporated, such as: Lateral load at the appearance of the first plastic joint F_y, development and location of plastic joints and plasticization of the elements, Ultimate capacity F_u and the corresponding displacements at the occurrence of the first plastic hinge Δ_y and ultimate displacement Δ_u.

In Figure 2. Pushover curve for the considered construction is shown. The segment bounded by points O-A describes the linear-elastic behavior up to the limit of occurrence of the first plastic hinge (point A). The lateral load at the appearance of the first plastic hinge is F_y=381.72kN, while the corresponding displacement is Δ_y=118mm. The total design seismic load (including the effects of accidental torsion) from the linear elastic analysis is

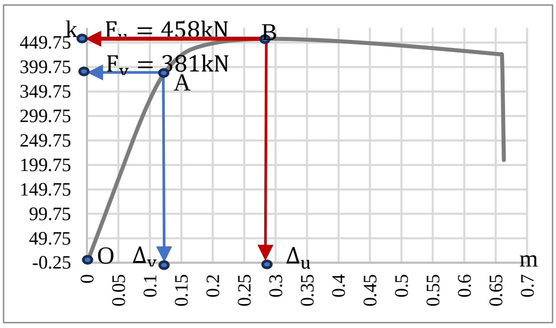

Figure 2. The Push-over curve for analysed steel moment frame.

S_{de}=188kN. Namely, for a seismic force almost 200% of the design action, the structure would remain in the linear elastic region.

The A-B portion of the curve is generated as a result of the plastic capacity to redistribute impacts until collapse is reached [6]. The ultimate state of the structure was reached from a total horizontal action of F_u=458kN and a corresponding displacement of Δ_y=291mm. The remaining part of the curve is called the softening branch and its gradient is functionally dependent on: The more sensitive the frame is to lateral loads, the steeper the drop will appear in the Push-Over Curve. In this case, a full mechanism is reached for a total displacement of 650mm.

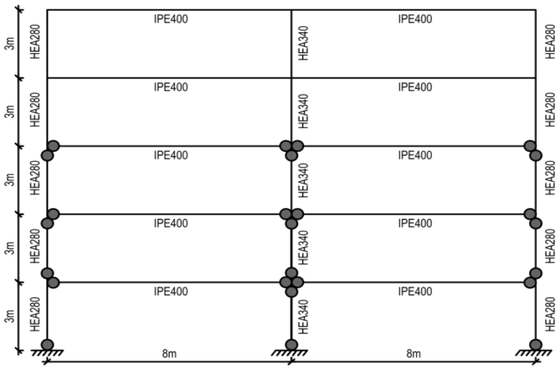

Figure 3. Distribution of plastic hinges over the MRF

Figure 3. shows the final stage of all plastic hinges in the construction while their development process is as follows: The first plastic hinge is registered in the first storey beam of the node with the peripheral column HEA280. The next plastic hinge is registered in the same vertical of the second storey. For columns, the first plastic joint is registered in the peripheral column HEA280 at the base of the structure. This is also a desirable way of initiating plasticization. Figure 3. shows the complete development of plastic joints and it can be easily noticed that in the last 2 levels of the frame, up to its ultimate state, no yielding of the elements was registered. This procedure leads to thinking about how to properly treat the "Strong columns/weak beams" criterion, which does not make a difference for the levels of the construction but must be fulfilled for each joint of the moment frame.

4.2 CALCULATION OF BEHAVIOUR FACTOR ACCORDING TO NONLINEAR STATIC ANALYSIS RESLUTLS

Based on the parameters extracted from Figure 2, the behavior factor for this construction is determined according to the Table 4. There is

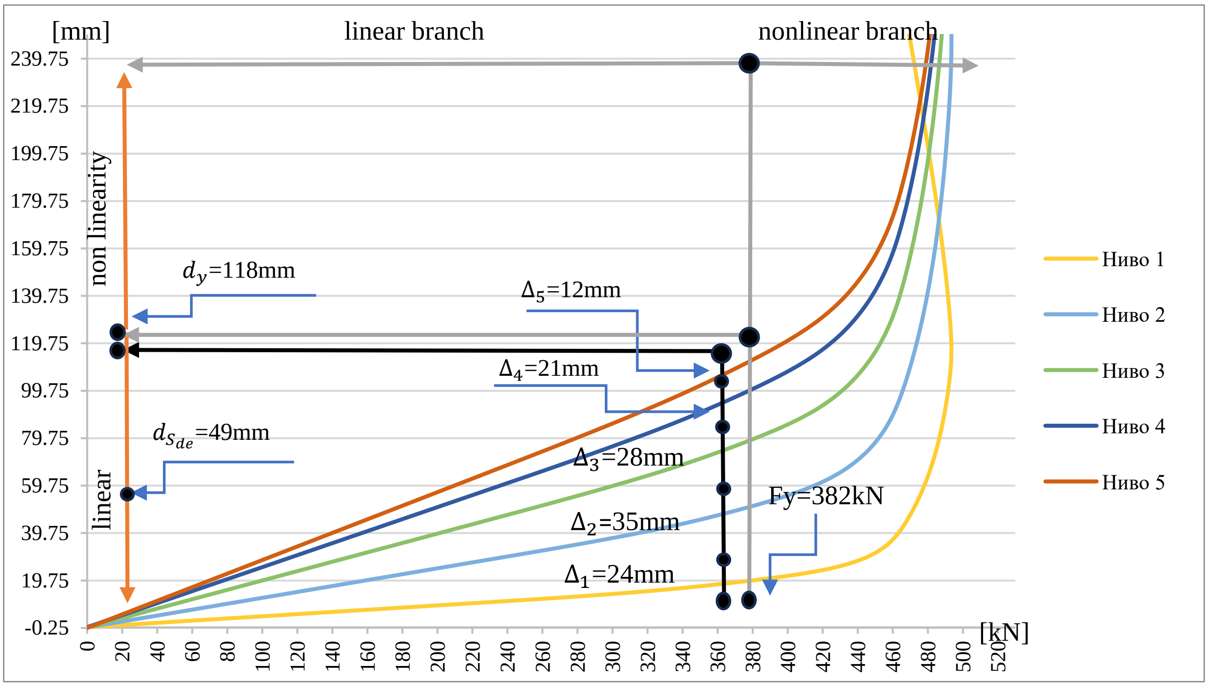

Figure 4. Distribution of lateral displacement of analyzed steel frame.

quite a large increase in the calculated value of the q-factor with the assumed value of q = 4. This is due to the solid ductility capacity of the system, which for structures with the period $T > T_c$ is calculated according to the expression: $\mu = \frac{\Delta_u}{\Delta_y} = 2.46$, as well as due to the over strength factor, α = 1.2.

Table 4. Values for θ coefficient according to linear static analysis

F_b [kN]	F_y [kN]	F_u [kN]	$\alpha = F_u/F_y$	Δ_y [mm]	Δ_u [mm]	**q**
188	381.72	458	1.202	118	291	**6.5**

4.3 ANALYSIS OF LATERAL DISPLACEMENTS BY NONLINEAR STATIC ANALYSIS

Figure 4 shows the horizontal displacements of each storey under the influence of a lateral horizontal load. It is observed that the maximum relative storey displacements are registered for storey 2. As the frame levels progress, the curves are closer together. In level 2, a relative storey displacement of 35mm is observed and at that moment, the structure is already in a linear phase. In the case of moment frames, this criterion, in a large number of cases, is crucial for the selection of the cross-sections of the columns, so its interpretation has a key implication of the economy of the constructive solution. The existing Yugoslavian standards [8] for limiting horizontal displacements of the structural system, i.e H/500 (H-total height of the structure), for this example would dictate maximum displacements of 30 mm. On the other hand, the structure up to displacements of 118 mm is completely in the linear elastic region, although there are no special restrictions due to non-structural elements, such an approach would lead to an uneconomical solution.

5. CONCLUCSION

Using the principles of non-linear static analysis, an analysis of a Steel Moment Frame was performed which was pre-solved according to the provisions proposed by the current generation of Eurocode 3 and 8.

From the performed analyses, the following conclusions were drawn: The coefficient for sensitivity to floor displacements, calculated according to the current Eurocode 8 for level 2, requires the inclusion of second-order effects through the amplification of internal forces. On the other hand, according to the non-linear static analysis, the global behavior of the system is linear even for horizontal force up to F=380kN (2.02 times the design seismic force S_{de}=188kN). If the current formulation for the θ-coefficient is used, for level 3 , the inter-story displacements of 12.5mm imply a θ-coefficient with a value of 0.1 (Table 2). This is at the limit of including second-order effects. On the other hand, in level 3, for a load of 305kN, the

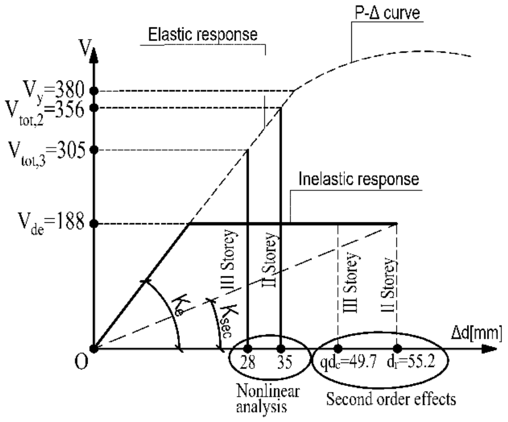

Figure 5. Display of horizontal displacements according to bilinear diagram (EC8) and non-linear static analysis.

considered structure (according to non-linear static analysis) behaves linearly (Figure 4 and Figure 5). This means that the relative storey displacements of 12.5(305/155.3) = 24.59 mm would belong to the area of linear elastic behavior. According to the conducted Pushover analysis, relative storey displacements up to 35mm imply elastic behavior of the structure in ULS condition. But according to current regulations, for displacements greater than 24.59 mm, it is necessary to take into account second-order effects through appropriate amplification (Table 2). Figure 5 shows how with this definition of secant stiffness according to EC8, displacements are obtained that lead to larger values for the θ-coefficient due to large values of inelastic displacements ($d_r = d_{eq}$).In the new generation of Eurocode 8, in the expression for θ, V_{tot} is amplified by the coefficient $k = q_S q_R$ which practically leads V_{tot} to the level of Significant Damage. For this example, significant damage can be considered the load close to the occurrence of first plastic hinge of the second storey beams. Given that the ratio V_y/S_{de} = 380/188 = 2.06, it follows that the initial values obtained for the θ-coefficient should be reduced by a value close to 2.

REFERENCES

[1] Denavit. M et al.: ,,Seismic performance factors for moment frames with steel- concrete compoiste columns and steel beam“. Earthquake engineering&structural dynamics. Wiley Online Library. DOI: 10.1002/eqe.2737.

[2] Eurocode 8: Design of Structures for Earthquake Resistance- Part 1: General rules, and rules for buildings.

[3] Eurocode 3: Design of steel structures - Part 1-1: General rules and rules for buildings.

[4] Gharakhanloo. A: “Distributed and concentrated inelasticity beam-column elements used in earthquake engineering”. Master Thesis, Civil and Environmental Engineering, NTNU-Tronheim, June 2014.

[5] Landolfo. R et al. :,,Design of steel structures for building in seismic areas”. Eurocode 8- Design of structures for earthquake resistance. Part 1-1- general rules, seismic actions and rules for buildings. ECCS. Wiley, 2016.

[6] Mazzolani. F, Piluso. V: ,,Theory and design of seismic resistanat steel frames”. Spon Press. Tylor & Francis Group, 1997.

[7] Menegotto M. et al (1973). „Method for analysis for cyclically loaded RC plane frames including changes in geometry and nonelastic behaviour of elements under combined normal forces and bending moments“. 15-22. Zurich, Switzerland.

[8] Pravilnik o Tehnickim Normativa Za Izgradnju Objekta Visokogradnje u Seizmickim Podrucijma, „Sluzbeni list SFRJ“. Br31/81, 49.82, 29/83,21/88 I 52/90.

[9] Yahmi. D, Branci. T, Bouchair. A and Fournely. E: “Evaluating the Behaviour Factor of Medium Ductile SMRF Structures”. Periodica Polytechnica, Civil Engineering. 62(2), pp. 375-385,2018.

[10] Yassin, M. H. M. (1994). “Nonlinear analysis of prestressed concrete structures under monotonic and cyclic loads.