MINISTRY OF EDUCATION AND RESEARCH

TECHNICAL UNIVERSITY OF CIVIL ENGINEERING BUCHAREST

FACULTY OF RAILWAYS, ROADS AND BRIDGES

RESEARCH REPORT No. 1

PhD. Candidate

Eng. Șimon Vlad-Mihai

Description of the seismic action in national, european and international standards. Methods of describing the seismic action

Scientific Supervisor

Prof. Dr. Eng. Nicolae POPA

BUCHAREST 2022

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PREFACE

Earthquakes belong to the category of particularly dangerous phenomena, as they are not accompanied by warning signs. For this reason, structures have suffered greatly because of them, which means that bridges are also subject to the effects of these events. Among the earthquake effects that lead to the degradation or even destruction of structures, the most significant are changes in the physical properties of the foundation soil, inertia forces induced in the structure, landslides, fires, etc.

This research report provides an overview of the codes describing seismic action, so that during an earthquake, design is carried out with an adequate level of safety to withstand design-level seismic action, maintaining a sufficient safety margin against deformation levels that lead to local or global collapse, thereby protecting human lives.

Regarding conditions in our country, given that Romania is a seismically active region, significant attention must be paid to protecting structures against damage that could occur during major earthquakes. The first seismic design code introduced in our country was P100-92. As global requirements for seismic protection evolved, improved versions of this code were issued in 2004, 2006, and 2013. These latter versions introduce significant changes, both in terms of seismic hazard levels and, especially, regarding structural details that provide higher safety factors.

In terms of international standards, the provisions of European Eurocode 8 and the American FEMA 356 standard are presented. These focus primarily on the influence of local soil conditions on seismic action, a critical factor to consider, as well as the methods used to describe seismic action in structural analysis.

As a case study, the modifications introduced by the P100-2006 and P100-2013 codes will be analyzed for a structure located in the Bucharest area designed under P100-92 guidelines, followed by proposed retrofitting solutions to satisfy the increased seismic protection requirements.

1. Description of the seismic action in national, european and international standards

The seismic action represents an event with a low probability of occurrence but catastrophic effects, serving as the main cause of casualties and material damage across large regions of the globe. The high frequency of destructive seismic events has driven changes in structural detailing and design principles, as well as the development of modern seismic codes.

The 1977 Vrancea Earthquake

The earthquake occurred on March 4, 1977, with a magnitude of 7.2 on the Richter scale, making it the second most powerful earthquake in Romania during the 20th century, after the November 1940 event. The epicenter was located in the Vrancea region, Romania's most active seismic zone, at a depth of 94 km.

As a result of the earthquake, approximately 1,600 people lost their lives, 90% of them in Bucharest, primarily due to building collapses. Over 11,000 people were injured, around 33,000 buildings were damaged, and 760 industrial units were affected. In Bucharest alone, more than 30 buildings collapsed.

A site-specific design spectrum is determined based on seismic hazard maps, characterized by the peak ground acceleration value a_g, and the soil stiffness, characterized by the corner period T_C.

At the time of the 1977 earthquake, the design spectrum was defined according to the "Code for the Design of Civil and Industrial Buildings in Seismic Regions," P13-70. The acceleration values reached during the earthquake were significantly higher than the design spectrum values specified in this code, causing extensive damage. Consequently, the 1977 Vrancea earthquake formed the basis for developing subsequent design spectra.

Figure 1-1 illustrates the comparison between the response spectrum of the 1977 earthquake, the design spectrum defined according to P13-70, and the design spectrum defined according to P100-2013.

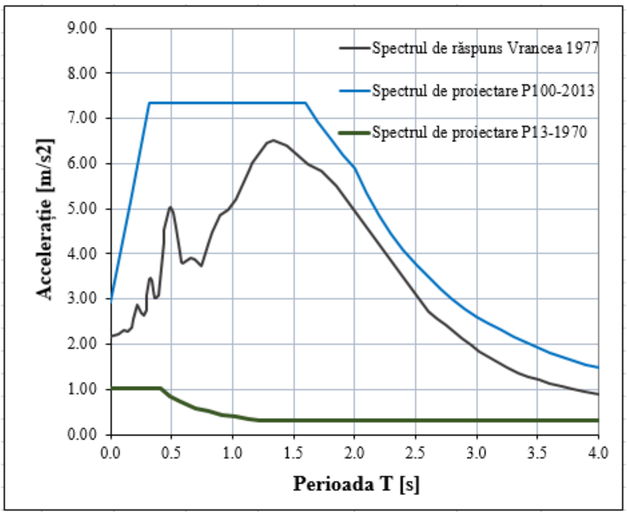

Figure 1-1 Comparison between the response spectrum of the March 1977 earthquake, the design spectrum in force at that time (P13-1970), and the current design spectrum (P100/2013)

The 1985 Michoacan Earthquake

The earthquake occurred on September 19, 1985, with a magnitude of 8.0 on the Richter scale, with its epicenter in Michoacán, Mexico. It was a major seismic event that resulted in a very large loss of human life and material damage, particularly in the capital, Mexico City, 350 km from the epicenter. The most severely affected structures were recently constructed high-rise buildings designed for earthquake resistance.

The collapse of these buildings was the result of local soil conditions in the area of the old lakebed on which Mexico City is built. The soft soil in this region led to an amplification of the seismic motion, where design accelerations were exceeded by up to four times. This was observed from accelerograms recorded at several stations across the city, with the most noticeable difference occurring between records from the UNAM station, located in the hill area, and the SCT station, situated on the former lakebed.

In the case of the UNAM station, the vibrations had a high frequency and lower amplitude, whereas at the SCT station, both the vibration period and amplitudes were significantly larger.

The Michoacán earthquake represented a defining turning point for Mexican seismic design codes, much like the Vrancea earthquake did for Romanian codes.

The figure below (Figure 1-2) shows a comparison between the response spectrum of the Michoacán earthquake, the design spectrum in force at the time of the event, and the current design spectrum.

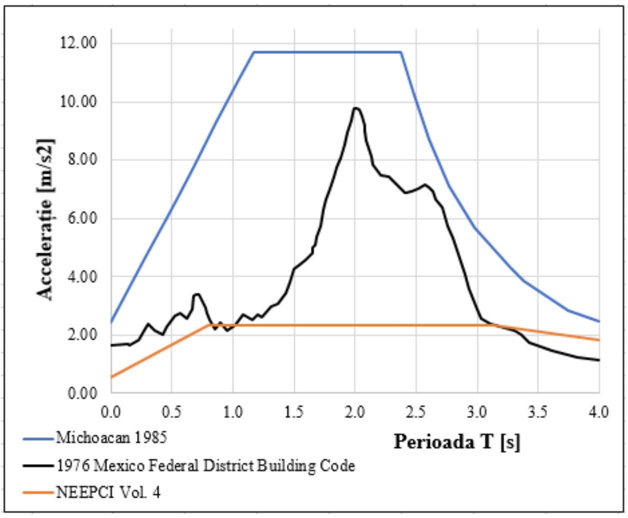

Figure 1-2 Comparison between the response spectrum of the Michoacan Earthquake, design spectrum in force at that time and the current design spectrum

The 2015 Nepal Earthquake

Also known as the Himalayan earthquake, it took place on April 25, 2015, with a magnitude of 7.8 on the Richter scale, and its epicenter was located in the Lamjung District, approximately 80 km from Nepal's capital, Kathmandu. The seismic motion lasted approximately 90–100 seconds, followed by 4 strong aftershocks:

- On April 25, with a magnitude of 6.6 on the Richter scale;
- On April 26, with a magnitude of 6.7 on the Richter scale;
- On May 12, with a magnitude of 7.3 on the Richter scale;

- On May 12, with a magnitude of 6.3 on the Richter scale.

As a result of the earthquake, approximately 8,600 people lost their lives and nearly 19,000 injuries were reported, the vast majority of which occurred in Nepal. Approximately 500,000 buildings were completely destroyed, while 260,000 were partially destroyed; out of 75 districts, 35 were affected across central and western Nepal.

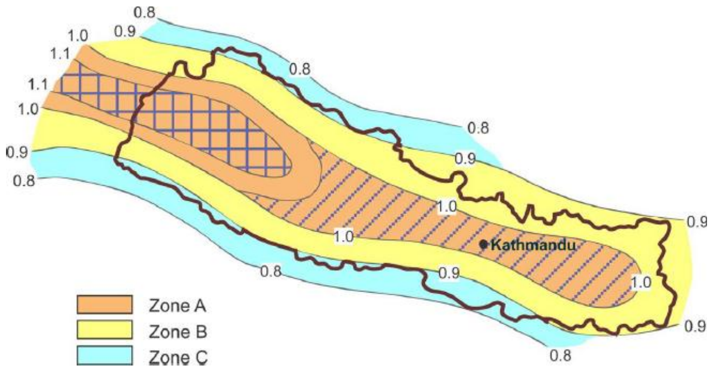

Figure 1-3 Seismic zoning of Nepal – seismic hazard is moderate-to-severe for aproximatelly 90% of the country's territory [14]

By comparing with the design spectrum in force at the time of the earthquake, it can be observed (Figure 1-4) that modifications to it are necessary due to the significantly higher amplifications in the response spectra of the recorded motions. Although the accelerations for the eastern horizontal component in the short-period range remain within the limits defined by the code, for periods greater than 1.00 s, the recorded accelerations are significantly higher than those defined by the design spectrum. Similarly, for the northern horizontal component, a large amplification is observed for periods around 0.50 s, which also exceeds the values specified by the standard in this case.

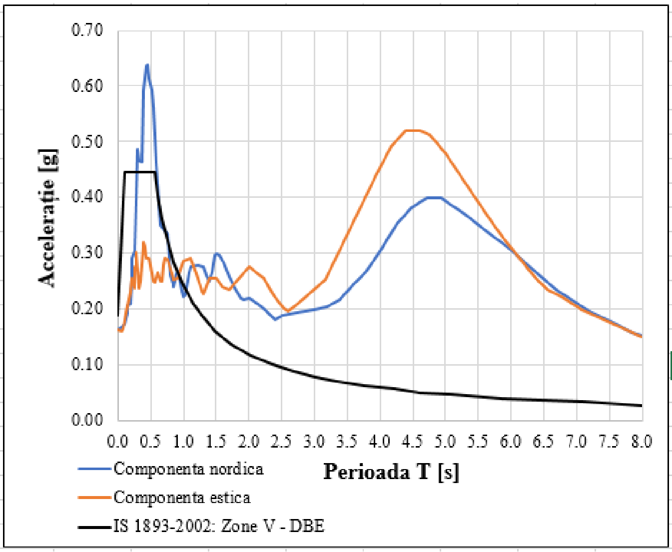

Figure 1-4 Comparison between the response spectra of the recorded motions and the design spectrum according to the Indian standards in force at that time [14]

1.1 Seismic action according to the P100-1992 Code

The "Code for the Seismic Design of residential, socio-cultural, agro-zootechnical, and industrial buildings - P100-92" came into force on May 1, 1992, replacing the code of the same name with designation P100-91.

Seismic design methods for load-bearing structures are classified into two main categories based on how the seismic action is modeled, the fidelity of the structural model, and the verification procedure:

Standard Design Method (Method "A")

This is a mandatory method for all structures covered by this code. Dynamic and non-linear behavior aspects are considered in a simplified manner. Method "A" involves the following main operations:

- determination of the design seismic force;
- structural analysis in the linear elastic domain under seismic loads and other loads from the special load combination;
- moment redistribution at the ends of structural elements to achieve a favorable energy dissipation mechanism;
- cross-sectional design and detailing of structural elements;
- verification of lateral displacement and ductility conditions.[1]

Design Method Based on Structural Non-linear Deformation Properties (Method "B")

This method is based on non-linear dynamic analysis. The seismic response is determined, subject to certain idealizations, at each time step under seismic excitation represented by accelerograms. The plastic hinge mechanism is selected through successive adjustments of the resistance and stiffness parameters of the structural elements.

Methods for Considering Seismic Action in Structural Analysis

The following modes of seismic action on structures are considered:

- inertial forces generated by structural oscillations resulting from acceleration at the soil-structure interface;
- internal forces generated by differential displacements imposed on parts of the structure due to seismic wave propagation through the ground;
- forces transmitted by support systems.

Code P100-1992 includes provisions for determining inertial forces, as well as guidelines for incorporating the mode of seismic action into analysis.

In dynamic analysis methods (whether linear or non-linear), seismic excitation is represented by site-recorded accelerograms or artificially generated accelerograms.

The relationships below establish the equivalent static loads used in calculations, implicitly accounting for inelastic response and deformation phenomena.

Horizontal seismic loads are determined using equations 1.1-1 and 1.1-2 for each mode of vibration.

$$S = c_r G \tag{1.1-1}$$

where:

$$c_r = a\, k_s\, \beta_r\, \psi\, \varepsilon_r \quad (1.1\text{-}2)$$

c_r – global seismic coefficient corresponding to vibration mode "r";

G – resultant of gravity loads for the entire structure;

a – importance factor of the building, based on importance classes (Table 1.1-1);

k_s – seismic zone coefficient for the site's design seismic zone;

β_r – dynamic amplification factor for vibration mode "r", based on the spectral composition of the seismic ground motion at the site;

ψ – reduction factor accounting for ductility, force redistribution capacity, structural overstrength reserves, and damping effects other than those inherent to the primary load-bearing structure;

ε_r – equivalence factor between the actual multi-degree-of-freedom system and a single-degree-of-freedom system corresponding to mode "r".

The factors "a" differentiate the level of seismic protection for structures based on their importance classes. Buildings are categorized into 4 importance classes (Table 1.1-1), with a safety factor assigned to each class.

Importance Class		Factor
I	Structures of vital importance to society, whose operational continuity during and immediately after an earthquake must be fully ensured (hospitals, fire stations, etc.)	1.40
II	Structures of special importance where damage limitation is required, considering the consequences of failure (schools, nursing homes, auditoriums/theaters, etc.)	1.20
III	Structures of normal importance (residential buildings)	1.00
IV	Structures of minor importance (single-story or two-story residential buildings)	0.80

Table 1.1-1 Importance classes and corresponding factors

The coefficient "k_s" represents the ratio of the peak ground acceleration of the seismic motion (considering a mean return period of 50 years) corresponding to the design seismic zone (Table 1.1-2, Figure 1.1-1).

Design seismic zone	A	B	C	D	E	F
k_S	0.32	0.25	0.20	0.16	0.12	0.08

Table 1.1-2 Values of the k_s coefficient

Figure 1.1-1 Seismic zoning of the territory of Romania in terms of k_s coefficient values [1]

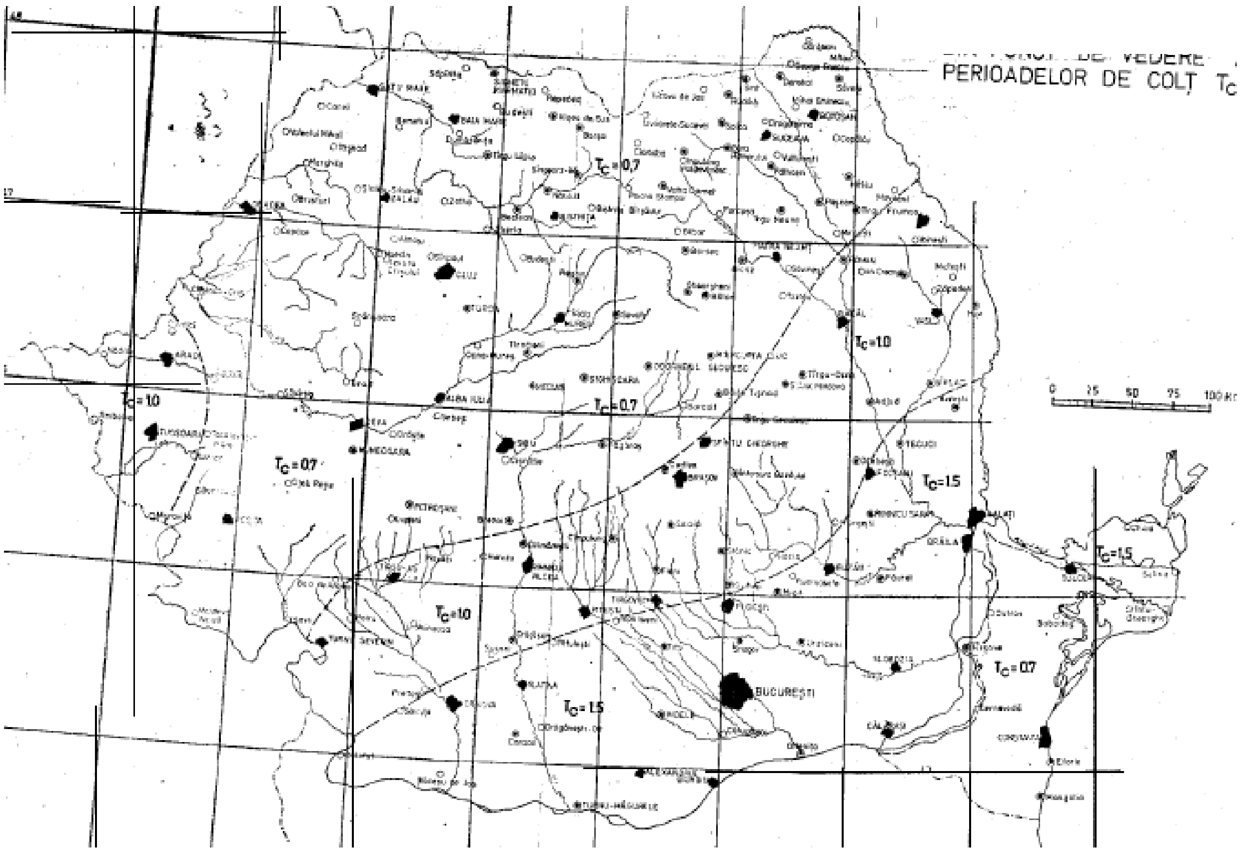

Figure 1.1-2 Seismic zoning of the territory of Romania in terms of corner period, T_c [1]

The dynamic amplification factor "β_r" is determined as a function of the fundamental vibration periods "T_r" of the structure and the seismic conditions of the region, which are characterized by the corner periods "T_C" (Figure 1.1-2), using the following equations:

$$\beta_r = 2.50 \text{ for } T_r \leq T_C \tag{1.1-3}$$

$$\beta_r = 2.50 - (T_r - T_C) \text{ for } T_r \leq T_C \tag{1.1-4}$$

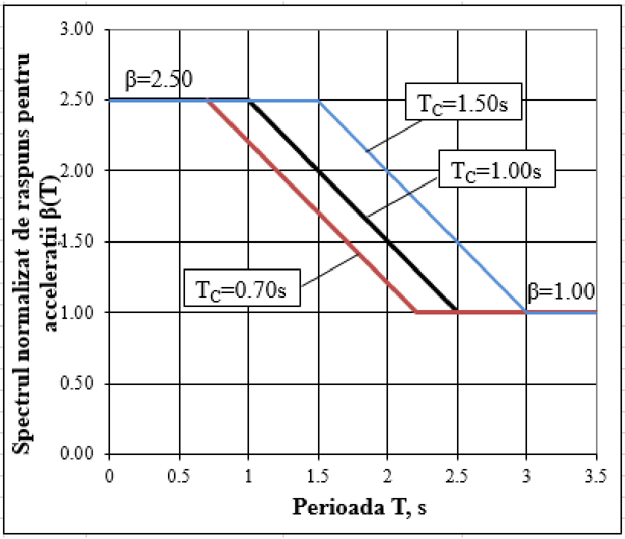

Figure 1.1-2 Normalized elastic response spectra of absolute accelerations

The coefficient "ψ" takes a wide range of values depending on the type of structure. In the case of bridges, energy dissipation mechanisms are assumed to develop at the base of the pier stems. For these elements, the coefficient "ψ" ranges between 0.25 and 0.35.

The coefficient "ε_r" is determined using the following relation:

$$\varepsilon_r = \frac{(\sum_{k=1}^{n} G_k u_{kr})^2}{G \sum_{k=1}^{n} G_k u_{kr}^2} \tag{1.1-5}$$

where: u_{kr} – component along degree of freedom "k" of the eigenvector corresponding to mode "r";

G_k – resultant of gravity loads at level "k".

The seismic load acting at level "k" along the direction of the corresponding degree of freedom for vibration mode "r" is determined using the following equation:

$$S_{kr} = c_{kr} G_k \tag{1.1-6}$$

where:

$$c_{kr} = a\, k_s\, \beta_r\, \psi\, \eta_{kr} \tag{1.1-7}$$

η_{kr} – distribution coefficient of seismic forces corresponding to level "k" and vibration mode "r"

$$\eta_{kr} = u_{kr} \frac{\sum_{k=1}^{n} G_k u_{kr}}{\sum_{k=1}^{n} G_k u_{kr}^2} \tag{1.1-8}$$

The total design internal force "N" in a cross-section of an element is determined using the following equation:

$$N = \sqrt{\sum_r N_r^2} \quad \text{1.1-8}$$

Determination of vertical seismic loads

Verification for vertical seismic actions is necessary for the following types of elements forming part of the primary load-bearing structure:

- elements subject to predominant axial forces (columns, tie-rods, etc.);
- beams and cantilevers subjected to high shear forces originating primarily from large concentrated vertical loads and long spans.

Vertical seismic loads are determined by multiplying the design gravity loads of the respective elements by the factor "c_v" and are superimposed on the gravity loads or directly on the corresponding internal forces.

Type of element	Modified load or internal force	Factor c_v
Element with predominant axial forces	Axial force of the element	$\pm$ k_s
Frame beams with high shear forces	Shear force resulting from gravity loads	$\pm$ 1.50 k_s
Beams with large concentrated vertical loads	Concentrated loads	$\pm$ 1.50 k_s
Flat slabs supported directly on columns, without beams	Support shear force at columns (for punching shear verification)	$\pm$ 2.00 k_s
Cantilevers with long spans	Total gravity load	$\pm$ 1.50 k_s

Table 1.1-3 Values of the factor c_v

1.2 Seismic action according to the P100-2006 Code

The P100-1/2006 seismic design code applies to the earthquake-resistant design of civil engineering structures. It aligns with SR EN 1998-1:2004 and represents a modernized revision of Romanian design prescriptions integrated into the European standard framework.

The implementation of code P100-1/2006 aims to guarantee adequate structural performance during seismic events in order to:

- prevent loss of human life;
- maintain uninterrupted operation of essential social and economic activities both during and after an earthquake;
- limit material damage;
- prevent explosions or the release of hazardous substances.

Performance requirements

Seismic design must satisfy the following fundamental performance requirements:

- *Life Safety requirement*

The structure must be designed to withstand seismic actions with an adequate safety margin against localized or global collapse. These seismic forces correspond to a reference mean return period of IMR=100 years [12].

For well-designed structures, satisfying the life safety criteria for IMR = 100 ani generally ensures compliance with collapse prevention requirements for an earthquake with IMR = 475 years [12].

- *Damage Limitation requirement*

The structure must be designed to resist seismic actions with a higher probability of occurrence than the design seismic action without suffering damage whose repair costs would be disproportionate to the value of the structure. These seismic actions correspond to a reference mean return period of IMR = 30 years.

Representation of seismic action for design

For structural design against seismic actions, the territory of Romania is divided into seismic hazard zones, with the seismic hazard level assumed to be constant within each zone.

Design seismic hazard is defined by the peak ground acceleration value, a_g, determined for the reference mean return period of IMR=100 years (Figure 1.2-1).

Ground motion at a point on the terrain surface is described by the elastic response spectrum for absolute accelerations.

Horizontal seismic action on buildings is represented by two independent, orthogonal horizontal components.

Normalized elastic acceleration response spectra are derived from the elastic acceleration response spectra by dividing the spectral ordinates by the peak ground acceleration value, a_g.

Local soil conditions are characterized by the values of the corner period, T_C, of the response spectrum for the project site. The control period, T_C, of the response spectrum represents the boundary between the plateau of maximum absolute acceleration values and the plateau of maximum relative velocity values.

For IMR=100 years , the zonation of Romania's territory in terms of the corner period, T_C, is presented in Figure 1.2-2.

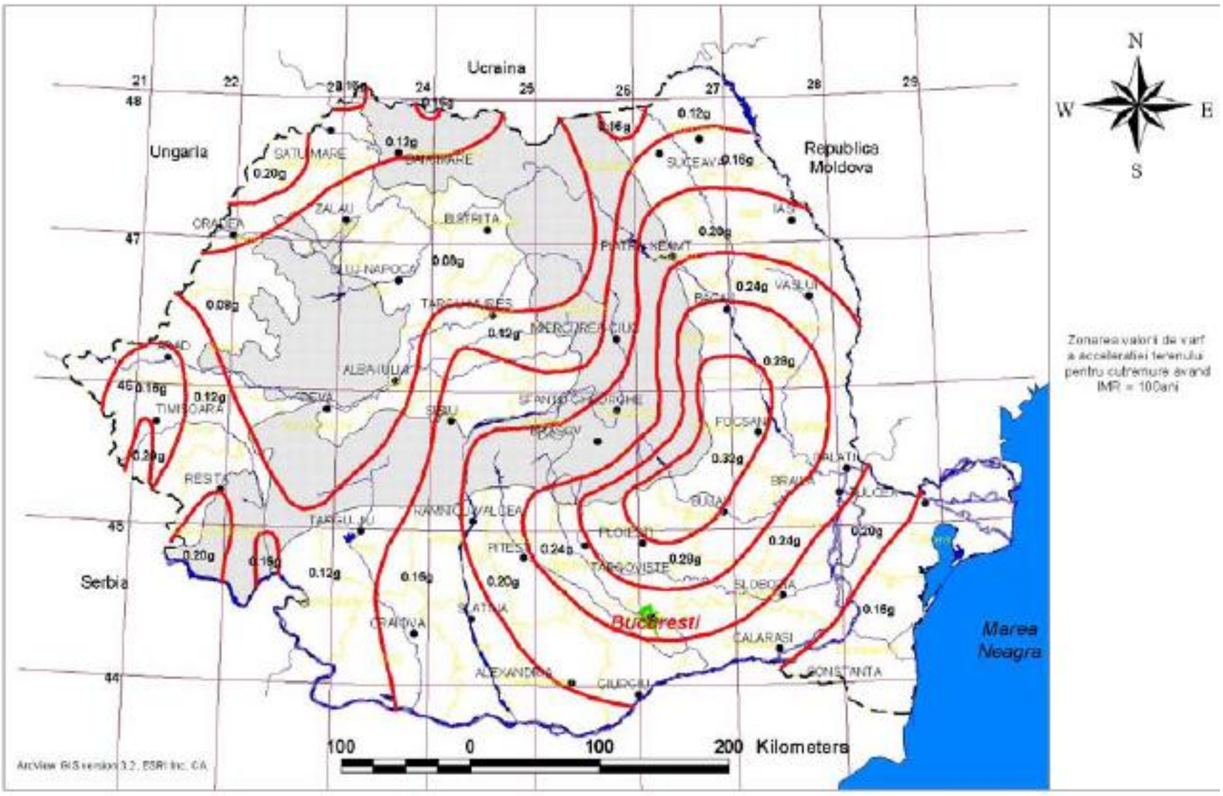

Figure 1.2-1 Zoning of the territory of Romania in terms of peak ground acceleration values for design, a_g, for earthquakes with IMR=100 years [12]

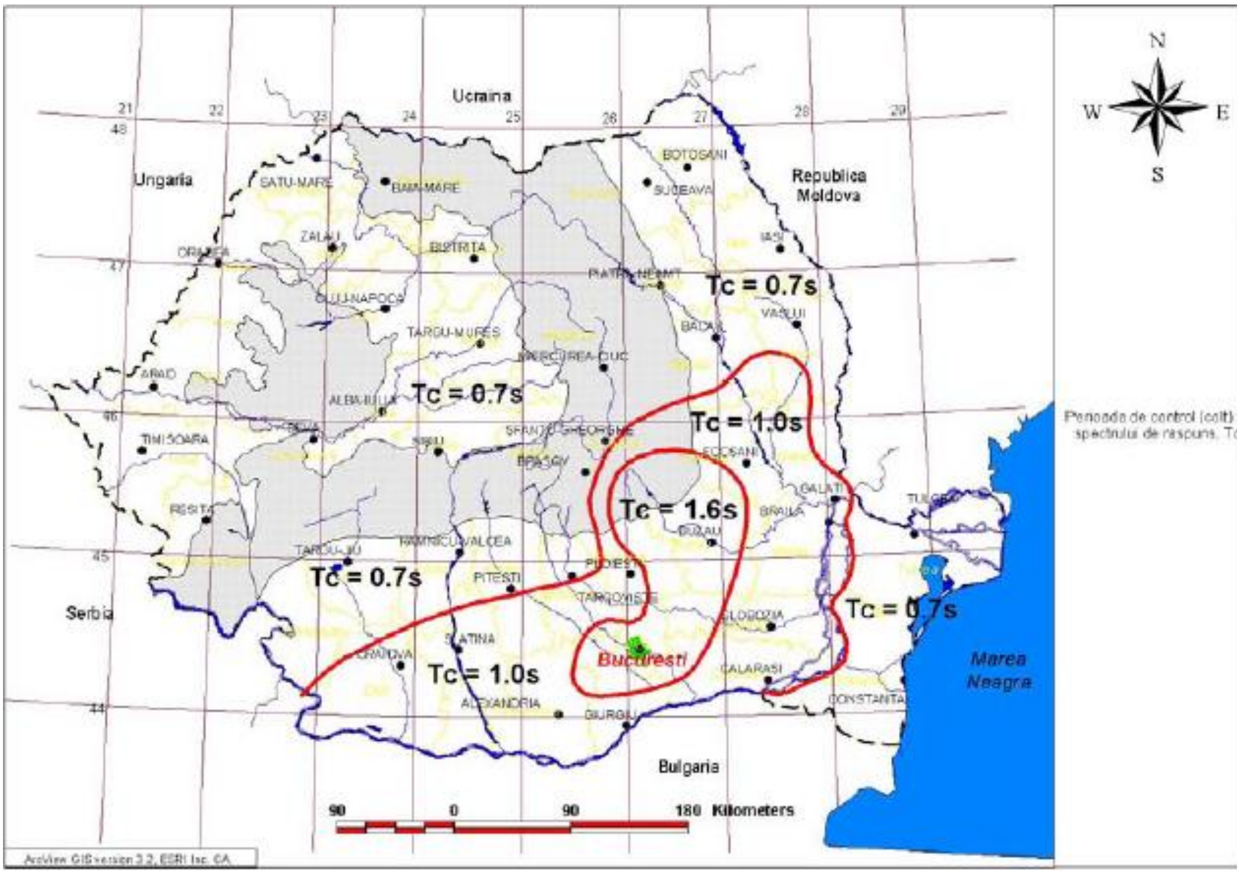

Figure 1.2-2 Zoning of the territory of Romania in terms of corner period, T_c [12]

Normalized elastic response spectrum

The normalized elastic response spectrum for horizontal ground acceleration components, *β(T)*, evaluated for a critical damping ratio of ξ=0.05 and as a function of the corner periods T_B, T_C and T_D is defined by equations (1.2-1) through (1.2-4).

$$\beta(T) = 1 + \frac{(\beta_0 - 1)}{T_B} T, \;\; for\; 0 \le T \le T_B \quad 1.2\text{-}1$$

$$\beta(T) = \beta_0, \;\; for\; T_B < T \le T_C \quad 1.2\text{-}2$$

$$\beta(T) = \beta_0 \frac{T_C}{T}, \;\; for\; T_C < T \le T_D \quad 1.2\text{-}3$$

$$\beta(T) = \beta_0 \frac{T_C T_D}{T^2}, \;\; for\; T > T_D \quad 1.2\text{-}4$$

where:

β(T) – normalized elastic response spectrum;

β_0 – factor of maximum dynamic amplification of horizontal ground acceleration by the structure;

T – vibration period of a single-degree-of-freedom structure with elastic response.

The corner period T_B can be expressed in simplified form as a function of T_C: $T_B = 0.10T_C$.

The corner period T_D of the response spectrum represents the boundary between the plateau of maximum relative velocity values and the plateau of maximum relative displacement values.

T_B și T_C represent the lower and upper bounds of the period range for which the spectral acceleration reaches its maximum values (Figure 1.2-3) [12].

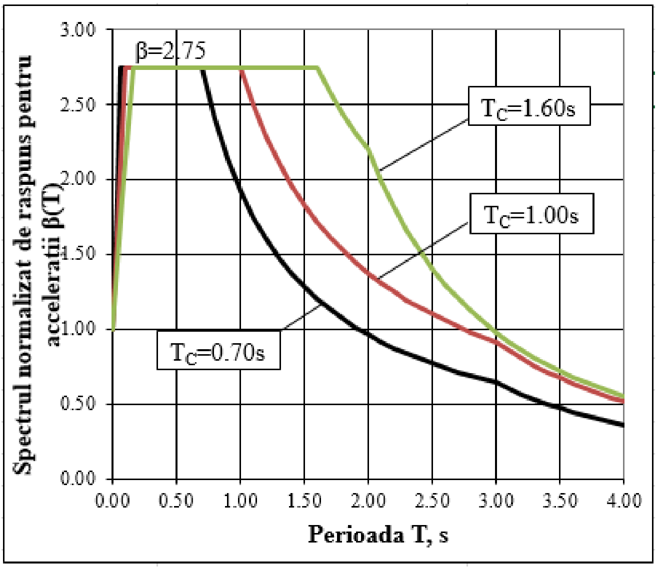

Figure 1.2-3 Normalized elastic response spectra of absolute accelerations

Figure 1.2-4 defines the normalized elastic response spectrum of absolute accelerations for the Banat region, corresponding to $a_g = 0.20g$ și $a_g = 0.16g$.

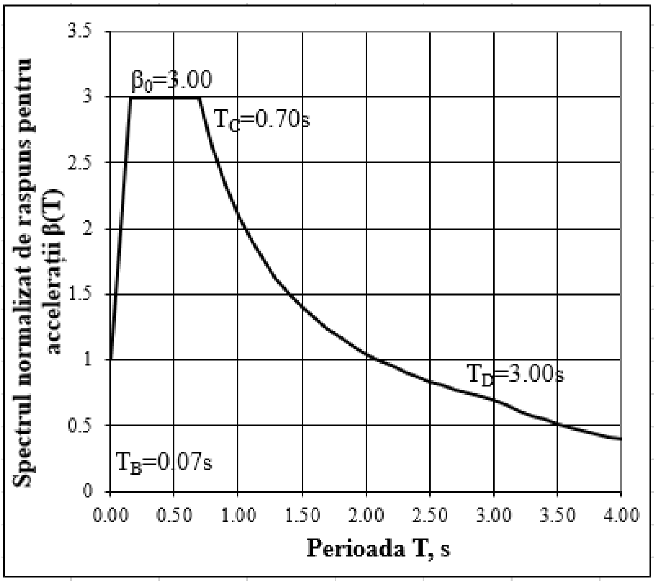

Figure 1.2-4 Normalized elastic acceleration response spectra for horizontal ground motion components for the Banat region, for $a_g = 0.20g$ și $a_g = 0.16g$

The elastic response spectrum for horizontal ground acceleration components at the site, $S_e(T)$, expressed in m/s^2, is defined by equation (1.2-5) [12].

$$S_e(T) = a_g\,\beta(T) \qquad 1.2\text{-}5$$

The elastic displacement response spectrum for horizontal ground motion components, $S_{De}(T)$, expressed in meters, is obtained by direct transformation of the elastic acceleration response spectrum $S_e(T)$ using the following relation:

$$S_{De}(T) = S_e(T)\left[\frac{T}{2\pi}\right]^2 \qquad 1.2\text{-}6$$

The vertical component of the seismic action is represented by the elastic acceleration response spectrum for the vertical ground motion. The normalized response spectrum for the vertical component, $\beta_v(T)$, for a critical damping ratio of ξ=0.05, as a function of the corner periods T_{Bv}, T_{Cv}, T_{Dv} is described by the equations below:

$$\beta_v(T) = 1 + \frac{(\beta_{0v}-1)}{T_{Bv}}T,\ \ for\ 0 \le T \le T_{Bv} \qquad 1.2\text{-}7$$

$$\beta_v(T) = \beta_{0v},\ \ for\ T_{Bv} < T \le T_C \qquad 1.2\text{-}8$$

$$\beta_v(T) = \beta_{0v}\frac{T_{Cv}}{T},\ \ for\ T_{Cv} < T \le T_{Dv} \qquad 1.2\text{-}9$$

$$\beta_v(T) = \beta_{0v}\frac{T_{Cv}T_{Dv}}{T^2},\ \ for\ T > T_{Dv} \qquad 1.2\text{-}10$$

Where β_{0v} is the factor of maximum dynamic amplification of vertical ground motion acceleration for a critical damping ratio of ξ=0.05.

The corner periods of the normalized response spectra for the vertical component of seismic ground motion are determined using the following relations:

$$T_{Bv} = 0.10\,T_{Bv} \qquad 1.2\text{-}11$$

$$T_{Cv} = 0.45\ T_C \tag{1.2-12}$$

$$T_{Dv} = T_D \tag{1.2-13}$$

The elastic response spectrum for the vertical component of ground acceleration at the site, $S_{ve}(T)$ is defined by the following equation:

$$S_{ve}(T) = a_{vg}\beta_v(T) \tag{1.2-14}$$

where $a_{vg} = 0.70\ a_g$ is the peak vertical ground acceleration.

Design Spectrum

This spectrum accounts for the ductility characteristics of the structure, is expressed in m/s^2, and is determined using the following formulas:

$$S_d(T) = a_g\left[1 + \frac{\frac{\beta_0}{q} - 1}{T_B} T\right],\ \ for\ 0 \leq T \leq T_B \tag{1.2-15}$$

$$S_d(T) = a_g\left[\frac{\beta(T)}{q}\right],\ \ for\ T > T_B \tag{1.2-16}$$

where

- q - behavior factor of the structure, which depends on the structural type and its energy dissipation capacity.

The design spectrum for the vertical component of seismic motion is derived in a similar manner, with the provision that the behavior factor q is taken as 1.50 for all structural types and materials.

Importance Classes and Importace Factors

Structures must be differentiated based on their importance and exposure to seismic risk. This differentiation involves assigning an importance class to which a corresponding importance factor (γ_I) is associated. The required safety levels are achieved by multiplying this importance factor by the parameters that define the reference seismic action.

Importance Class		γ_I
I	Buildings with essential functions whose integrity during earthquakes is vital for civil protection (police stations, fire stations, emergency hospitals).	1.40
II	Buildings whose seismic resistance is important due to the severe consequences associated with structural collapse or heavy damage (penitentiaries, nursing homes, schools, residential and public buildings accommodating more than 400 people in the total exposed area).	1.20
III	Ordinary buildings that do not belong to the other categories.	1.00
IV	Structures of minor importance (agricultural buildings or low-occupancy structures).	0.80

Table 1.2-1 Importance Classes and their corresponding factors

1.3 Seismic action according to the P100-1/2013 Code

This standard was issued in May 2013, titled *"Seismic Design Code – Part I, Design Provisions for Buildings"*, and applies to the seismic design of buildings and similar structures located in Romania.

The provisions of this code aim to ensure that, during and after an earthquake:

- human life and physical integrity are protected;
- essential social and economic activities continue unimpeded;
- explosions or releases of hazardous substances are prevented;
- material damage is limited.

Performance requirements

Seismic design must satisfy two fundamental performance requirements:

- *Life Safety requirement*

The structure must be designed to withstand seismic actions with an adequate safety margin against localized or global collapse. The design seismic load corresponds to a reference mean recurrence interval of IMR=225 years (20% probability of exceedance in 50 years) [15].

- *Damage Limitation requirement*

The structure must be designed to withstand seismic actions that have a higher probability of occurrence than the design earthquake, without sustaining damage whose repair costs would be disproportionate to the cost of the structure. This seismic action corresponds to a reference mean recurrence interval of IMR=40 years (20% probability of exceedance in 10 years).

Similarly to the P100-2006, seismic hazard for design is quantified by the peak ground acceleration value, a_g, determined for the reference mean recurrence interval (IMR=225 years). The spatial distribution of a_g across the territory of Romania for IMR=225 years is illustrated in Figure 1.3-1.

The design seismic action, A_{Ed}, is determined by multiplying the characteristic seismic action value, A_{Ek}, by the importance factor, γ_I:

$$A_{Ed} = \gamma_I \cdot A_{Ek} \qquad 1.3\text{-}1$$

Compared to the previous code (P100-1/2006), the increase in seismic hazard levels was necessary to enhance occupant safety and align with the seismic hazard benchmarks recommended by SR EN 1998-1 (Eurocode 8).

The elastic response spectrum for horizontal ground acceleration components at the site, $S_e(T)$, expressed in m/s^2, is defined by equation (1.3-2) [15]:

$$S_e(T) = a_g\, \beta(T) \qquad 1.3\text{-}2$$

where $\beta(T)$ is the normalized elastic response spectrum for absolute horizontal accelerations.

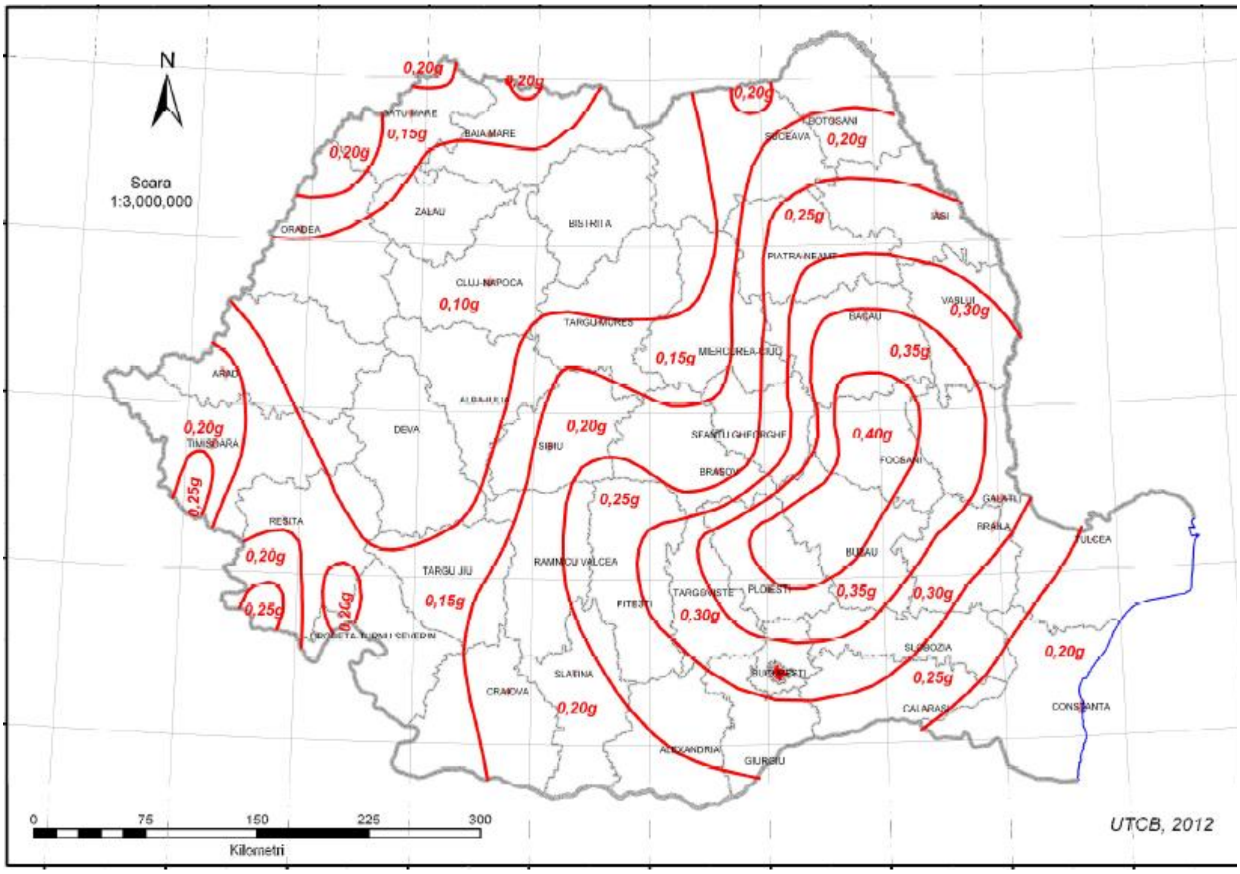

Figure 1.3-1 Zoning of the Romanian territory in terms of peak ground acceleration design values, a_g, for earthquakes with IMR = 225 years and a 20% probability of exceedance in 50 years [15].

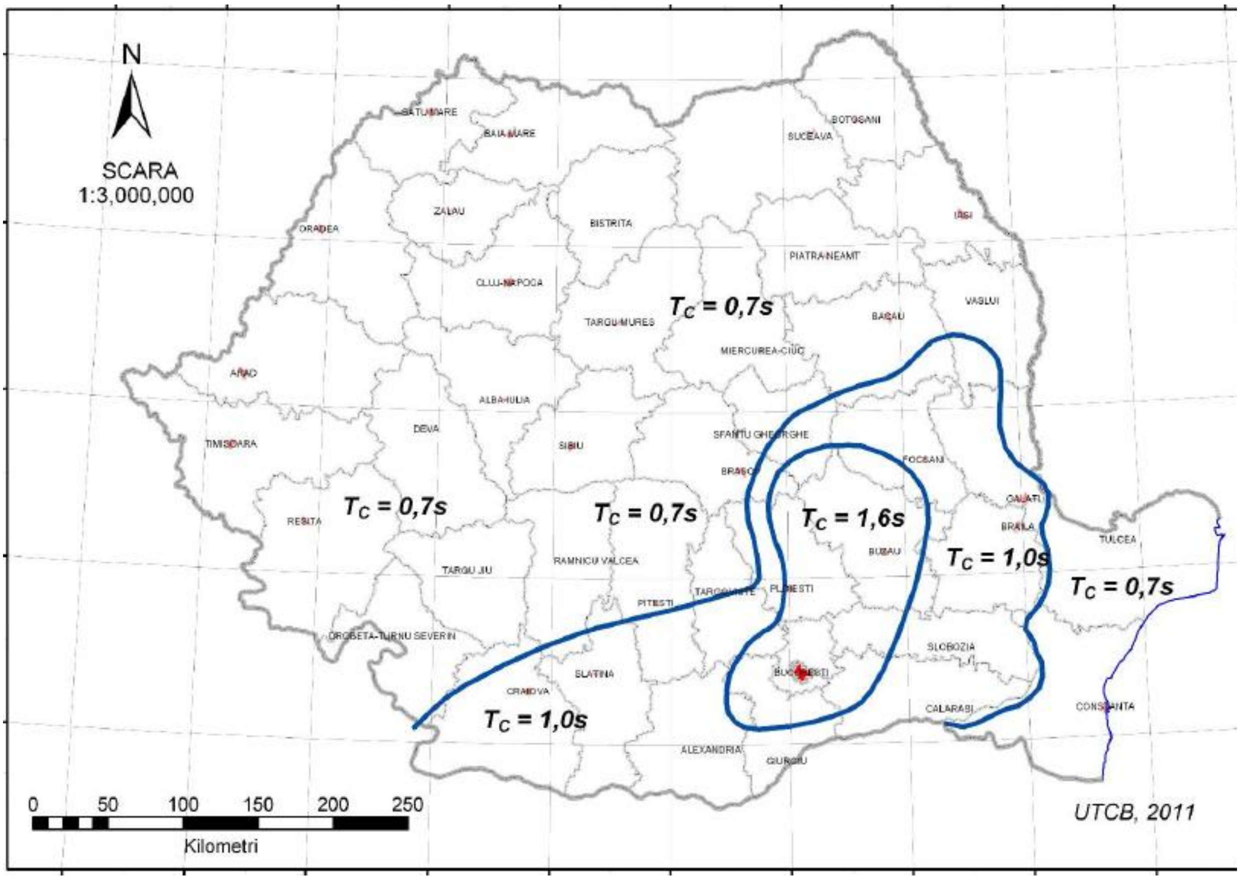

Figure 1.3-2 Zoning of the Romanian territory in terms of corner period, T_c [15]

The formulas used to determine the normalized elastic response spectra of absolute accelerations for the horizontal components of ground motion, *β(T),* for a critical damping ratio of ξ=0.05 (Figure 1.3-3) are defined in the same manner as in P100-1/2006, according to equations (1.2-1) – (1.2-4), with the terms having the same meanings.

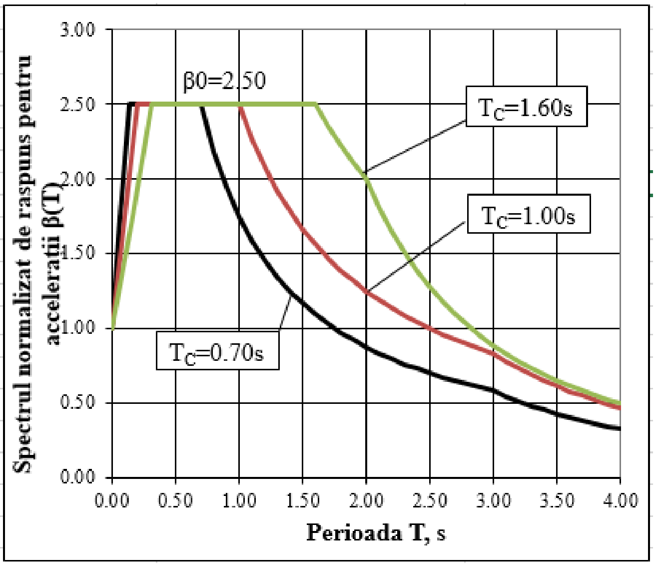

Figure 1.3-3 Normalized elastic response spectra of absolute accelerations

The differences in the normalized response spectra compared to P100-1/2006 consist of using a maximum dynamic amplification factor of $\beta_{0\_2013} = 2.50$, compared to $\beta_{0\_2006} = 2.75$, while the corner periods T_B are equal to $0.20T_c$, compared to $0.10T_c$ as defined in the previous standard.

Both the elastic relative displacement response spectrum for horizontal ground motion components, $S_{De}(T)$, and the elastic absolute acceleration response spectrum for the vertical ground motion component, $S_{ve}(T)$, are defined in the same manner as in P100-1/2006, according to equations (1.2-6) for $S_{De}(T)$ and (1.2-7) – (1.2-10) for $S_{ve}(T)$, respectively.

1.4 Seismic action according to Eurocode 8

Within the EN 1998 framework, the seismic motion at a given point on the ground surface is represented by an elastic response spectrum for ground acceleration.

The seismic action is described for two safety levels, the ultimate limit state (no-collapse requirement) and the damage limitation requirement, and the same shape of the response spectrum is considered for both levels.

Horizontal Elastic Response Spectrum

The elastic response spectrum $S_e(T)$ is defined by the following expressions, according to [13]:

$$S_e(T) = a_g \cdot S \cdot \left[1 + \frac{T}{T_B} \cdot (\eta \cdot 2.50 - 1)\right],\ for\ 0 \le T \le T_B \quad 1.4\text{-}1$$

$$S_e(T) = a_g \cdot S \cdot \eta \cdot 2.50,\ for\ T_B < T \le T_C \quad 1.4\text{-}2$$

$$S_e(T) = a_g \cdot S \cdot \eta \cdot 2.50 \cdot \left[\frac{T_C}{T}\right],\ for\ T_C < T \le T_D \quad 1.4\text{-}3$$

$$S_e(T) = a_g \cdot S \cdot \eta \cdot 2.50 \cdot \left[\frac{T_C T_D}{T^2}\right], \ for\ T_D < T \leq 4s \quad 1.4\text{-}4$$

where:

$S_e(T)$ – elastic response spectrum;

T - vibration period of a linear single-degree-of-freedom system;

a_g - design ground acceleration on Type A ground;

T_B - lower limit of the period of the constant spectral acceleration branch;

T_C - upper limit of the period of the constant spectral acceleration branch;

T_D - value defining the beginning of the constant displacement response range of the spectrum;

S - soil factor;

η - damping correction factor ($\eta = \sqrt{10/5+\xi} \geq 0.55$, where ξ is the critical damping ratio, in percent).

The soil factor (Table 1.4-1) is selected based on the ground conditions at the site. Ground types range from Class A, representing rock or rock-like formations, to Class E, representing surface alluvium layers over a stiffer material. There are also two special ground categories, S_1 and S_2, which consist of high-plasticity clay deposits or liquefiable soils requiring special site-specific studies to determine the corner periods T_B, T_C, T_D , and the soil factor S.

If deep geology is not accounted for, the use of two spectrum types is recommended: Type 1 and Type 2.

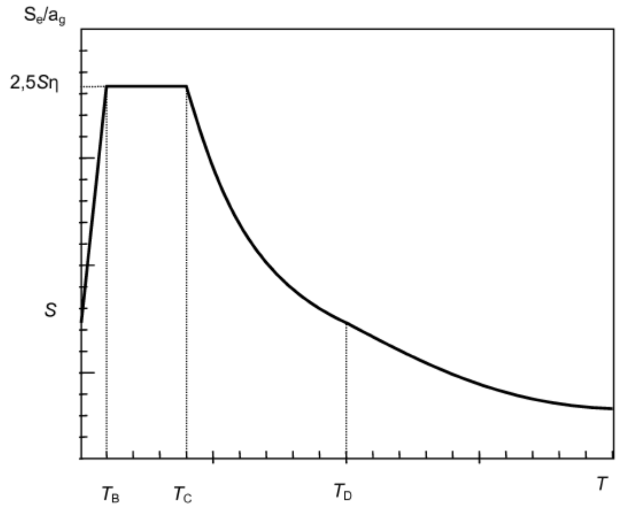

Figure 1.4-1 Elastic response spectrum, according to SR EN 1998-1 [13]

Soil Class	S	T_B [s]	T_C [s]	T_D [s]
A	1.00	0.15	0.40	2.00
B	1.20	0.15	0.50	2.00
C	1.15	0.20	0.60	2.00
D	1.35	0.20	0.80	2.0
E	1.40	0.15	0.50	2.0

Table 1.4-1 Parameter values defining the recommended Type 1 elastic respoonse spectrum[13]

To determine the elastic response spectrum, it is necessary to account for the viscous damping correction factor η, which is determined using equation (1.4-5):

$$\eta = \sqrt{10/(5+\xi)} \geq 0.55 \quad \text{1.4-5}$$

where ξ is the critical damping ratio, expressed as a percentage.

Vertical elastic response spectrum

The elastic response spectrum corresponding to the vertical component of the seismic action, $S_{ve}(T)$, is defined by the following expressions, according to [13]:

$$S_{ve}(T) = a_{vg} \cdot \left[1 + \frac{T}{T_{Bv}} \cdot (\eta \cdot 3.0 - 1)\right],\ for\ 0 \leq T \leq T_{Bv} \quad \text{1.4-6}$$

$$S_{ve}(T) = a_{vg} \cdot \eta \cdot 3.00,\ for\ T_{Bv} < T \leq T_{Cv} \quad \text{1.4-7}$$

$$S_{ve}(T) = a_{vg} \cdot \eta \cdot 3.00 \cdot \left[\frac{T_{Cv}}{T}\right],\ for\ T_{Cv} < T \leq T_{Dv} \quad \text{1.4-8}$$

$$S_{ve}(T) = a_{vg} \cdot \eta \cdot 3.00 \cdot \left[\frac{T_{Cv}T_{Dv}}{T^2}\right],\ for\ T_{Dv} < T \leq 4s \quad \text{1.4-9}$$

Spectrum	a_{vg}/a_g	T_{Bv} [s]	T_{Cv} [s]	T_{Dv} [s]
Type 1	0.90	0.05	0.15	1.00
Type 2	0.45	0.05	0.15	1.00

Table 1.4-2 Parameter values defining the vertical elastic response spectrum[13]

For both the horizontal and vertical elastic response spectra, if the earthquakes contributing most to the seismic hazard defined for the site have a surface-wave magnitude $M_s \leq 5.50$, the adoption of the Type 2 spectrum is recommended. Otherwise, the Type 1 spectrum is adopted.

Design Spectrum for Elastic Analysis

Structures generally have the capacity to resist seismic actions in the non-linear range, which permits design for seismic forces lower than those corresponding to a linear elastic response [13].

Considering that performing a non-linear analysis involves a higher degree of complexity, the ductile behavior of structures is accounted for by conducting an elastic analysis based on a response spectrum reduced by means of the behavior factor q. The behavior factor represents an approximation of the ratio between the seismic forces that the structure would experience if the response were completely elastic with 5% viscous damping and the seismic forces that may be used in design using a conventional elastic structural model.

The design spectrum for the horizontal components of seismic action, $S_d(T)$, is defined below:

$$S_d(T) = a_g \cdot S \cdot \left[\frac{2}{3} + \frac{T}{T_B} \cdot \left(\frac{2.50}{q} - \frac{2}{3}\right)\right],\ for\ 0 \leq T \leq T_B \quad \text{1.4-10}$$

$$S_d(T) = a_{vg} \cdot S \cdot \frac{2.50}{q},\ for\ T_B < T \leq T_C \quad \text{1.4-11}$$

$$S_d(T) = a_g \cdot S \cdot \frac{2.50}{q} \cdot \left[\frac{T_C}{T}\right],\ for\ T_C < T \leq T_D \quad \text{1.4-12}$$

$$S_d(T) = a_g \cdot S \cdot \frac{2.50}{q} \cdot \left[\frac{T_C T_D}{T^2}\right], \ \ for\ T_D < T \leq 4s \qquad 1.4\text{-}13$$

Time-History representation

Another method for representing seismic action consists of describing ground acceleration as a function of time (Time-History).

When performing analysis on a spatial structural model, seismic motion is represented by 3 accelerograms acting simultaneously, with the requirement that the same accelerogram cannot be used simultaneously along both horizontal directions. The accelerograms used may be artificially generated, recorded, or simulated [13].

Artificial accelerograms

Artificial accelerograms must satisfy the following rules:

- they must be generated to match the elastic response spectrum for a viscous damping ratio of 5%;
- a minimum of 3 accelerograms must be generated;
- the mean value of spectral acceleration must not be lower than the value $a_g S$ for the site considered;
- no value of the mean elastic spectrum with 5% damping, calculated across all generated accelerograms, should be less than 90% of the corresponding value from the 5% damped elastic response spectrum.

Intended seismic behavior

From the perspective of structural performance under seismic action, the response must be limited ductile/essentially elastic or ductile, depending on the site seismicity. Ductile or limited ductile behavior is characterized by the global force-displacement relationship of the structure, as illustrated in Figure 1.4-2.

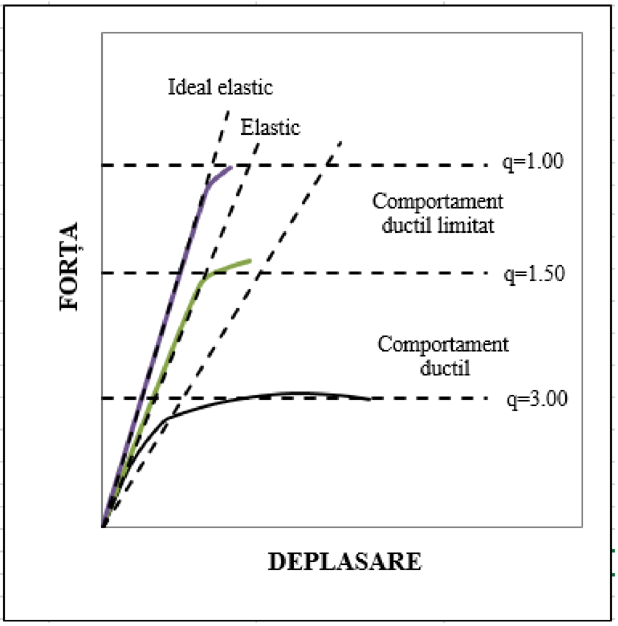

Figure 1.4-2 Force-displacement relationship for the intended seismic behavior [13]

For economic and safety reasons, it is preferable in regions of moderate to high seismicity to design structures for a ductile behavior, which entails implementing measures to ensure the dissipation of a significant amount of induced energy during severe earthquakes.

In the case of bridges, design requires energy dissipation to occur through the formation of flexural plastic hinges, developing a stable partial or complete mechanism. Generally, these primary energy-dissipating elements form in the bridge piers; however, they do not need to form in all piers, although optimal seismic ductile behavior for a bridge is achieved when plastic hinges develop simultaneously in as many piers as possible.

For structures with limited ductile behavior, the formation of a yield zone with a significant reduction in secant stiffness is not required. For this type of behavior, a behavior factor of $q=1.50$ used.

For a single-degree-of-freedom system with an idealized elastic-perfectly plastic force-displacement relationship, the design value of the structural ductility factor is defined as the ratio of ultimate limit state displacement (d_u) to yield displacement (d_y). The ultimate displacement d_u is defined as the maximum displacement provided the structure remains capable of sustaining at least 5 complete deformation cycles without failure of the confining reinforcement in reinforced concrete sections or local buckling of steel members, and without a reduction in resistance force for ductile steel elements or with no more than a 20% reduction in ultimate resistance force for ductile reinforced concrete elements.

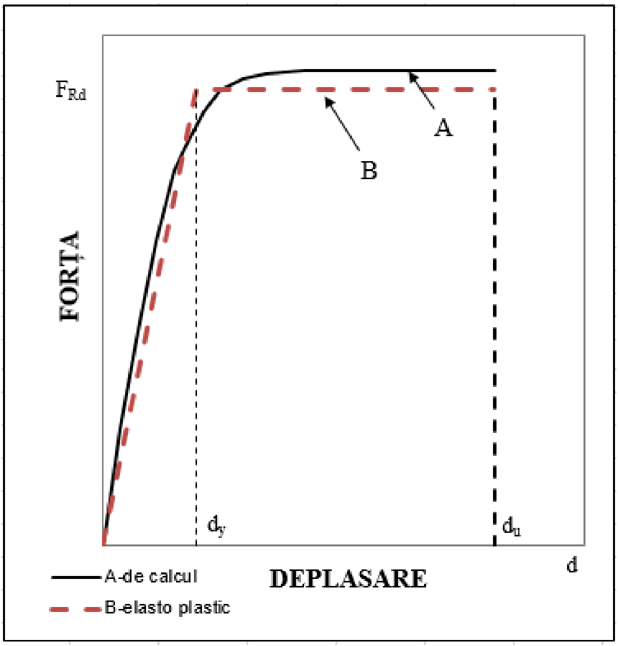

Figure 1.4-3 Global force-displacement diagram (monotonic loading)

Spacial variability of seismic action

This aspect must be taken into account if either of the following two conditions is met:

- the ground exhibits different properties along the total length of the bridge;
- the length of the continuous deck is greater than a corresponding minimum length, $L_{\lim} = L_g/1.50$ (L_g is defined in Table 1.4-5), even if the ground properties are uniform.

Soil spatial variability can lead to a progressive loss of correlation between motions at different support locations, involving complex wave reflections and refractions. This phenomenon, along with the random nature of seismic wave propagation, represents a crucial aspect that must be incorporated into the model describing the spatial variability of seismic action.

The simplified method described in Eurocode 8 assumes that the spatial variability of seismic action can be estimated through the pseudo-static effects of corresponding sets of displacements imposed at the bridge piers and abutments. These sets are selected to induce the maximum effects of the represented seismic action.

For each analyzed horizontal direction, the displacement sets must be applied at the foundation level of the substructures, and the effects of these two displacement sets must not be combined.

<u>Displacement Set A</u>

Set A consists of relative displacements:

$$d_{ri} = \varepsilon_r L_i \leq d_g\sqrt{2} \qquad 1.4\text{-}14$$

where $\varepsilon_r = \frac{d_g\sqrt{2}}{L_g}$

These displacements must be applied simultaneously with the same sign to all bridge substructures in the considered horizontal direction.

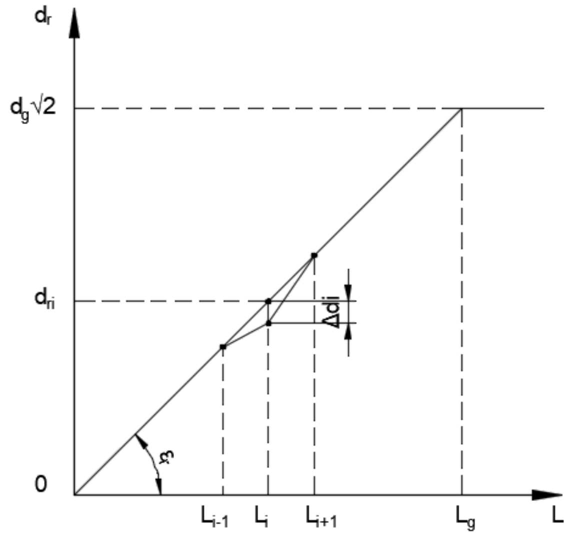

Figure 1.4-4 Displacement set A [13]

where:

d_g – design ground displacement corresponding to the ground type;

L_i – horizontal distance from support i to the reference support $i = 0$, usually selected at an abutment location;

L_g – distance beyond which ground motions may be considered completely uncorrelated.

Soil type	A	B	C	D	E
L_g(m)	600	500	400	300	500

Table 1.4-3 Distance beyond which ground motions may be considered uncorrelated [13]

Displacement Set B

Set B refers to ground displacements occurring in opposite directions at adjacent piers and is accounted for in the analysis by imposing displacements Δd_i (Equation 1.4-15) at each intermediate support, while the displacement at adjacent supports is taken as zero.

$$\Delta d_i = \pm\beta_r \varepsilon_r L_{\alpha v,i} \tag{1.4-15}$$

where:

$L_{\alpha v,i}$ – average of the distances $L_{i-1,i}$ and $L_{i,i+1}$ from intermediate support i to the adjacent supports $i-1$ and $i+1$, respectively;

β_r – factor care ține seama de amplitudinea deplasărilor terenului care apar în direcție opusă la reazemele adiacente.

1.5 Seismic action according to the American Standard FEMA 356/2000

Seismic Hazard

In the United States of America, earthquakes have been recorded in nearly all regions, affecting buildings across all 50 states. The severity of earthquake effects can be expressed in two ways: quantitatively, through magnitude, and qualitatively, through intensity.

Earthquake magnitude (M) is an objective parameter defining the strength of the shock at the focal point and relates to the amount of strain energy released when the seismic shock occurs.

Magnitude was defined by Charles Richter as the decimal logarithm of the maximum amplitude measured 100 km from the epicenter, expressed in microns, recorded by a standard seismometer with a natural period of 0.80s, a damping coefficient c=0.80 and a magnification factor f=2800:

$$M = lgA_{max} \tag{1.5-1}$$

Based on magnitude, earthquakes are classified into:

- Major earthquakes: M=7-9;
- Moderate earthquakes: M=5-7;
- Minor earthquakes: M=3-5;
- Microearthquakes: M=0-3.

A clear distinction must be made between earthquake magnitude and intensity.

Intensity refers to a metric that evaluates the effects of an earthquake at the Earth's surface on people and property. This makes earthquake intensity a subjective parameter, as it relies on human assessment. Earthquake intensity is determined using intensity scales, the most commonly used being the Modified Mercalli (MM) scale.

The Modified Mercalli (MM) scale divides earthquakes into 12 intensity levels based on observed effects and is used in Western Europe and the USA.

Intensity	Description of Observed Effects
I	Not felt except by a very few under especially favorable conditions.
II	Felt only by a few persons at rest, especially on upper floors of buildings.

III	Felt quite noticeably indoors, especially on upper floors. Vibration like passing of a truck. Duration estimated.
IV	Felt indoors by many, outdoors by few.
V	Felt by nearly everyone. Plaster cracks, unstable objects overturn.
VI	Felt by all, causing panic. Heavy furniture moves; plaster falls. Slight damage to poorly built structures.
VII	Causes general panic; people rush outdoors. Negligible damage in well-designed/built structures; slight to moderate damage in ordinary structures; considerable damage in poorly built or designed structures.
VIII	Slight damage in specially designed structures; considerable damage in ordinary buildings; collapse of poorly executed structures.
IX	Considerable damage in designated seismic structures; well-designed frame structures thrown out of plumb; damage to poorly built structures; ground cracked; underground pipes broken.
X	Most masonry and frame structures destroyed along with foundations; ground badly cracked; significant landslides.
XI	Few, if any, structures remain standing; surface faulting; broad fissures and massive landslips.
XII	Total damage; waves seen on ground surfaces; lines of sight and level distorted; objects thrown upward into the air.

Table 1.5-1 Modified Mercalli (MM) Seismic Intensity Scale [8]

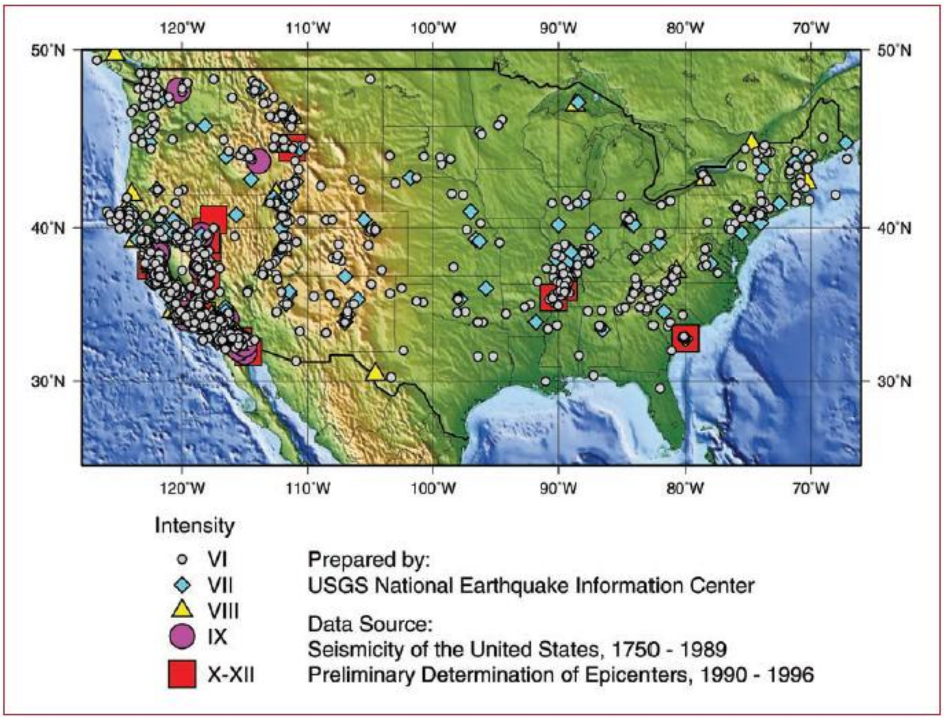

Figure 1.5-1 Location of earthquake epicenters in the United States between 1750 and 1996 [8]

The FEMA 356/2000 standard defines seismic hazard through deterministic or probabilistic methods.

The deterministic approach involves developing a specific earthquake scenario on the basis of which ground motion is evaluated at the site of interest. The probabilistic approach determines the annual probability of exceedance or the return period of a given level of ground motion at the site. Probabilistic analysis provides seismic probability maps in the form of contours of expected peak ground motion values (intensity, peak ground acceleration) for specified return periods [9].

The seismic hazard level is divided into two levels: Basic Safety Earthquake 1 (BSE-1) and Basic Safety Earthquake 2 (BSE-2). For each seismic hazard level, it is necessary to define normalized elastic response spectra for a 5% critical damping ratio, for both short periods (0.2s) and long periods (1.0s).

The seismic hazard maps used in this standard were developed for:

- seismic ground motions with a 10% probability of exceedance in 50 years;
- seismic ground motions with a 10% probability of exceedance in 100 years;
- seismic ground motions with a 10% probability of exceedance in 250 years.

These correspond to earthquakes expected to occur once every 500, 1000, and 2500 years, respectively. Past recorded ground motions are referred to as deterministic motions and were used to produce the Maximum Considered Earthquake (MCE) maps [10].

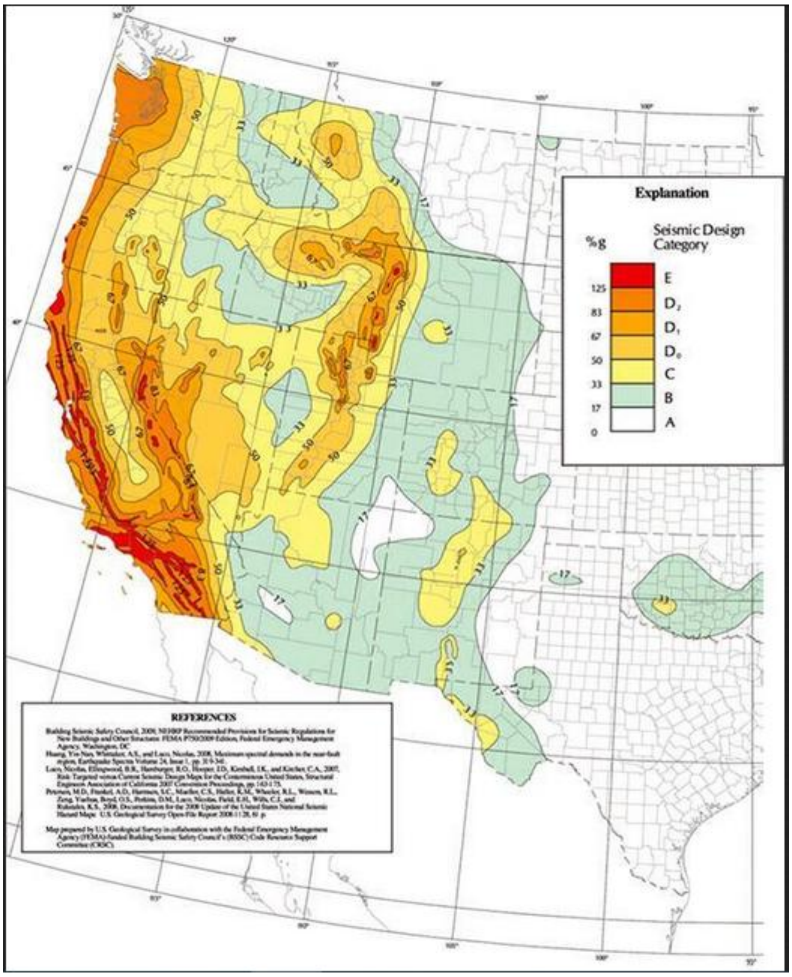

Figure 1.5-2 Zoning of peak ground acceleration values, S_s, for the western region of the USA [11]

Elastic Response Spectrum for the BSE-2 Seismic Hazard Level

The amplification factors for short periods, S_{XS}, and for a 1.0 s period, S_{X1}, are determined based on the S_S and S_1 values from the official MCE peak ground acceleration zonation maps and the ground site class. The parameters S_S and S_1 are obtained either by interpolating between two values adjacent to the considered site or by selecting the maximum value between the two [10].

In most regions of the US, the BSE-2 seismic hazard level corresponds to a 2% probability of exceedance in 50 years. In known fault zones characterized by earthquakes with magnitudes $M > 6.0$, the seismic hazard level is considered for a probability of exceedance greater than 2% in 50 years.

Elastic Response Spectrum for the BSE-1 Seismic Hazard Level

For this seismic hazard level, the parameters S_{XS} and S_{X1} are selected as the smaller values between:

- the S_S and S_1 values obtained from the official MCE maps for a 10% probability of exceedance in 50 years, modified according to the ground site class;
- two-thirds of the values obtained for the BSE-2 seismic hazard level.

Modification of Parameters for Other Seismic Hazard Levels

For probabilities of exceedance between 2% in 50 years and 10% in 50 years, the parameters S_S and S_1 are determined using Equation (1.5-2), provided that the parameter for periods under 1.0 s (S_S) is less than $1.50g$:

$$\ln(S_i) = \ln(S_{i10/50}) + [\ln(S_{iBSE-2}) - \ln(S_{i10/50})] * [0.06\ln(P_R) - 3.73] \quad 1.5\text{-}2$$

where: $\ln(S_i)$ = natural logarithm of acceleration ("$i = s$" for $T < 1.00$ s or "$i = 1$" for $T = 1.00$ s) for the target probability of exceedance;

$\ln(S_{i10/50})$ = natural logarithm of acceleration ("$i = s$" for $T < 1.00$ s or "$i = 1$" for $T = 1.00$ s) for a 10% probability of exceedance in 50 years;

$\ln(S_{iBSE-2})$ = natural logarithm of acceleration ("$i = s$" for $T < 1.00$ s or "$i = 1$" for $T = 1.00$ s) for the BSE-2 seismic hazard level;

$\ln(P_R)$ = natural logarithm of the mean recurrence interval corresponding to the desired probability of exceedance.

If the peak ground acceleration value $S_S > 1.50g$, the parameters S_S and S_1 for probabilities of exceedance between 2% in 50 years and 10% in 50 years are determined using Equation (1.5-3):

$$S_i = S_{i10/50}\left(\frac{P_R}{475}\right)^n \quad 1.5\text{-}3$$

where "n" is an exponent with values given in Table 1.5-1.

For probabilities of exceedance greater than 10% in 50 years and $S_S < 1.50g$, the parameter values S_S and S_1 are determined using Equation (1.5-3), with the values of the exponent "n" taken from Table 1.5-2. If $S_S \geq 1.50g$, the values of "n" are likewise taken from Table 1.5-2.

Region	S_S	S_1
California	0.29	0.29
Nord-West Pacific	0.56	0.67
Intermountain Zone	0.50	0.60
Central US	0.98	1.09
Eastern US	0.93	1.05

Table 1.5-1 Values of exponent "n" for seismic hazard levels with a probability of exceedance between 10%/50 years and 2%/50 years, with $S_S \geq 1.50g$ [10]

Region	S_S	S_1
California	0.29	0.29
Nord-West Pacific	0.56	0.67
Intermountain Zone	0.50	0.60
Central US	0.98	1.09
Eastern US	0.93	1.05

Table 1.5-2 Values of exponent "n" for seismic hazard levels with a probability of exceedance greater than 10%/50 years, with $S_S < 1.50g$ [10]

Region	S_S	S_1
California	0.29	0.29
Nord-West Pacific	0.56	0.67
Intermountain Zone	0.50	0.60
Central US	0.98	1.09
Eastern US	0.93	1.05

Table 1.5-3 Values of exponent "n" for seismic hazard levels with a probability of exceedance greater than 10%/50 years, with $S_S \geq 1.50g$ [10]

Identification of Site Classes

The design spectrum value for short periods $T < 1.0$ s, S_{XS}, as well as the value for the period $T = 1.0$ s, S_{X1}, are determined using Equations (1.5-4) and (1.5-5):

$$S_{XS} = F_a S_s \tag{1.5-4}$$

$$S_{X1} = F_v S_1 \tag{1.5-5}$$

where F_a and F_v are site coefficients that depend on the site class and the peak ground acceleration parameters S_S and S_1 for the selected mean recurrence interval, as determined from Tables 1.5-4 and 1.5-5.

Six site classes are distinguished:

- Class A: hard rock with average shear wave velocity $v_s > 1524$ m/s;
- Class B: rock with 762 m/s $< v_s \leq 1524$ m/s;
- Class C: very dense soil and soft rock, with 366 m/s $< v_s \leq 762$ m/s or standard penetration resistance $N_{\text{SPT}} > 50$ or undrained shear strength $s_u > 96$ kPa;
- Class D: stiff soil profile with 183 m/s $< v_s \leq 366$ m/s or $15 \leq N_{\text{SPT}} \leq 50$ or undrained shear strength 48 kPa $\leq s_u \leq 95$ kPa;
- Class E: any profile with more than 3.00 meters of clay with plasticity index $I_p > 20$, moisture content $w \geq 40\%$, undrained shear strength $s_u < 25$ kPa, or a soil profile with average shear wave velocity $v_s < 183$ m/s;
- Class F: soils requiring site-specific evaluation, such as soils vulnerable to potential failure or collapse under seismic loading (liquefiable soils, highly sensitive clays, highly organic clays, or high-plasticity clays).

If average shear wave velocity v_s values are available for the analyzed site, they shall be used to establish the site class. When average shear wave velocity values are not provided, N_{SPT} values shall be used for classifying cohesionless soils, and undrained shear strength s_u values shall be used for classifying cohesive soils [10].

Site class	$S_S \le 0.25$	$S_S = 0.50$	$S_S = 0.75$	$S_S = 1.00$	$S_S \ge 1.25$
A	0.80	0.80	0.80	0.80	0.80
B	1.00	1.00	1.00	1.00	1.00
C	1.20	1.20	1.10	1.00	1.00
D	1.60	1.40	1.20	1.10	1.00
E	2.50	1.70	1.20	0.90	0.90
F	*	*	*	*	*
* Site-specific geotechnical investigation and dynamic site response analyses shall be performed.					

Table 1.5-4 Values of F_a defined as a function of site class and acceleration value S_S [10]

Site class	$S_S \le 0.10$	$S_S = 0.20$	$S_S = 0.30$	$S_S = 0.40$	$S_S \ge 0.50$
A	0.80	0.80	0.80	0.80	0.80
B	1.00	1.00	1.00	1.00	1.00
C	1.70	1.60	1.50	1.40	1.30
D	2.40	2.00	1.80	1.60	1.50
E	3.50	3.20	2.80	2.40	2.40
F	*	*	*	*	*
* Site-specific geotechnical investigation and dynamic site response analyses shall be performed.					

Table 1.5-5 Values of F_v defined as a function of site class and acceleration value S_1 [10]

<u>*Elastic Response Spectrum for absolute accelerations*</u>

The horizontal spectrum (Figure 1.5-2) is constructed for a critical damping ratio ξ = 5.00% and is defined by the following expressions [10]:

$S_a = S_{XS}\left[\left(\frac{5}{B_S} - 2\right)\frac{T}{T_S} + 0.4\right]$, for 0 < T < T0 1.5-6

$S_a = \frac{S_{XS}}{B_S}$, for T < T_S 1.5-7

$S_a = \frac{S_{X1}}{B_1 T}$, for T > T_S 1.5-8

where T_S și T_0 are determined by the following expressions:

$T_S = \frac{S_{X1} B_S}{S_{XS} B_1}$ 1.5-9

$T_0 = 0.20 T_S$, B_s and B_1 are determined from Table 1.5-6

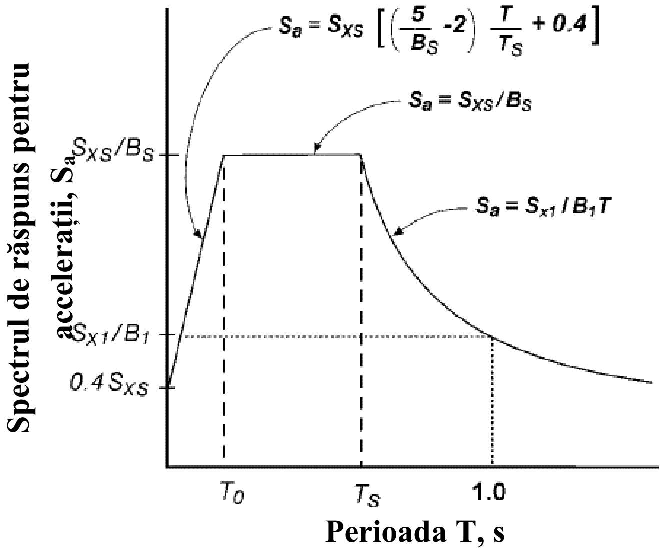

Figure 1.5-2 Elastic response spectrum of absolute accelerations for horizontal components of ground motion [10]

Damping ratio (% of critical)	B_S	B_1
≤2	0.80	0.80
5	1.00	1.00
10	1.30	1.20
20	1.80	1.50
30	2.30	1.70
≥50	3.00	2.00

Table 1.5-6 Damping coefficients as a function of the critical damping ratio

The elastic response spectrum of absolute accelerations for the vertical component of ground motion shall be taken, at any period, as two-thirds of the horizontal elastic response spectrum value.

Time-History representation

Time-history analyses must include at least 3 sets of ground motion accelerograms (each set consisting of two horizontal components and one vertical component), which must be scaled for at least three seismic events. If recorded accelerograms are not available for the specific site, the seismic motion may be defined using synthetic (artificial) accelerograms.

Synthetic accelerograms must be generated such that they match the site-specific elastic response spectrum for a viscous damping ratio of 5%.

If only three accelerogram sets are used for analysis, the structural response (internal forces, displacements) shall be taken as the maximum value obtained among the three dynamic analyses.

If the structural response is obtained from at least 7 dynamic analyses, the design response parameter is defined as the mean value of the responses resulting from all these analyses.

Seismicity zones

High seismicity zones are defined as those for which the elastic response spectrum values S_{XS} and S_{X1} for a 10% probability of exceedance in 50 years satisfy the following conditions (1.5-10):

$$S_{XS} \geq 0.50g \tag{1.5-10}$$

$$S_{X1} \geq 0.20g$$

Moderate seismicity zones are defined according to Equation 1.5-11.

$$0.167g \leq S_{XS} < 0.50g$$
$$0.067g \leq S_{X1} < 0.20g \quad (1.5\text{-}11)$$

Low seismicity zones are defined according to Equation 1.5-12.

$$S_{XS} < 0.167g$$
$$S_{X1} < 0.067g \quad (1.5\text{-}12)$$

1.6 Structural Analysis methods for seismic actions

In current structural design, analysis methods are chosen based on the importance category and structural characteristics of the building. Two main methods are primarily distinguished:

- the lateral force method associated with the fundamental mode of vibration, in which the dynamic nature of the seismic action is represented through a distribution of static equivalent forces;
- the modal response spectrum analysis method.

In addition to these two methods, nonlinear static analysis (pushover), linear dynamic analysis, or nonlinear dynamic analysis (time-history) may be applied to obtain a more precise structural response [15].

Equivalent Static Lateral Force Method

This method is recommended for structures where the fundamental mode of vibration has a dominant contribution to the total seismic response, and the fundamental period of vibration does not exceed 1.50 s.

For each principal horizontal direction, the base shear force corresponding to the fundamental mode of vibration is determined using Equation (1.3-3):

$$F_b = \gamma_I \cdot S_d(T_1) \cdot m \cdot \lambda \quad (1.3\text{-}3)$$

where

$S_d\,(T_1)$ - ordinate of the design response spectrum corresponding to the fundamental period of vibration T_1;

m - total seismic mass of the structure;

γ_I - importance factor of the building;

λ - correction factor accounting for the contribution of the fundamental vibration mode to the total response, taken as 0.85 if $T_1 \leq T_C$, or 1.00 otherwise.

Seismic lateral forces must be applied at each story level, and the force corresponding to level i is calculated using Equation (1.3-4):

$$F_i = F_b \frac{m_i s_i}{\sum_{j=1}^{n} m_j s_j} \quad (1.3\text{-}4)$$

where

F_i - equivalent static seismic force at level i;

s_i, s_j - displacement component of the fundamental mode shape along the translational dynamic degree of freedom at level i or j;

m_i, m_j - story mass associated with level i or j;

n - total number of story levels of the structure.

Modal Response Spectrum Analysis Method

This method applies to structures that do not satisfy the conditions specified above for the simplified equivalent static lateral force method.

The seismic action is evaluated based on the response spectra determined as described previously.

In the case of complex structures requiring a 3D spatial model, the seismic action must be applied along the relevant horizontal directions and along the principal orthogonal axes.

For the analysis, it is necessary to determine the dynamic vibration characteristics of the structure (natural modes of vibration, modal mass participation factors). Only those modes that satisfy the following conditions shall be considered:

- the sum of the effective modal masses accounts for at least 90% of the total mass of the structure;
- all natural modes with an effective modal mass greater than 5% of the total mass of the structure must be taken into account.

When consecutive natural vibration modes are independent of each other ($T_{i+1} \leq 0.90T_i$), the total maximum effect is obtained using the SRSS (Square Root of the Sum of Squares) combination rule, according to the expression below:

$$E_E = \sqrt{\sum {E_{E,i}}^2} \tag{1.3-4}$$

If the condition mentioned above is not satisfied, other modal combination rules for maximum response must be used, such as CQC (Complete Quadratic Combination).

Linear Dynamic Analysis Method (Time-History)

The linear seismic response over time is obtained through direct integration of the differential equations of motion, which express instantaneous dynamic equilibrium along the directions of the dynamic degrees of freedom defined in the model [15].

Unlike the previously described methods, the seismic input in this case is characterized by time-history accelerograms that satisfy the local site conditions as well as the design seismic action requirements.

If recorded accelerograms are not available for the considered site, synthetic accelerograms generated based on the site-specific response spectrum may be used. The design forces and displacements are obtained from the structural response scaled by the behavior factor q.

Nonlinear Static Analysis (Pushover)

Nonlinear static analysis consists of monotonically increasing horizontal loads while gravity loads remain constant. It can be applied to both new and existing structures for the following purposes:

- establishing or refining the ratio between the base shear force corresponding to the plastic collapse mechanism and the base shear force corresponding to the formation of the first plastic hinge (the overstrength factor α_u/α_1);
- serving as a design alternative to linear elastic analysis using seismic forces modified by the behavior factor q;
- evaluating the seismic response of critical/important structures;
- identifying expected plastic hinge mechanisms and damage patterns;
- assessing the global collapse margin of the structure.

Lateral loads must be applied using either a uniform pattern (with lateral forces proportional to story masses) or a modal pattern.

The overstrength factor (α_u/α_1) is calculated for both lateral load distribution patterns, and the design base shear force is chosen based on the minimum value obtained between the two.

Nonlinear Dynamic Analysis (Time-History)

Nonlinear dynamic response is obtained through direct step-by-step integration of the equations of motion, with seismic action represented by recorded or artificially generated ground motion accelerograms.

Hysteretic rules must be assigned to model post-elastic loading-unloading behavior realistically, capturing energy dissipation within the range of displacement amplitudes expected for the design earthquake [15].

The design seismic response parameter is taken as the average of the peak responses obtained across all accelerograms when using at least 7 ground motion records, or as the most critical response value when using fewer than 7 accelerograms.

Combination of seismic action effects

Simultaneous action of the orthogonal horizontal components of seismic ground motion must be accounted for. The maximum structural response is obtained using the Square Root of the Sum of Squares (SRSS) rule applied to the effects of each orthogonal horizontal component.

When identifying the principal structural directions is difficult, the seismic action effects may be evaluated using the following spatial combinations of horizontal components (X and Y) alongside the vertical component (Z), according to Equation (1.3-5):

$$\begin{gathered} E_{Edx} + 0.30E_{Edy} + 0.30E_{Edz} \\ 0.30E_{Edx} + E_{Edy} + 0.30E_{Edz} \\ 0.30E_{Edx} + 0.30E_{Edy} + E_{Edz} \end{gathered} \tag{1.3-5}$$

2. <u>CASE STUDY: THE INFLUENCE OF SEISMIC DESIGN CODE REVISIONS ON EXISTING BRIDGES IN ROMANIA</u>

Romania is a seismically active country, characterized by two primary seismic regions: a region defined by intermediate-depth earthquakes, such as those originating in Vrancea, and a region characterized by shallow-focus earthquakes, such as the Banat events. Earthquakes in the former category are more severe, featuring a recurrence period of 40–50 years and hypocenters located at depths of 100 - 150 km. One of the largest earthquakes recorded in the country occurred on March 4, 1977, with a magnitude of 7.2 on the Richter scale, claiming 1,500 lives and causing significant structural damage [14].

The first Romanian design code dedicated to structural seismic design was the *"Code for the Earthquake-Resistant Design of Residential, Socio-Cultural, Agro-Zootechnical, and Industrial Buildings - Code Designation P100-92"*. The objective of this case study is to design a bridge for seismic action as specified by P100-92 and subsequently evaluate the capacity of this structure to resist ground motion specified by more recent codes, P100-2006 and P100-2013, respectively, which introduced major updates regarding peak ground acceleration a_g and the corner period T_C.

Description of the structure

The superstructure is configured statically as a continuous beam over three spans measuring 40.00 m + 60.00 m + 40.00 m. The cross-section consists of three steel plate girders topped by a cast-in-place reinforced concrete deck slab. The steel girders are fabricated from grade S355 steel.

The total width of the superstructure is 12.00 m, providing a clear roadway width of 7.80 m. The roadway is flanked by H4b safety barriers. The sidewalks are fitted with pedestrian railings along the outer edges.

The reinforced concrete deck slab is cast using C35/45 grade concrete. Transversal slopes are integrated directly into the slab geometry, eliminating the need for an additional slope concrete layer.

The substructures comprise embedded abutments and piers, supported by deep foundations consisting of large-diameter drilled piles.

The piers and abutments are constructed using C20/25 grade concrete.

The pier bents consist of two circular columns with a diameter of 1.60 m and a clear height of 12.00 m.

The bearings are standard elastomeric bearing pads, composed of neoprene with steel shim plates. These are positioned beneath each girder at both the abutments and intermediate piers.

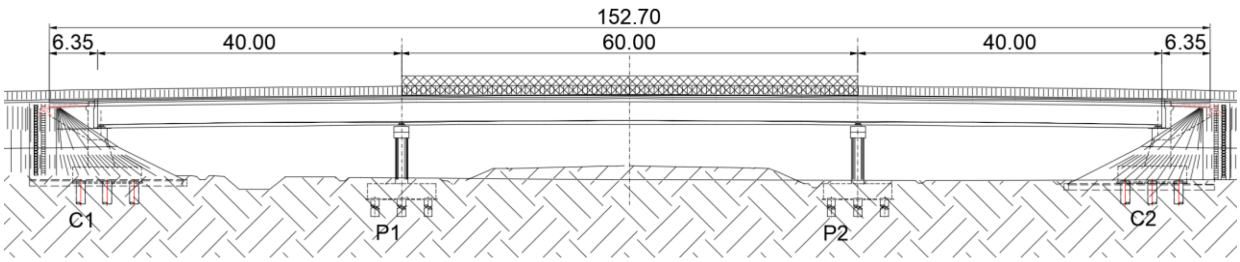

Figure 2.1. Structure elevation

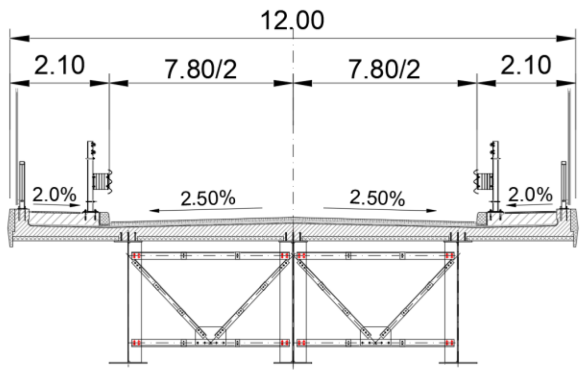

Figure 2.2. Cross section

Finite element model of the structure

To investigate the dynamic response of the structure under seismic action, a finite element model was created.

The piers were modeled using frame elements, each node possessing six degrees of freedom. The steel girders and the reinforced concrete deck slab were discretized using 4-node shell elements, while the cross-girders (diaphragms) were represented by frame elements. Composite action between the steel girders and the concrete deck slab was modeled using rigid links.

Soil-structure interaction was neglected, as the pile cap and drilled shaft foundation assembly is sufficiently rigid to justify assuming fixed support conditions at the base of the pier columns. Modeling the foundation as a fixed boundary condition yields conservative results regarding internal forces in the piers.

The conventional neoprene elastomeric bearings were modeled using linear link elements. Their vertical stiffness is high, as these devices must support significant gravity loads, a direct consequence of the small thickness of the individual rubber layers. The horizontal stiffness arises solely from the shear deformation of the rubber layers, as the steel shim plates provide confinement and prevent lateral bulging or flexural deformation. Their response remains linear up to a shear strain limit of 100%.

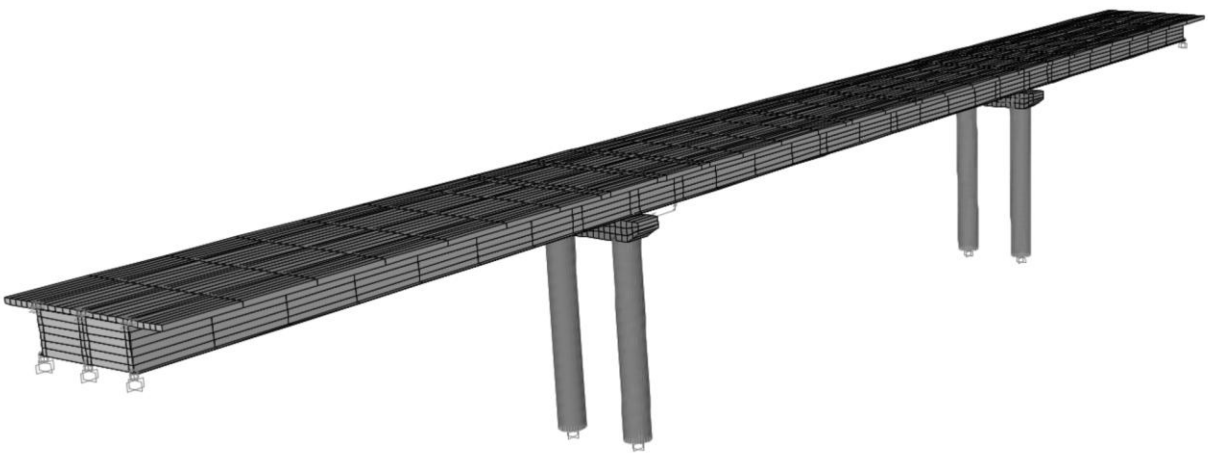

Figure 2.3. 3D view of the finite element model of the structure

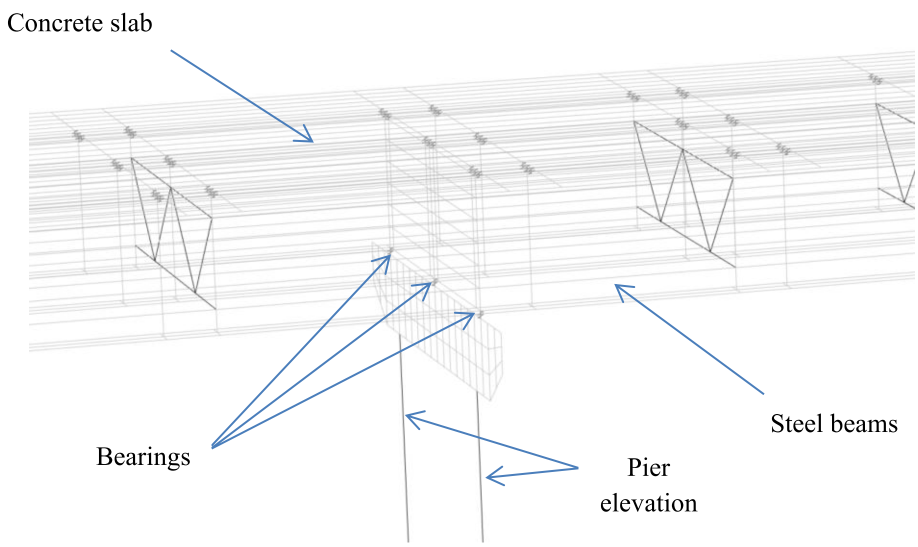

Figure 2.4. Finite element model of the structure

2.1 Structural response to the seismic action defined according to P100-92

Representation of Seismic Action

For the seismic design of structures, the territory of Romania is divided into seismic hazard zones. The seismic hazard level within each zone is assumed, in a simplified manner, to be constant [1].

Seismic hazard is defined by the peak ground acceleration of the horizontal seismic motion, a_g, evaluated for a specified mean recurrence interval. According to P100-1992, the mean recurrence interval is 50 years.

The parameter a_g represents the characteristic value of the horizontal peak ground acceleration used to compute the characteristic design seismic force, S.

The parameters governing the seismic action were established considering the bridge site located near the city of Bucharest. The design acceleration spectrum was calculated using a peak ground acceleration $a_g = 0.20g$, a corner period $T_C = 1.50$ s, and a seismic action reduction factor accounting for structural ductility, $\psi = 0.35$. The normalized elastic response spectrum for absolute horizontal ground accelerations, $\beta(T)$, corresponding to a standard critical damping ratio $\xi = 0.05$, is defined by the following expressions:

$$\beta(T) = 2.50, \text{ for } T \leq T_C$$
$$\beta(T) = 2.50 - (T - T_c) \geq 1.00, \text{ for } T > T_C \quad (2.1\text{-}1)$$

where T is the vibration period of a single-degree-of-freedom structure, and T_C is the corner period associated with the bridge site.

The corner period for the bridge location analyzed in this case study is $T_C = 1.50$ s.

Figures 2.4 - 2.6 show the normalized response spectrum $\beta(T)$, the elastic horizontal acceleration response spectrum $S_e(T)$, and the design response spectrum $S_d(T)$ for the selected site.

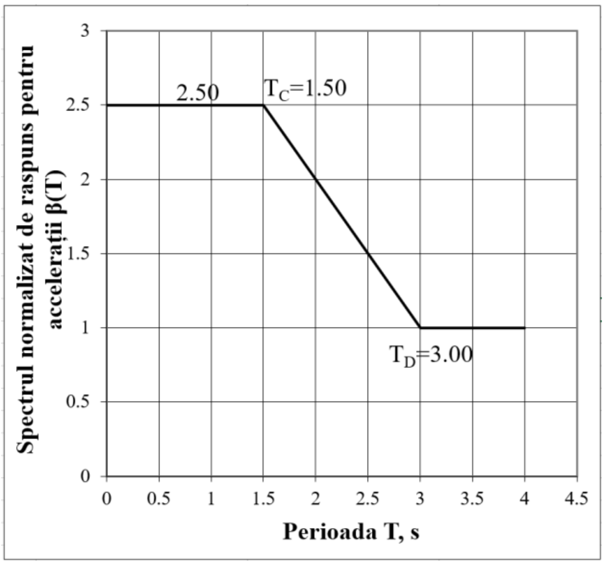

Figure 2.5. Normalized response spectrum for accelerations $\beta(T)$

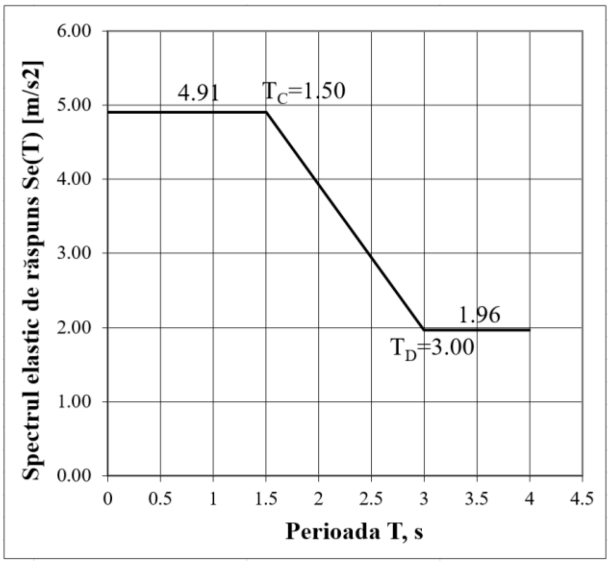

Figure 2.6. Elastic response spectrum $S_e(T)$

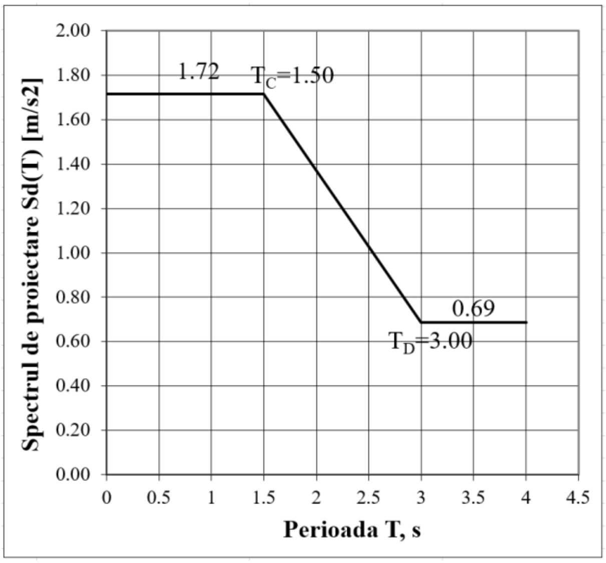

Figure 2.7. Design spectrum $S_d(T)$

Analysis and Results

To evaluate the seismic response of the structure, a linear dynamic analysis, specifically the Response Spectrum Method, was performed. This involves an elastic calculation of the dynamic peak responses for all significant vibration modes of the structure, utilizing the ordinates of the previously defined design response spectrum. The total structural response is obtained through a statistical combination of the maximum modal contributions. The primary dynamic parameters of the structure (natural period T and natural frequency f) are determined via an eigenvalue and eigenvector modal analysis. Figures 2.7 through 2.9 illustrate the first three fundamental mode shapes of the structure, showing that the dynamic response is dominated by translations along the two principal axes as well as in-plane rotation of the bridge deck.

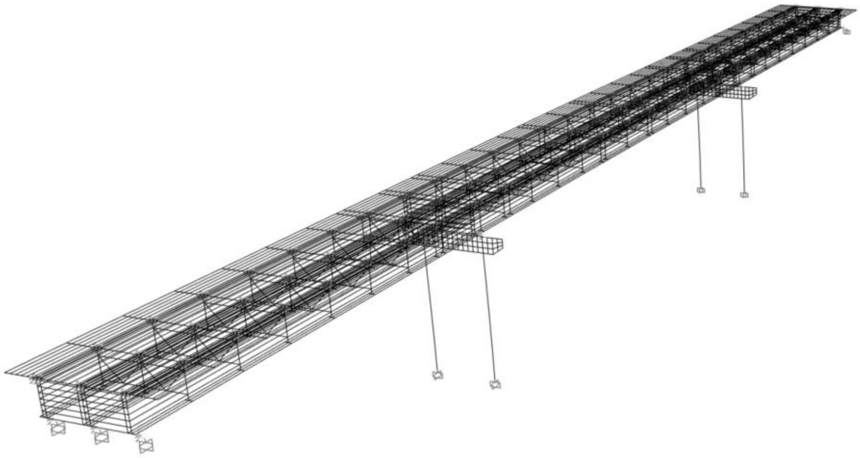

Figure 2.8. Mode 1 of vibration (transverse translation), T_1=1.55s, f_1=0.65 Hz

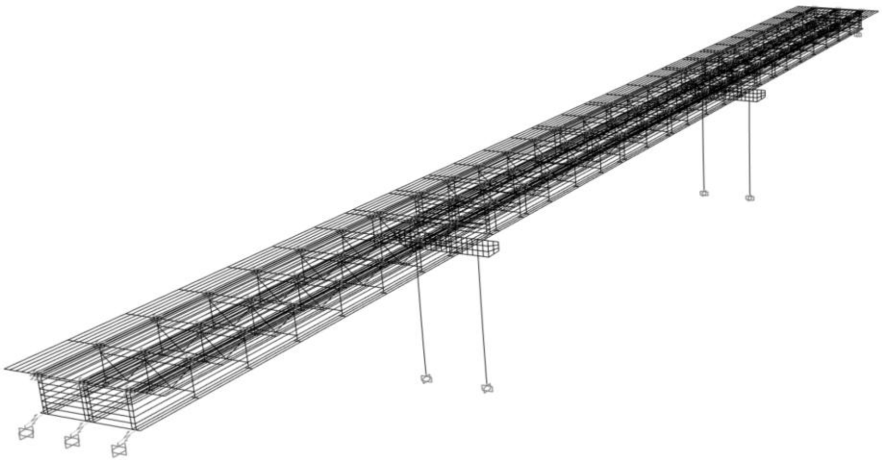

Figure 2.9. Mode 2 of vibration (longitudinal translation), T_2=1.50s, f_2=0.66 Hz

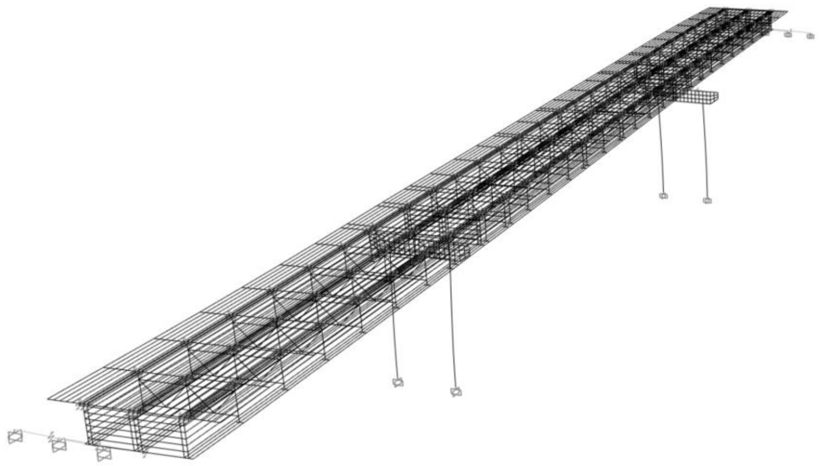

Figure 2.10. Mode 3 of vibration (plan rotation), T_3=1.06s, f_3=0.94 Hz

Linear dynamic analysis was performed considering the design spectrum described in Figure 2.6. Spectral ordinates were determined assuming limited ductile behavior for the piers, resulting in the dissipation of earthquake-induced energy through their capacity to deform in the post-elastic range.

The fundamental period of the structure (T_1 = 1.55s ≈ T_c = 1.50s) indicates that the global stiffness of the structure is significant, leading to substantial seismic forces being attracted, as nearly all response spectrum ordinates fall on the maximum absolute acceleration plateau. The maximum internal forces at the base of the piers and the maximum displacements at the superstructure level are presented in Table 2.1-1. These values were evaluated considering seismic action acting exclusively in the longitudinal direction of the bridge.

Maximum bending moment [kN*m]	6635.00
Maximum shear force [kN]	487.00
Maximum bearing displacement [cm]	6.50

Table 2.1-1

The cross-section of the pier columns was sized based on these governing internal forces. Accordingly, the pier column has a diameter of 1.60 m and is constructed from C20/25 grade concrete ($f_{ck} = 20$ N/mm^2, $E_{cm} = 30000$ N/mm^2). Longitudinal reinforcement consists of 40 $\phi 32$ bars of PC52 grade steel ($f_{yk} = 335$ N/mm^2, $E = 200000$ N/mm^2).

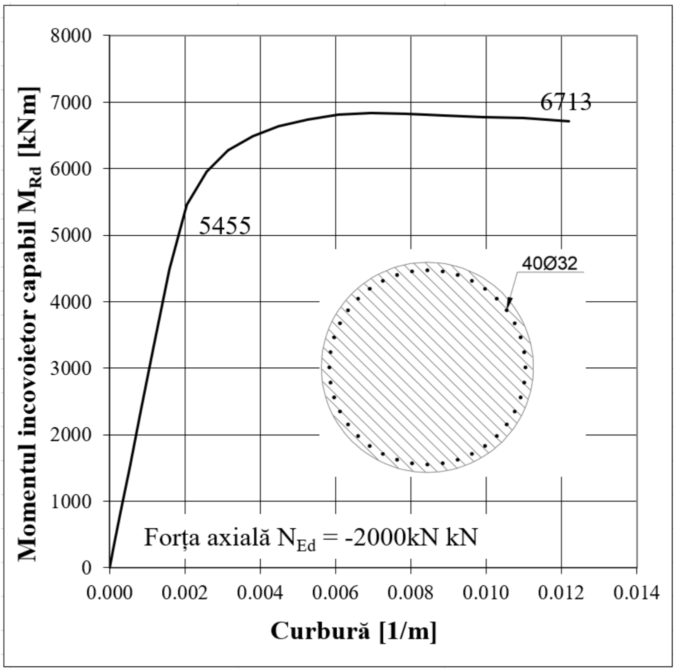

Figure 2.11. Moment-curvature diagram of the pier cross-section

According to the moment-curvature diagram, the flexural resistance of the pier cross-section is $M_{Rd} = 6713$ kNm.

Regarding the maximum displacement of the elastomeric bearings ($\Delta_{max} = 6.50$cm), it does not exceed the maximum displacement capacity of 8.50cm.

Next, the capacity of this structure to resist seismic actions defined in accordance with the P100-2006 and P100-2013 design codes will be evaluated.

2.2 Structural response to the seismic action defined according to P100-2006

The acceleration design spectra were established for a peak ground acceleration $a_g = 0.24g$, a corner period $T_C = 1.60$ s, and behavior factors $q_1 = 1.50$ and $q_2 = 2.50$, respectively. The mean recurrence interval is MRI $= 100$ years.

The normalized elastic response spectrum for absolute horizontal ground motion accelerations, $\beta(T)$, corresponding to a standard critical damping ratio $\xi = 0.05$ and expressed as a function of the control periods T_B, T_C, and T_D, is defined by the following expressions:

$$\begin{aligned}
&\beta(T) = 1 + \frac{(\beta_0 - 1)}{T_B} T, \ \ for\ 0 \leq T \leq T_B \\
&\beta(T) = \beta_0, \ \ for\ T_B \leq T \leq T_C \\
&\beta(T) = \beta_0 \frac{T_C}{T}, \ \ for\ T_C \leq T \leq T_D \\
&\beta(T) = \beta_0 \frac{T_C T_D}{T^2}, \ \ for\ T \geq T_D
\end{aligned} \qquad 2.2\text{-}1$$

Figures 2.11 - 2.14 show the normalized response spectrum $\beta(T)$, the elastic response spectrum for horizontal components of ground acceleration $S_e(T)$, and the two design response spectra $S_d(T)$.

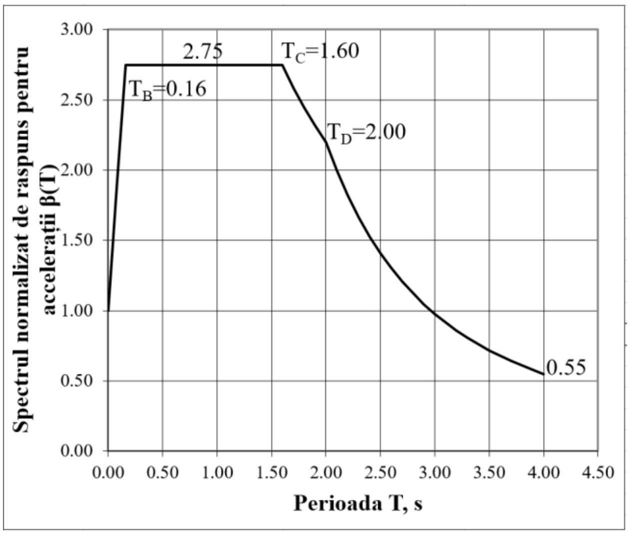

Figure 2.12. Normalized response spectrum for accelerations β(T)

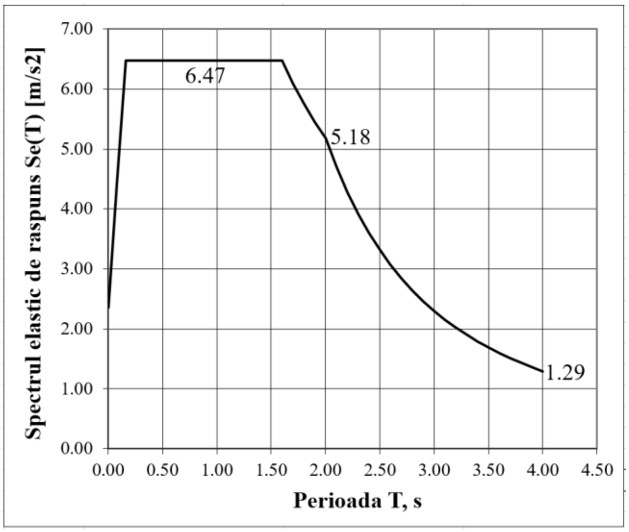

Figure 2.13. Elastic response spectrum $S_e(T)$

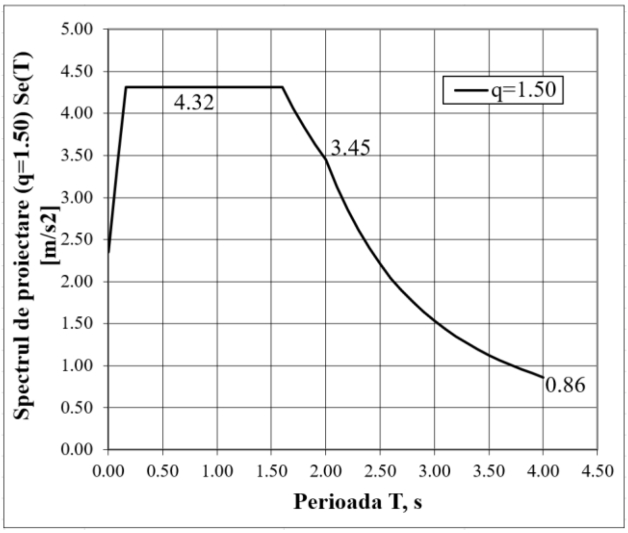

Figure 2.14. Design spectrum $S_d(T)$, (q=1.50)

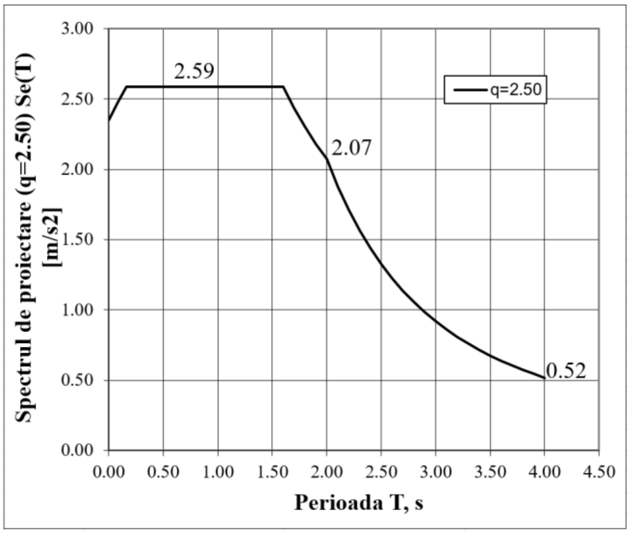

Figure 2.15. Design spectrum $S_d(T)$, (q=2.50)

<u>Analysis and results</u>

To evaluate the seismic response of the structure, two linear dynamic analyses were performed using the Response Spectrum Method. This involves an elastic calculation of the peak dynamic responses for all significant vibration modes of the structure, utilizing the ordinates of the previously defined design response spectra. The total structural response is obtained through a statistical combination of the maximum modal contributions. The fundamental dynamic parameters of the structure (natural period T and natural frequency f) were established in the preceding section, as the mode shapes remain identical.

The two analyses were conducted considering the design spectra shown in Figures 2.13 and 2.14. Spectral ordinates were evaluated assuming both limited ductile behavior ($q = 1.50$) and ductile behavior ($q = 2.50$) for the piers, reflecting their capacity to dissipate seismic energy through post-elastic deformation.

The maximum internal forces at the base of the piers and the maximum displacements at the superstructure level are presented in Table 2.2-1.

	q=1.50	q=2.50
Maximum bending moment [kN*m]	16100.00	9540.00
Maximum shear force [kN]	1210.00	750.00
Maximum bearing displacement [cm]	14.00	9.50

Table 2.2-1. Maximum internal forces and displacements

2.3 Structural response to the seismic action defined according to P100-2013

The acceleration design spectra were established for a peak ground acceleration $a_g = 0.30g$, a corner period $T_C = 1.60$ s, and behavior factors $q_1 = 1.50$ and $q_2 = 2.50$, respectively. The mean recurrence interval is MRI $= 225$ years.

The ordinates of the response spectrum are determined using the relationships previously described in Section 2.2-1.

Figures 2.15 - 2.18 show the normalized response spectrum $\beta(T)$, the elastic response spectrum for horizontal components of ground acceleration $S_e(T)$, and the two design response spectra $S_d(T)$.

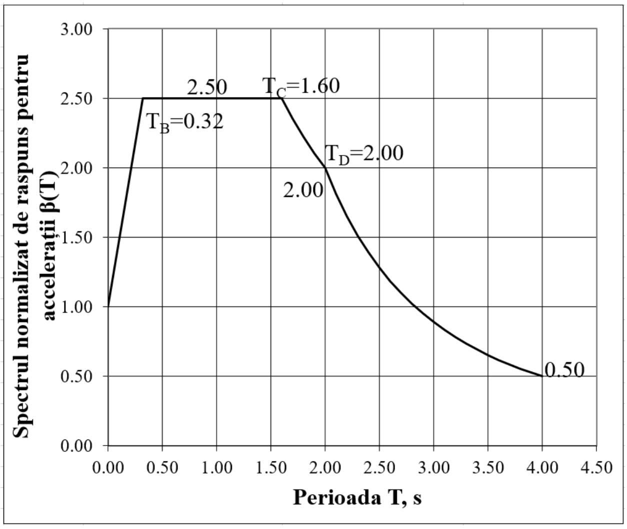

Figure 2.16. Normalized response spectrum for accelerations β(T)

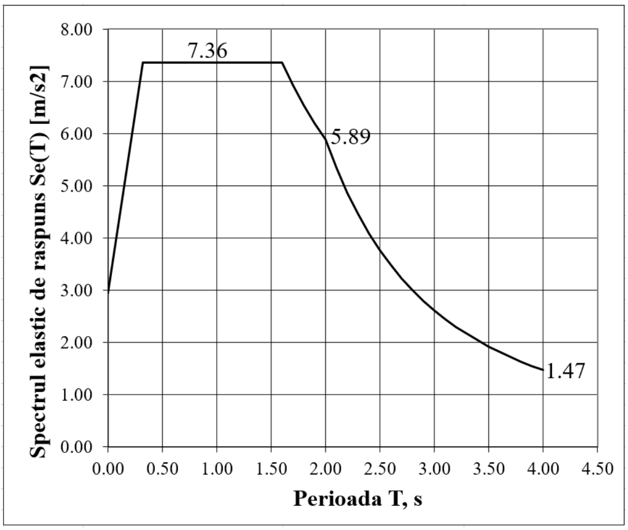

Figure 2.17. Elastic response spectrum $S_e(T)$

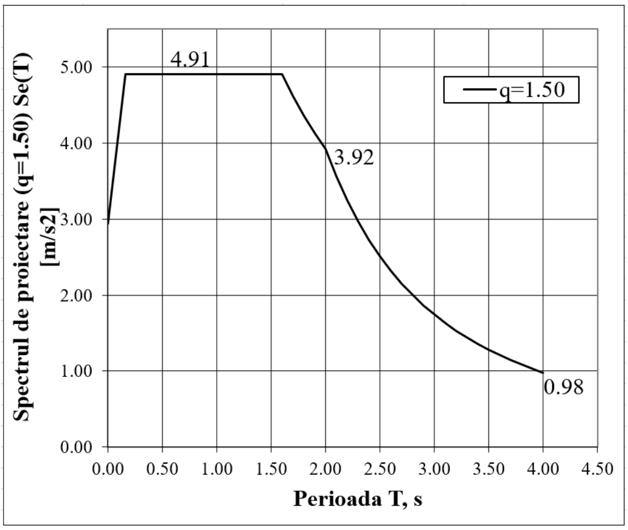

Figure 2.18. Design spectrum $S_d(T)$, (q=1.50)

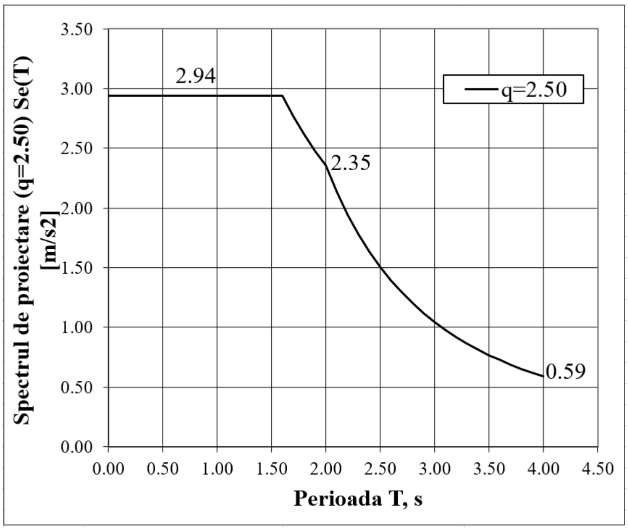

Figure 2.19. Design spectrum $S_d(T)$, (q=2.50)

Analysis and results

To evaluate the seismic response of the structure, two linear dynamic analyses were performed using the Response Spectrum Method. This involves an elastic calculation of the dynamic peak responses for all significant vibration modes of the structure, utilizing the ordinates of the previously defined design response spectra. The total structural response is obtained through a statistical combination of the maximum modal contributions. The primary dynamic parameters of the structure (natural period T and natural frequency f) were established previously, as the mode shapes remain identical.

The two analyses were conducted considering the design spectra shown in Figures 2.17 and 2.18. Spectral ordinates were evaluated assuming both limited ductile behavior ($q = 1.50$) and ductile behavior ($q = 2.50$) for the piers, reflecting their capacity to dissipate seismic energy through post-elastic deformation.

The maximum internal forces at the base of the piers and the maximum displacements at the superstructure level are presented in Table 2.3-1.

	q=1.50	q=2.50
Maximum bending moment [kN*m]	18100.00	10800.00
Maximum shear force [kN]	1390.00	840.00
Maximum bearing displacement [cm]	17.20	10.80

Table 2.3-1. Maximum internal forces and displacements

2.4 Comparison of results

From a seismic hazard perspective, it can be observed that with the implementation of more recent standards, design demands have become increasingly severe. For the selected site, the peak ground acceleration a_g increased from $0.20g$ (under P100-92) to $0.24g$ (under P100-2006), and reached $0.30g$ (under P100-2013). Considering that a large number of existing bridges in Romania were designed for seismic action according to P100-92, this case study investigated the impact of elevated seismic hazard levels on such a structure.

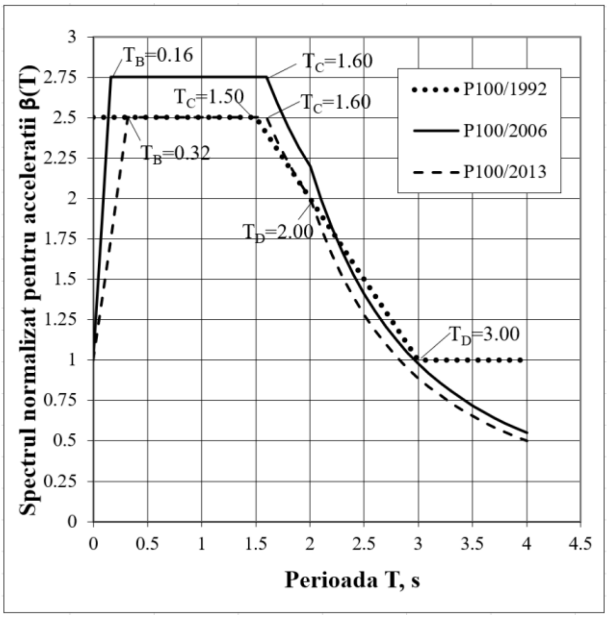

Figure 2.20. Comparison of the normalized response spectrum

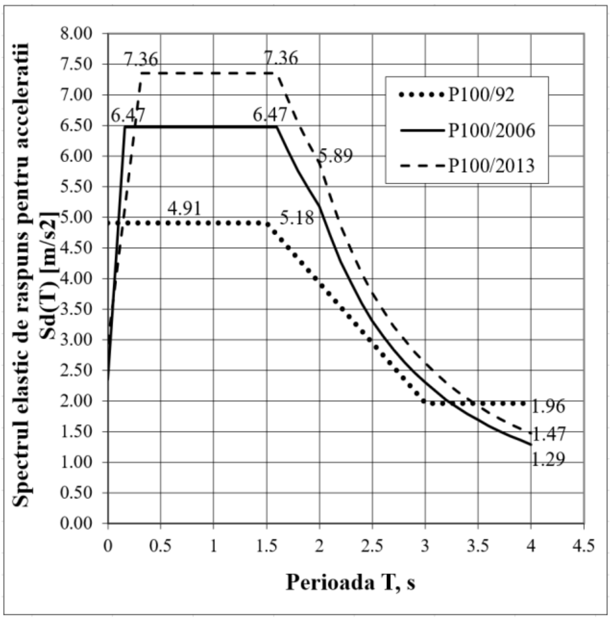

Figure 2.21. Comparison of elastic response spectra for absolute accelerations

As can be observed in Figure 2.19, regarding the normalized response spectra, the differences among the three code editions are minor. Both the corner period values T_C and the maximum dynamic amplification factors for horizontal peak ground acceleration β_0 are approximately equal. The discrepancies become significantly more pronounced when examining the elastic response spectra: the peak spectral ordinates increase by 20% under P100-2006 and by 50% under P100-2013 compared to the initial baseline established by P100-92.

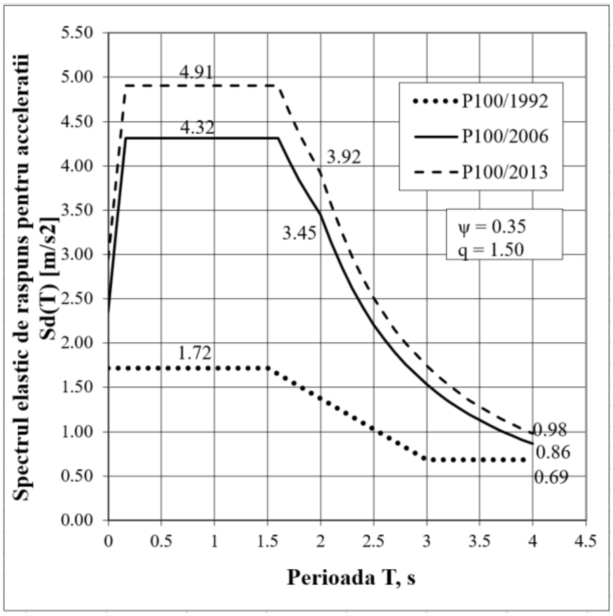

Figure 2.22. Comparison of the design spectrum for ψ=0.35 și q=1.50

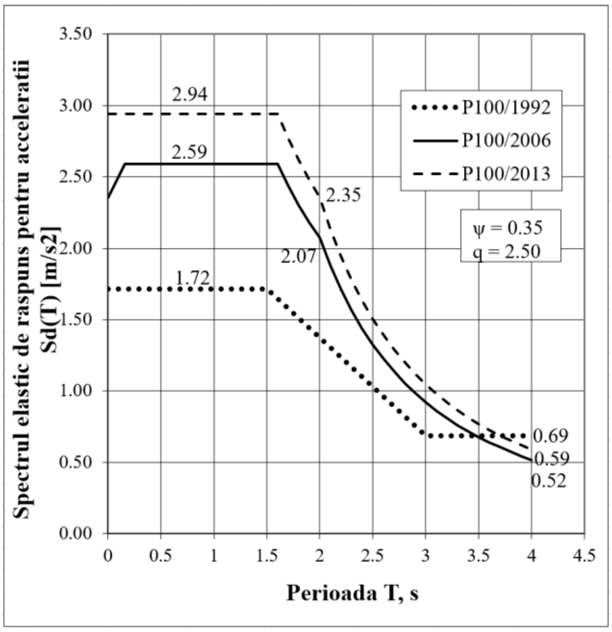

Figure 2.23. Comparison of the design spectrum for ψ=0.35 și q=2.50

Regarding the design spectra, the discrepancies are even more pronounced. For the design spectrum evaluated using a behavior factor of $\psi = 0.35$ (under P100-92) or $q = 1.50$ (under P100-2006 and P100-2013), the maximum ordinates increase by 150% under P100-2006 and by 185% under P100-2013. When assuming a behavior factor of $\psi = 0.35$ (P100-92) versus $q =$

2.50 (P100-2006 and P100-2013), the maximum values increase by 51% under P100-2006 and by 70% under P100-2013.

These significant differences in the design spectra stem from both the higher peak ground accelerations introduced with each successive code release and, more critically, the shift in how structural ductility is accounted for. Under P100-92, selecting a seismic reduction coefficient $\psi = 0.35$, which was assigned to components such as bridge piers, implicitly assumes a ductile structural response characterized by the formation of plastic hinges that dissipate seismic energy.

Equating the reduction coefficient ψ to the behavior factor q reveals that piers designed according to P100-92 correspond to an equivalent behavior factor $q = 2.85$. This implies a ductile structural behavior providing greater energy dissipation compared to both the limited ductility scenario ($q = 1.50$) and the modern ductile classification ($q = 2.50$).

A comparison of the internal forces at the base of the most heavily loaded pier across all three dynamic analyses, as well as the superstructure-level displacements, is presented below.

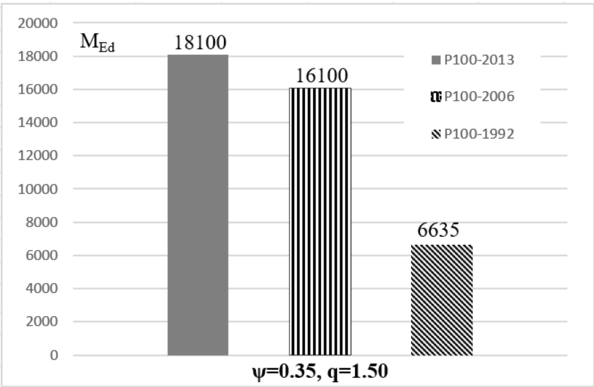

Figure 2.24. Comparison of bending moments at the base of the most heavily loaded pier ($\psi = 0.35, q = 1.50$) [kN·m]

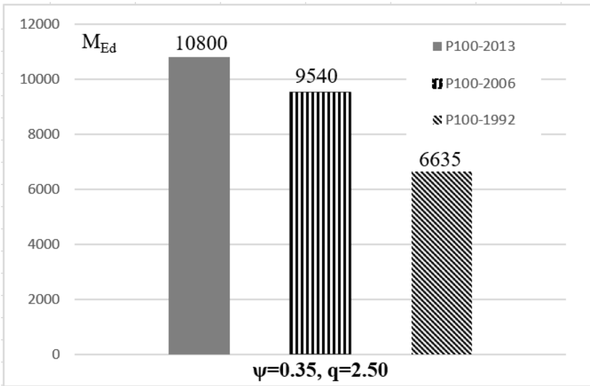

Figure 2.25. Comparison of bending moments at the base of the most heavily loaded pier ($\psi = 0.35, q = 2.50$) [kN·m]

	P100-1992	P100-2006	P100-2013
M_{Ed} [kN*m] (ψ=0.35 și q=1.50)	6635.00	16100.00	18100.00
Difference from P100-1992 [%]	0%	142%	172%

Table 2.4-3. Comparison of the bending moments between the codes analyzed, for q=1.50

	P100-1992	P100-2006	P100-2013
M_{Ed} [kN*m] (ψ=0.35 și q=2.50)	6635.00	9540.00	10800.00
Difference from P100-1992 [%]	0%	44%	63%

Table 2.4-4. Comparison of the bending moments between the codes analyzed, for q=2.50

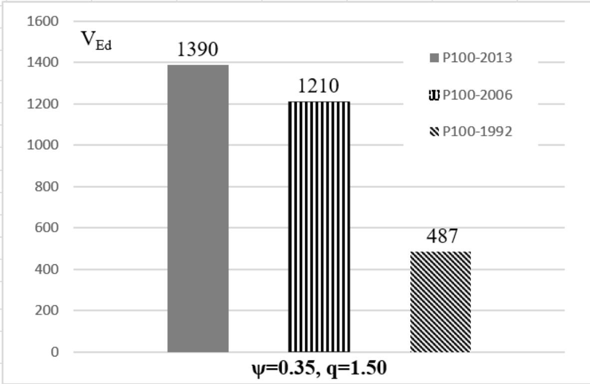

Figure 2.26. Comparison of shear forces at the base of the most heavily loaded pier (ψ=0.35, q=1.50) [kN]

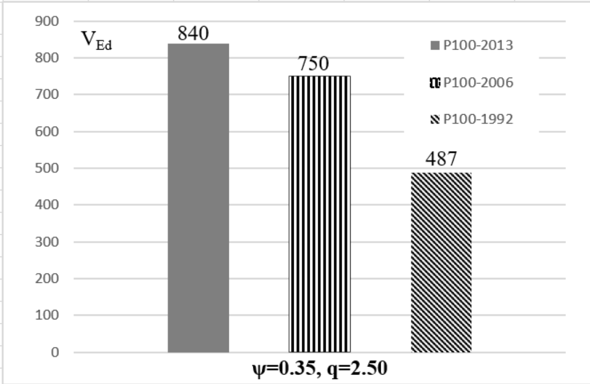

Figure 2.27. Comparison of shear forces at the base of the most heavily loaded pier (ψ=0.35, q=2.50) [kN]

	P100-1992	P100-2006	P100-2013
V_{Ed} [kN*m] (ψ=0.35 și q=1.50)	487.00	1210.00	1390.00
Difference from P100-1992 [%]	0%	144%	180%

Table 2.4-3 Comparison of the shear forces between the codes analyzed, for q=1.50

	P100-1992	P100-2006	P100-2013
V_{Ed} [kN*m] (ψ=0.35 și q=2.50)	487.00	750.00	840.00
Difference from P100-1992 [%]	0%	51%	70%

Table 2.4-4 Comparison of the shear forces between the codes analyzed, for q=2.50

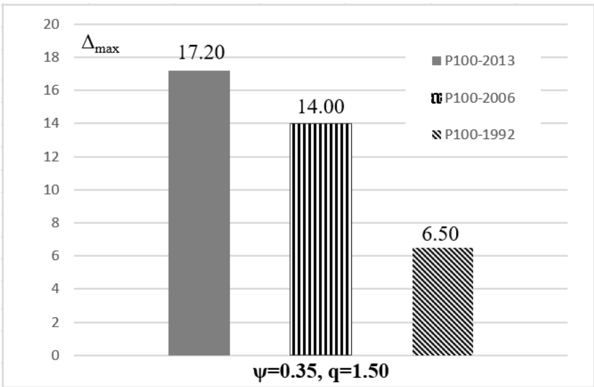

Figure 2.28. Comparison of maximum superstructure displacements (ψ=0.35, q=1.50)[m]

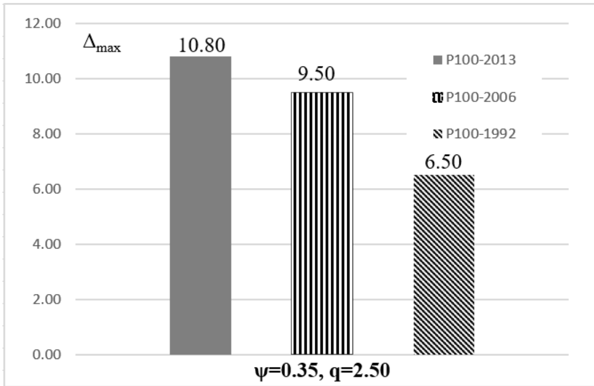

Figure 2.29. Comparison of maximum superstructure displacements (ψ=0.35, q=2.50)[m]

	P100-1992	P100-2006	P100-2013
Δ_{max} [m] (ψ=0.35 și q=1.50)	6.50	14.00	17.20
Difference from P100-1992 [%]	0%	115%	164%

Table 2.4-5 Comparison of the displacements between the codes analyzed, for q=1.50

	P100-1992	P100-2006	P100-2013
Δ_{max} [m] (ψ=0.35 și q=2.50)	6.50	9.50	10.80
Difference from P100-1992 [%]	0%	46%	66%

Table 2.4-6 Comparison of the displacements between the codes analyzed, for q=2.50

As illustrated in the figures and tables above, the piers sized according to the P100-1992 code lack the capacity to resist the internal forces generated in the event of stronger earthquakes, such as those defined in P100-2006 and especially P100-2013. Maximum bending moments increase by 44% to 172%, depending on the seismic hazard level and the behavior factor considered in the analysis—values that significantly exceed the ultimate flexural resistance of the pier cross-section, $M_{Rd} = 6713\ \mathrm{kN \cdot m}$ (Figure 2.10).

Furthermore, regarding superstructure displacements, the response exceeds the maximum allowable bearing displacement capacity of 8.50 cm by 12% to 102%.

Considering these aspects, it is evident that the structure lacks the required capacity to withstand a seismic event stronger than the one defined according to P100-1992.

Consequently, for the majority of bridges in Romania, it is necessary to implement measures aimed at enhancing their seismic performance, ensuring they can withstand earthquakes that could otherwise cause severe damage, including loss of human life and economic losses. The following sections outline several methods to improve the structural response of the analyzed bridge, both by increasing the load-bearing capacity of the piers and by replacing the elastomeric bearings with seismic energy-dissipating devices, thereby reducing the forces transferred to the pier stems.

2.5 Measures for improving the seismic behavior of the structure

Modern engineering practice relies on seismic isolation by introducing specialized devices designed to attenuate the structural seismic response. Response reduction is achieved either by lengthening the fundamental vibration period of the structure, which decreases seismic inertia forces while increasing displacements, or by increasing damping, which controls displacements and can also reduce forces. Ideally, an isolation system combines both mechanisms.

Next, a retrofitted structural model is evaluated by replacing the existing elastomeric bearings with Lead Rubber Bearings (LRB). These devices serve to extend the fundamental period while increasing equivalent damping, typically reaching values of up to 25% at maximum bearing displacement.

Because these devices exhibit non-linear behavior, a Non-Linear Time-History Dynamic Analysis is required, with the seismic input defined by accelerograms that are either recorded or artificially generated based on the site-specific elastic response spectrum $S_e(T)$. For this case study, three spectrally matched accelerograms were generated (Figure 2.27), and the governing structural response is taken as the maximum value obtained across the three time-history analyses.

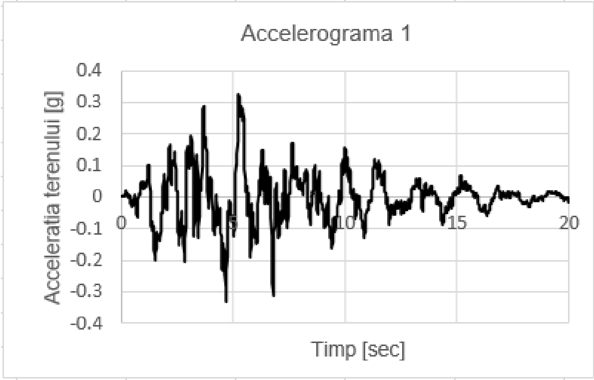

Figure 2.30. Artificial accelerogram generated based on the elastic response spectrum defined according to P100-2006

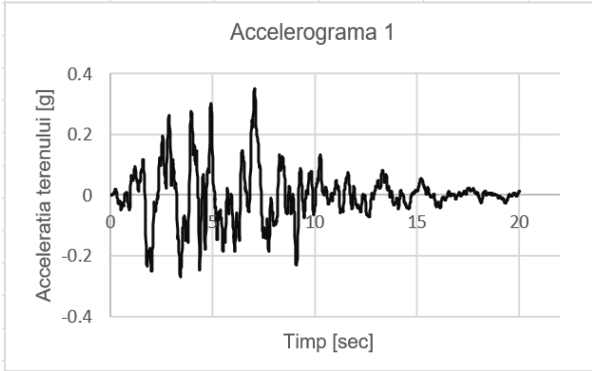

Figure 2.31. Artificial accelerogram generated based on the elastic response spectrum defined according to P100-2013

The primary dynamic characteristics of the structure (natural period T and natural frequency f) are determined through an eigenvalue dynamic modal analysis. Figures 2.29 - 2.31 display the first three fundamental vibration modes of the structure, illustrating that the overall dynamic response is dominated by translation along the two principal horizontal axes, accompanied by in-plane deck rotation.

Figure 2.32. Mode 1 of vibration (longitudinal translation), T_1=2.14s, f_1=0.47 Hz

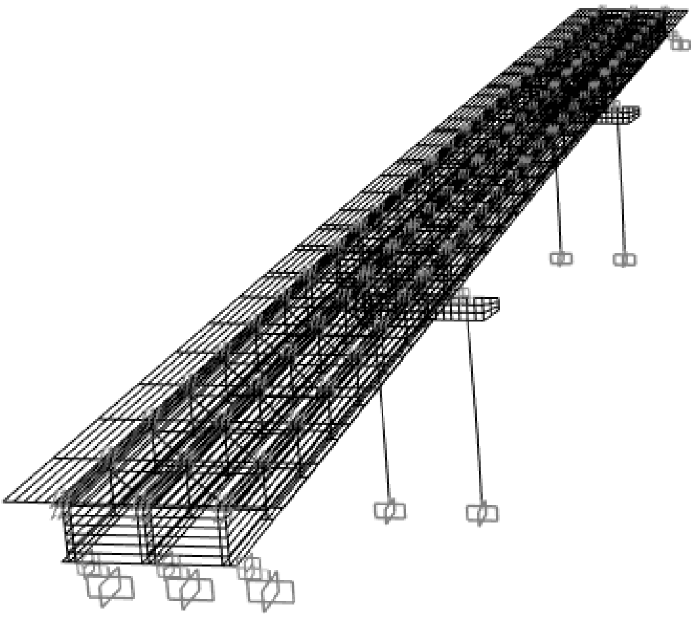

Figure 2.33. Mode 2 of vibration (transverse translation), T_2=2.12s, f_2=0.46 Hz

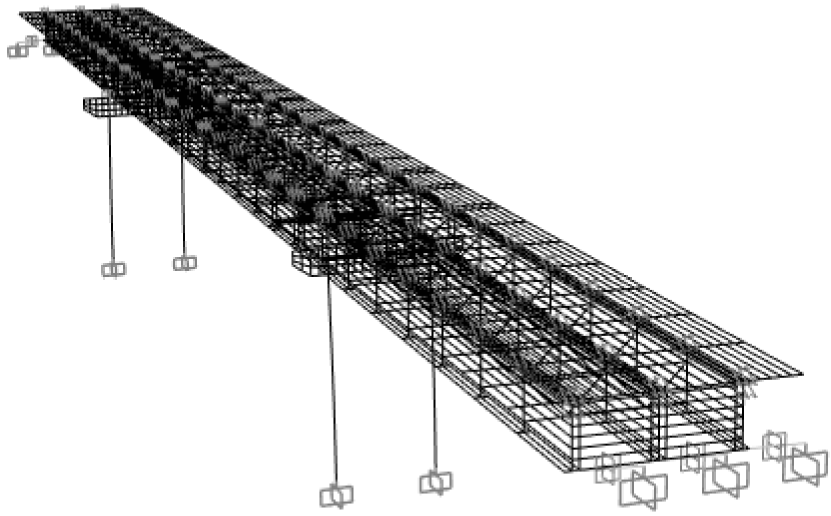

Figure 2.34. Mode 3 of vibration (plan rotation), T_3=1.57s, f_3=0.64 Hz

It can be observed that by utilizing more flexible bearings, the fundamental vibration period of the structure increases ($T_1 = 2.14$ s). Consequently, the internal forces induced in the structure are reduced, as the dynamic amplification factor for horizontal ground acceleration is approximately 30% lower than the maximum dynamic amplification factor (Figure 2.32).

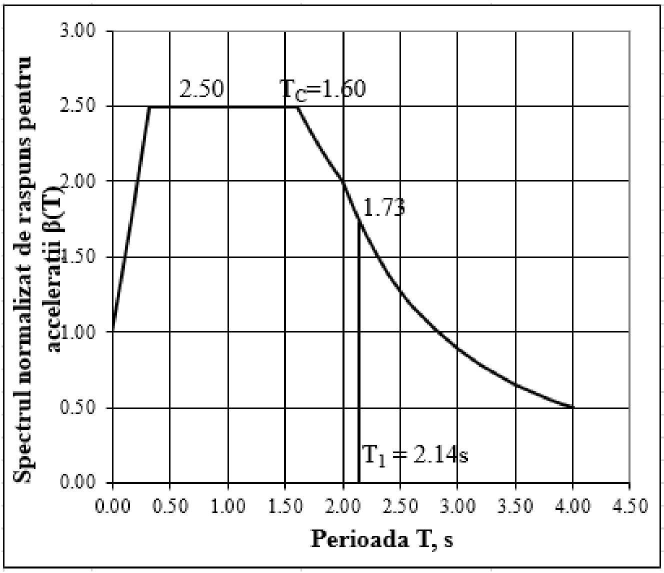

Figure 2.35. Reduction of the dynamic amplification factor through lengthening of the fundamental vibration period (spectrum according to P100-2013)

In the figures and tables below, the internal forces (bending moments and shear forces) at the base of the most heavily loaded pier, as well as the maximum displacements of the bridge superstructure, are presented and compared against those obtained for the same structure evaluated according to the provisions of P100-1992.

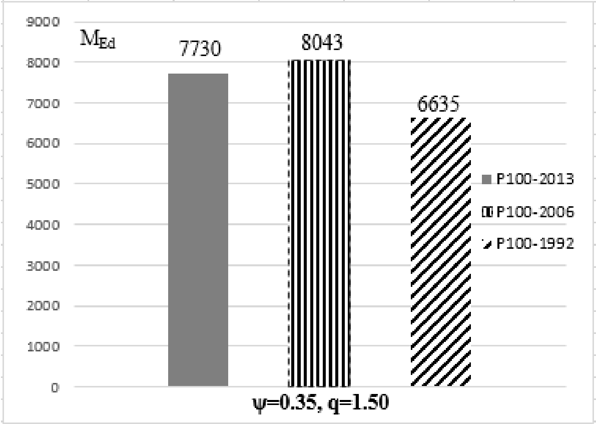

Figure 2.36. Comparison of bending moments at the base of the most heavily loaded pier ($\psi = 0.35, q = 1.50$) [kN·m]

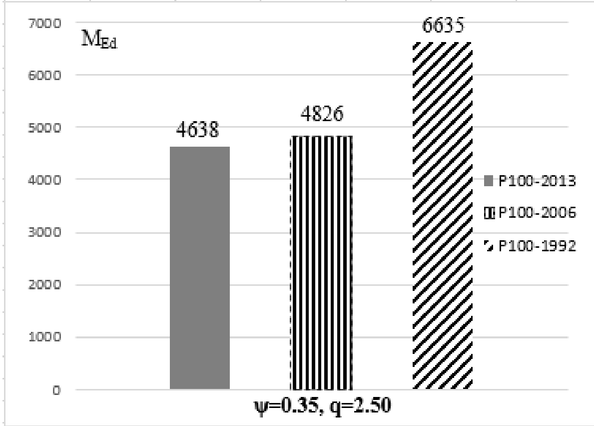

Figure 2.37. Comparison of bending moments at the base of the most heavily loaded pier ($\psi = 0.35, q = 2.50$) [kN·m]

	P100-1992	P100-2006	P100-2013
M_{Ed} [kN*m] (ψ=0.35 și q=1.50)	6635.00	8043.00	7730.00
Difference from P100-1992 [%]	0%	21.00%	16.50%

Table 2.5-1 Comparison of the bending moments between the codes analyzed, for q=1.50

	P100-1992	P100-2006	P100-2013
M_{Ed} [kN*m] (ψ=0.35 și q=2.50)	6635.00	4826.00	4638.00
Difference from P100-1992 [%]	0%	-27%	-30%

Table 2.5-2 Comparison of the bending moments between the codes analyzed, for q=2.50

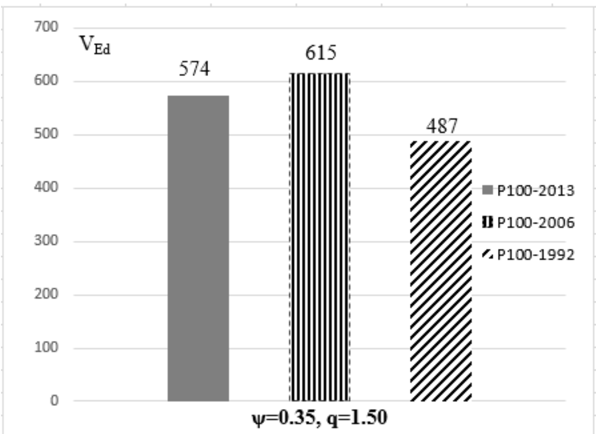

Figure 2.38. Comparison of shear forces at the base of the most heavily loaded pier ($\psi = 0.35, q = 1.50$) [kN·m]

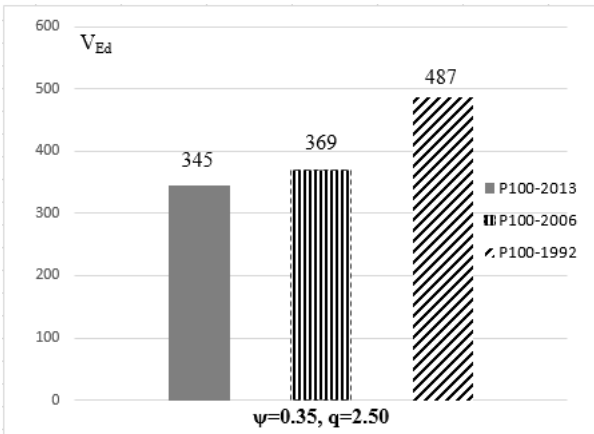

Figure 2.39. Comparison of shear forces at the base of the most heavily loaded pier ($\psi = 0.35, q = 2.50$) [kN·m]

	P100-1992	P100-2006	P100-2013
V_{Ed} [kN*m] (ψ=0.35 și q=1.50)	487.00	615.00	574.00
Difference from P100-1992 [%]	0%	26%	18%

Table 2.5-3 Comparison of the shear forces between the codes analyzed, for q=1.50

	P100-1992	P100-2006	P100-2013
V_{Ed} [kN*m] (ψ=0.35 și q=2.50)	487.00	369.00	345.00
Difference from P100-1992 [%]	0%	-24%	-29%

Table 2.5-4 Comparison of the shear forces between the codes analyzed, for q=2.50

From the figures and tables above, it can be observed that replacing standard elastomeric neoprene bearings with Lead Rubber Bearing (LRB) seismic isolators yields a substantial reduction in seismic response under P100-2006 and P100-2013 code analyses.

Regarding internal forces under P100-2006 and P100-2013 provisions, a reduction of approximately 50% is achieved relative to the un-isolated structure when assuming limited ductile behavior for the piers ($q = 1.50$). Although the integration of seismic isolators significantly lowers force demands, the resulting bending moments still exceed the flexural load capacity of the piers. A further reduction in forces, below the load-bearing capacity limits of the pier. can be attained by designing the structure assuming ductile pier behavior ($q = 2.50$), characterized by plastic hinge formation at the base of the piers to dissipate induced seismic energy.

If limited ductile behavior is assumed, additional measures are required to enhance the load-bearing capacity of the piers.

A first solution consists of retaining the original neoprene bearings and jacketing the pier cross-section with an additional concrete layer and a substantial amount of reinforcement, thereby raising its flexural resistance above the maximum design bending moment corresponding to P100-2013 demands ($M_{Ed} = 18{,}100\ \mathrm{kN \cdot m}$).

Accordingly, applying a 10 cm concrete jacket along with 40 $\phi 32$ rebar ($B500C, f_{yk} = 500\ \mathrm{N/mm^2}, E = 200{,}000\ \mathrm{N/mm^2}$) increases the flexural resistance of the new cross-section to $M_{Rd} = 18{,}192\ \mathrm{kN \cdot m}$, which exceeds the demand values obtained under both P100-2006 and P100-2013. Figure 2.37 shows the updated pier cross-section alongside its moment-curvature diagram.

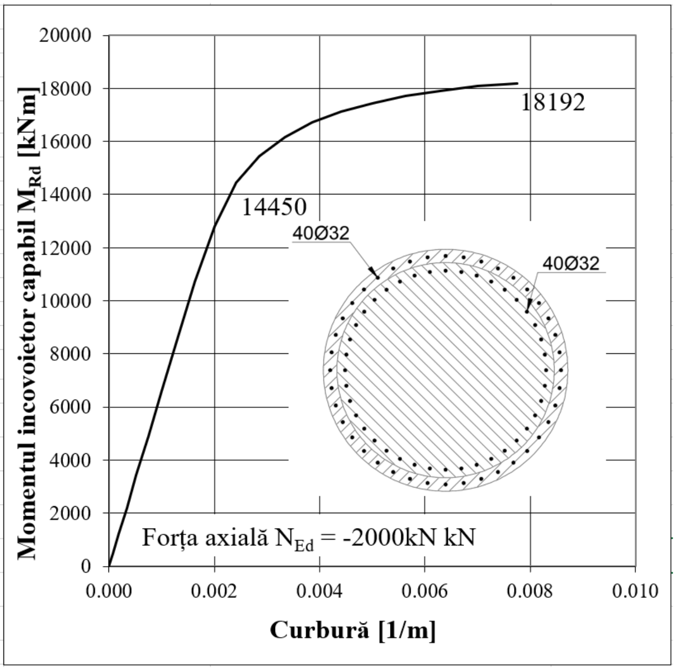

Figure 2.40. Moment-curvature diagram of the new cross section of the pier (Solution 1)

A *second solution* also relies on increasing the load-bearing capacity of the pier; however, in this case, it is implemented in conjunction with the seismic isolation of the structure by replacing the bearings with Lead Rubber Bearing (LRB) seismic isolators. Consequently, the updated pier cross-section must be enhanced to resist the bending moments specified in Table 2.5-1. Unlike the first solution, while the cross-section is similarly enlarged with an additional 10 cm concrete jacket, the required reinforcement is significantly reduced to 20 $\phi 20$ bars ($B500C, f_{yk} = 500\ \text{N/mm}^2, E = 200{,}000\ \text{N/mm}^2$). The new load-bearing capacity of the pier reaches $M_{Rd} = 10{,}282\ \text{kN} \cdot \text{m}$, and Figure 2.38 illustrates the modified cross-section along with its moment-curvature diagram.

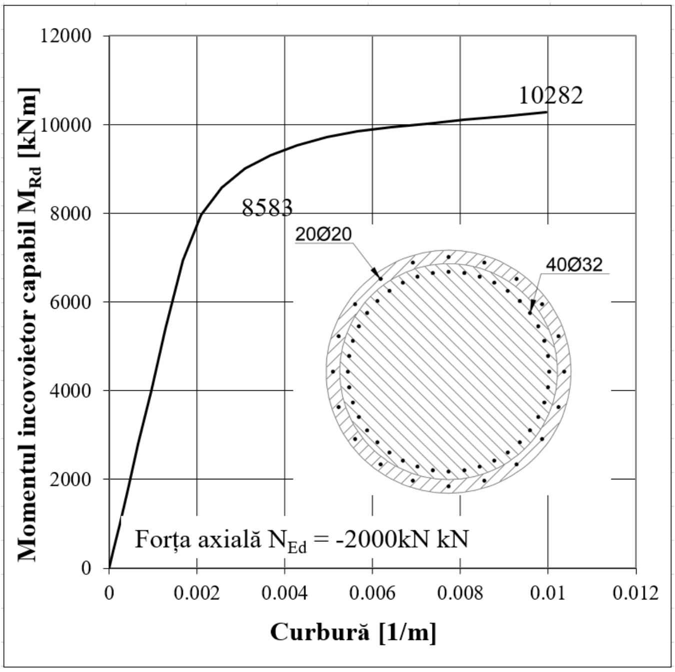

Figure 2.41. Moment-curvature diagram of the new cross section of the pier (Solution 2)

3. CONCLUSIONS

The objective of this research report is to investigate the representation of seismic action according to both Romanian seismic design codes in force to date and international standards (Eurocode 8, FEMA 356).

For the seismic design of structures, the territory of each country or region is mapped into seismic hazard zones. Seismic hazard is characterized by the peak ground acceleration a_g established for a specified reference mean recurrence interval (MRI).

Local ground conditions are represented by the corner period T_C of the response spectrum associated with the site location. The corner (control) period T_C marks the boundary between the constant acceleration response plateau and the constant velocity response spectral region.

In the international codes presented in this report, the influence of local soil conditions on seismic action is explicitly accounted for through ground classifications defined by the average shear wave velocity ($v_{s,30}$) or N_{SPT} values. In Romanian codes, local site effects are accounted for in a simplified manner through the corner period T_C of the site-specific response spectrum.

As a case study, the differences in seismic action modeling across the three Romanian seismic design codes published to date were investigated. The impact of the modifications introduced by P100-2006 and P100-2013 on a bridge originally designed under the requirements of P100-92 was evaluated.

It was observed that while the differences in normalized response spectra are minimal, the resulting design spectra exhibit substantial discrepancies. This is primarily driven by elevated seismic hazard levels (a_g) and, more significantly, by revisions to the behavior factor (q) governing structural ductility. Equating the seismic reduction coefficient ψ to the behavior factor q demonstrates that piers sized under P100-92 correspond to an equivalent behavior factor of $q = 2.85$. This reflects a fully ductile structural response that provides higher energy dissipation compared to both limited ductile behavior ($q = 1.50$) and modern ductile classification ($q = 2.50$).

Regarding internal forces, the existing structure lacks the capacity to resist demands generated by earthquakes stronger than the design event defined by P100-92. Consequently, structural retrofitting or seismic isolation via the replacement of elastomeric neoprene bearings with energy-dissipating devices is required. Seismic isolation substantially improves structural response, reducing internal pier demands by approximately 50% under identical hazard levels. However, force demands remain above the original section capacities. To address this, two retrofit solutions were proposed: (1) structural jacketing of the pier stems with a 10 cm concrete layer and substantial additional reinforcement to raise section resistance, and (2) combining seismic isolation with pier jacketing requiring a significantly lower reinforcement ratio.

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