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EVALUATION OF SEISMIC FORCES ACCORDING TO EUROCODE 8 AND SNiP II-7-81. COMPARATIVE ANALYSIS

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Abstract. The evaluation of seismic forces is very important for the design of all types of structures. In the present, Republic of Moldova is in a period of transition in the construction industry from the national design norms to the European design norms. The comparison provides an interesting and imperative approach in design of structures. During the design process it is very important to be aware of all the forces acting on a structure, especially the accidental forces. This paper presents a comprehensive analysis of a concrete frame structure, subjected to seismic action. A calculation is performed according to the current national design code SNiP II-7-81 and according to Eurocode 8. The purpose of the paper is the quantitative comparison of the values of the seismic forces and the qualitative evaluation of the factors that influence these forces.

Keywords: *Eurocode, Earthquake, Seismic forces.*

Rezumat. Evaluarea forțelor seismice este foarte importantă pentru proiectarea tuturor tipurilor de structuri. Având în vedere faptul că Republica Moldova se află într-o perioadă de tranziție în industria construcțiilor de la normele naționale de proiectare la normele europene de proiectare acest studiu comparativ este foarte important. În timpul procesului de proiectare este foarte importantă conștientizarea tuturor forțelor care acționează asupra unei structuri, în special forțele accidentale. Această lucrare prezintă o analiză a unei structuri de cadru din beton, supusă acțiunii seismice. Este prezentat un calcul efectuat conform codului național de proiectare actual SNiP II-7-81 și conform Eurocod 8. Scopul lucrării este compararea cantitativă a valorilor forțelor seismice și evaluarea calitativă a factorilor care influențează aceste forțe.

Cuvinte cheie: *Eurocod, cutremur, forțe seismice.*

1. Introduction

Design earthquake resistant structures has always been a challenge for engineers. In order to reduce the destructive effects on the constructions, certain constructive measures were applied. Initially, these measures were only intuitive principles, such as reducing the height of structures or reducing their mass [1]. Later, certain constructive provisions received recommendation status, without being mandatory for implementation. It was not until the beginning of the 20th century that mandatory seismic design codes began to be developed in earthquake-prone regions [2]. These design codes differed depending on the region in which they were developed [3].

One widely recognized set of seismic provisions is the Eurocode, which is a series of European standards for the design of structures [4]. In the United States, seismic provisions are outlined in the International Building Code (IBC) [5] and the American Society of Civil Engineers (ASCE) standards [6,7]. The seismic regulations in Japan are outlined in the "Building Standard Law" and the "Building Standard Law Enforcement Order" [8]. China has implemented seismic provisions and building codes to address the seismic risks prevalent in certain regions of the country. Seismic design standards are primarily outlined in the "Code for Seismic Design of Buildings" (GB50011) [9].

These provisions are designed to mitigate the impact of seismic forces on buildings, bridges, and other infrastructure, with the ultimate goal of protecting human life and minimizing damage to property [10].

In the Republic of Moldova, 2 seismic design codes are relevant - SNiP II-7-81 [11] and SM EN 1998 [12], known as Eurocode 8. SNiP II-7-81 was adopted as a national design standard in 1982. SM EN 1998 is in the process of implementation.

2. Comparison of design codes

2.1. Seismic hazard

Eurocode 8 considers the seismic action in terms of PGA – peak value of ground acceleration, for a ground class A – a_{gr}. [3] The reference peak value of the ground acceleration, corresponds to the reference return period of the seismic action (T_{NCR}).

SNiP II-7-81 describes seismic action in terms of intensity, according to the MSK-64 intensity scale [13]. The MSK-64 intensity scale is based on an analysis of seismic action results and allows estimating the intensity of seismic action using statistical data. In the current version of SNiP, the parameter that describes the intensity of seismic action according to the MSK-64 scale – seismicity, measured in degrees. For each degree of seismicity (intensity) could be assigned maximum value of peak value of ground acceleration represented by the design intensity factor (I_p).

2.2. Classification of the terrain

Seismic action is directly influenced by ground conditions.

Eurocode 8 classifies the soil in 4 categories, depending on the value of the average speed of the shear waves ($v_{s,30}$), considered until a depth of 30 meters. If the value of the average shear wave velocity is not known, the standard penetration test shall be used to determine the ground characteristics [4].

SNiP II-7-81 classifies the soil into 3 categories, depending on the consistency index, porosity ratio and other physical-mechanical properties of the soil [11]. The terrain category directly influences the seismicity of the site, by amplifying or reducing the reference intensity of the site.

2.3. Elastic response spectrum

The main parameter that determines the impact of seismic action on structures is the elastic response spectrum [14].

According to **Eurocode 8**, the elastic spectrum is defined by the relationship:

$$\frac{S_e}{g} = S \cdot S_e(T), \tag{1}$$

where: $S_e(T)$ – elastic response spectrum defined by Eurocode with different calculation formulas depending on the fundamental period of the structure.
S – the terrain factor.

According to **SNiP II-7-81** the design spectrum is defined by the following relationship:

$$\frac{S_e}{g} = \beta \cdot k_{soil}, \tag{2}$$

where: β – dynamic coefficient, which is determined according to the terrain category.

2.4. Building behaviour factor

The main design concept for seismic actions of structures consists in energy dissipation, through the formation of plastic joints [15]. The amount of energy dissipated is directly influenced by the structural configuration [16]. According to **Eurocode 8** the behavior factor (q) represents the ratio between the design forces in the linear elastic domain for the critical damping fraction of 5% and the inelastic one, which takes into account the plastic joints.

Within **SNiP II-7-81**, the behavior factor (k_1) assumes the limitation of degradations, respectively the formation of plastic joints. Thus, in the situation when degradations are not allowed, the behavior factor will be equal to 1; when certain structural degradations are allowed, which do not affect the integrity of the people occupying this structure, the value of the safety factor will be lower than 1.

3. Comparative example

3.1. Initial data

A comparative numerical analysis of a three-level structure according to Figure 1 will be considered.

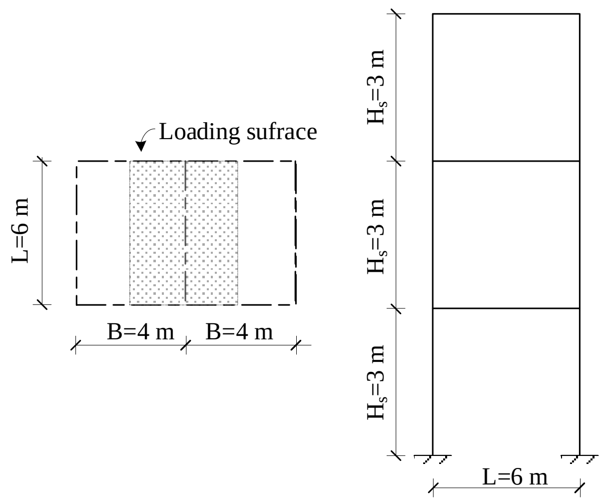

Figure 1. The 3-story frame structure.

The structure will be planned for both Sculeni in Iasi County and Sculeni in Ungheni district. These two localities are situated within a close proximity of less than 10 kilometers, as illustrated in Figure 2 sourced from Google Maps.

All pertinent data such as frame span, span length, story height, materials used, column sections, and other relevant information are centralized in Table 1 for ease of reference.

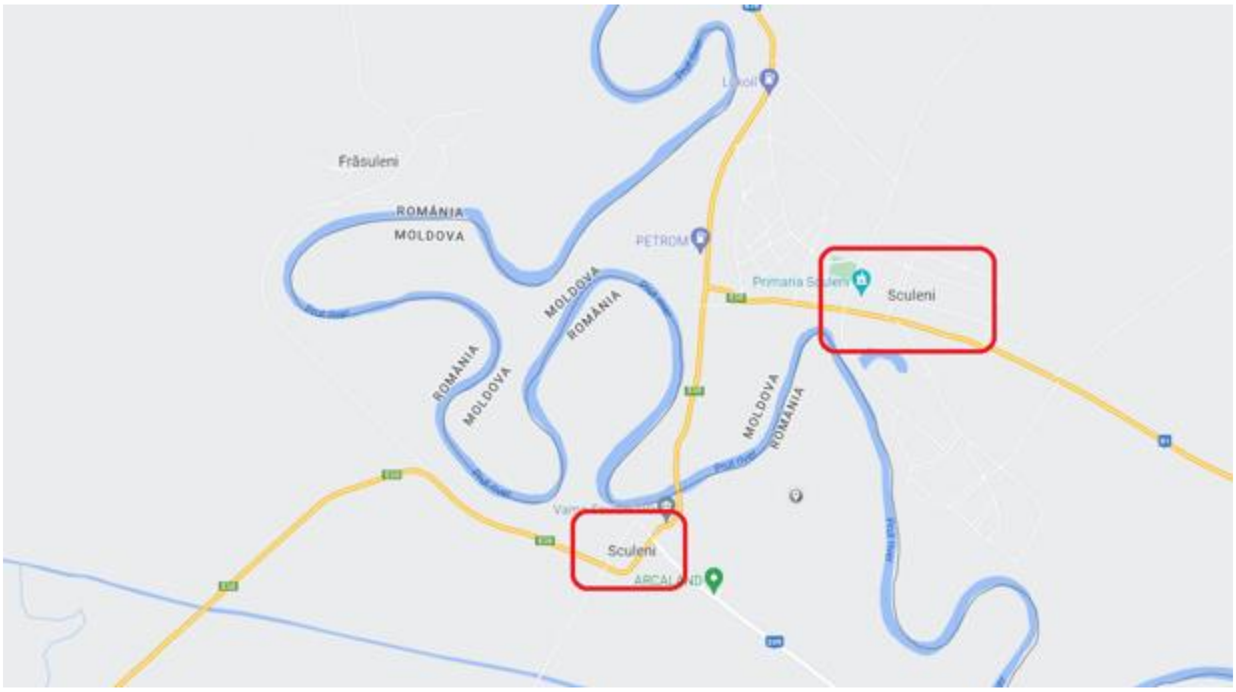

Figure 2. Position of the considered localities.

Table 1

Initial data

Description	Unit	Value
Frame span, L	m	6
Span length, B	m	4
Story height, H	m	3
Slab thickness, δ	cm	18
Material used	-	Concrete C20/25
Column section, b×h	cm	40×40
Beam section, b×h	cm	40×50
Live load	kPa	1.5
Site intensity according to MSK-64	-	8
Soil category according to SNiP II-7-81	-	III
PGA	$\mathrm{m/s^2}$	0.16g
Soil type according to EC-8	-	Type C

Note: MSK - Medvedev–Sponheuer–Karnik scale

The same design loads will be considered for both cases, except for the snow load, which differs in design code of each country (Republic of Moldova and Romania). It is important to note that the snow loads are listed in Table 2, highlighting the variations between the two regions.

Table 2

Snow load classification

Snow load	Unit	Design value
Eurocode 1	$\mathrm{kN/m^2}$	2.00
SNiP II.I-07.85		0.50

Building loads, are conveniently compiled in Table 3 for easy access and comparison.

Table 3

Building loads

Snow load	Unit	Design value
Total permanent load slab	kN/m^2	4.96
Total permanent load roof	kN/m^2	5.26
Permanent load on other elements		
Beam weight (*b*×*h*=40×50 cm)	kN/m^2	5.28
Colum weight (*b*×*h*=40×40 cm)	kN/m^2	4.22
Total live load	kN/m^2	1.50

3.2. Calculation of concentrated masses

The structure will be reduced to a plane frame, to simplify the design. Later using the assumption of infinite stiffness of the slab in the horizontal plane, the plane frame will be reduced to an inverted pendulum with masses concentrated at the level of the story according to Figure 3.

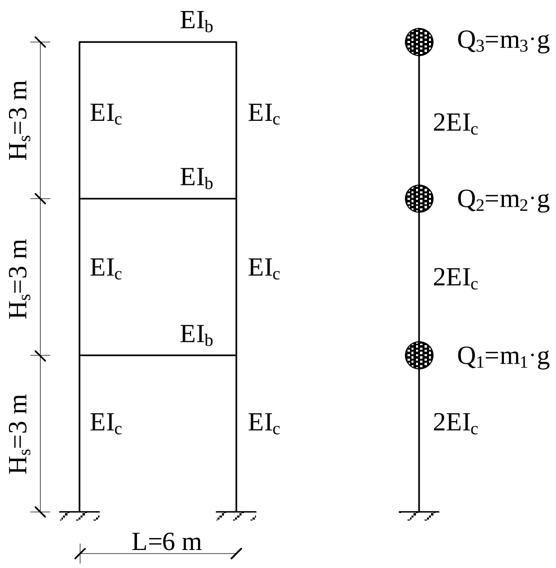

Figure 3. Inverted pendulum with masses concentrated at the level of the story.

$$Q_{perm,1} = Q_{perm,2} = q_{sl} \cdot B \cdot L + q_b \cdot L + q_b \cdot B \cdot 2 + 2 \cdot q_c \cdot H_s = 209.28\ kN \tag{3}$$

$$Q_{var,1} = Q_{var,2} = q_{var} \cdot B \cdot L = 36.00\ kN \tag{4}$$

$$Q_{perm,3} = q_{sl} \cdot B \cdot L + q_b \cdot L + q_b \cdot B \cdot 2 + 2 \cdot q_c \cdot H_s/2 = 204.96\ kN \tag{5}$$

$$Q_{var,3} = q_{var} \cdot B \cdot L = 36.00\ kN \tag{6}$$

Calculation of concentrated masses according **SNiP II-7-81:**

$$Q_1 = Q_2 = 0.9 \cdot Q_{perm,1} + 0.5 \cdot Q_{var,1} = 206.352\ kN \tag{7}$$

$$Q_3 = 0.9 \cdot Q_{perm,1} + 0.5 \cdot Q_{var,1} + 0.5 \cdot Q_{zăpadă} = 208.464\ kN \tag{8}$$

Calculation of concentrated masses according **Eurocode 8**

$$Q_1 = Q_2 = 1.35 \cdot Q_{perm,1} + 1.5 \cdot Q_{var,1} = 336.528\ kN \tag{9}$$

$$Q_3 = 1.35 \cdot Q_{perm,1} + 1.5 \cdot Q_{var,1} + 1.05 \cdot Q_{zăpadă} = 381.096\ kN \tag{10}$$

3.3. Calculation of the dynamic parameters of the structure

The similarity between the flexibility and stiffness matrices in both cases is due to the consistent use of the same materials and structural dimensions. Here, the terms $\boldsymbol{\delta_{ij}}$ represent the displacement along the degree of freedom $\boldsymbol{i}$ when a force equal to unity is applied solely along the dynamic degree of freedom $\boldsymbol{j}$, under conditions where the dynamic degrees of freedom remain unconstrained. The lateral stiffness matrix can also be determined by inversing the lateral flexibility matrix.

The flexibility matrix:

$$[U] = \begin{bmatrix} \delta_{11} & \delta_{12} & \delta_{13} \\ \delta_{21} & \delta_{22} & \delta_{23} \\ \delta_{31} & \delta_{32} & \delta_{33} \end{bmatrix} = \begin{bmatrix} 70.313 & 175.783 & 281.253 \\ 175.783 & 562.504 & 984.380 \\ 281.253 & 984.380 & 1898.446 \end{bmatrix} \cdot 10^{-9} \left[\frac{m}{N}\right] \tag{11}$$

The stiffness matrix:

$$[K] = [U]^{-1} = \begin{bmatrix} 8.752 \cdot 10^7 & -5.032 \cdot 10^7 & 1.313 \cdot 10^7 \\ -5.032 \cdot 10^7 & 4.814 \cdot 10^7 & -1.75 \cdot 10^7 \\ 1.313 \cdot 10^7 & -1.75 \cdot 10^7 & 7.659 \cdot 10^6 \end{bmatrix} \left[\frac{N}{m}\right] \tag{12}$$

The mass matrix:

- SNiP II-7-81

$$[M] = \begin{bmatrix} 2.104 \cdot 10^4 & 0 & 0 \\ 0 & 2.104 \cdot 10^4 & 0 \\ 0 & 0 & 2.126 \cdot 10^4 \end{bmatrix} [kg] \tag{13}$$

- Eurocode 8

$$[M] = \begin{bmatrix} 3.432 \cdot 10^4 & 0 & 0 \\ 0 & 3.432 \cdot 10^4 & 0 \\ 0 & 0 & 3.886 \cdot 10^4 \end{bmatrix} [kg] \tag{14}$$

The motion equation is:

$$[M]\{\ddot{U}\} + [C]\{\dot{U}\} + [K]\{U\} = -[M]\{1\}\ddot{u_g}(t) \tag{15}$$

The eigenvalue problem can be solved using the following relationships:

$$([K] - \omega^2[M])[\Phi] = 0 \tag{16}$$

$$|[K] - \omega^2[M]| = 0 \tag{17}$$

Spectral matrix and mode shape matrix:

- SNiP II-7-81

$$\Omega^2 = \begin{bmatrix} 5963.411 & 0 & 0 \\ 0 & 824.623 & 0 \\ 0 & 0 & 19.123 \end{bmatrix} \left[\left(\frac{rad}{s}\right)^2\right] \tag{18}$$

$$\Phi = \begin{bmatrix} 1 & 1 & 1 \\ 3.399 & 1.191 & -0.699 \\ 6.395 & -0.781 & 0.213 \end{bmatrix} \tag{19}$$

- Eurocode 8

$$\Omega^2 = \begin{bmatrix} 3644.906 & 0 & 0 \\ 0 & 494.535 & 0 \\ 0 & 0 & 10.730 \end{bmatrix} \left[\left(\frac{rad}{s}\right)^2\right] \tag{20}$$

$$\Phi = \begin{bmatrix} 1 & 1 & 1 \\ 3.408 & 1.217 & -0.697 \\ 6.425 & -0.708 & 0.189 \end{bmatrix} \tag{21}$$

Frequencies and natural periods of structures are:

- SNiP II-7-81

$$\omega = \begin{bmatrix} 4.373 & 0 & 0 \\ 0 & 28.716 & 0 \\ 0 & 0 & 77.223 \end{bmatrix} \left[\frac{rad}{s}\right] \tag{22}$$

$$T = \begin{bmatrix} 1.4368 & 0 & 0 \\ 0 & 0.2188 & 0 \\ 0 & 0 & 0.0814 \end{bmatrix} [s] \tag{23}$$

- Eurocode 8

$$\omega = \begin{bmatrix} 3.276 & 0 & 0 \\ 0 & 22.238 & 0 \\ 0 & 0 & 60.373 \end{bmatrix} \left[\frac{rad}{s}\right] \tag{24}$$

$$T = \begin{bmatrix} 1.9181 & 0 & 0 \\ 0 & 0.2825 & 0 \\ 0 & 0 & 0.1040 \end{bmatrix} [s] \tag{25}$$

3.4. Calculation of seismic forces

In this section, we will calculate the seismic forces according to SNiP II-7-81 and Eurocode 8. Initially, we will perform the calculation following the provisions of SNiP II-7-81.

- SNiP II-7-81

$$S_{ik} = K_1 \cdot K_2 \cdot Q_k \cdot A \cdot \beta_i \cdot K_\Psi \cdot \eta_{ik} = K \cdot Q_k \cdot \beta_i \cdot \eta_{ik}, \tag{26}$$

where:
$K_1 = 0,25$ – behavior factor;
$K_2 = 1$ – coefficient that takes into account the structure type;
$A = 0,2$ – coefficient that takes into account the seismicity of the site;
$K_\Psi = 1$ – shape coefficient.

Multiple coefficient K:

$$K = 0.25 \cdot 1 \cdot 0.2 \cdot 1 = 0.05 \tag{27}$$

In accordance with p.2.6 of SNiP II-7-81 for soil category III and vibration periods $T_i > 0.5$, the dynamic coefficient is computed by following expression:

$$\beta_1 = \frac{1.35}{T_1} \leq 0.8 \tag{28}$$

$$\beta_1 = \frac{1.35}{1.4368} = 0.94 \rightarrow \beta_1 = 0.8 \tag{29}$$

In accordance with p.2.6 of SNiP II-7-81 for soil category III and vibration periods $0.1 < T_i < 0.5$, the dynamic coefficient is computed by following expression:

$$T_2 = 0.2188\, s \rightarrow \beta_2 = 2.7 \tag{30}$$

In accordance with p.2.6 of SNiP II-7-81 for soil category III and vibration periods $T_i < 0.1$, the dynamic coefficient is computed by following expression:

$$T_3 = 0.0814\ s \ \rightarrow \ \beta_3 = 17 \cdot T_i + 1 = 2.384 \tag{31}$$

Relation for computing form coefficients can be found in SNiP II-7-81, p. 2.7.

$$\eta_{ik} = \frac{X_i(x_2)\sum_{j=1}^{2} Q_j X_i(x_j)}{\sum_{j=1}^{2} Q_j X_i^2(x_j)} \tag{32}$$

- For I mode of vibration:

$$\eta_{11} = 0.202,\ \ \eta_{12} = 0.685,\ \ \eta_{13} = 1.296 \tag{33}$$

- For II mode of vibration:

$$\eta_{21} = 0.462,\ \ \eta_{22} = 0.550,\ \ \eta_{23} = -0.361 \tag{34}$$

- For III mode of vibration:

$$\eta_{31} = 0.336,\ \ \eta_{32} = -0.235,\ \ \eta_{33} = 0.072 \tag{35}$$

Seismic force for each mode:

- For I mode of vibration:

$$S_{11} = 1.664\ [kN],\ \ S_{12} = 5.655\ [kN],\ \ S_{13} = 10.807\ [kN] \tag{36}$$

- For II mode of vibration:

$$S_{21} = 12.87\ [kN],\ \ S_{22} = 15.328\ [kN],\ \ S_{23} = -10.154\ [kN] \tag{37}$$

- For III mode of vibration:

$$S_{31} = 8.274\ [kN],\ \ S_{32} = -5.783\ [kN],\ \ S_{33} = 1.78\ [kN] \tag{38}$$

The resulted vector of forces on each story:

$$S_1 = S_{11} + S_{12} + \ S_{13} = 18.126\ [kN] \tag{39}$$

$$S_2 = S_{21} + S_{22} + \ S_{23} = 18.044\ [kN] \tag{40}$$

$$S_3 = S_{31} + S_{32} + \ S_{33} = 4.271\ [kN] \tag{41}$$

Following that, we will conduct the calculation in accordance with the guidelines outlined in Eurocode 8.

The seismic force, as the base shear force, in the k mode of oscillation is calculated with the relation:

$$F_{b,k} = \gamma_I \cdot S_d(T_k) \cdot m_k, \tag{42}$$

where:

γ_I – the factor of the building importance;

$S_d(T_i)$ – the design value of elastic response spectrum of the k mode of oscillation for the horizontal components of the ground motion, $[m/s^2]$;

m – the modal mass associated with the eigenmode of oscillation and is determined with the relation:

$$m_k = \frac{\left(\sum_{i=1}^{n} m_i s_{i,k}\right)^2}{\sum_{i=1}^{n} m_i {s_{i,k}}^2}, \tag{43}$$

$\gamma_I = 1$ importance class II.

The behavior factor $\boldsymbol{q}$ is defined as a function of the dissipation capacity of the structural system, through its base value $\boldsymbol{q_0}$ and the ratio $\boldsymbol{\alpha_u/\alpha_1}$ due to the redundancy or overresistance of the structure:

$$q = q_0 \frac{\alpha_u}{\alpha_1} \tag{44}$$

Multi-story frames or dual structures equivalent to frames: $\alpha_u/\alpha_1 = 1.3$.

For medium ductility class structures (DCM):

$$q = 3{,}0 \cdot \alpha_u/\alpha_1 = 3.9 \tag{45}$$

T_1, T_2, T_3 are natural period of vibration for fundamental modes of vibrations.

For $T_1 = 1.918\ s$

$$S_d(T_1) = a_g \cdot S \cdot \frac{2.5}{q} \cdot \left(\frac{T_c}{T}\right) = 0.24 \cdot 9.81 \cdot 1 \cdot \frac{2.5}{3.9} \cdot \frac{0.7}{1.918} = 0.551 \tag{46}$$

For $T_2 = 0.2825\ s$

$$S_d(T_1) = a_g \cdot S \cdot \frac{2.5}{q} = 0.24 \cdot 9.81 \cdot 1 \cdot \frac{2.5}{3.9} = 1.509 \tag{47}$$

For $T_3 = 0.1041\ s$

$$S_d(T_1) = a_g \cdot S \cdot \frac{2.5}{q} = 0.24 \cdot 9.81 \cdot 1 \cdot \frac{2.5}{3.9} = 1.509 \tag{48}$$

Calculation of the effective modal mass associated with the eigenmode of vibration:

$$m_k = \frac{\left(\sum_{i=1}^{n} m_i s_{i,k}\right)^2}{\sum_{i=1}^{n} m_i {s_{i,k}}^2} \tag{49}$$

$$m_1 = 78919.974\ [kg],\ \ m_2 = 35928.448\ [kg],\ \ m_3 = 6010.410\ [kg] \tag{50}$$

Seismic force, as the base shear force:

$$F_{b,1} = 43.484\ [kN],\ \ F_{b,2} = 54.216\ [kN],\ \ F_{b,3} = 9.114\ [kN] \tag{51}$$

The results of the comparative calculation of seismic forces according to SNiP and Eurocode are centralized in Table 4.

Table 4

Comparison of seismic forces

Mode	**Forces at story according to SNiP II-7-81, kN**	**Forces at story according to EC 8, kN**	**Ratio, $\frac{EC-8}{SNiP\ II-7-81}$**
1 mode	18.126	43.484	2.398
2 mode	18.044	54.216	3.004
3 mode	4.271	9.114	2.133

In this chapter, we embark on a comparative analysis of seismic calculations, aiming to elucidate the nuances of seismic design methodologies and their implications for structural outcomes. The study commenced with the utilization of two different regulatory standards: SNiP II-7-81 and Eurocode 8. The structural model, materials, and site characteristics remained consistent throughout the analysis, ensuring a fair and accurate comparison.

Seismic calculations were performed following the prescribed procedures outlined in each standard. It was observed that the Eurocode 8 approach tends to result in higher seismic forces in certain structural elements, reflecting a more conservative approach to seismic design. Conversely, the SNiP II-7-81 methodology, with its unique coefficients, demonstrates a nuanced consideration of the seismic environment.

4. Conclusion

The SNiP II-7-81 elaborated in 1981 do not have any significant modifications for over 40 years. On other side, Eurocode 8 that consists of 6 parts provides detailed information on every step of design.

Both normative have different approach of quantifying the seismic action i.e. ground motion. Nevertheless, the basis on which the hazard maps are made is the same – probabilistic seismic hazard analysis.

The soil classification is different in both codes. In Eurocode the soils are classified in categories by the shear wave velocities, SNiP II-7-81 that divides by mechanical proprieties of soil. The elastic response spectrum has similar shapes, which means that the structures with lower natural period of structure have higher acceleration values, and structures with high natural period that will have smaller acceleration values.

Behavior factor is as important coefficient in both codes. In Eurocode 8 the behavior factor is described more accurately, for each type of structure, and take into account energy absorption during a seismic event. In SNiP II-7-81 the behavior factor imposes the possibility or avoid the dissipation of the energy by plastic hinges or another plastic deformation in the structure. Even plastic hinges are allowed, the people safety have to be satisfied.

A comparative example had been performed for a simple 3 story structure. The result shown in table 9 denotes that final result using EN 1998 normative are higher than the results using SNiP II-7-81. The main cause is in coefficient that are used for load case combination. Other reason could be the impact of the response spectrum. The dynamic coefficient values are higher in SNiP II-7-81 then the seismic acceleration from spectrum response of Eurocode 8.

Conflicts of Interest: The authors declare no conflict of interest.

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