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Genealogy of Development and Codification of Yugoslavian Earthquake Resistant Design

Zoran Milutinovic¹, Radmila Salic Makreska¹, Daniel Tomic¹, Jovan Trajceviski¹

Abstract

This contribution aims to clarify for exposure modellers and vulnerability and seismic risk assessors the basis on which seismic lateral loads have been established in all Yugoslavian seismic design codes since 1948. The particular focus is on JUS39/64 and JUS31/81 seismic design codes since they controlled the seismic safety of Macedonian (and Yugoslavian) exposure for 16 and 41 years, respectively. The analyses and discussions are centred on lateral force coefficients as a primary factor for representing the features of each design level.

Presented is in all detail the developed K-Quotient method that efficiently compares inter-code lateral force levels. The method provides the closed solution for simplified lateral load calculation applicable to “stiff” buildings ($T \leq 0.5s$). The presented results are indicative of flexible buildings requiring further research on the subject.

Keywords Seismic design evolution • Lateral force levels • K-Quotient method • Yugoslavia • Seismic design codes • JUS standards

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Introduction

A building code is a set of minimum regulations intended to safeguard building occupants' public health, safety, and general welfare. The development, adoption, and enforcement of building codes vary widely from one country to another. In some countries, the United States, for example, the authority to regulate building construction is delegated to local jurisdictions. In other countries, building codes are developed by governmental agencies and are enforced nationwide. Yugoslavia used a combined model by enforcing seismic design codes nationwide but also delegating authority to regulate building construction to Federal units (Republics).

An understanding of the level of design of a given building class is fundamental for the development of vulnerability models that are capable of representing the features of each design level (Borzi et al. 2008; Verderame et al. 2010; Romao et al. 2019; Crowley et al. 2021) in national seismic risk assessment models.

The Institute of Earthquake Engineering and Engineering Seismology (IZIIS), Ss. Cyril and Methodius University in Skopje, with the assistance and the support of the National Coordinator for Implementation of the National Platform for Disaster Risk Reduction, based on the data of last, the 2021 census (if available), intends to develop a high resolution national exposure model.

Works along this line are already ongoing on the European scale. The European Commission's Horizon 2020 SERA is the first concerted effort among the research community to produce a uniform European risk model (ESRM20). The European seismic risk assessment exposure model is presented in Crowley et al. (2020a and b). In support of Crowley et al., 2020b, Crowley et al., 2021 presents the "Model of seismic design lateral force levels for the existing reinforced concrete European building stock," Republic of North Macedonia (RNM) included.

Yugoslavia enforced the first JUS61/48 earthquake design regulations in 1948, Table 1. The second (JUS39/64) was introduced in 1963/1964 following the 26th July 1963 Skopje earthquake, the time when IAEE² recorded 25 to 27 seismic design codes worldwide. The third, JUS31/81, was introduced in 1981 (in effect from August 1982). Out of 63 recorded nowadays, it was the 31st Code recorded by IAEE, Fig. 1. By OGoRNM No. 211 (Official Gazette of RNM) of 02.09.2020, Macedonia enforced Eurocode Design System MKC EN 1990 – MKC EN 1999, allowing JUS31/81 and all other related standards in parallel use for a period of 3 years.

Fig. 1 Growth in Worldwide number of seismic design codes (Data source: Reitherman, 2008)

The paper presents in necessary detail the spatial and temporal evolution of seismic design requirements in Yugoslavia over the last 70-75 years. The subject is scarcely treated and published in detail. To the authors' best knowledge, a brief review of Croatian earthquake-resistant design from 1948 to 2021 is presented in Zamolo, 2021 (in Croatian). However, the subject is not treated in such technical details as this paper does.

² IAEE: International Association for Earthquake Engineering <https://www.iaee.or.jp/>

Table 1 Evolution of Yugoslavian technical regulations for earthquake resistant design

Regulation	Abbreviated as	Level
<i>Provisional Technical Regulations for Building Loading</i> (Official Gazette of FNRY No. 61/48)	JUS61/48 (PTP2)	National FNRY
<i>Regulations for Design and Construction of Structures in Seismic Regions</i> (Official Gazette of SRS No. 18/63 of June 13, 1963)	SI18/63	Republic, Slovenia
<i>Regulations for Design and Construction of Structures in Seismic Regions</i> (Official Gazette of SRM No. 33/63 of 24 September 1963) Translation of SI18/63	MK33/63	Republic, Macedonia
<i>Provisional Technical Regulations for Construction in Seismic Regions</i> (Official Gazette of SFRY No. 39/64 of 30 September 1964)	JUS39/64	National SFRY
<i>Technical Regulations for Building Construction in Seismic Regions</i> (Official Gazette of SFRJ No. 31/81 of 5 June 1981, including amendments: No. 49/82 of 13 August 1982, No. 29/83 of 10 June 1983, No. 21/88 of 1 April 1988, and No. 52/90 of 7 September 1990)	JUS31/81	National SFRY
<i>Technical Regulations for Repair, Strengthening and Reconstruction of Building Damaged by Earthquakes, and for Reconstruction and Revitalization of Buildings;</i> (Official Gazette of SFRY No. 52 of 4 October 1985)	JUS52/85 (Not discussed in this contribution)	National SFRY
<i>Regulations on Technical Norms for Design and Calculation of Civil Engineering Facilities in Seismic Regions;</i> (1986, Draft)	JUSxx/86 (Not discussed in this contribution)	National SFRY
Eurocode Design System MKC EN 1990 – MKC EN 1999 (OGORNM No. 211 of 02.09.2020)	MKC EN 1998-1	National RNM
<i>Eurocode 8: Design of structures for earthquake resistance - Part 1: General rules, seismic actions and rules for buildings</i>		

57

58 The lateral force coefficient K , Eq. 1, follows Crowley et al., 2021 formulation that coincides with the
59 JUS31/81 definition.

$$60 \quad K = K_0 \cdot K_S \cdot K_P \cdot K_D \quad (1)$$

61 K_0 is a coefficient of building importance, (K_S) is a coefficient of seismic intensity, (K_P) is a coefficient
62 that considers the energy-dissipation capacity of the structure (in JUS codes termed coefficient of
63 damping and ductility; in other codes very often termed as a structural coefficient) and K_D is a dynamic
64 coefficient or coefficient of dynamic response calculated from the period of vibration of the structure.
65 Adopting the same K formulation enables a direct comparison of findings of this contribution with those
66 presented in Crowley et al., 2021.

67 The contribution also tends to resolve the “memory hole” effect viz that generations trained in ongoing
68 Code have inappropriate experience in previous codes. While it is not an issue for design practice, it is
69 of utmost importance for vulnerability and seismic risk modellers and analysts to understand and model
70 the level of design of a given building class in national/regional/European seismic risk assessment
71 models. Accordingly, the second primary objective of this contribution is to clarify and save specific
72 evolutionary facts for seismic design in RNM and SFRY from the memory hole.

73

74 **2 Development of Equivalent Static Lateral Force Method**

75 **2.1 Worldwide Beginning**

76 Workable equivalent static lateral force design methods (the “seismic coefficient” or “seismic ratio”
77 methods) are developed independently in Italy and Japan following the 1908 Reggio-Messina, Italy and
78 1906 California, United States earthquakes, respectively. Japanese method is based on the work of Eng.
79 Toshikata Sano, who visited San Francisco after the 1906 San Francisco Earthquake, in 1914 submitted
80 his doctoral thesis entitled "Earthquake Resistance of Buildings," in which he proposed the use of
81 equivalent static horizontal load in seismic design of buildings.

82

The fundamental difference between the Italian and Japanese concepts is in the definition of the design lateral force. Sano's "shindo" coefficient is based on the intensity of ground motion, while the Italian is based on the estimated lateral resistance of structures that survived the earthquake (Otani, 2008). The "rapporto sismico" in Italy, or the "shindo" in Japan, later became "seismic coefficient" or "seismic ratio."

The essence of Italian and Japanese seismic coefficient ideas is contained in the $F = ma$ inertial concept of Isaac Newton (1642-1727) built on earlier work of Galilei Galileo (1554-1642), Reitherman 2008. The significance of Toshikata Sano in the world history of earthquake engineering merits some attention to his work as presented in his 1916 treatise. Sano presents his coefficient, *shindo*, as $k = a/g$. The term a is the estimated acceleration of the ground motion to be used in the design, and when divided by gravitational acceleration, g , $g=9.81m/s^2$, it conveniently results in a dimensionless coefficient K that can be applied to a building weight (W) to give a value in terms of units of force, Eq. 2.

$$F = K \cdot W = \frac{a}{g} W \quad (2)$$

In response to the destruction caused by the great 1923 Kanto earthquake, Japan introduced the seismic design of buildings in 1924 through the revision of the Urban Building Law promulgated in April 1919 (Otani, 2008). It is the first official implementation of Sano's criterion requiring design seismic forces equal to 10 percent of the floor weight.

Substantial refinements of the method beyond these Italian and Japanese precedents in the USA came much later, Reitherman, 2000. In California, the Santa Barbara earthquake of 1925 motivated several communities to adopt codes with C' (equivalent to K_s) as high as 20 percent of gravity. The first edition of the U.S. Uniform Building Code (UBC), published in 1927 by the Pacific Coast Building Officials (PCBO), contained an optional seismic appendix that adopted Sano's criterion. It allowed variations in C' depending on the region and foundation material. For building foundations on soft soil in earthquake-prone regions, the UBC's optional provisions corresponded to a lateral force coefficient equal to the Japanese value. The adopted lateral force coefficient was 7.5 percent for buildings on hard ground.

According to the genesis of UBC³, the lateral force system structural factors (K_p) and coefficient of dynamic response (K_d) are included starting by UBC edition 1961. The building importance factor (K_o) is included in UBC edition 1976, while the seismic intensity coefficient (K_s) is included in UBC edition 1949 following the adoption of the first, the 1948 seismic hazard map, elaborated by US Coast and Geodetic Survey [<https://www.earthquakecountry.org/library/12706.pdf>].

2.2 Yugoslavian Prospective

2.2.1 Prior to the 1963 Skopje earthquake

Pre-WWII period, Slovenia

Within the territory of Yugoslavia, the first earthquake prevention measures were instituted in Slovenia following the Ljubljana earthquake of 1895 ($I \approx VIII$ degrees in MCS scale). In the absence of methods for earthquake resistance analyses of structures, the seismic resistance of dominating massive brick/stone masonry structures was accomplished by a top-down increase of thicknesses of bearing walls, assuring: (1) the lower centre of gravity of the structure and, (2) that the strongest, the base bearing sections, receive the highest earthquake loading stresses, Bubnov, 1981.

Between the two World Wars, Slovenia adopted standards for masonry building loading requiring very low horizontal loads for seismic analysis, i.e., the uniform horizontal load of 1/50 of the structure's weight, Bubnov, 1981. According to Bubnov's definition "uniform horizontal load of 1/50 of the weight of the structure," it seems that Slovenia followed Sano's shindo concept (constant inertial force along with the height of a building) relevant for perfectly rigid structures.

³ Maintained and published by the International Conference of Building Officials, 610 South Broadway, Madison 8-2314, Los Angeles 14, California

1948 *Provisional Regulations for Building Loading* JUS61/48 (PTP2)

Following the Second World War (WWII), by OGoFNRY No. 61/48, Yugoslavia in 1948 adopted the Provisional Technical Regulations for Building Loading JUS61/48 referred to nationally as PTP2. PTP2 dedicates two sections to the definition of lateral loads. The first one defines the minimum horizontal loading H_{min} by structural type, Table 2. The presented H_{min} 's may be interpreted as K_P coefficients for URM masonry structures that at the time were the predominant construction typology. The second section allocates the coefficient of seismic intensity K_S to three mapped "seismological" zones, Table 3.

The product of (3x1) K_P^{48} row vector and (1x3) K_S^{48} column vector is (3x3) K^{48} seismic coefficient matrix defined by Eq. 3 and presented in Table 4.

$$S_k = K^{48} W_k = K_P^{48} \cdot K_S^{48} \cdot W_k \quad (3)$$

where S_k is story $\langle k \rangle$ seismic action and W_k is a total vertical load at and above the story $\langle k \rangle$.

Table 2 Minimum horizontal loading H_{min} or K_P^{48}

Structural Type	(in % of W) = K_P^{48}
Buildings with massive walls and massive floors (roofs)	1.0
Buildings with massive walls and light floors (roofs)	1.5
Buildings with light walls and light floors	2.0

Table 3 Coefficient of seismic intensity K_S^{48}

Seismological zone	K_S^{48}
Zone of minor damages ($I_{MCS} = VII$)	1.0
Zone of significant damages ($I_{MCS} = VIII$)	1.5
Zone of catastrophic destruction ($I_{MCS} \geq IX$)	2.0

Table 4 K^{48} Seismic coefficient (in % of W)

Structural Type		Seismic Zone		
		VII	VIII	$\geq IX$
Buildings with massive walls and massive floors (roofs)		1.0	1.5	2.0
Buildings with massive walls and light floors (roofs)		1.5	2.25	3.0
Buildings with light walls and light floors		2.0	3.0	4.0
JUS31/81 K^{81}	Reinforced masonry ($K_{P2} = 1.3$)	3.25	6.5	13.0
	Masonry strengthened by vertical RC cerclages ($K_{P3} = 1.6$)	4.0	8.0	16.0
	Ordinary* masonry structures ($K_{P4} = 2.0$)	5.0	10.0	20.0

The term "ordinary" is undefined; most probably, it refers to URM

K^{48} coefficients range from 1.0% to 4.0% and depending on the type of the masonry and the seismic zone, they are 2.5 to 5 times lower than K^{81} .

Slovenian Regulations for Design and Construction of Structures in Seismic Regions

Shortly after adopting PTP2 regulations, the Slovenian academic and professional community concluded that they are not adequate for ensuring satisfactory resistance of structures to earthquake loadings and that new regulations on the subject must be drawn up. This finding has been supported by the effects of a number of small earthquakes affecting Slovenia at that time and achievements and developments in earthquake engineering Worldwide. This stimulated the Slovenian Ministry for the industry to create a special governmental commission to prepare updated earthquake-resistant design regulations. Government of NR Slovenia in 1963 officially adopted *Regulations for Design and Construction of Structures in Seismic Regions* SI18/63 (Official Gazette of SRS 18/63No. 18/63 of June 13, 1963). The model code for SI18/63 was the seismic design code of the USSR: "The norms and rules for construction in seismic regions," SN-8-57.

2.2.2 Post 1963 Skopje earthquake

History demonstrated Reitherman 2000 and 2006 that the impetus of national development of earthquake engineering and the subsequent codification of the developments is usually an earthquake event that brings immense casualty and loss of material property and sets new socio-economic priorities that, depending on the size and economic power of the country, can even change the overall course of national development policies.

In the case of Yugoslavia, they were:

- The Skopje Mw6.1 July 26, 1964, Earthquake, killing 1,070 and injuring 3,000-4,000 citizens, bringing about 150-200,000 homelessness, 80% of the City destruction, loss of about 84.2% of the administrative value of the City; 15% of Yugoslavian GDP for the year of 1962 or 2 GDPs' of the Socialist Republic of Macedonia; and,
- The Montenegro Mw6.9 April 15, 1979, Earthquake killing 101 citizens in Montenegro (and 35 in Albania), bringing about 100,000 homelessness and destruction estimated at 18% of Yugoslavian GDP for the year of 1978.

In response to destruction and loss caused by the Skopje earthquakes, Macedonia and Yugoslavia adopted MK33/63 and JUS39/64 (Table 1) earthquake resistant design regulations. The Montenegro earthquake accelerated the completion of the revision of JUS39/64 that started in 1975. The new code, JUS31/81, regulating the earthquake resistant design of buildings, was published in 1981 and entered in force in 1982 following the adoption of a new seismic zoning map in 1982.

SI18/63, MK33/63, and JUS39/64 code patterned itself after at the time contemporary USSR practice, such as SN-8-57 and SNiP II-A.II 12.62 codes. JUS31/81 largely patterned itself after, at the time, contemporary US practice, the UBC76, and UBC79 Codes.

The most direct response to the Montenegro earthquake was an elaboration of the Code for “Repair, Strengthening and Reconstruction of Building Damaged by Earthquakes, and for Reconstruction and Revitalization of Buildings.” It was enforced in 1985.

By the mid-1980s, IZIIS completed the development of the Code for “Design and Calculation of Civil Engineering Structures and other Facilities in Seismic Regions.” In 1986 it reached the status of 'Draft.' Although unofficially used since 1986, it has not been officially enforced.

In September 2020, RNM adopted the Eurocode design system. Since then, JUS31/81 has been in parallel use for 3 years (OGORNM No. 20/211 of 2.09.2020).

Characteristics of MK33/63 and SI18/63 Earthquake Resistant Design Regulations

The 1963 Skopje Earthquake took place just 40 days following the enforcement of the SI18/63. Immediately following the earthquake, the Government of Macedonia enforced the *Law on the Procedure for Retrofitting the Social Standard Facilities Damaged by Skopje Earthquake* (Official Gazette of SRM No. 31/63 of 6 September 1963) accompanied by the following recommendations:

- Recommendation for Eliminating the Consequences of the Earthquake in Skopje in the Field of Education (the same Official Gazette); and,
- Recommendation on Measures for Elimination of the Consequences of the Earthquake at the University of Skopje (the same Official Gazette).

To avoid pure retrofit being estimated as an unjustifiable investment in earthquake prone areas and to assure the feasibility and sustainability of all technical interventions during the initial phase of Skopje reconstruction, the Government of Macedonia enforced *Regulations for Design and Construction of Structures in Seismic Regions MK33/63* (Official Gazette of SRM No. 33/63 of 24 September 1963), which was translated version of Slovenian regulations SI18/63.

Both codes define $\langle i^{th} \rangle$ mode seismic action at $\langle k^{th} \rangle$ story, S_{ik}^{63}

$$S_{ik}^{63} = K_i^{63} \cdot w_k = K_S^{63} \cdot K_{Di}^{63} \cdot K_{\eta}^{63} \cdot w_k \quad (4)$$

where, K_i^{63} is the modal seismic coefficient, K_{Di}^{63} = dynamic coefficient (originally denoted by β), w_k = $\langle k^{th} \rangle$ story tributary weight, and $K_{in}^{63} = w_k$ and height position-dependent coefficient of vertical distribution, in the following, referred to as modal shape coefficient.

The characteristics of both 1963 codes are:

- The structures constructed in I_{MCS} earthquake zones of VII, VIII, and IX are obligatorily designed and executed according to provisions of SI18/63 and MK33/63 rulebooks;
 - The design and execution of structures in $I_{MCS} < VII$ zones shall be, irrespectively of site-specific ground conditions, designed by K_S^{63} assigned to $I_{MCS}=VII$ for good ground conditions (Article 2);
 - Recognition of the importance of the structures (coefficient K_p) prescribing that temporary buildings not intended for the residence of people be designed for one seismic degree lower than the degree of seismicity in which they are located (Article 3);
 - Define K_S^{63} coefficient of seismic intensity for good, medium, and weak ground types (GT) by introducing S^{63} soil factor, Table 5;
 - Introduces the concept of elastic design spectra K_{Di}^{63} by defining it as a product of S^{63} soil factor and the 5% damped elastic spectral shape S_{ie}^{63}
- $$0.5 \leq S_{ie}^{63} = \frac{0.75}{T_i} \leq 1.5 \quad (5)$$
- Introduces the energy dissipation concept (coefficient K_p^{63}) by requesting (Article 12) that seismic action for low damping structures (industrial chimneys, high towers, still columns, and similar) is increased by a factor up to 1.6
 - Proposes two methods for “stress control,” the “approximate” and the “more accurate”:
 - Approximate: requesting that if the period of the structure is not calculated according to empirical formulas, it should be set to $T=0.5s$ and coefficient $K_{in}^{63} = 1.0$ used
 - More accurate: Requires a maximum of three modes but does not provide mode combination criteria.

The SI18/63 and MK33/63 regulations were in effect until 30 September 1964, when, at the Federal level, they were replaced by the National *Provisional Technical Regulations for Construction in Seismic Regions (Official Gazette of SFRJ No. 39/64 of 30 September 1964)* – in the following referred to as JUS39/64.

By enforcement of JUS39/64, Macedonia immediately withdrew the *Law on the Procedure for Retrofitting Social Standard Facilities Damaged by Skopje Earthquake (Official Gazette of SRM No. 44/64 of 31 December 1964)*.

1963 Republic codes, adopted initially by Slovenia and following the 1963 Skopje earthquake by Macedonia, controlled the design and construction in these republics for a relatively short period, about 15 months in Slovenia and one year in Macedonia. Both were succeeded by JUS39/64 in September 1964.

Table 5 K^{63} seismic coefficient

Ground Type (GT)		Description	Imcs Seismic zone			S^{63} factor	Diff
			VII	VIII	IX		
Good	GT_1	Hard rock, homogeneous gravel deposits	0.015	0.03	0.06	0.75	-25%
Medium	GT_2	Homogeneous sand deposits; over consolidated clayey and marl deposits; moderately heterogeneous clayey and sandy-gravel deposits	0.020	0.04	0.08	1.00	$\pm 0\%$
Week	GT_3	Heterogeneous deposits; soft marls and clays	0.025	0.05	0.10	1.25	+25%

3 Detail Comparative Analysis of JUS39/64 and JUS31/81 Seismic Coefficients

3.1 Definition of Base Shear

For flexible structures, Table 10, JUS39/64 (Article 1.3) define $\langle i^{th} \rangle$ mode seismic action at $\langle k^{th} \rangle$ story, S_{ik}^{64}

$$S_{ik}^{64} = K_i^{64} \cdot w_k = K_S^{64} \cdot S^{64} \cdot S_{ie}^{64} \cdot K_{i\eta}^{64} \cdot w_k \quad (6)$$

where

$$K_{i\eta}^{64} = x_{ik} \frac{\sum_{j=1}^n w_j x_{ij}}{\sum_{j=1}^n w_j x_{ij}^2} \quad (7)$$

and combine the modal responses by

$$E_k^{64} = \sqrt{0.5 [(E_{i,max}^{64})^2 + \sum_{i=1}^3 E_{ik}^2]} \quad (8)$$

for defining the seismic action effect under consideration (force, displacement, etc.). Please note that x_{ij}, x_{ik} are the displacements of storey tributary weights in the first three mode shapes. In essence, the JUS39/64 method for determining the seismic action is the classic Modal Response Spectra (MRS) method limited to three modes with stipulated participation of the maximum seismic action effect.

JUS39/64 for stiff structures allows the use of alternate simplified method prescribing $S_e^{64} = 1.5$ and fundamental mode shape approximated by horizontal displacements increasing linearly along the height of the building

$$K_{k\eta}^{64} = x_k \frac{\sum_{j=1}^n w_j h_{ij}}{\sum_{j=1}^n w_j h_{ij}^2} \quad (9)$$

Irrespective of the method used, the base shear is a sum of story shears

$$F_b^{64} = \sum_{k=1}^n F_k^{64} \quad (10)$$

One of the shortcomings of JUS39/64 is the failure to define criteria defining the stiff structures to which the alternative simplified method is prescribed.

Following the UBS pragmatic concept JUS31/81 directly determines the base shear

$$F_b^{81} = K^{81} \cdot W = K_0^{81} \cdot K_P^{81} \cdot K_S^{81} \cdot S^{81} \cdot S_e^{81} \cdot K_{i\eta}^{81} \cdot W \quad (11)$$

and then distribute it along the height of the building with the following approximate formula:

$$S_k = (F_b^{81} - S_T) \frac{w_k h_k}{\sum_{j=1}^N w_j h_j} \quad \begin{cases} N < 5, S_T = 0 \\ N \geq 5, S_T = 0.15 F_b^{81} \end{cases} \quad (12)$$

For (1) all out-of-category buildings and (2) prototypes of prefabricated buildings or structures which are produced industrially in large series (except for wooden structures), JUS31/81 prescribes the dynamic analysis (nonlinear RTH) method.

Common to both JUS codes is a request that seismic action effects shall be determined using two planar models, one for each main horizontal direction.

3.2 Seismic Weight (w, W)

The seismic weight (w, W) for buildings all JUS's defined as a sum of all dead (q), probable live loads (p), and snow (s) loading. Probable live loading is considered to be 50% of the loading specified in the loading regulations (JUS U.C7.121). If live loading is a significant factor (e.g., in the case of warehouses, silos, libraries, archives, similar), then the seismic loads are determined for the most

unfavourable case of maximum or minimum actual loading. JUS U.C7.121 is identical with ISO 2103-1986.

JUS63/48 and 1963 SI/MK Republic codes in a body text prescribe only $q+0.5p$; however, the seismic weight was formulated according to a table similar to JUS U.C7.121 requirements. The genesis of defining the building seismic weight is presented in Table 6.

Table 6 General features of JUS codes

Code	Design Concept	Code type	Methods of analysis	Seismic Weight (W, w)
JUS61/48	Working stress	Force based	ELF	$q^{(1)}+0.5p^{(2)}$
SI18/63 MK33/63	Working stress	Ductility	ELF, RSM (3modes)	$q+0.5p^{(3)}$
JUS39/64	Working stress ULS* allowed	Ductility	ELF, RSM (3modes), RTH allowed	$q+0.5p^{(4)}+s^{(5)}+e^{(6)}+c^{(7)}$
JUS31/81	ULS	Ductility	ELF, RTH	$q+0.5p^{(8)}+s+e$

* ULS: Ultimate strength

(1) Self weight of the structure

(2) Live load

(3) For tanks, cisterns, bunkers, silos, etc., the whole live load should be taken into account

(4) If there is a real probability that buildings will be loaded with the total live load in time of the earthquake, it should be considered in the total amount. This is especially true of buildings where live load makes up a significant part of the total load (libraries, archives, etc.);

(5) Snow

(6) Full weight of immovable (permanently installed) equipment

(7) Horizontal forces, which belong to the primary loads of the building, and whose action can realistically be expected during the earthquake, as well as the influence of inertial forces from the oscillation of liquids during the earthquake (water pressure, soil, materials in tanks, silos, etc.).

(8) If live loading is a significant factor (e.g., in the case of warehouses, silos, libraries, archives, etc.), then the seismic loads shall be determined for the most unfavourable case of maximum or minimum actual loading.

3.3 The Seismic Intensity Coefficient K_S^{C64}, K_S^{C81}

Both JUSs prescribe the same seismic intensity coefficient K_S for mapped seismic intensity zones of $I_{MCS} = VII, VIII$ and IX degrees, Table 7. Characteristic to both seismic intensity coefficients is that they, in an implicit way, mask the coefficient D_L that is solely dependent only on the level of allowable structural damage. For $D_L = 4$, PGA values corresponding to $I_{MCS} = VII, VIII$, and IX degree zones are 0.1, 0.2, and 0.4, respectively. Accordingly, from 1963 onwards, the $K_S - PGA$ relation has not changed, thus so far is constant. More details are presented and discussed in Section 3.5.

Table 7 $K_S^{C64}, K_S^{C81} - PGA$ relation from 1963 onwards

K_S^{CODE} / PGA^{CODE}	I_{MCS} Zone		
	VII	VIII	IX
K_S^{C64}, K_S^{C81}	0.025	0.05	0.1
PGA^{C64}, PGA^{C81}	0.1	0.2	0.4

3.4 Building Importance Factor K_O^{C64}, K_O^{C81}

The specific pattern of early SNiPs is the relation between the design I_{DSG} and the-site specific I_{MCS} seismicity.

The site-seismicity is based on statistical (SZM1950 and SZM1982) or PSHA (SZM1987) analyses of either intensities (I) or magnitudes (M) and their mapping over the entire territory of SFRY in the form of seismic zoning maps (SZM). Term design seismicity is self-explanatory.

SZM1950 accompanied JUS39/64. SZM1982 accompanied JUS31/81 from August 1982 till September 1990, when it was abandoned and replaced by SZM1987.

The relation between the design I_{DSG} and the mapped I_{MCS} JUS39/64 seismicity is presented in Table 8. With the increase of I_{MCS} for one degree, the seismic coefficient K_S^{64} (or corresponding PGA) doubles its value, Table 7. K_O^{64} factors, calculated as follows, are presented in parentheses in Table 8.

For each prescribed design seismicity degree, the values of K_O^{64} coefficients are calculated as quotient between the values of seismic coefficients that would correspond to the prescribed design seismicity I_{DSG} ($K_{S,DSG}^{64}$) and the values of seismic coefficients that correspond to mapped seismicity $K_{S,MCS}^{64} = K_S^{64} = K_S^{81}$, Table 7.

$$K_O^{64} = \frac{K_{S,DSG}^{64}}{K_{S,MCS}^{64}} \quad (13)$$

Hence, the JUS39/64 design seismicity matrix formulated in I_{MCS} terms is a product between the (3x1) K_S^{64} transposed row vector (Table 7) and (3x3) K_O^{64} importance matrix, Table 9.

$$K_{S,DSG}^{64} = [K_S^{64}]^T K_O^{64} \quad (14)$$

Table 8 JUS39/64 design seismicity (I_{DSG}) matrix

Importance class	Type of Building or Structure	Design Seismicity (I_{DSG})		
		Mapped Seismicity (I_{MCS})		
		VII	VIII	IX
---	Out of category buildings and structures			
I	Buildings that contain rooms for assembly use (schools, cinemas, theatres, gymnasiums, etc.); buildings with function of particular importance in earthquake aftermath (hospitals and fire stations etc.); important buildings of federal and republic relevance; industrial buildings containing particularly valuable equipment; buildings containing objects of exceptional cultural and artistic value (museums etc.); buildings and structures whose collapse could cause further catastrophic consequences.	VIII ($K_O^{64} = 2.0$)	IX (2.0)	IX+ (1.5)
II	Residential buildings; hotels; restaurants; railway, tram, trolleybus and bus depots; museums (not included in I); libraries, institutes, administrative and communal buildings, industrial production buildings (not included in I); PTT buildings (not included in I); storages of valuable equipment and goods, engineering structures (antenna towers, industrial chimneys, bridges, reservoirs, silos, retaining walls, and other similar structures); water distribution and sewage system installations; all other structures not included in I, II and IV.	VII (1.0)	VIII (1.0)	IX (1.0)
III	Auxiliary industrial buildings on which the primary production does not depend; energy facilities of local significance; stables for cattle on community farms.	VII (1.0)	VII (0.5)	VIII (0.5)
IV	Buildings whose collapse cannot endanger human lives and cause damage to expensive equipment; temporary structures	Seismic influences shall not be taken into account		

Table 9 JUS39/64 and JUS31/81 importance factors

Building Category	K_O^{64} Importance factor matrix			K_O^{81} Importance factor vector
	VII	VIII	IX	
I	2.0	2.0	1.5	1.5
II	1.0	1.0	1.0	1.0
III	1.0	0.5	0.5	0.75

K_O^{64} , K_O^{81} values are shown comparatively in Table 9. This table highlights the conceptual differences between JUS38/64 and JUS31/81 formulations of importance coefficient. K_O^{64} is a function of building category and seismic zone, whereas K_O^{81} solely of building category.

Referring to the presented values, K_0^{64} relative to K_0^{81} for 33.3⁴% stipulate the value of importance coefficient for Category-I buildings located in I_{MCS} zones of VII and VIII degrees. Also, it depreciates it for -33.3% for Category-III buildings in I_{MCS} zones of VIII and IX degrees.

3.5 Ductility and Damping Coefficients and R_p Factor

Generations of designers and researchers in RNM are convinced that JUS39/64 is force based code and that JUS31/81 is ductility code. The latter is because JUS39/64 expression for seismic coefficient K does not include a coefficient equivalent to K_p (Ductility and Damping in JUS31/81) that takes care for energy dissipation, viz damping and ductility capacity of the structure. Moreover, by no single line does JUS39/64 discuss the energy dissipation capacity of structures.

Both JUSs are classical K_S -format codes assigning the non-dimensional coefficient K_S to zoned territories, Table 7.

Neither JUSs nor early SNIps discussed the background of K_S . The first is the SNIp II-7-81 that transited from K_S - to PGA-format code, i.e., assigned PGA values (in g , $g=9.81m/s^2$) to zoned territories. For that, SNIp II-7-81 introduced coefficient K_1 which in this contribution is used and discussed in its reciprocal form $D_L=1/K_1$. D_L is a coefficient solely dependent on the level of allowable structural damage. For Importance class-II building typology, its reference value is $D_L = 4^5$ and is comparable with EN 1998-1 behaviour factor $q=3.9$ for DCM design. In essence, the $D_L = 4$ is the nominal energy dissipation coefficient that is hidden in the formulation of K_S^{64} and K_S^{81} coefficients of seismic intensity.

Table 10 Values of JUS31/81 ductility and damping coefficient (K_p) and corresponding R_p values

Structural Type		JUS39/64		JUS31/81	
		K_p	R_p	K_p	R_p
K_{P1}	All modern reinforced-concrete (RC) structures, all-steel structures except those listed under item (2) of this table, and all modern wooden structures, except those listed under item (3) of this table.	1.0	4.0	1.0	4.0
K_{P2}	All reinforced masonry structures and braced still frame structures	1.0	4.0	1.3	~3.1
$K_{P3.1}$	Masonry buildings strengthened by vertical RC cerclages; All RC shear wall structures with aspect* ratios of each wall i lower than 2 (Article 68).	1.0	4.0	1.6	2.5
$K_{P3.2}$	Very high and slender low damping structures such as high industrial chimneys, antenna towers, masts, water-towers, and other structures with a fundamental natural vibration period of vibration $T \geq 2.0s$.	1.6	2.5	1.6	2.5
K_{P4}	Structures with a flexible ground story or any other story, or for structures where there are dramatic changes in stiffness, or with a sudden change in stiffness, as well as ordinary (URM) masonry structures.	1.0	4.0	2.0	2.0

* h_{wi}/l_{wi} ; h_{wi} = height of the wall; l_{wi} = length of the wall i

The actual energy dissipation factor of JUS codes, R_p , become visible with the transformation of JUS's seismic coefficient K in R-code format

$$K = K_0 \cdot K_p \cdot K_S \cdot S \cdot S_e(T) \cdot K_\eta = K_0 \cdot K_p \cdot \frac{PGA}{D_L} \cdot S \cdot S_e(T) \cdot K_\eta = K_0 \cdot \frac{PGA}{R_p} \cdot S \cdot S_e(T) \cdot K_\eta \quad (15)$$

where

$$R_p = \frac{D_L}{K_p} \quad (16)$$

JUS39/64 Article 2.5.3 for structural typology 3.2, Table 10, prescribes $K_{P3.2} = 1.6$. Other structural typologies are not mentioned, which implies that their $K_p = 1.0$. It also does not recognise

⁴ 33.3 Underlined number is repetend

⁵ $D_L=1.0$ For buildings and structures in which permanent deformations and local damage are not allowed, and $D_L=8.3$ Buildings and structures in which significant residual deformations, cracks, damage to individual elements, their displacement is allowed (SNIp II-7-81, Table 3, page 7)

structural typology 3.1 recognized later by JUS31/81. To avoid repetition of Table 10 for JUS39/64 and JUS31/81 structural typology, the JUS31/81 structural typology K_{P3} is split into two categories $K_{P3,2}$, included in JUS39/64 only, and both, $K_{P3,1}$ and $K_{P3,2}$, included in JUS31/81 structural typology.

JUSs R_p coefficient, Eq. 16, are physically equivalent to EN 1998-1 behaviour factor q . Their values are tabulated in Table 10.

3.6 Design Response Spectral Shapes K_D^{64} , K_D^{81}

JUS39/64 and JUS31/81 define the site-specific design response spectral shapes K_D as a product between the soil coefficient S and 5%-damped response spectral shape S_e . Such spectra, as in every national code, are aimed to properly quantify and codify the influence of local ground conditions on the amplitude (S) and frequency S_e modification of regional ground motions expressed and mapped either in terms of intensity or acceleration.

$$K_D(T) = S \cdot S_e(T) \quad (17)$$

Such disaggregation is necessary and allows discussing both coefficients separately. In Eqs. 11 and 15 K_D of Eq. 1 is represented in its disaggregated form defined by Eq. 17.

For discriminated ground types (Table 11), both design response spectral sets are presented comparatively in Fig. 2.

3.7 5%-damped Spectral Shapes S_e^{64} , S_e^{81}

The amplitude and frequency modification of regional motions both codes achieve with different, ground type dependent S_e^{64} and S_e^{81} soil factors.

Characteristic to the JUS39/64 S_e^{64} (Article 2.5.3) is the use of one spectral shape defined by Eq. 18 and a single constant acceleration region plateau extending over the $T = 0-0.5s$ period range.

$$0.50 \leq S_e^{64} = \frac{0.75}{T_i} \leq 1.5 \quad (18)$$

JUS31/81 defines three 5%-damped spectral shapes by the following expression:

$$\frac{2 T_{C,GTi}^{81}}{3} \leq S_{e,GTi}^{81} = \frac{T_{C,GTi}^{81}}{T_i} \leq 1 \quad (19)$$

where $T_{C,GTi}^{81}$ is the upper, ground type-dependent corner period of the constant spectral acceleration branch denoted by subscript $\langle C \rangle$.

The basis for the development of S_e^{81} spectral shapes are $S_e^{64} \cdot S_e^{64}$ spectral shapes:

- $S_{e,GT1}^{81}$ is constructed by normalizing the corresponding $S_{e,GT1}^{64}$ with itself;
- $S_{e,GT2/3}^{81}$ for GT_2 and GT_3 ground types are constructed by intersecting corresponding $S_{e,GT2/3}^{64}$ with $S_{e,GT1}^{64/81} = 1$ ordinate;
- the GT_2 intersection point (the upper corner period of the constant spectral acceleration branch, $T_{C,GR2}$) is moved left for 0.05s in order to
 - capture better the effects recognized by spectral analyses of ground motion data obtained from the 15 April 1979 Montenegro earthquake; and,
 - assure equidistant (0.2s) plateau extensions by a decrease of ground stiffness;
- extract the GT_4 ground category from the Code provisions.

One of the problems of such construction of S_e^{81} design response spectral shape set is the extraction of ground-type dependent amplitude influence on modification of regional motions over the entire constant acceleration region plateaus.

Parameters S , T_C , T_D defining the shape of horizontal elastic response spectra are tabulated in Table 12. The elastic response spectral shapes $S^{64} \cdot S_e^{64}$ and $S^{81} \cdot S_e^{81}$ are comparatively presented in Fig. 2.

Values of corner periods of the constant spectral acceleration ranges $T_{C,GTi}$ are estimated as too high (Poceviski, 1982), causing lengthy plateaus “so that most of the buildings will need to be designed to the maximum value of seismic forces.” He proposed shortening of spectral plateaus for 0.2 to 0.3 seconds, justifying it by the fact that such definition of S_e^{81} leads to an unjustifiably high cost of construction. While it is true, the longer plateaus provide increased design requirements for flexible structures, hence are in favour of higher seismic safety of the construction of flexible buildings.

Fig. 2 Comparative presentation $S^{64} \cdot S_e^{64}$ and $S^{81} \cdot S_e^{81}$ elastic response spectral shapes

3.8 Soil Factor S^{64} , S^{81}

Table 2 of JUS39/64 defines the "Coefficient of seismicity" K_S as a function of mapped seismicity I_{MCS} and the ground type (GT). It recognizes three ground types (good, medium and week) as presented in Table 11. However, Article 2.5.3 requests that "In the case of soft soil of considerable thickness, the equation $(1.5/T_i)$ should be used instead of equation $(0.75/T_i)$." De facto, it is the introduction of an additional, the fourth ground type described as "Soft deposits of considerable thickness" with factor S_4 equal to 2.

It is interesting to note that in response to the 1985 Mexico City earthquake, UBC edition 1988 added a fourth soil profile type, S_4 , for very deep soft soils, with the factor $S_4 = 2$.

JUS39/64, Table 2, defines the design seismic intensity coefficient (K_{SD}^{64}) as a function of I_{MCS} zone and ground type, Table 11, that mask the S^{64} soil factor.

The seismic zoning in Yugoslavia is historically conducted in terms of I_{MCS} degrees associated with the "characteristic ground" conditions defined as clayey-sandy or sandy-clayey deposits with groundwater table at about 4 meters from the surface EB73, 1973. The EB87, 1987 renamed it "average ground

conditions" for a given seismic zone. However, neither JUS39/64 nor JUS31/81 use the term "average ground conditions." Instead, both use the term "medium ground conditions" as an equivalent to "average ground conditions."

To disaggregate K_{SD}^{64} into S^{64} and K_S^{64} coefficients, the K_{SD}^{64} table is normalized to medium ground conditions (GT_2). The results of normalization are I_{MCS} independent S^{64} soil factor (4x1 column, Table 11) vector and seismic intensity coefficient K_S^{64} (1x3 row vector, Table 7). The matrix product defined by Eq. 20 fully reconstructs the K_{SD}^{64} table presented in Table 11.

$$K_{SD}^{64} = S^{64} [K_S^{64}]^T \quad (20)$$

Table 11 K_{SD}^{64} design seismic intensity coefficient table and disaggregated derivatives K_S^{64} and S^{64}

Ground Type (GT)		Description	K_{SD}^{64} for I_{MCS} Zone			S^{64} factor	
			VII	VIII	IX		
Good	GT_1	Hard rock, homogeneous gravel deposits	0.02	0.04	0.08	S_1	0.8
Medium	GT_2	Homogeneous sand deposits; over consolidated clay and marl deposits; moderately heterogeneous clayey and sandy-gravel deposits	0.025	0.05	0.10	S_2	1.0
Weak	GT_3	Heterogeneous deposits; soft marls and clays	0.03	0.06	0.12	S_3	1.2
Very weak	GT_4	Soft deposits of considerable thickness	0.05	0.10	0.20	S_4	2.0
K_S^{64} , K_S^{81} Seismic coefficients before transformation to PGA, Table 7			0.025	0.05	0.10	---	

Table 12 Behaviour of S^{81} soil factor over the $0-T_D$ + period range

round Type	S^{81}	T_C (s)	T_D (s)	$0-T_{C,GT1}$	$T_{C,GT1} - T_{C,GT2}$	$T_{C,GT2} - T_{C,GT3}$	$T_{C,GT3} - T_D$
Good	S_1	0.5	1.5	1.0	Eq. k.2 (N ¹)	0.714 (-28.6%)	
Medium	S_2	0.7	1.5	1.0			
Week	S_3	0.9	1.5	1.0		Eq. k,3 (L ²)	1.286 (+28.6%)
Very weak	S_4	---	---	1.0	(N ¹) Nonlinear, (L ²) Linear relation		

JUS31/81, as discussed above, defines three different, ground-type dependent design response spectral shapes. Each spectral shape is characterised by its own $0-T_{c,GTi}$ plateau, Fig. 2. The ground type-dependent corner periods of the constant spectral acceleration branch $T_{c,GTi}$ are presented in Table 12. The plateau for good ground type extends up to 0.5s, whereas for medium and weak ground typology is extended by an equidistant increment of 0.2s, to 0.7s and 0.9s, respectively.

Since JUS31/81 defines three GT dependent design response spectral shapes, it is logical to assume that the influence of ground typology is built-in their $S_{e,GTi}^{81}$ shapes. The $S_{e,GTi}^{81}/S_{e,GT2}^{81}$ normalization of design response spectral shapes defines the spectral behaviour of S_i^{81} soil factors, which are presented in Fig. 3 and tabulated in Table 12.

S_2^{81} over all spectral regions (acceleration, velocity, and displacement) equals 1.0. S_1^{81} is equal to 1.0 over its acceleration region (0-0.5s), from where nonlinearly decrease to a value of 0.714 ($= T_{c,GT1}^{81}/T_{c,GT2}^{81}$) until the T_{GT2}^{81} corner period of 0.7s and maintain it over the rest of T_{GT2}^{81} velocity and the entire T_D + displacement region. S_3^{81} equals to 1.0 over the entire T_{GT2}^{81} acceleration region. From T_{GT2}^{81} to T_{GT3}^{81} it linearly increases to a value of 1.268 ($= T_{c,GT3}^{81}/T_{c,GT2}^{81}$) and maintains it over the rest of T_{GT3}^{81} velocity and the entire T_D + displacement region.

Relative to GT_2 ground type, the maximum S_1^{81} and S_3^{81} variations (Fig. 3, Table 12) are $\pm 28.6\%$ and are slightly greater than S_1^{64} and S_3^{64} ones that are $\pm 25.0\%$.

446 The $K_{D,GTi}^{81}$ expressed in S_{GT2}^{81} and $S_{e,GTi}^{81}$ terms is

$$447 \quad K_{D,GTi}^{81} = \frac{T_{GT2}^{81}}{T} \begin{cases} S_1^{81} \text{ (Eq. 22)} \\ 1 \\ S_3^{81} \text{ (Eq. 23)} \end{cases} \quad (21)$$

448 where

$$449 \quad S_1^{81} = \begin{cases} 1 & 0 \leq T \leq T_{C,GT1} \\ 1 - \frac{T - T_{C,GT1}}{T_{C,GT2} - T_{C,GT1}} & T_{C,GT1} \leq T \leq T_{C,GT2} \\ T_{C,GT1}/T_{C,GT2} & T_{C,GT2} \leq T \leq \infty \end{cases} \quad (22)$$

$$450 \quad S_3^{81} = \begin{cases} 1 & 0 \leq T \leq T_{C,GT2} \\ 1 + \frac{T - T_{C,GT2}}{T_{C,GT3} - T_{C,GT2}} & T_{C,GT2} \leq T \leq T_{C,GT3} \\ T_{C,GT3}/T_{C,GT2} & T_{C,GT3} \leq T \leq \infty \end{cases} \quad (23)$$

451

Fig. 3 Spectral behaviour of S^{81} soil factors

452

453 **4 Comparative Analysis of K^{64} and K^{81} Seismic Coefficients**

454 Due to conceptual differences between the formats of JUS39/64 and JUS31/81 codes for the definition
 455 of base shear, the direct comparison of coefficients constituting the seismic coefficient for flexible
 456 buildings ($H > 20\text{m}$, or $n > 5$ stories) cannot be made. The comparisons of computer-aided results can be
 457 made; however, the insight in behaviour and contribution of some coefficients constituting the seismic
 458 coefficient will be lost.

459 For stiff buildings ($H \leq 20\text{m}$, or $n \leq 5$ stories), both codes prescribe the use of a simplified method.
 460 JUS39/64 calculate story shears according to Eq. 6 and then by Eq. 10 sums them in F_b^{64} . The period of
 461 $n \leq 5$ story buildings, calculated by $T = 0.1n$, is $T \leq 0.5\text{s}$ and fully corresponds to the JUS39/64 definition
 462 of “stiff structure.”

463

464 **4.1 K – Quotient method**

465 The most efficient way to compare or scale the values of base shears defined by two or more codes is
 466 taking the appropriate ratio. The ratio, RK_b , defined by Eq. 24 expresses the F_b^{81} base shear in terms of
 467 F_b^{64}

$$RK_b = \frac{F_b^{81}}{F_b^{64}} = \frac{K^{81}}{K^{64}} = \frac{K_O^{81}}{K_O^{64}} \frac{K_P^{81}}{K_P^{64}} \frac{S^{81}}{S^{64}} \frac{S_e^{81}}{S_e^{64}} \frac{K_S^{81}}{K_S^{64}} \frac{K_\eta^{81}}{K_\eta^{64}} = RK_O RK_P RS RS_e RK_S RK_\eta \quad (24)$$

If, for a particular Seismic Zone (K_S), Building Importance Category (K_O), site Ground Conditions (S), Building Type (K_P) and Number of Stories (n), the RK_b is:

- $RK_b > 1$, JUS31/81 base shear force is *larger* than that prescribed by JUS39/64; and,
- $RK_b < 1$, JUS31/81 base shear force is *smaller* than that prescribed by JUS39/64.

It is also true for all RK_b constituents. If the constituent $<i> RK_i > 1$, it will tend to increase the JUS31/81 base shear relative to that prescribed by JUS39/64, and vice versa, if $RK_i < 1$, the constituent $<i>$ will tend to decrease the JUS31/81 base shear relative to that prescribed by JUS39/64.

The RK_b of Eq. 24 is a product of six quotients (ratios). Accordingly, this contribution terms it $K - Quotient$ method.

The method provides a closed-form solution for stiff structures, thus a single value. For flexible structures, it is necessary to conduct structural analysis and then compare the final results.

4.1.1 Stiff Buildings

According to JUS39/64 Article 2.6, the application of a simplified method is recommended for stiff structures ($H \leq 20$ metres, Article 1.3). It is conditioned by: (1) doubling the value of the coefficient of dynamic response (instead $0.75/T$ use of $1.5/T$), and (2) calculation of modal shape factor according to Eq. 9. For the same structural category ($n \leq 5$ storeys), JUS31/81 calculates the F_b^{81} by Eq. 11.

The period of $n \leq 5$ story buildings is in the range 0-0.5s. It is comprised of all three S_e^{81} plateaus and the S_e^{64} plateau. Thus $S_e^{81} = S_e^{64} = 1$. As discussed above, for using a simplified method JUS39/64 (Article 2.6) request that S_e^{64} spectral shape is multiplied by a factor of 1.5. Thus,

$$RS_e = \frac{S_e^{81}}{S_e^{64}} = \frac{1}{1.5} = \frac{2}{3} \quad (25)$$

The coefficient of seismic intensity, irrespectively of its formulation (Table 7), is the same for both codes, hence

$$RK_S = \frac{K_S^{81}}{K_S^{64}} = \frac{PGA^{81}}{PGA^{64}} = 1 \quad (26)$$

The K_η^{64} shape coefficient is a sum of story shape coefficients $K_{k\eta}^{64}$ defined by Eq. 9 and equals to

$$K_\eta^{64} = \sum_{k=1}^n K_{k\eta}^{64} = 1.5 \frac{n(n+1)}{2n+1} \quad (27)$$

JUS31/81 base shear is expressed in W (total weight of the structure), not by storey tributary weights $w = w_i$ as in JUS39/64. Substituting W with $n \cdot w$ and cancelling out all other ratios except K_η^{81}/K_η^{64} defines $RK_\eta = n/K_\eta^{64}$. The reminder n is de facto the K_η^{81} coefficient, and RK_η ratio is

$$RK_\eta = \frac{K_\eta^{81}}{K_\eta^{64}} = \frac{2}{3} \frac{2n+1}{n+1} \quad (28)$$

Defined RK_O , RS , RK_P , and RK_η ratios are displayed in Tables 13 to 16, respectively. Based on Eqs. 26 and 27, Eq. 24 reduces to:

$$RK_b = \frac{2}{3} RK_O RK_P RS RK_\eta \quad (29)$$

Table 13 JUS39/64-JUS31/81 RK_0 ratios

Building Importance Category		VII	I_{MCS} VIII	IX
1	RK ₀₁	0.75	0.75	1.00
2	RK ₀₂	1.00	1.00	1.00
3	RK ₀₃	0.75	1.50	1.50

Table 14 SDC64-SDC81 RS ratios

Ground Category			S^{64}	S^{81}	RS
I	S_1	Good	1.2	1.0	0.8333
II	S_2	Medium	1.0	1.0	0.6667
III	S_3	Weak	1.2	1.0	0.5556
IV	S_4	Very Weak	2.0	1.0	0.3333

Table 15 JUS39/64-JUS31/81 RK_P ratios

Building Typology		K_P^{64}	K_P^{81}	RK_P
(1)	RK_{P1}	1.0	1.0	1.0
(2)	RK_{P2}	1.0	1.3	1.3
(3.1)	$RK_{P3.1}$	1.0	1.6	1.6
(3.2)	$RK_{P3.2}$	1.6	1.6	1.0
(4)	RK_{P6}	1.0	2.0	2.0

Table 16 JUS39/64-JUS31/81 RK_η ratio

# of stories	K_η^{81}	K_η^{64}	RK_η
6	1.0	0.8077	1.2381
5	1.0	0.8182	1.2222
4	1.0	0.8333	1.2000
3	1.0	0.8571	1.1667
2	1.0	0.9000	1.1111
1	1.0	1.0000	1.0000

503

Fig. 4 Behaviour of K_b by number of stories, ground types, and structural typology; Plots by Ground types Fig. 5 Behaviour of K_b by number of stories, Ground Categories, and structural typology Plots by structural typology

From Tables 13 to s.16, it is observable that RK_p and RK_η ratios, both ≥ 1 tend to increase RK_b . On the other hand RS and RS_e (Eq. 25) ratios tend to decrease it. RK_O ratio, Table 13, for Importance Class I buildings decrease RK_b in I_{MCS} VII and VIII seismic zones whereas stipulate it for Importance Class III buildings in I_{MCS} VIII and IX zones.

Behaviour of RK_b by ground categories are presented in Figs. 4 and by structural typology in Figs. 5. Data used to generate Figs. 3 and 4 are tabulated in Table 17.

From Figs. 4 and 5, it is evident that JUS31/81 is superior only for all K_{p4} structures on GT_1 ground category and $n=3-5$ stories $K_{p3.1}$ structures on GT_2 ground category.

Table 17 K conversion coefficient values

VIII # of Stories	Soil Factor S				Structural Type K_p			
	S_1	S_2	S_3	S_4	K_{p1}	K_{p2}	$K_{p3.1}$	K_{p4}
$RK_{p1} = 1.0$					$S_1 = 0.8333$			
5	0.6790	0.5432	0.4527	0.2713	0.6790	0.8827	1.0864	1.3580
4	0.6666	0.5334	0.4445	0.2664	0.6666	0.8666	1.0666	1.3333
3	0.6481	0.5185	0.4321	0.2590	0.6481	0.8426	1.0370	1.2962
2	0.6173	0.4939	0.4116	0.2467	0.6173	0.8024	0.9876	1.2345
1	0.5555	0.4445	0.3704	0.2220	0.5555	0.7222	0.8889	1.1111
$RK_{p2} = 1.3$					$S_2 = 0.6667$			
5	0.8827	0.7062	0.5885	0.3527	0.5432	0.7062	0.8692	1.0865
4	0.8666	0.6934	0.5778	0.3463	0.5334	0.6934	0.8534	1.0667
3	0.8426	0.6741	0.5618	0.3367	0.5185	0.6741	0.8297	1.0371
2	0.8024	0.6420	0.5350	0.3207	0.4939	0.6420	0.7902	0.9877
1	0.8827	0.7062	0.5885	0.3527	0.4445	0.5778	0.7111	0.8889
$RK_{p3.1} = 1.6$					$S_3 = 0.5556$			
5	1.0864	0.8692	0.7243	0.4341	0.4527	0.5885	0.7243	0.9054
4	1.0666	0.8534	0.7112	0.4262	0.4445	0.5778	0.7112	0.8890
3	1.0370	0.8297	0.6914	0.4144	0.4321	0.5618	0.6914	0.8643
2	0.9876	0.7902	0.6585	0.3947	0.4116	0.5350	0.6585	0.8231
1	0.8889	0.7111	0.5926	0.3552	0.3704	0.4815	0.5926	0.7408
$RK_{p4} = 2.0$					$S_4 = 0.3333$			
5	1.3580	1.0865	0.9054	0.5427	0.2713	0.3527	0.4341	0.5427
4	1.3333	1.0667	0.8890	0.5328	0.2664	0.3463	0.4262	0.5328
3	1.2962	1.0371	0.8643	0.5180	0.2590	0.3367	0.4144	0.5180
2	1.2345	0.9877	0.8231	0.4933	0.2467	0.3207	0.3947	0.4933
1	1.1111	0.8889	0.7408	0.4440	0.2220	0.2886	0.3552	0.4440
JUS31/81 base shear prevail JUS39/64								
Cases calculated below								

The proposed K-Quotient method is easily applicable for comparing ELF-based base shears. Fig. 6 is a tabular presentation of the procedure proposed by Eqs. 24 and 29.

$RK_b =$	RS_e	SZ	RK_S	RK_O	VII	VIII	IX	S	RS	K_p	RK_p	K_η	RK_η
	2/3	VII	1.0	RK_{01}	0.75	0.75	1.00	S_1	0.8333	K_{p1}	1.0	η_5	1.2222
		VIII	1.0	RK_{02}	1.00	1.00	1.00	S_2	0.6667	K_{p2}	1.3	η_4	1.2000
		IX	1.0	RK_{03}	0.75	1.50	1.50	S_3	0.5556	$K_{p3.1}$	1.6	η_3	1.1167
								S_4	0.3333	$K_{p3.2}$	1.0	η_2	1.1111
										K_{p4}	2.0	η_1	1.0000

(Eq. 25)

(Eq. 26)

(Table 13)

(Table 14)

(Table 15)

(Table 16)

Fig. 6 Brake down on RS_e , RK_S , RK_O , RS , RK_p and RK_η ratios constituting K transformation coefficient

For example, for Category II ($RK_{01} = 1.00$, Table 13), 5 story ($n = 5$, $\eta_5 = 1.2222$, Table 16), RC structure ($K_{p1} = 1.00$, Table 15), located in the seismic zone of VIII degree intensity

($RK_S = 1.00$, Eq. s, 23), sited on medium ground conditions ($S_2 = 0.6667$, Table 14), RK_b calculated by order as presented in Fig. 6, is:

$$RK_b = (2/3) \cdot 1.00 \cdot 1.00 \cdot 0.667 \cdot 1.00 \cdot 1.2222 = 0.5432, \text{ and } F_b^{81} = 0.5432 F_b^{64}$$

If the building is with stiffness discontinuity (e.g., flexible or soft story, $K_{P4} = 2.00$), the RK_b would be:

$$RK_b = (2/3) \cdot 1.00 \cdot 1.00 \cdot 0.667 \cdot 2.00 \cdot 1.2222 = 1.0865, \text{ and } F_b^{81} = 1.0865 F_b^{64}$$

For four story ($n=4$, $\eta_4 = 1.2000$) masonry building strengthened by vertical RC cerclages ($K_{P3.1} = 1.6$, Table 10) and all other coefficients as in the previous case, RK_b is

$$RK_b = (2/3) \cdot 1.00 \cdot 1.00 \cdot 0.667 \cdot 1.60 \cdot 1.2000 = 0.8534, \text{ and } F_b^{81} = 0.8534 F_b^{64}$$

Relative to JUS31/81, the JUS39/64 simplified method is over-conservative for stiff building. The largest conservatism is associated with lateral force coefficient associated to GT_3 and GT_4 ground conditions (0.5556 and 0.3333, respectively, Table 17), then to GT_2 (0.6667), and finally to GT_1 (0.8333).

4.1.2 Flexible Buildings

The RK_b ratio (or RK_b -quotient) is calculated for two selected flexible buildings located in I_{MCS} zone of IX degrees with $K_S = 0.10$. Both are reinforced concrete space moment-resisting frame (skeleton) buildings symmetric in both orthogonal horizontal directions ($K_P = 1$; $R_P = 4$). They are residential buildings with the ground floor (GF) hosting commercial uses, hence importance category II ($K_0 = 1$).

DTB (Tomic, 2018) building is 43 metres in height GF+ME+12 story tower with $T_x=T_y=1.848s$. JTB (Trajcevski, 2018) building is a GF+ME+4 story block type building 19.3 metres in height with $T_x=1.108s$ and $T_y=1.048s$. ME is the mezzanine story. The floor plan of both buildings is presented in Figs. 7 and .8. Both were designed and executed according to JUS39/64 at the beginning of the 1970s'. In addition, analysed is 10 storey 5 bay (in longitudinal) and 3 bay (in transverse) direction. The building (B10) is a dual RC frame/RC wall system with a shear wall placed in the middle span of two internal frames.

Fig. 7 JTB Building, characteristic floor plan

Fig. 8 DTB Building, characteristic floor plan

The analyses are conducted for all three ground types. The results are presented in Table 18. RK_b values are of the same order, irrespective of the fact that JUS39/64 are derived from 3 mode MRS and JUS31/81 by ELF method. Common to both JTB and DTB buildings is the inverse trending of ground type-dependent RKs relative to trends outlined by the simplified method. For GT_1 ground type, both RK_b s are ≤ 1 , hence JUS38/64 did prescribed the higher base shear than JUS31/81. The $RK_b \geq 1.0$ are attributed to GT_3 ground typology demonstrating that JUS31/81 overwhelms the JUS39/64 base shear.

Comparatively JUS39/64 and JUS31/81 base shears are about the same for GT_2 . However, the results for B10 building, although following the same pattern of GT dependency, are all above the $RK_b \geq 1.0$.

Table 18 Lateral force ratio (RK_b) for JTB and DTB buildings

# of stories	GT	W [t]	Longitudinal Direction LN (X)						Transverse Direction TR (Y)					
			T [s]	JUS39/64		JUS31/81		RK_b	T [s]	JUS39/64		JUS31/81		RK_b
				Fb [kN]	%W	Fb [kN]	%W			Fb [kN]	%W	Fb [kN]	%W	
JTB 6	1	1,802	1.108	92.37	5.13	81.29	4.51	0.880	1.048	95.66	5.31	85.97	4.77	0.899
	2			115.02	6.4	113.81	6.3	0.989		118.79	6.6	120.36	6.7	1.013
	3			138.02	7.7	146.33	8.1	1.060		142.55	7.9	154.76	8.6	1.086
DTB 14	1	6,920	1.848	238.20	3.44	230.67	3.33	0.968	Direction not analysed, Dual system (K_{P1}) symmetric like JTB building					
	2			297.75	4.3	322.93	4.7	1.085						
	3			357.30	5.2	415.20	6.0	1.162						
B10 10	1	2,880	1.069	124.16	4.3	134.70	4.7	1.085	Direction not analysed, Dual system (K_{P1}) symmetric like JTB building					
	2			155.20	5.4	188.58	6.5	1.215						
	3			186.24	6.5	242.45	8.4	1.302

Fig. 9 Spectral behaviour of lateral force ratio (RK_b) for stiff and flexible Buildings

The analyses are conducted for all three ground types. The results are presented in Table 18. RK values are of the same order, irrespective of the fact that JUS39/64 are derived from 3 mode MRS and JUS31/81 by ELF method. Common to both JTB and DTB buildings is the inverse trending of ground type-

dependent RK_b s relative to trends outlined by the simplified method. For GT_1 ground type, both RK_b s are ≤ 1 , hence JUS38/64 did prescribed the higher base shear than JUS31/81. The $RK_b \geq 1.0$ are attributed to GT_3 ground typology demonstrating that JUS31/81 overwhelms the JUS39/64 base shear. Comparatively JUS39/64 and JUS31/81 base shears are about the same for GT_2 . However, the results for B10 building, although following the same GT dependency patterns, are all above the $RK_b \geq 1.0$.

To further elucidate the behaviour of RK_b over the velocity and displacement spectral regions, a numerical triage has been performed on JTB, RTB, and B10 buildings. For all three buildings, formulated are a number of computational models by simply adding or cancelling out stories. The JTB building from six was upgraded up to 14, DTB from 14 to 20, and B10 to 15 stories by replicating the top story mass and stiffness characteristics of the basic 6, 14, and 10 story computational models. Downscaling of computational models is realized by simply removing stories down to the 3rd story. In such a way, formulated are 43 models in total, 18 for DTB building, 13 for B10, and two times 12 (for each direction separately) computational models that have been analysed for all three ground types. All computations, 165 in total, were conducted by in-house developed Matlab script CodeSDB. The Authors are fully aware that formulation of such computational models influence their Eigenvalue characteristics in general and the behaviour of K_η^{64} coefficient in particular.

Apart from indicated deficiency of models used, the analyses, Fig. 9

- Confirmed that in the period range 0.9 - 2.0s+ JUS31/81 overwhelms the JUS39/64 base shear for GT_3 ground type. Hence over the entire velocity and displacement spectral regions of GT_3 spectral shape the $RK_b > 1$ reaching the values of 1.3+ for B10 and 1.2 for the DTB building. The directional values of the JTB building are slightly lower, but both exceed 1.1;
- For GT_1 ground type, the trends are inverse. Over the entire $T_{C,(GT_2/GT_3)} - 2.0+$ seconds range JUS39/64 overwhelms the JUS81/81 base shear. For DTB building with the first period being in the displacement spectral region ($T_1=1.848$ s), RK_b converge to 1, whereas for JTB building ($T_{1x}/T_{1y}=1.108/1.048$ s) it converges to value close to 0.95s. The exception is the B10 building for which $RK_b > 1$ over the entire $T_{C,GT_2} - 2.0$ s period range;
- The order of RK_b relations over the $T_{C,(GT_2/GT_3)} - 2.0$ s+ period range is $RK_{b,GT_3} > RK_{b,GT_2} > RK_{b,GT_1}$;
- Over the $0.5-T_{C,(GT_2/GT_3)}$, the RK_b relations reorder themselves so that $RK_{b,GT_1} > RK_{b,GT_2} > RK_{b,GT_3}$;
- The simplified calculation of base shear for “stiff” buildings is over conservative relative to calculating them by MRS method; and,
- The sloping of RK_{b,GT_i} in post T_{Ci} velocity and over the entire displacement ($T_D \geq 1.5$ s) regions is influenced by $K_\eta^{81} = n$ coefficient. The effect is particularly pronounced in $T_D \geq 1.5$ s range where RK_b increases monotonically and almost linearly.

Table 19 Computed RK_{b,GT_i} quotients for JTB, DTB, and B10 buildings for closest period larger than 0.5s

Building	T [s]	$R_{P1} = 4$				$R_{P2} = \sim 3.1$	$R_{P3,(1/2)} = 2.5$
		GT_1	GT_2	GT_3	$min GT_i$		
JTB, x	0.640	0.847	0.867	0.723	0.723	0.940	1.157
JTB, y	0.580	0.858	0.796	0.664	0.664	0.863	1.062
DTB	0.514	0.918	0.755	0.629	0.629	0.818	1.006
B10	0.580	1.045	0.970	0.809	0.809	1.052	1.294

All discussed values are relevant only for K_{P1} structural typology ($K_{P1} = 1, R_{P1} = 4$, Table 10), which was and still is dominantly implemented RC typology in RNM and Yugoslavia since the 1964 year. The minimum calculated $K - Quotients$ for all ground types at periods slightly over 0.5s are presented in Table 19. $min GT_i$ values for K_{P1} structural typology ($K_{P1} = 1$) are all < 1 causing the full prevalence of JUS39/64 over JUS31/81 as well as for three cases of K_{P2} structural typology ($R_{P2} = \sim 3.1$). Irrespective

of the ground typology, for one K_{p2} case and all K_{p3} ($R_{p3} = 2.5$) and K_{p4} ($R_{p4} = 2$) the JUS31/81 will prevail JUS39/64 base shear.

As shown, the behaviour of RK_b is quite irregular over the analysed spectral range of 0.5-2.0s and does not provide a clear picture to decide on the seismic design level of both codes. RK_b is a method of analysis and ground type dependent. For GT_2 located buildings JUS31/81 consistently overwhelms the JUS39/64 base shear. For other ground types the RK_b behaviour is variable. For some of analysed buildings $RK_b > 1$ whereas for other $RK_b < 1$. The result of this irregularity is that RK_b cannot discriminate which code is superior to the other. Hence the irregular behaviour of $K - Quotient$ over the entire analysed spectral range does not allow its exclusive use as a metric for scaling JUS31/81 and JUS39/64 seismic design levels.

5 Challenges for Vulnerability Assessors and Exposure Modellers

Due to discussed irregular behaviour the $K_b - Quotient$ and resulting base shears are not suitable parameters for deciding mutual seismic design levels of JUS39/64 and JUS31/81 codes.

The novelty of JUS31/81 relative to JUS39/64 code was that it:

- prescribed the deformation control (Articles 16 and 41). Article 16 limits the maximum lateral elastic deflection of a building top f_{max} to $H/600$, where H is the height of the building. Article 41 limit the interstory drift d_i for linear behaviour of the structure to $d_i \leq h_i/350$, and to $d_i \leq h_i/150$ for nonlinear behaviour of the structure subjected to design level earthquake;
- for RC columns and walls prescribes verification of normalised design axial force v_d against the following limits: ≤ 0.35 (for columns, Article 61) and ≤ 0.20 (for shear walls, Article 73); and
- improved reinforcement detailing to assure better confinement in columns' top and bottom critical regions and prescribes 50% rebar lapping outside the zones of plastic hinges (critical regions) and in areas where the tensile stresses are the least (about the central part of the column).

On this ground, rather than on the value of lateral force coefficients or $K - Quotient$ behaviour, Milutinovic and Trendafiloski, 2003 adopted and assigned four design levels to seismic design codes historically used in Macedonia and Yugoslavia, Table 20.

Table 20 Design and construction eras based on enforcement time of JUS, SZM, and CRC standards

	Era	Enforcement span	JUS	SZM	CRC
	Pre-code	< 1948		n/a	
CDL	Low code	1948-1964	JUS61/48	SZM1948	CRC46/47
CDM	Moderate-code	1965-1982	JUS39/64	SZM1950	
CDH	High code	1983-2023	JUS31/81	SZM1982	CRC51/71
				SZM1990	CRC11/87
	Eurocode system	≥ 2020	MKS EN 1998-1	MKS EN 1998-1/NA	MKS EN 1992-1-1

The further complexity that vulnerability assessors and exposure modellers should be aware of is the non-simultaneous enforcement of seismic design codes and codes for Concrete and Reinforced Concrete (CRC). By rule, enforcement of CRCs' was delayed for 6 to 7 years, Table 20. The implication of such non harmonized enforcement timing is that the JUS39/64 and JUS31/81 are both associated with two CRC codes, hence with potentially two vulnerability levels dictated solely by their provisions.

6 Conclusions

The contribution presents Yugoslavian seismic resistant design requirements evolution, focusing on lateral force levels prescribed by two Yugoslavian seismic design codes, the JUS39/64 and the JUS31/81. In all details is presented a comparative analysis of five coefficients (K_0, K_p, K_s, S, K_η) and

two sets of 5%-damped elastic response spectral shapes (S_e). Each section is appropriately concluded, so a number of conclusions will not be repeated herein.

The most relevant findings are as summarized below:

- Eq. 11 is not suitable for analysing and comparing Europe wise the lateral force coefficients (K), although for comparing JUS39/64 and JUS31/81, it may be;
- The appropriate K comparison format is the R-code format (Eq. 15). In particular, since EN 1998-1 (Eurocode 8) uses it, all lateral force coefficients and its constituents of previous codes can efficiently be compared with it

$$K = \gamma_I \cdot a_{gR} \cdot S \cdot \frac{1}{q} \cdot S_e(T) \quad (30)$$

where γ_I is building importance factor, a_{gR} reference peak ground acceleration on type A ground, S soil factor, q behaviour factor and, $S_e(T)$ elastic horizontal ground acceleration response spectrum shape.

- $K - Quotient$ due to its irregular and ground-type-dependent behaviour over the entire spectral range does not allow its exclusive use as a metric for scaling JUS31/81 and JUS39/64 seismic design levels.
- Proposes and documents K-Quotient method that efficiently compares and intrinsically screens the formulation of all K constituents;
- Confirms that both JUS39/64 and JUS31/81 codes are ductility based codes with nominal ductility factor $D_L = 4$ that latter is modified with K_P coefficient. R-code R_P coefficient, equivalent to EN 1998-1 behaviour factor, take care of both, the D_L and K_P JUS's coefficients;
- Both codes allow alternate simplified methods of analysis for stiff buildings. Relative to JUS31/81, the JUS39/64 simplified method is over-conservative. Depending on the ground type, for K_{P1} structural typology it prescribes from 1.2 to 3 times greater base shear;
- K-Quotient behaviour of “flexible” buildings varies with ground typology. For GT_2 JUS31/81 base shear is consistently greater than of JUS39/64. For other ground conditions, K-Quotient varies in the range of 0.8 to 1.2. The discussed values are relevant only for K_{P1} structural types (Table 10). For other structural types, JUS31/81 and JUS39/64 base shear prevalence is alternative;
- The minimum calculated $K - Quotients$ for periods slightly over 0.5s (Table 19) and ground types, for K_{P1} structural typology ($K_{P1} = 1$) are all <1 causing the full prevalence of JUS39/64 over the JUS31/81 as well as for three cases of K_{P2} structural typology ($R_{P2} = \sim 3.1$). Irrespective of the ground typology, for one K_{P2} case and all K_{P3} ($R_{P3} = 2.5$) and K_{P4} ($R_{P4} = 2$) cases the JUS31/81 prevails JUS39/64 base shear.
- K_{P1} structural types are mostly implemented systems in RNM and Yugoslavia since the 1964 year, hence the irregularity in the behaviour of $K - Quotient$ over the 0.5-2.5s+ prevent its exclusive use as a metrics for scaling JUS31/81 and JUS39/64 seismic design levels; and,
- The development of $K - Quotient$ values is based on actual data from only three buildings and related numerical simulations. The influence of CRC codes on vulnerability and resistance of the exposure is practically unknown. Further research along these two lines is indispensable.

Over the last decade, the SERA Consortium made significant efforts to collect, classify, and analyse building typology across Europe to establish the 2020 exposure model that contains relevant data on approximately 145 million residential, commercial, and industrial buildings in Europe. Recent efforts Crowley et al., 2021, have focused on the spatial and temporal evolution of seismic design across Europe to better understand and classify the seismic design lateral force levels for the existing RC European building stock. Further development of such continent wise model, Russia excluded, is a significant effort exclusively dependent on qualified but also on semi qualified national inputs. Maintenance of

such a model is a dynamic issue that requires a permanent update of all information controlling the national seismic design lateral force levels. We hope that this contribution provides sufficiently detailed, compact, and valuable technical information on temporal evolution on seismic design models used in Macedonia and Yugoslavia from 1948 until they were replaced by Eurocode Design System EN 1990 – EN 1999.

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Appendix

This appendix provides the full details for the design codes referred to in this contribution.

- JUS61/48: *Provisional Technical Regulations for Building Loading* (Official Gazette of FNRJ No. 61/48, in Serbo-Croatian)
- SN-8-57: СН-8-57 „Нормы и правила строительства в сейсмических районах (*The Norms and Rules for Construction in Seismic Regions*)“. ГОССТРОЙ СССР, Москва 1958 (in Russian).
- SNiP II-A.II 12.62: СНиП II-A.II 12.62 „Строительные нормы и правила, Часть II, раздел А: Нормы проектирования, Глава 12: Строительство в сейсмических районах (*The Construction Norms and Rules, Part II, Section A: Design Norms, Chapter 12: Construction in Seismic regions*)“. ГОССТРОЙ СССР, Москва 1963 (in Russian).
- SI18/63: *Regulations for Design and Construction of Structures in Seismic Regions* (Official Gazette of SRS No. 18/63 of June 13, 1963, in Slovenian)
- GS18/63: “Dimenzioniranje Gradbenih objektov v potresnih območjih (*Dimensioning of buildings in seismic areas*)” Slovenian association of structural engineers and technicians, Ljubljana, 1963. (in Slovenian)
- MK33/63: *Regulations for Design and Construction of Structures in Seismic Regions* (Official Gazette of SRM No. 33/63 of 24 September 1963, in Macedonian)
- JUS39/64: *Provisional Technical Regulations for Construction in Seismic Regions* (Official Gazette of SFRY No. 39/64 of 30 September 1964, in Serbo-Croatian).
- EB73: *Explanatory Booklet for Map of Seismic Regionalization of the Territory of SR Serbia* (scale 1:500 000), Seismological Institute, Belgrade, Serbia, 1973
- Technical Regulations for Building Construction in Seismic Regions (Official Gazette of SFRJ No. 31/81 of 5 June 1981, including amendments: No. 49/82 of 13 August 1982, No. 29/83 of 10 June 1983, No. 21/88 of 1 April 1988, and No. 52/90 of 7 September 1990, in Macedonian)
- JUS52/85: *Technical Regulations for Repair, Strengthening and Reconstruction of Building Damaged by Earthquakes, and for Reconstruction and Revitalization of Buildings*; (Official Gazette of SFRY No. 52 of 4 October 1985, in Serbo-Croatian)

790 JUSxx/86: *Regulations on Technical Norms for Design and Calculation of Civil Engineering*
 791 *Facilities in Seismic Regions*; (1986, Draft, in Serbo-Croatian)
 792 *The rulebook for Amending the Rulebook for Design Standards and Norms for Design* (Official Gazette
 793 of RNM No. 211 of 02.09.2020, in Macedonian)
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