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SEISMIC RESPONSE OF STEEL FRAME STRUCTURES

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ABSTRACT :

The paper deals with analysis of the dynamic response of steel orthogonal frames with non-linear semi-rigid connections under the seismic loads. Chen-Lui exponential curve was used for the mathematical modelling of the non-linear beam-column connections. Direct integration of the equations of motion was performed with the so-called α-procedure. The general numerical procedure was programmed in MATLAB and used for numerical example presented in this paper.

KEYWORDS: Steel frames, seismic load, non-linear semi-rigid connections, non-linear dynamic analysis, soil stiffness, programming.

1. INTRODUCTION

The general accepted concept of seismic designing is based on the model of a ductile structure. This design approach assumes a partial and controlled plasticity of the structure under the load of the designed earthquake (the strongest expected one). Ductile frames accept horizontal loads primary with bending their columns and beams, and in the case of the designed earthquake they should obtain form of beam mechanisms, which means they should obtain appearance and development of plastic hinges in the beams ends areas, but not in the columns or in the frame nodes. As the jointing of the columns to the beams is obtained with beam-column connections, the nonlinear deformations can be located in the beams and/or connections, as functions of the bearing capacity of the connections (or beams). The location of the plastic hinges can be controlled with inclination of the bearing capacity of ones in relation to the others with the specified overstrength factors.

This paper is based on the usage of the capacity design method in seismic designing. Namely, with the nonlinear behaviour of the connections beam-column, the seismic forces are limited in the structure, in that way the members of the structure are kept in the desirable (usually linear) range, while the largest part of the seismic energy is spent in the hysteresis behaviour of the connections. In this way, nonlinear beam-column connections present some kind of safety devices, consciously designed bad points in the structure, which, including the necessary rotation capacity, make the structure to move like a mechanism in limited case, during the formation of the plastic hinges in the frame fixity, and while the elements of the complex structure are in the desirable (linear) range of stresses.

The paper keeps itself to the nonlinear behaviour of the connections, while the other part of the structure is designed for linear range of stresses. The problem is materially and geometrically nonlinear. Geometrical nonlinearity is the consequence of the nonlinear behaviour of the connections, which has direct influence on the structural rigidity using their variable stiffness. Linearization of the element stiffness matrix of the system during the numerical solution of the equations of motion is performed according to the tangent stiffness for each time step of integration.

2. CALCULATIVE ASSUMPTIONS

The following assumptions are made during the formation of the numerical model:
- the influence of the axial forces on the deformation is neglected,
- the mass of the finite elements is treated as consistent,
- the connection beam-column is modelled as nonlinear one,"Chen-Lui exponential model" (Figure 1)

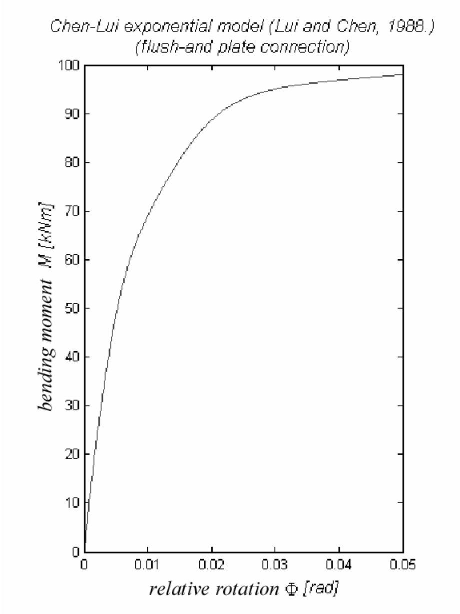

Figure 1 Chen-Lui exponential model (Lui and Chen 1988)

Consistent matrix of the mass **M** (Aleksić, S., Kolundžija, B., 2005), is defined as:

$$\mathbf{M} = \frac{\mu l}{420}\begin{bmatrix} 156 & 22l & 54 & -13l \\ 22l & 4l^2 & 13l & -3l^2 \\ 54 & 13l & 156 & -22l \\ -13l & -3l^2 & -22l & 4l^2 \end{bmatrix} \tag{2.1}$$

where μ is distributed mass, and *l* the element's length.

The stiffness matrix **K** of the finite element with the semi-rigid connections (Aleksić, S., Kolundžija, B., 2005), is defined according:

$$\mathbf{K} = \begin{bmatrix} 12\zeta_3 \frac{EI}{l^3} & 6\zeta_1 \frac{EI}{l^2} & -12\zeta_3 \frac{EI}{l^3} & 6\zeta_2 \frac{EI}{l^2} \\ 6\zeta_1 \frac{EI}{l^2} & 4\zeta_4 \frac{EI}{l} & -6\zeta_1 \frac{EI}{l^2} & 2\zeta_5 \frac{EI}{l} \\ -12\zeta_3 \frac{EI}{l^3} & -6\zeta_1 \frac{EI}{l^2} & 12\zeta_3 \frac{EI}{l^3} & -6\zeta_2 \frac{EI}{l^2} \\ 6\zeta_2 \frac{EI}{l^2} & 2\zeta_5 \frac{EI}{l} & -6\zeta_2 \frac{EI}{l^2} & 4\zeta_6 \frac{EI}{l} \end{bmatrix} \tag{2.2}$$

During the programming of the numerical procedure, the matrix of the mass [**M**] and the stiffness matrix of the system [**K**] are formed with the procedure of the code numbers. Under the proportional damping assumption, the damping matrix [**C**] is shown as a linear combination of the mass matrix [**M**] and the stiffness matrix [**K**]:

$$[\mathbf{C}] = \alpha\,[\mathbf{M}] + \beta\,[\mathbf{K}]$$

$$\alpha = \frac{2\xi\omega_1\omega_2}{\omega_1+\omega_2} \qquad \beta = \frac{2\xi}{\omega_1+\omega_2} \tag{2.3}$$

ω_i- circular frequency during *i*- mode of oscillation
ξ_i- relative damping during *i*-mode of oscillation

Direct integration of the equations of motion is done using the α-procedure, with the linear law of the acceleration variation during the observed time interval. Unconditional stability of the procedure, which means integration of the equations of motion with the arbitrary length of the time increment Δt, is obtained if the procedure parameters α, β and γ are chosen according to Brčić, S. (1998).

3. NUMERICAL EXAMPLE

The complete numerical procedure is programmed in the higher programming language MATLAB. Program codes are of the general character for the accepted marking system, and the program itself is very suitable for subtle analysis of the issued problem.

Using the abovementioned program, numerical examples are done with an input data according to the figure 2. Relative damping is $\xi = 0.1$, and the time step of integration corresponds to the time step of the accelerogram $\Delta t = 0.01$s. The structure is under the influence of two accelerograms of two different frequency bands, according to the following graph, and aiming for a deeper insight into the behaviour of the time history response of the structure. The amplitude of the ground acceleration in the case of the Campano Lucano earthquake in 1980 is $1.52 m/s^2$, and in the case of Montenegrin earthquake in 1979, it is $2.2 m/s^2$, which is greater for 44.7%.

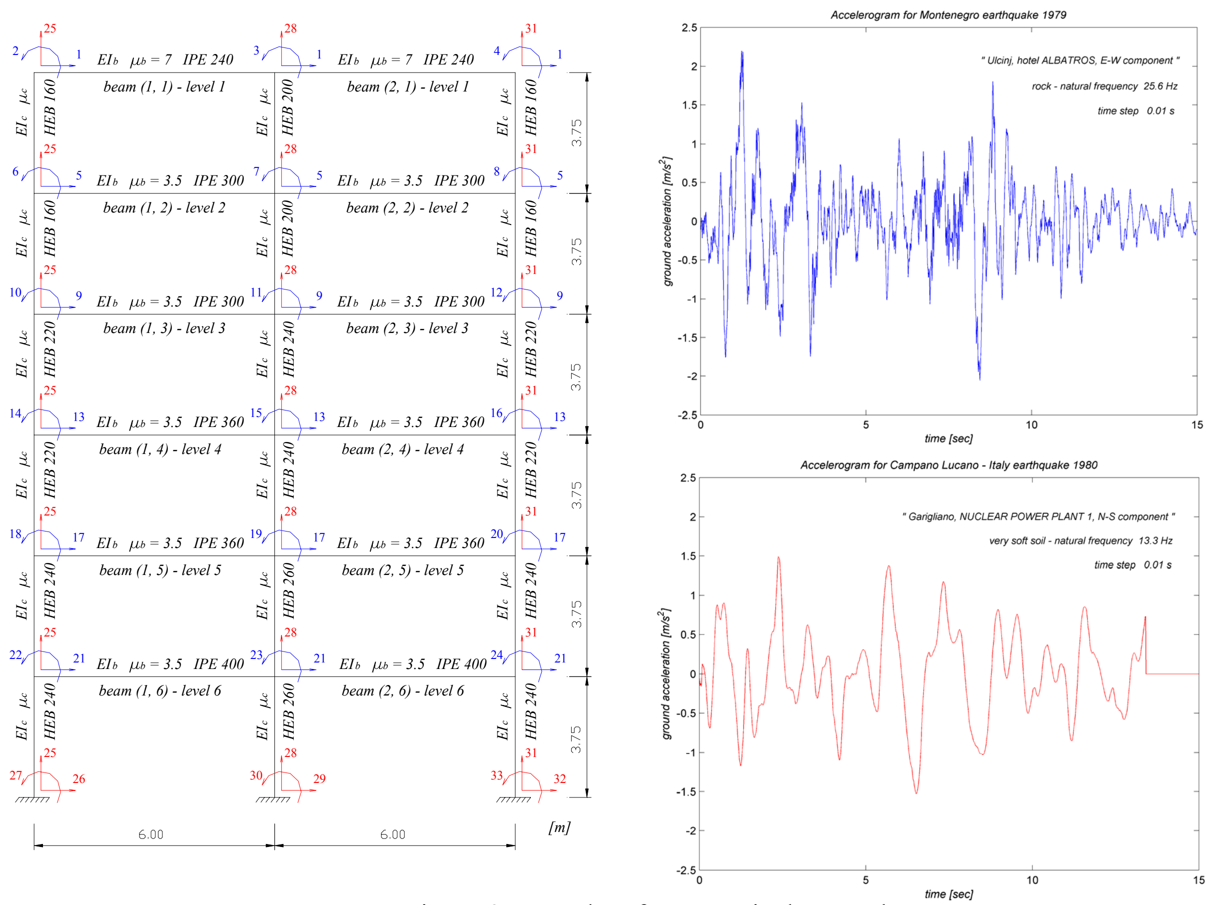

Figure 2 Input data for numerical example

For insight into the behaviour of dynamic characteristics of the frames, the rotation stiffness of the beam-column connection is varied and the following values are obtained for the periods of oscillations in the first three modes (Table 3.1).

Table 3.1. Periods of oscillations in the first three modes for different values of the rotation stiffness S_{r0}

S_{ro} [kNm/rad]	100	5000	12 343*	50 000	100 000
First mode	14.90	5.101	3.948	3.085	2.909
Second mode	2.811	1.526	1.278	1.074	1.030
Third mode	1.039	0.746	0.651	0.566	0.547

* Chen-Lui exponential model

For the confirmation of the results of the developed analysis, as well as for the justification of the given assumptions, the numerical example with the same input data and using the software SAP2000 is done, where the following values for the periods of the first three modes of oscillation are obtained:

$$SAP2000 \Rightarrow T = \begin{Bmatrix} 2.841 \\ 1.010 \\ 0.543 \end{Bmatrix}; S_{ro} \quad \rightarrow \quad \infty \tag{3.1}$$

The graphical presentation of the time history floor displacements during the nonlinear response of the structure is given in the figure 3. Displacements are presented as the absolute ones in relation to the referent stationary point (left), as well as the relative ones in relation to the ground displacement (right). The maximal frame top displacement in the case of the Montenegrin earthquake is 16.82cm, and in the case of the earthquake Campano Lucano is 23.1cm, which is greater for 37.5%, in spite of the quite smaller amplitude of the ground acceleration.

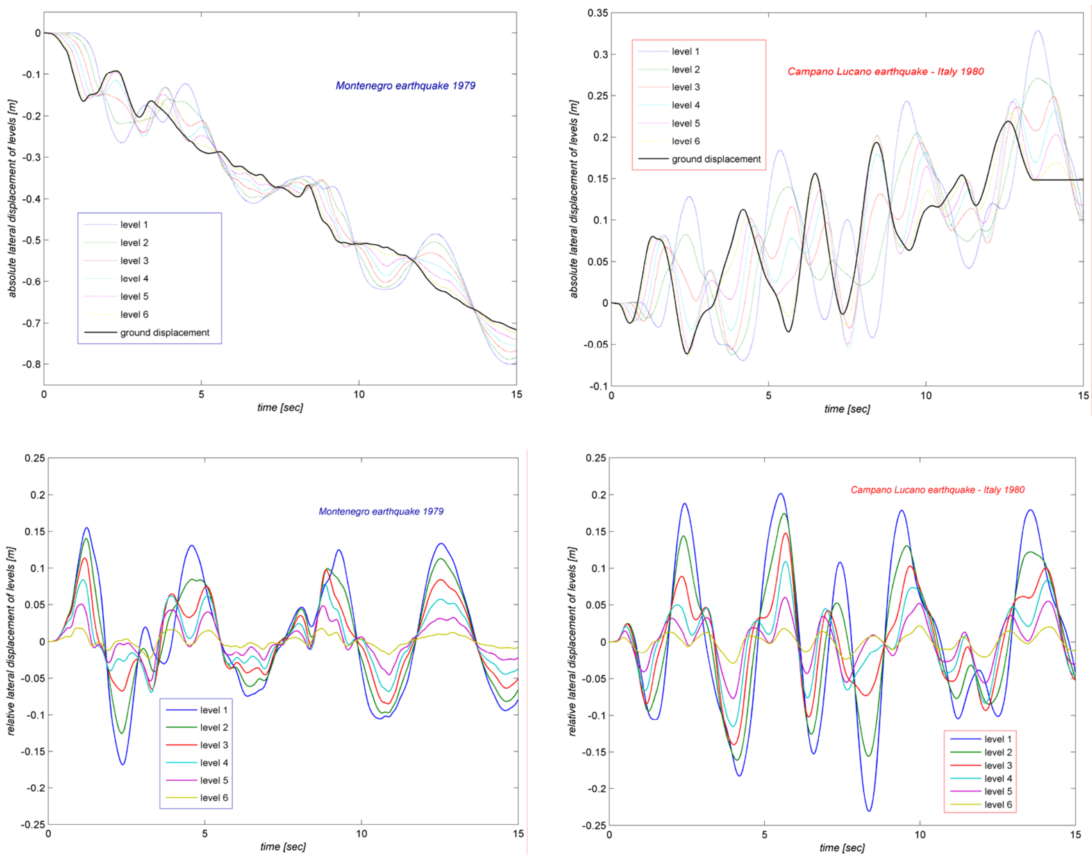

Figure 3 Time history floor displacements during the nonlinear response of the structure

The following diagrams (Figure 4) describe time history analysis of the relative displacements of the levels (floors) of the frame. The graphics show the response of the structure during the nonlinear analysis in comparison with the linear behaviour of the beam-column connections. For the rotation stiffness of the beam-column connections, in the case of a linear response of the system, the primary tangent stiffness of the accepted " *Chen-Lui* " exponential curve is used.

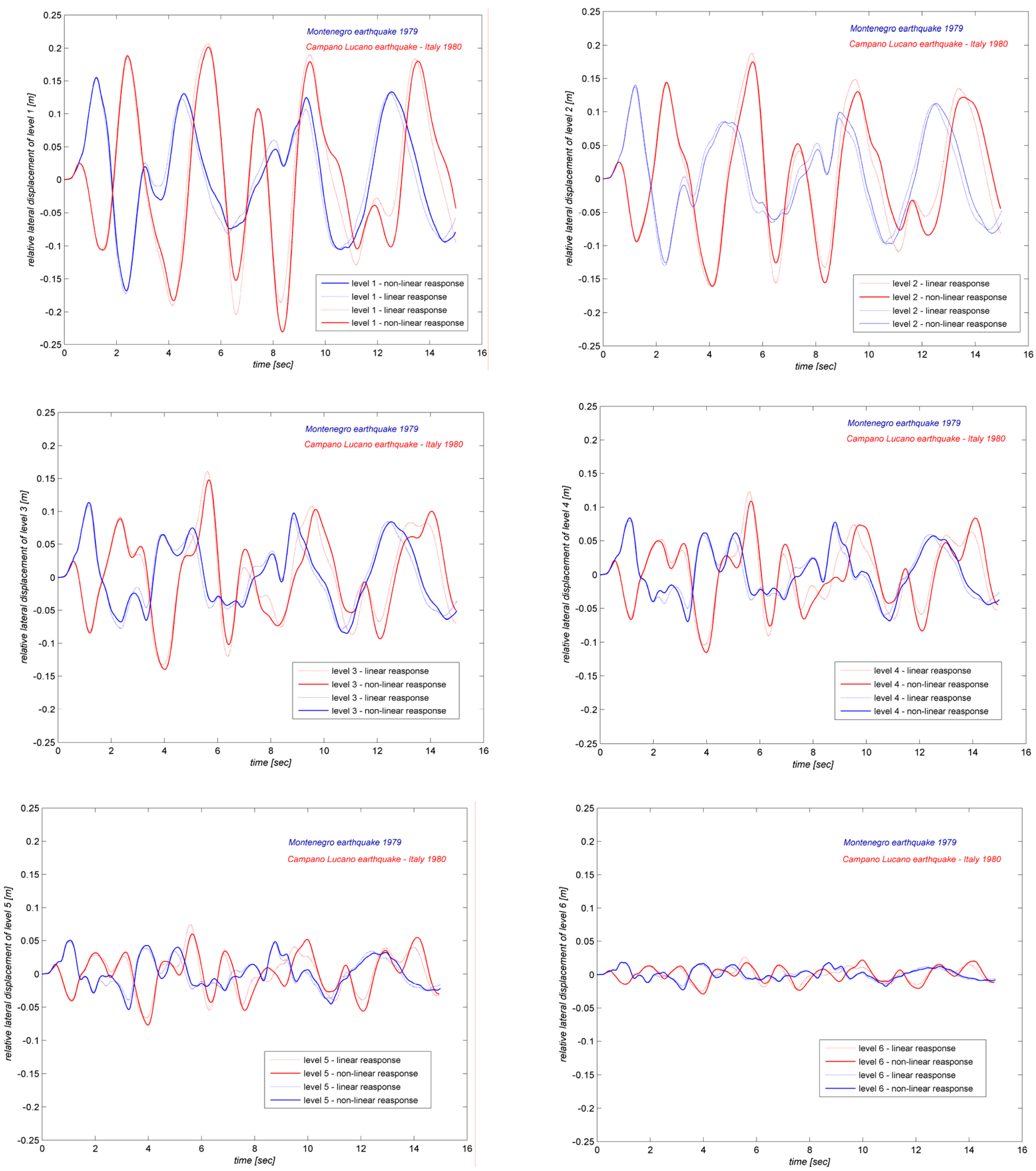

Figure 4 Parametric analysis of the relative displacements of the levels (floors) of the frame

In the case of the both earthquakes, as it can be seen from the following diagrams (Figure 5), the structure enters into the range of nonlinear deformations. The nonlinearity is more emphasised in the case of load with accelerogram Campano Lucano, despite the quite smaller amplitude of the ground acceleration. The necessary rotation ductility of the beam-column connection in the case of accelerogram Campano Lucano is about eight, while in the case of the Montenegrin accelerogram, the connections did not enter into the deep plastic behaviour, which can be concluded with the above supplied *"Chen-Lui"* exponential curve.

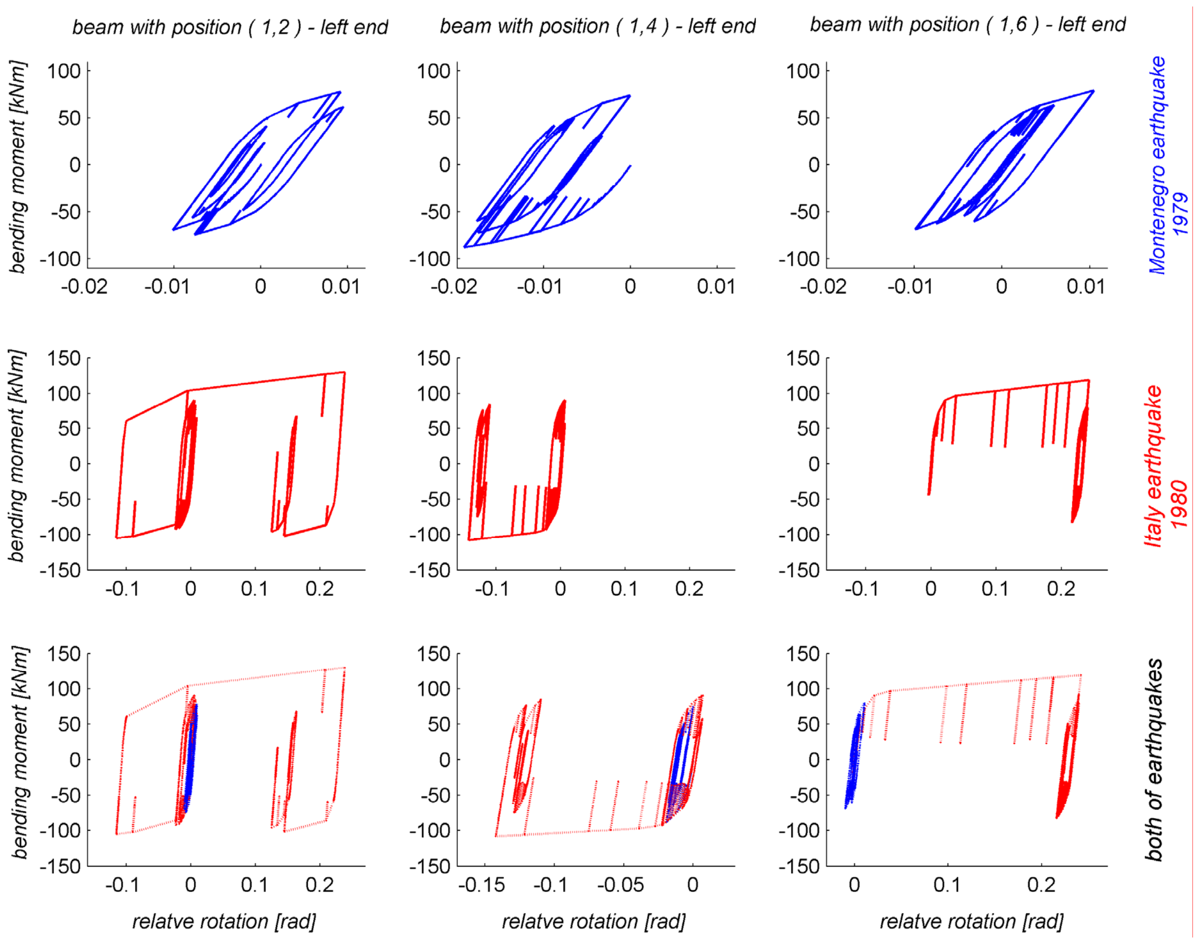

Figure 5 Moment-relative rotation diagrams for characteristic nodes

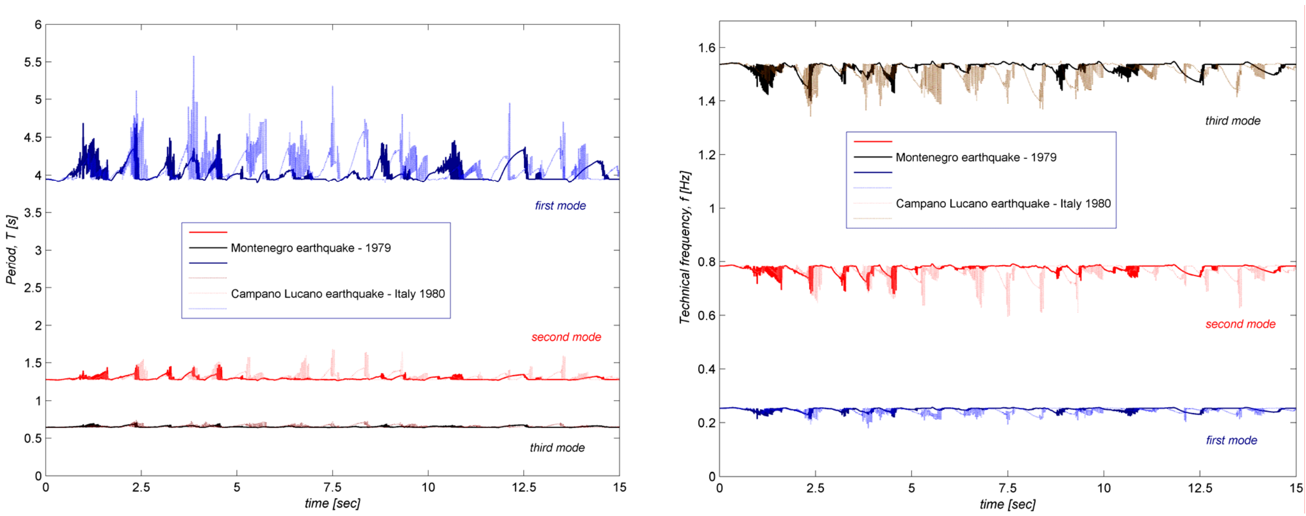

Figure 6 The periods of oscillation (left) and technical frequencies (right) for the first three modes of oscillation

As nonlinear semi-rigid connections beam-column with their variable rigidity have direct influence on the structure stiffness (as a function of the stress level), the basic dynamic characteristics of the system (circular frequencies and periods of oscillation) are exposed to variation during the earthquake action. The periods of oscillation are shown in the Figure 6 (left), as well as the technical frequencies (right) for the first three modes of oscillation, and during the total accelerogram duration of 15 seconds. The maximal change of the shown periods of oscillation in the case of accelerogram of Campano Lucano is about 40%, 30% and 15% for the first, the second and the third mode of oscillation, respectively. In the case of the Montenegrin earthquake those differences are about 20%, 15% and 10%, for the I, II and III mode of oscillation, respectively.

4. CONCLUSION

Taking into consideration the difference in the amplitudes of the ground acceleration of 44.7%, the difference in the amplitudes of the frame top displacement of 37.5%, the changes in the basic dynamic characteristics of the structure under the earthquake load of about 40.0%, as well as the great difference in the necessary rotation ductility of the beam-column connections (obvious from the supplied diagrams moment-rotation), the conclusion is that the relation of the dynamic characteristics of the structure in respect to the dynamic characteristics of the seismic loads has a key role during the analysis of the seismic resistance of the structure.

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