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Nonlinear seismic analysis of steel frame with semi-rigid joints

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Subject review

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The nonlinear seismic analysis of steel frame without diagonals is presented in the paper. The real behaviour of semi-rigid joints is taken into account through prior numerical simulation of the selected type of joint. The resulting bending moment and rotation curve is substituted with trilinear approximation and incorporated in seismic analysis according to the nonlinear static N2 method. Absolute and relative frame displacements are determined, and the corresponding results are compared with results for the steel frame with absolutely rigid joints.

Key words:
seismic analysis, steel frame, semi-rigid joints, numerical analysis, nonlinear static N2 method

Pregledni rad

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Nelinearna seizmička analiza čeličnog okvira s djelomično krutim priključcima

U radu je prikazana nelinearna seizmička analiza čeličnog okvira bez dijagonala, pri čemu je uzeto u obzir stvarno ponašanje realnih djelomično krutih priključaka na način da je za odabrani priključak prethodno provedena numerička simulacija. Rezultat toga je dijagram odnosa momenta savijanja i rotacije priključka, koji je aproksimiran kao trilinearan, i uključen u seizmičku analizu primjenom nelinearne statičke metode N2. Određeni su apsolutni i relativni pomaci okvira te izvršena usporedba dobivenih rezultata s rezultatima za čelični okvir s apsolutno krutim priključcima.

Ključne riječi:
seizmička analiza, čelični okvir, djelomično kruti priključci, numerička analiza, nelinearna statička metoda N2

Übersichtsarbeit

Paulina Krolo, Mehmed Čaušević, Mladen Bulić

Nichtlineare seismische Analyse eines Stahlrahmens mit teilweise steifen Anschlüssen

In dieser Arbeit wird die nichtlineare seismische Analyse eines Stahlrahmens ohne Diagonalen dargestellt, wobei das reale Verhalten teilweise steifer Anschlüsse in Betracht gezogen wird, so dass für den ausgewählten Anschluss zuvor eine numerische Simulation durchgeführt wurde. Als Resultat wurde das Verhältnis von Biegemoment und Krümmung ermittelt, das durch eine trilineare Approximation in der seismischen Analyse, die auf der nichtlinearen statischen N2 Methode beruht, angewandt wurde. Absolute und relative Verschiebungen des Rahmens wurden ermittelt und ein Vergleich mit Resultaten für einen Stahlrahmen mit vollständig steifen Anschlüssen wurde durchgeführt.

Schlüsselwörter:
seismische Analyse, Stahlrahmen, teilweise steife Anschlüsse, numerische Analyse, nichtlineare statische Methode N2

1. Introduction

The effects of seismic action can be defined in the following ways: by using an equivalent static action when the structural response is such that the contribution of higher modes of oscillation is not significant (only the first mode is taken into account); by modal analysis through response spectra, which is applied without limitations, i.e. for all structures where the influence of higher modes of oscillation on structural response is present; by nonlinear static pushover analysis, in which only the first mode is taken into account; and by nonlinear dynamic time history analysis.

While two of the above mentioned methods are linear and well known, the need was increasingly felt to introduce nonlinear static methods that would be relatively easy to use. Several methods of this type are currently available, and their review is given in [1-4]. They have been introduced in all modern international codes regulating seismic analysis of structures, e.g. in [5, 6]. The nonlinear static N2 method was introduced in European standards [6], and it is the result of the long-standing work of a group of researchers from Slovenia [7-9]. The basic formulation of this N2 method has been continuously developed and complemented and, in this way, its use has been extended considerably. Thus, from initial idea when the method was used for regular systems only, i.e. for systems in which a significant influence of higher oscillation modes is not present, the nonlinear static method was first extended to non-regular systems in which higher modes have to be taken into account [10], and also to systems with the pronounced influence of torsion [11]. The basic feature of all these extensions of the original nonlinear static N2 method is that the method has still remained relatively simple, and that in many cases it is more favourable to use this extended N2 method instead of the nonlinear dynamic time history analysis. In the research that is partly presented in this paper, the nonlinear static N2 method is applied in the analysis of steel frame structures in which semi-rigid joints are taken into account. In fact, this paper is a continuation of this research, some results of which have already been presented in [12, 13].

In the traditional approach to the design of steel frame structures, joints are considered either as pinned that are devoid of any resistance and stiffness, or as absolutely rigid elements offering full resistance. Although such an approach simplifies the structure analysis procedure, it does not describe the real behaviour of structures. In reality, both cases can be incorrect, as they are in fact only limit cases of real behaviour. Rotational behaviour of the existing joints is most often in the area that lies in between these two extremes. The bending moment and the relative rotation of a semi-rigid joint are linked together by the relationship depending on properties of the joint. When semi-rigid joints are used instead of rigid or pinned joints they do not only modify the displacement value but they also influence the distribution and size of internal forces and bending moments within the structure. Joints have their real rigidity and are therefore classified as semi-rigid joints. Their behaviour greatly influences the bearing capacity, rigidity and stability of the entire structure, and also the dissipation of the seismic energy that is introduced into the structure during an earthquake action.

A considerable similarity exists between classification of cross sections of structural elements and classification of joints. Cross sections are classified into four classes depending on their capacity to withstand local instability, when they are partly or fully exposed to compression and/or bending. Their resistance ranges from fully plastic resistance to elastic resistance or a reduced elastic resistance. For joints, the notion of rotational capability is equivalent to the notion of ductility as applied to cross sections. Consequently, joints are also classified according to ductility or rotational capability. This classification is a measure of their ability to withstand a premature local instability and a premature brittle failure (especially due to bolt failure), which is directly related to the ability to dissipate seismic energy [14, 15].

Criteria for correct seismic analysis strive toward enabling the structure to gain resistance and rigidity at moderate seismic actions with a small return period, or ductility and seismic energy dissipation ability in case of stronger earthquakes with a considerable return period. In case of moment resisting frames, semi-rigid joints located at end-parts of elements represent an efficient mechanism for the dissipation of seismic energy. On the other hand, concentrically braced frames are characterised with a sufficient resistance and rigidity, but their inelastic cyclic behaviour shows a reduced energy dissipation capability. A structural typology of eccentrically braced frames has recently been applied. It is based on stiffening the moment resisting frame using the eccentrically joined diagonals. Here, the rigidity of the concentrically braced frame is combined with the ductility and energy dissipating capability of the moment resisting frame [16, 17]. The seismic energy dissipation mechanism that guarantees ductile behaviour differs in the moment resisting frame with semi-rigid joints, as compared to the concentrically or eccentrically braced frame. In case of the concentrically or eccentrically braced frame, the joints must guarantee the ultimate strength that must not be lower that the element yield strength in which formation of plastic hinge is anticipated.

Rules for the analysis and shaping of joints according to their rotational capacity are provided in Eurocode 8 [6] where it is also indicated that joints should be designed in such a way that the rotational capacity of the plastic hinge zone should not be lower than 35 mrad for structures belonging to the high class of ductility (DCH), or 25 mrad for structures belonging to the medium class of ductility (DCM). However, detailed guidelines are not given as to the way in which the real behaviour of joints in the steel frame should be accounted for.

The analysis of a steel frame structure with semi-rigid joints is presented in this paper. The nonlinear static N2 method is used in the analysis of this frame subjected to seismic actions. As detailed guidelines about the analysis of real behaviour of joints during analysis of steel frames are not given in the European standard [6], the way in which this can be achieved is presented in this paper, together with the effect of semi-rigid joints on the overall behaviour of steel frames exposed to seismic actions. First the behaviour of a semi-rigid welded joint is analysed through numerical simulation using the finite element method [18]. The results of this analysis are presented in the diagram showing the relationship between the bending moment and rotation of a joint. In order to include the behaviour of joints into the steel frame analysis, the nonlinear relationship between the bending moment and rotation is idealised through a trilinear curve. Finally, the steel frame model with a trilinear behaviour of joints is formed, and the pushover analysis is conducted for this model using the SeismoStruct program.

2. Numerical model of semi rigid joints

In order to take into account real behaviour of joints in the steel frame analysis, the numerical simulation of a typical semi-rigid welded joint was first conducted using the finite element method [18]. The numerical analysis result obtained in form of a diagram showing relationship between the bending moment and joint rotation was then incorporated in the steel frame structure, for which the analysis of seismic actions was then conducted. The schematic view of the analysis of frame with semi-rigid joints is presented in Figure 1.

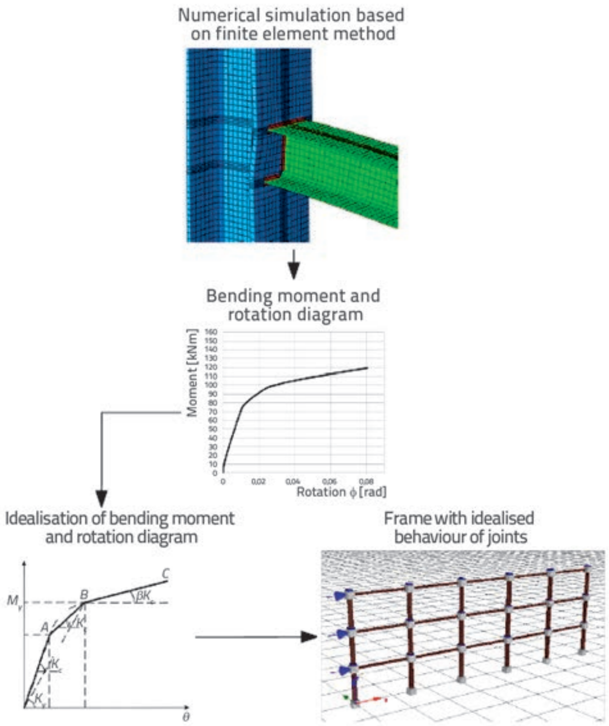

Figure 1. Analysis of frame with semi-rigid joints

The analysis of behaviour of joints must be conducted before the computation of frame with semi-rigid joints with regard to earthquake actions. The numerical approach involving the finite element method was selected for that purpose, and the simulation of joint behaviour was made, which describes complex interactions between its individual elements. The results obtained through this analysis were checked against the results of experimental and numerical tests [19]. Geometrical and material properties of the beam to column joint, boundary conditions, loading plan, and numerical simulation results, are presented in the following sections.

2.1. Geometrical properties of joints

The numerical simulation involving the use of the finite element method [18] was conducted for welded joints of the column HEB 300 and beam HEB 200, with respect to the influence of monotonous bending. The weld thickness next to the beam flange and web amounts to 6 mm and 5 mm, respectively. The selected column and beam cross sections, with characteristic dimensions, are given in Table 1. The form of numerical model was defined based on the form of the experimental model from [19].

Table 1. Cross sectional dimensions of column and beam (mm)

Cross-section properties	Height of cross section	Flange width	Web thickness	Flange thickness
Beam	200	200	9	15
Column	300	300	11	19

2.2. Material properties

When designing welded joints for columns and beams, the weld failure must be avoided by any means, and it does not contribute to the total rotation of the joint. Thus two types of materials and two material models were used in the analysis. The nonlinear material model was selected for the column and beam, as shown in Figure 2. The presented diagram was obtained by experimental testing of the sample made of steel grade S235. The bilinear material model was selected for welds, as shown in Figure 3. A better quality of material was selected so as to avoid its failure prior to failure of material in the beam or column.

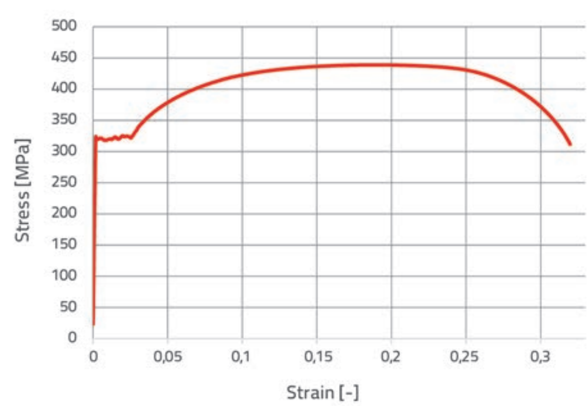

Figure 2. Material properties of steel for column and beam

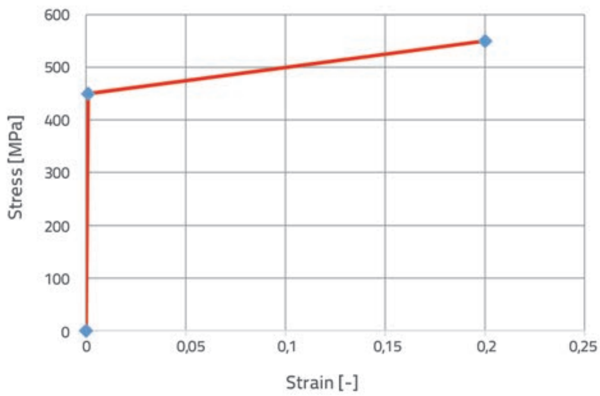

Figure 3. Material properties of steel for welds

2.3. Boundary conditions and loading plan

The form of the model used in numerical simulation of joints was selected according to [19], as shown in Figure 4. The column is 1521 mm in height, and beam is 1055 mm in length. The bending strength of the model was analysed by means of fourteen concentrated forces acting from the distance of 1000 mm from the centre of the joint, i.e. 850 mm from the joint. The position of the concentrated force is on the top flange of the beam, and the force acts monotonously in 22 steps, the value of the force in last step being 140 kN. The bottom part of the column was realized as a pinned support, while to top part was realized as a sliding support enabling free vertical displacement, Figure 4.

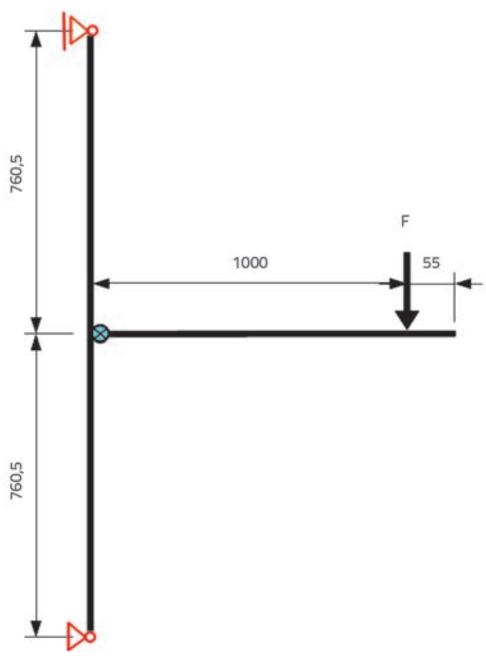

Figure 4. Form of the system with boundary conditions and loading position

The column to beam contact was realized over welds only. During establishment of the finite element mesh, finite elements of the column had to be connected with the weld, and the beam had to be connected with the weld. The mesh was made denser in the contact area, and finite elements (C3D8R) with eight nodes were selected for modelling the column, beam, and weld. The connection detail with the fully formed finite element mesh is shown in Figure 5.

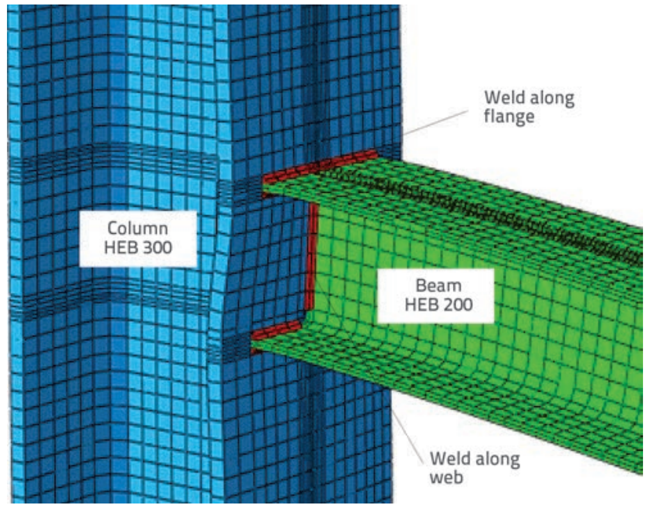

Figure 5. Connection detail with the fully formed finite element mesh

2.4. Numerical analysis result

The diagram showing relationship between the bending moment and rotation, and enabling a credible presentation of joint behaviour in case of a monotonous bending action, was obtained as the result of numerical simulation of the joint. The total rotation of joint (*Rot b*) can be calculated from the vertical displacement of the point below the direct action of vertical load δ_1, Figure 7. The joint rotation ϕ can be obtained from the Eq. (1):

$$\phi = Rot\ b - b_{el} - Rot\ H1 \tag{1}$$

where:

Rot b - total rotation of the joint with elastic rotation of the beam b_{el} that is calculated according to Eq. (2),

b_{el} - elastic rotation of the beam that is calculated according to Eq. (3),

*Rot H*1 - rotation of the column web panel due to shear that is calculated according to Eq. (4).

$$\text{Rot } b = \frac{\delta_1}{850} \tag{2}$$

where

δ_1 - vertical displacement in the point below the load action;

$$b_{el} = \frac{FL_F^2}{2EI_b} \tag{3}$$

where

F - concentrated force,

L_F - distance between the load action and the external surface of column to which the beam is connected,

E - Young modulus of elasticity for steel,

I_b - moment of inertia for beam,

$$\text{Rot } H_1 = \frac{\delta_3 - \delta_2}{z} \tag{4}$$

where

δ_2 i δ_3 - horizontal displacements of column flanges due to shear action, Figure 6.

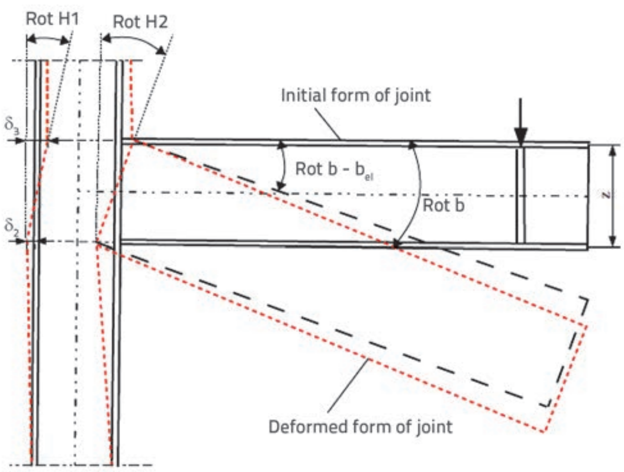

Figure 6. Initial and deformed shape of joint

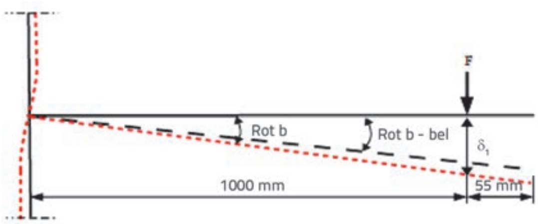

Figure 7. Total rotation of joint

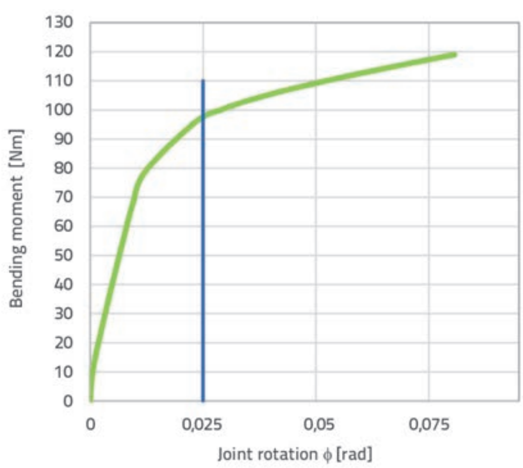

Figure 8. Bending moment and rotation diagram for a welded semi-rigid joint with a standard allowable rotation of 25 mrad (DCM) according to [6]

The final result is presented in form of diagram, as a relationship between the bending moment and rotation, as shown in Figure 8. The initial rotational rigidity amounts to 6464.5 kNm/rad. It has been accepted in applicable regulations [6] that the joint that is almost rigid or almost pinned can still be considered a fully rigid or fully pinned in the dimensioning process. In this respect, joints are classified as rigid, semi-rigid or pinned, depending on the comparison between the rigidity of joint and rigidity of girder, which depends on the moment of inertia and girder length. Figure 9 shows that the analysed joint belongs to the zone of semi-rigid joints, which is defined by the upper and lower bounds of a semi-rigid joint.

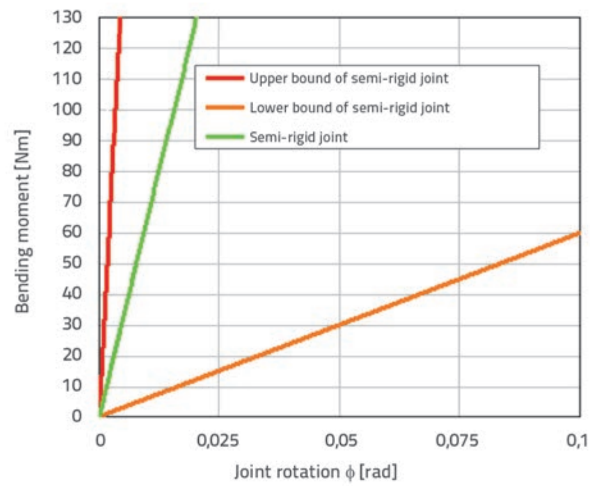

Figure 9. Joint classification according to rigidity [6]

3. Seismic analysis of frame

As indicated in the introductory part of the paper, the analysis of frames has so far focused on extreme cases of joint behaviour, pinned or rigid, although these limit cases do not reflect real behaviour of the frame. As the nonlinear static N2 method does not provide detailed guidelines about the way in which the real or semi-rigid behaviour of joints should be considered, this section provides a steel frame analysis based on the N2 method, which takens into account properties of real joints when subjected to monotonous bending. The previously conducted numerical simulation provides insight into behaviour of a welded column and beam joint, as incorporated into a steel frame structure and, in this way, more realistic values of absolute and relative (interstorey) displacements were obtained. Displacement results obtained by analysis of the same frame with rigid joints were used in the comparison of results.

3.1. Properties of frame and seismic load

The procedure was conducted using the "Swedish model" of the steel frame structure. A typical Swedish model is composed of steel frames with the reinforced concrete core. However, in order to obtain a nonlinear behaviour, the structure was "softened" by replacing the reinforced-concrete core with the steel bracing system, Figure 10.

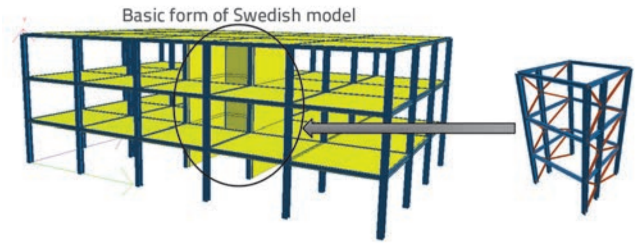

Figure 10. "Swedish model" softening

The seismic analysis was conducted on a replacement three-storey in-plane longitudinal frame, as shown in Figure 11, while the seismic load properties are given in Table 2.

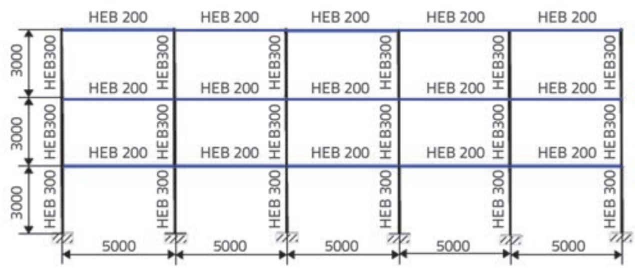

Figure 11. Geometric properties of in-plane frame with presentation of cross sections

Table 2. Seismic load properties

Earthquake spectrum	*Type* 1
Ground type	*B*
Soil parameters	S = 1,2; T_B = 0,15 s; T_C = 0,5 s; T_D = 2,0 s
Peak base acceleration	a_g = 0,254 g
Damping	ξ = 4 %

3.2. Nonlinear static analysis based on N2 method

The procedure of forming two steel frame models for pushover analysis, conducted using the SeismoStruct software, is presented below. The first steel frame is modelled in a traditional way that takes into account rigid joints, while the other frame considers the real behaviour of joints for which numerical simulations based on the finite element methods were previously conducted. The behaviour of joints presented in form of the bending moment and rotation diagram is idealised via the trilinear curve according to [20]. The obtained trilinear approximation of joint behaviour is incorporated in the frame analysis. "Shear building" criteria according to [1] are used in the first case, while storey rotations are additionally taken into account in the second case. The degrees of freedom with zero mass values are eliminated using static condensation [3]. Nonlinear static N2 method results are given in the way the procedure is presented in [3, 7-9]. fundamental period values for two frame types with different joint stiffness values are presented in Table 3. The fundamental period value amounting to *0.802 s* was obtained for the case with rigid joints, while the value of *1.200 s* corresponds to the frame with semi-rigid joints.

Table 3. Fundamental period values dependent on type of joint in frame

Type of frame	Steel frame with rigid joints	Steel frame with semi-rigid joints
Fundamental period T_1 [s]	0,802 s	1,200 s

The position of periods obtained on the elastic response spectrum curve formed according to Table 2 and [6] is shown in Figure 12. It can be seen that the fundamental period increases and the seismic load decreases with the reduction in rigidity.

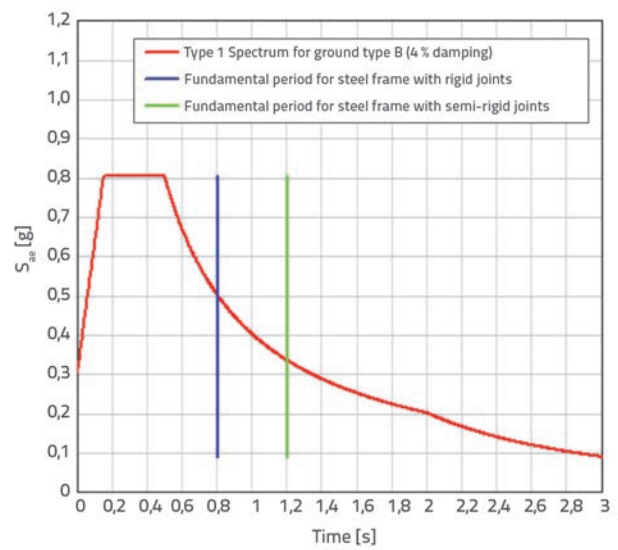

Figure 12. Position of frame periods on the elastic response spectrum curve

The steel frame is subjected to the monotonously increasing lateral load (pushover) representing inertia forces that occur in the structure during the earthquake due to acceleration of foundation soil. The triangular distribution of lateral load, standard in the N2 method, is selected. The lateral load vector{P} is determined according to Eq. (5).

$$\{P\} = p\,[m]\,\{\phi\} \tag{5}$$

where
p - intensity of lateral load,
[m] - diagonal mass matrix,
{ϕ} - assumed form of displacement.

The analysis was conducted using the *SeismoStruct* software. The frame model with the schematic presentation of lateral load is given in Figure 13. The model is subjected to the monotonously increasing lateral load for the controlled roof displacement of 100 cm. The mass matrix is defined in Eq. (6), while the assumed form of displacement is given in Eq. (7).

$$[m]=\begin{bmatrix} m_1 & 0 & 0 \\ 0 & m_2 & 0 \\ 0 & 0 & m_3 \end{bmatrix}=\begin{bmatrix} 209{,}1 & 0 & 0 \\ 0 & 209{,}1 & 0 \\ 0 & 0 & 213{,}6 \end{bmatrix}[t] \tag{6}$$

$$\phi^{T}=\{0{,}33\ 0{,}67\ 1{,}00\} \tag{7}$$

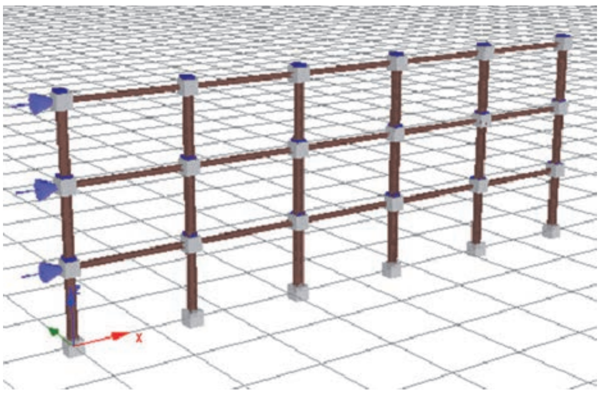

Figure 13. Steel frame model with application of lateral load

The result of the pushover analysis are capacity curves representing the relationship between the transverse force *V* at the top edge level of foundations and the roof displacement *d* at the top of the steel frame. Capacity curve diagrams for the case of steel frame with rigid joints, and steel frame with semi-rigid joints, are presented in Figure 14.

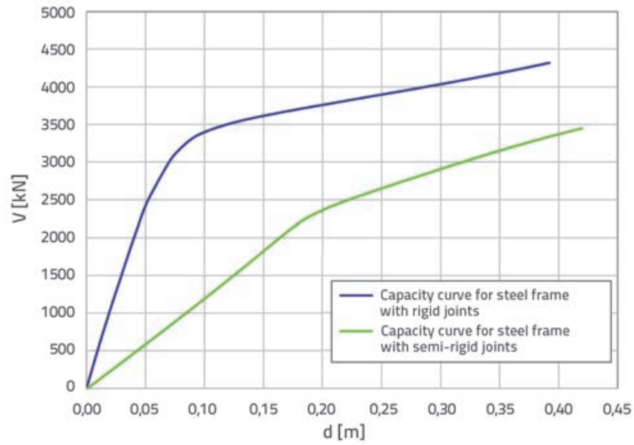

Figure 14. Capacity curves obtained by pushover analysis

As the seismic demand is given for the systems with a single degree of freedom, the system with three degrees of freedom is transformed into the system with a single degree of freedom. The equivalent mass of the system with a single degree of freedom is obtained using the Eq. (8). The transformation factor Γ, that controls the system's transition form several degrees of freedom to a system with a single degree of freedom is defined by the Eq. (9).

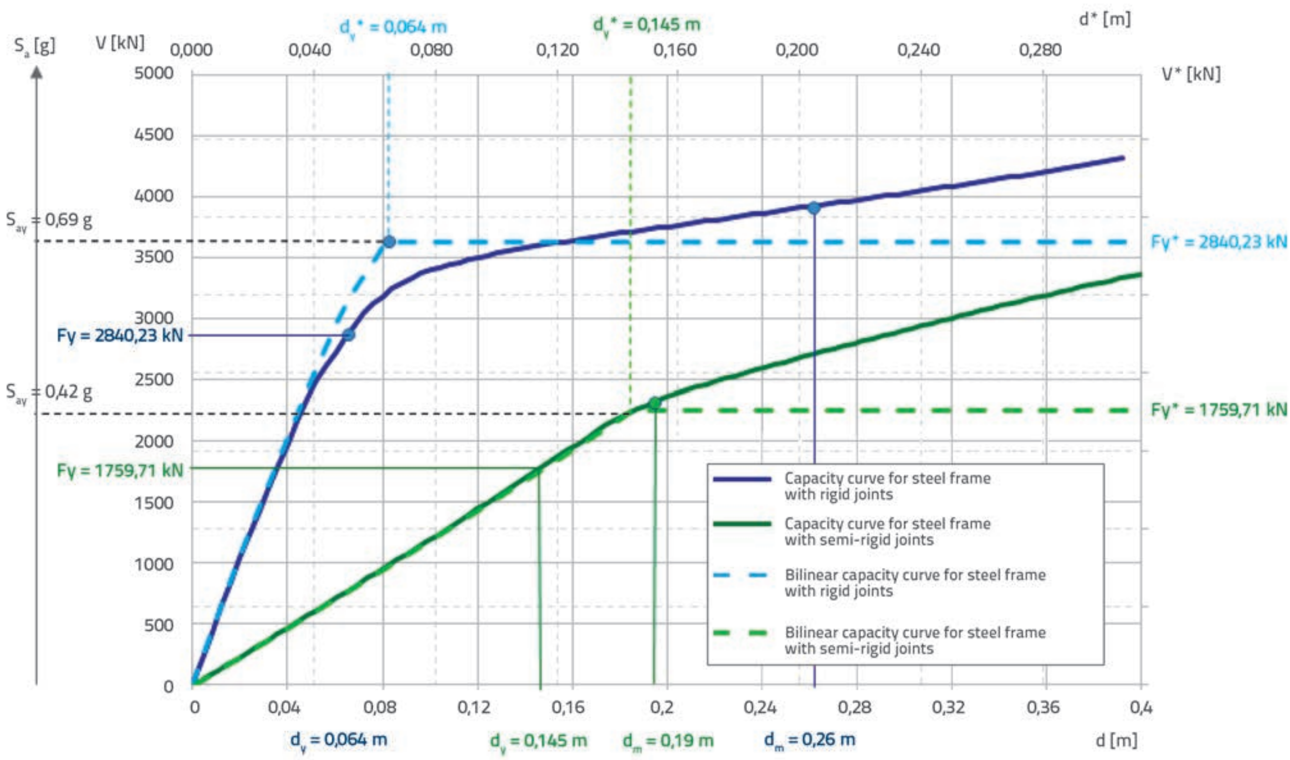

Figure 15. Formation of elastoplastic (bilinear) capacity curves

$$m^* = \sum m_i \varphi_i = m_1\varphi_1 + m_2\varphi_2 + m_3\varphi_3 = 209{,}1 \cdot 0{,}33 + 209{,}1 \cdot 0{,}67 + 213{,}6 \cdot 1{,}00 = 422{,}7 \text{ t} \quad (8)$$

$$\Gamma = \frac{\{\varphi\}^T [m]\{1\}}{\{\varphi\}^T [m]\{\varphi\}} = \frac{\{0{,}33\ 0{,}67\ 1{,}00\}\begin{bmatrix}209{,}1 & 0 & 0\\ 0 & 209{,}1 & 0\\ 0 & 0 & 213{,}6\end{bmatrix}\begin{Bmatrix}1\\1\\1\end{Bmatrix}}{\{0{,}33\ 0{,}67\ 1{,}00\}\begin{bmatrix}209{,}1 & 0 & 0\\ 0 & 209{,}1 & 0\\ 0 & 0 & 213{,}6\end{bmatrix}\begin{Bmatrix}0{,}33\\0{,}67\\1{,}00\end{Bmatrix}} = 1{,}28 \quad (9)$$

The capacity curve, i.e. the force-displacement diagram (*V-d*), as defined for the system with several degrees of freedom, is also valid for the equivalent system with a single degree of freedom (*V*-d**) providing that both the force and the displacement are divided with the transformation factor Γ. The initial rigidity of the equivalent system with a single degree of freedom is equal to the initial rigidity of the system with several degrees of freedom, and is determined from the diagram *V-d*, Figure 15. Instructions for defining the simplified curve of the elastic – ideally plastic link between the force and displacement are given in Appendix B of the Eurocode 8-1 according to [6]. The failure of frame with rigid joints occurs for the value of F_y = 2840.23 kN, in which case the roof displacement amounts to d_y = 0.064 m, while these values for the frame with the semi-rigid joints amount to F_y = 1759.71 kN and d_y = 0.145 m.
Elastic periods T^* of equivalent systems with a single degree of freedom, for the bilinear force and deformation relationship, are defined according to (10). The first value T^* = 0.61 s corresponds to the steel frame with rigid joints, while the other value T^* = 1.17 s corresponds to the steel frame with semi-rigid joints.

$$T^* = 2\pi\sqrt{\frac{m^* d_y^*}{F_y^*}} = 2\pi\sqrt{\frac{422{,}7 \cdot 0{,}064}{2840{,}23}} = 0{,}61\ s$$

$$T^* = 2\pi\sqrt{\frac{m^* d_y^*}{F_y^*}} = 2\pi\sqrt{\frac{422{,}7 \cdot 0{,}145}{1759{,}71}} = 1{,}17\ s \quad (10)$$

The target displacement of the top of the structure is calculated depending on period values obtained, i.e. for structures with a short period ($T^* < T_c$) while ($T^* > T_c$) for structures with medium-sized and long periods. The respective rules are defined in Appendix B of the Eurocode 8-1 according to [6]. Both periods of equivalent systems T^* are greater than T_c = 0,5 s and they belong to the range of medium and long periods, and so the rule of equal displacements according to Eq. (11) is applied.

$$d_t^* = d^* \quad (11)$$

The acceleration at failure limit is obtained from capacity curve in AD format by dividing the force F^* with the equivalent mass m^* according to Eq. (12). The first value S_{ay} = 0.69 g corresponds to the frame with rigid joints, while the value S_{ay} = 0.42 g is related to the steel frame with semi-rigid joints.

$$S_{ay} = \frac{F_y^*}{m^*} = \frac{2840{,}23}{422{,}7} = 0{,}69\ g$$
$$S_{ay} = \frac{F_y^*}{m^*} = \frac{1759{,}71}{422{,}7} = 0{,}42\ g \quad (12)$$

All values can be obtained numerically but, for better visualisation and understanding, a graphical procedure is presented in Figure 16. The required spectrum and bilinear capacity curve are presented on the same diagram. Radial straight lines

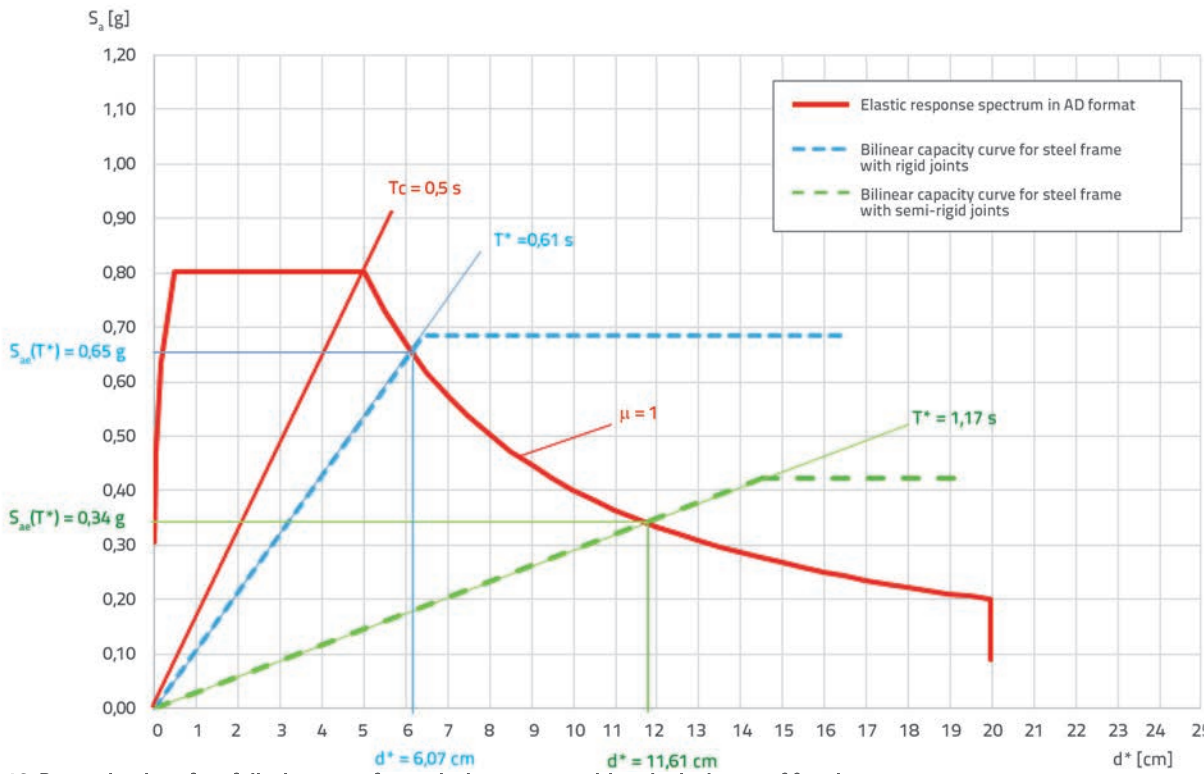

Figure 16. Determination of roof displacement for equivalent system with a single degree of freedom

Table 4. Absolute and relative displacement values

Storey	Storey height [m]	Absolute displacements [cm]		Relative displacements (interstorey displacements) [cm]	
		Steel frame with rigid joints	Steel frame with semi-rigid joints	Steel frame with rigid joints	Steel frame with semi-rigid joints
3	9	7.8	14.9	1.5	4.7
2	6	6.3	10.2	3.0	6.2
1	3	3.3	4.0	3.3	4.0

corresponding to elastic periods T^* of idealised bilinear systems spread from the origin to the point with coordinates that can be determined according to Eq (13).

$$\left(d^*, S_{ae}\left(T^*\right)\right) = \left(S_{ae}\left(T^*\right)\left(\frac{T^*}{2\pi}\right)^2, S_{ae}\left(T^*\right)\right)$$

$$\left(d^*, S_{ae}\left(T^*\right)\right) = \left(0{,}65g\left(\frac{0{,}61}{2\pi}\right)^2,\ 0{,}65g\right) = \left(6{,}07\ cm,\ 0{,}65g\right) \quad (13)$$

$$\left(d^*, S_{ae}\left(T^*\right)\right) = \left(0{,}34g\left(\frac{1{,}17}{2\pi}\right)^2,\ 0{,}34g\right) = \left(11{,}61\ cm,\ 0{,}34g\right)$$

where

d^* - target displacement of the equivalent system with a single degree of freedom

$S_{ae}(T^*)$ - required elastic acceleration.

Target displacement values for an equivalent system with a single degree of freedom amount to 6.07 cm for the system with rigid joints, i.e. 11.61 cm for the system with semi-rigid joints. The required ductility μ s equal to the reduction factor R_μ and is obtained from Eq. (14).

$$R_\mu = \frac{S_{ae}\left(T^*\right)}{S_{ay}} = \frac{0{,}65g}{0{,}685g} = 0{,}95; \quad \mu = R_\mu = 0{,}95$$

$$R_\mu = \frac{S_{ae}\left(T^*\right)}{S_{ay}} = \frac{0{,}34g}{0{,}424g} = 0{,}80; \quad \mu = R_\mu = 0{,}80 \quad (14)$$

Target displacement values for systems with a single degree of freedom are transformed into a global system, and the following values are obtained according to Eq. (15).

$$d = d^*\Gamma = 6{,}07 \cdot 1{,}28 = 7{,}8\ cm$$

$$d = d^*\Gamma = 11{,}61 \cdot 1{,}28 = 14{,}9\ cm \quad (15)$$

The target displacement values of 7.8 cm and 14.9 cm correspond to the frame with rigid joints and frame with semi-rigid joints, respectively. The local earthquake requirement, i.e. absolute and relative storey displacements are obtained after the frames are once again subjected to monotonously increasing lateral load (pushover analysis), but for target displacement values Eq. (15). The results are presented in Table 4 and Figure 17.

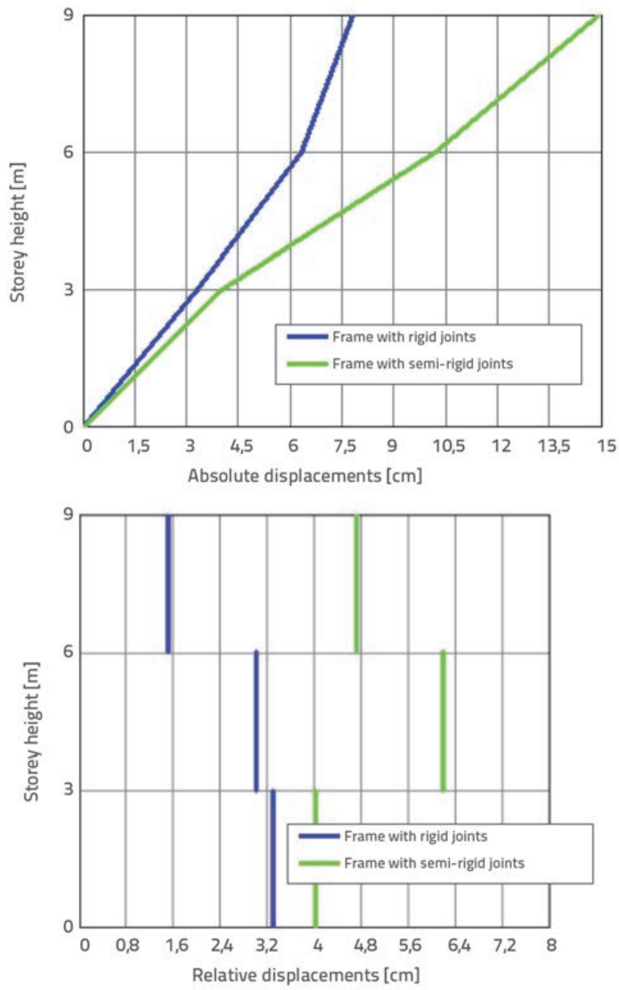

Figure 17. Presentation of absolute and relative frame displacements

It can be seen from the results that absolute and relative displacements of the steel frame with semi-rigid joints are greater when compared to displacements of the steel frame with rigid joints. The roof displacement is by 52.3 % greater in case of the frame with semi-rigid joints, compared to the roof displacement of the frame with rigid joints. The highest relative displacement of the frame with rigid joints is on the first storey, while in case of the frame with semi-rigid joints the values decrease at the first storey, and increase on other storeys with respect to the frame with rigid joints. According to the factor of reduction R_μ values given in Eq. (14), which are due to ductility, i..e. hysteretic dissipation of energy in case of ductile structures, it can be seen that values smaller than one are obtained for both cases. This can also be seen in Figure 16, which means that the behaviour remains linear. Welded joints do not allow failure in the joint itself, i.e. at the weld point. In order to meet this requirement, the welds must be sufficiently resistant so that the weld failure due to joint rotation can be avoided.

4. Critical comment of results and description of further research

For the welded joint, the numerical simulation was first made by means of the finite element analysis using the Abaqus software. Numerical simulation of joints results are presented in the bending moment and rotation diagram, through which a reliable behaviour of joints under influence of monotonous bending is obtained. The obtained diagram is idealised with a trilinear curve, and the joint behaviour is included in this form in the steel frame analysis using the nonlinear static N2 method. Displacement results obtained by analysing the same frame with rigid joints were used for the comparison of results. The comparison of results shows that significantly greater absolute displacements are obtained in case of steel frames with semi-rigid joints, when compared to steel frames with rigid joints. Relative displacement are the greatest on the first storey in case of frames with rigid joints, while in case of frames with semi-rigid joints the greatest relative displacement is on the second storey, and the smallest displacement is on the first storey.

Further research will focus on a greater differentiation of joints, which will enable a better comparison of results. Parametric analyses of joints will be made, and it will be established which are the significant factors in semi-rigid joints that change the seismic response of the frame structure. In addition, further studies will include analyses of the increase in dissipation of seismic energy in case when real semi-rigid joints are included in the analysis. Such an analysis has not so far been published in literature. Finally, nonlinear dynamic analyses, based on selected real earthquake records, will be made instead of nonlinear static analyses [4].

5. Conclusion

Absolute and relative frame displacement values are presented, and results for the steel frame with absolutely rigid joints are compared with results for the steel frame with semi-rigid welded joints. Results are also compared with allowable values as given in Eurocodes.

The analysis of the frame with semi-rigid joints has revealed a reduced capacity of the structure, but also an increase in its fundamental period, which results in a smaller value of seismic load acting on the structure. The results obtained can be generalised for the welded joint only. Based on results obtained by numerical simulation for the semi-rigid joint, it is possible to estimate the capacity of seismic energy dissipation in the joint itself, and in the structure taken as a whole, which will be the subject of further studies.

The use of fully welded moment resisting frames, as related to seismic load, has been extensively studied in recent times, as many welded steel structures have suffered damage precisely in the zone of joints. More than 150 structures have exhibited this type of damage during earthquakes in Northridge, 1994 and, Kobe, 1995 [21]. The weld damage mostly occurred due to the use of low-strength welds in combination with many other joint details, material properties, etc. Most cases of damage that did not cause collapse of structures, now present a serious risk in case of a repeated seismic action. The repair of such damage is associated with considerable costs. Many studies have been conducted worldwide to enable a better analysis, design, and realization of such joints. Numerous studies have been made in relation to the Northridge earthquake, 1994 [22-25] and in these studies the focus has mainly been on the behaviour of welded joints.

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