Genealogy of development and codification of Yugoslavian earthquake resistant design

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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 Yugoslav) 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 ≤ 0.5 s). 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

1 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. 2021a, b) 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 the 2021 census, intends to develop a high resolution national exposure model, if the 2021 census data are rendered publicly available.

Works along this line are already ongoing on the European scale. The European Commission’s Horizon 2020 SERA project is the first concerted effort among the European 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; 2021a, b). In support of Crowley et al. (2020b), Crowley et al. (2021a, b) present the “Model of seismic design lateral force levels for the existing reinforced concrete European building stock,” Republic of North Macedonia (RNM) included. All relevant and up-to-date exposure, vulnerability, and risk resources are available at the European seismic risk services website: http://risk.efehr.org.

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 IAEE1 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 32nd 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.

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 only in Zamolo (2021, in Croatian). However, the subject is not treated in such technical details as this paper does.

For the benefit of readers of this manuscript, in the beginning, we define and explain certain abbreviations used consistently in the paper. FNRY (the Federal People’s Republic of Yugoslavia, 1945–1963), SFRY (the Socialist Federal Republic of Yugoslavia, 1963–1992). Internally, the Yugoslav federation was divided into six constituent states (Federal units, or Republics), now sovereign countries, by alphabetic order: BA (Bosnia and Herzegovina), HR (Croatia), Macedonia (MK, presently the Republic of North Macedonia in this paper referred as RNM), MN (Montenegro), SI (Slovenia) and RS (Serbia). Country abbreviations comply with the ISO alpha-2 country coding system. JUSxx/yy is an abbreviation for Yugoslavian standard (JUS), < xx > is a number of Official Gazette, and < yy > are the last two digits of the year of publishing. OGoXXXX stands for Official Gazette of XXXX = FNRY/SFRY. The Republic standards are referred to as ISOalpha-2xx/yy with < xx/yy >.

We also strongly note that the discussions involving RNM are valid for the entire territory of FNRY/SFRY and vice versa, except for discussions related specifically to some Republic standards, like SI18/63 (Slovenia) and MK33/63 (Macedonia).

1 IAEE: International Association for Earthquake Engineering https://www.iaee.or.jp/.
Table 1  Evolution of FNRY/SFRY and subsequently RNM technical regulations for earthquake resistant design

| Regulation                                                                 | Abbreviated as | Level                        |
|---------------------------------------------------------------------------|----------------|------------------------------|
| *Provisional Technical Regulations for Building Loading* (Official Gazette of FNRY No. 61/48) | JUS61/48 (PTP2) | National FNRY               |
| *Regulations for Design and Construction of Structures in Seismic Regions* (Official Gazette of SRS No. 18/63 of June 13, 1963) | SI18/63        | Republic, Slovenia           |
| *Technical Regulations for Construction in Seismic Regions* (Official Gazette of SFRY No. 39/64 of 30 September 1964) | JUS39/64       | National SFRY               |
| *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) | JUS31/81       | National SFRY               |
| *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) | JUS52/85       | National SFRY               |
| *Regulations on Technical Norms for Design and Calculation of Civil Engineering Facilities in Seismic Regions*; (1986, Draft) | JUSxx/86       | National SFRY               |
| Eurocode Design System MKC EN 1990 – MKC EN 1999 (OGoRNM No. 211 of 02.09.2020) *Eurocode 8: Design of structures for earthquake resistance—Part 1: General rules, seismic actions and rules for buildings* | MKC EN 1998–1 | National RNM               |

Eurocode Design System MKC EN 1990 – MKC EN 1999 (OGoRNM No. 211 of 02.09.2020) *Eurocode 8: Design of structures for earthquake resistance—Part 1: General rules, seismic actions and rules for buildings*
The lateral force coefficient $K$, Eq. (1), follows Crowley et al. (2021a, b) formulation that coincides with the JUS31/81 definition:

$$K = K_O \cdot K_S \cdot K_P \cdot K_D$$  \hspace{1cm} (1)

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

Hereinafter, the coefficients $K$, $K_O$, $K_S$, $K_P$, $K_D$ and other defined subsequently in the text will be presented by a two-digit superscript to denote the year of the code.

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

2 Development of equivalent static lateral force method

2.1 Worldwide beginning

Workable equivalent static lateral force design methods (the “seismic coefficient” or “seismic ratio” methods) were developed independently in Italy and Japan following the 1908 Reggio-Messina, Italy and 1906 California, United States earthquakes, respectively. Japanese method is based on the work of Eng. Toshikata Sano, who visited San Francisco after the 1906 San Francisco Earthquake, and in 1914 submitted his doctoral thesis entitled "Earthquake Resistance of Buildings," in which he proposed the use of equivalent static horizontal load in the seismic design of buildings.
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 shindo coefficient as $k = a/g$. The term $a$ is the estimated acceleration of the ground motion to be used in the design, that when divided by gravitational acceleration, $g$, $g = 9.81 \text{ m/s}^2$, conveniently results in a dimensionless coefficient $K$, which applied to a building weight ($W$) give a value in terms of units of force, Eq. (2).

$$F = K \cdot W = \frac{a}{g} W$$  

(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 UBC\textsuperscript{2}, the lateral force system structural factors ($K_p$) and coefficient of dynamic response ($K_d$) are included starting by the 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 the 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

2.2.1.1 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 on the 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 of “uniform horizontal load of 1/50 = 0.02% 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.

### 2.2.1.2 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 unreinforced masonry structures (URM) that were the predominant construction typology at the time. The second section allocates the coefficient of seismic intensity $K_S$ to three mapped “seismological” zones, Table 3.

The product of $(3 \times 1) K^{48}_P$ row vector and $(1 \times 3) K^{48}_S$ column vector is $(3 \times 3) K^{48}$ seismic coefficient matrix presented in Table 4.

For masonry, Table 4, the $K^{48}$ coefficients range from 1.0%W to 3.0%W, and depending on the type of the masonry and the seismic zone, they are 5 to 20 times lower than $K^{81}$.

The $k^{th}$ story seismic action $S_k$ is a product between the $K^{48}$ and the total vertical load at and above the story $<k>$, $W_k$, i.e.:

$$S_k = K^{48}_P W_k = K^{48}_P \cdot K^{48}_S \cdot W_k$$

When JUS61/48 was introduced in 1948, the use of RC building typology was exceptionally rare, almost non-existent. Therefore it was not recognized in Tables 2 and 4; hence neither $K^{48}_P$ nor $K^{48}$ coefficients were defined for this building typology.

According to JUS61/48 Article 2332, structures in Skopje (wind zone II, Fig. 10) had to be designed to resist a wind forces corresponding to characteristic wind pressure $w = 75$ to...
| Structural type | Seismic zone | VII | VIII | ≥ IX |
|-----------------|--------------|-----|------|------|
| Buildings with massive walls and massive floors (roofs) | 1.0 | 1.5 | 2.0 |
| Buildings with massive walls and light floors (roofs) | 1.2 | 1.8 | 2.4 |
| Buildings with light walls and light floors | 1.5 | 2.25 | 3.0 |
| JUS31/81 Reinforced masonry ($K_p^v = 1.3$) | 3.25 | 6.5 | 13.0 |
| Masonry strengthened by vertical RC cerclages ($K_p^m = 1.6$) | 4.0 | 8.0 | 16.0 |
| Ordinary* masonry structures ($K_p^w = 2.0$) | 5.0 | 10.0 | 20.0 |

*The term “ordinary” is undefined; most probably, it refers to classic URM*
105 kN/m², Table 22. Depending on the height of the building and the type of its exposure, the characteristic wind pressure is augmented by a factor of 1.2 (pressure of 0.8w on the windward side and suction of 0.4w on the leeward side). The wind code did not prescribe the height of buildings above which the wind design should be applied.

National and international studies on the damaging effects of the Skopje earthquake documented that all ≥ GF+6 buildings were designed to resist wind loading since all of them possessed vertical elements (shear walls, stairwell walls, elevator shafts, or combined) to assist frame members in the lateral force resistance. The studies also revealed the fact that Macedonian design and construction of post-WWII masonry and RC frame buildings generally ignored the seismic code provisions (Muto et al. 1963, Sozen 1964, Berg 1964 and 1965, Kapsarov 1972), as well as that three GF+13 RC frame towers in Karpos II settlement, were the only structures in Skopje for which the seismic provisions of the JUS61/48 code were considered in the design.

In summary:

- Unreinforced masonry proved totally inadequate to resist the lateral forces imposed upon them by the earthquake. Several such buildings instantly collapsed, and most of them were so severely damaged as to make repair economically unfeasible if not impossible. The highest mortality and injury are associated with this building type. Nearly half of those who died (more than 460 of 1,070 officially declared, FEC1963/1963) did so in five large buildings which collapsed immediately—one of these being the Hotel Macedonia (70 bodies retrieved) which was widely publicized because it accommodated foreigners. The greatest fatality toll (195 persons) for one building is associated with the half-building collapse of a modern apartment block, Block 13, in the Karpos II settlement.
- Low rise, ≤ GF + 5 RC frame structures, designed without regard to lateral forces, suffered severe damage. The most dramatic of these was the collapsed main exhibition hall at the fairgrounds. Due to the time of the earthquake (05:17), no fatalities were associated with this collapse, neither with any building of this typology; and,
- Midrise, ≥ GF + 6 up to GF + 13, RC frame structures designed to resist lateral wind forces behaved unexpectedly well.

Most of the buildings in Skopje had no lateral resistance to earthquake forces which was obvious from the type of damage they suffered. Hence, it is not surprising that the 1963 earthquake cost Skopje, Macedonia, and SFRY 1,402 Million 1963 US$ or 82.45% of its administrative value evaluated at 1,157 Million 1963 US$, FEC1963 (1963). Presented figures are calculated with the rate of 1 1963 US$ = 300 old Yu Dinars.

What is surprising is that many RC buildings, which had been designed without consideration of lateral forces, survived the earthquake with minor damage. The numerous examples of slightly damaged buildings in Skopje are positive evidence that structures of reinforced concrete in Skopje were able to withstand strong earthquake forces, even though they were not designed specifically to withstand such forces, Ambraseys, 1968. However, the Skopje earthquake is not an exception. Structures designed for 0.1 g or less performed successfully in May 18, 1940, El-Centro earthquake with maximum ground acceleration PGA = 0.32 g and maximum ground velocity PGV = 35.56 cm/s, Milutinovic (1986).

The above discussion, including the entire discussion in Appendix B, is relevant for Skopje and Macedonia since the characteristics of RC frame building typology in other Federal units of FNRY could be modified by local interpretation and implementation of JUS61/48 seismic and wind provisions.
A brief discussion on the structural characteristics and damage patterns of pre-1963 Skopje’s building stock is presented in Appendix B. Moreover, for seismicity zone $I_{MC5} = IX$ (Skopje) and wind zone II (Fig. 10, Appendix B) calculate and compare JUS61/48 wind base shear with seismic base shears demanded by JUS39/64 and JUS31/81 seismic design codes.

Comparatively, for flexible buildings presented in Sect. 4.1.2, the ratios between JUS39/64 and JUS31/81 $I_{MC5} = IX$ seismic base shears relative to JUS61/48 wind base shear are 2.40 to 3.62 and 2.63 to 3.8, respectively. For seismic zone $I_{MC5} = VIII$, the presented values should be halved, and for seismic zone $I_{MC5} = VII$ be quartered and vary from 0.6–0.90 and 0.66–0.95, respectively. In other words, in the $I_{MC5} = VII$ zone, wind base shear would exceed the seismic base shear.

The seismic resistant design in FNRY/SFRY commenced administratively by enforcement of JUS61/48, with which design and construction practice was implemented for 16 years. Although somewhat primitive in its seismic provisions, it is essential to emphasize that JUS61/48 recognized the seismicity of the country, specified minimum seismic design coefficients, and requested seismic action, as a special load, be combined with gravitational loads (all dead $q/d$ and $1/2$ of probable live $p/l$) for defining the design value of the action effects. Unfortunately, as already discussed, the design of low rise ($\leq GF + 5$) buildings in general, and masonry in particular, ignored JUS61/48 seismic provisions. Had the JUS61/48 building code been followed, the 1963 Skopje earthquake might have been much less of a disaster.

2.3 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 inadequate 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. The government of NR Slovenia 1963 officially adopted Regulations for Design and Construction of Structures in Seismic Regions SI18/63 (Official Gazette of SRS 18/63 No. 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.

2.3.1 Post 1963 Skopje earthquake

History demonstrated (Reitherman 2000, 2006) that the impetus for 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 killed 1,070 and injured 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 1978.

In response to the 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 practices, 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 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, and this will continue for a total of 3 years (OGoRNM No. 20//211 of 2.09.2020).

2.4 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 $S_{ik}^{63}$ mode seismic action at $k$th story.
where $K_{63}^{i}$ is the modal seismic coefficient, $K_{DS}^{63}$ = IMSC and ground type (GT) dependent coefficient of seismic intensity, $K_{Di}^{63}$ = dynamic coefficient (originally denoted by $\beta$), $w_k$ = < $k$th > story tributary weight, and $K_{/u1D702}^{63}$-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 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 the site-specific ground conditions, designed by $K_{DS}^{63}$ assigned to I_MCS = VII for good ground conditions (Article 2), i.e., value 0.015 from Table 5;
- Recognition of the importance class of the structures (coefficient $K_O$) 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 $K_{DS}^{63}$ coefficient of seismic intensity for good, medium, and weak ground types (GT) by introducing $S_{63}^{GT}$ soil factor, Table 5;
- Introduces the concept of elastic design spectra $K_D$ by defining it as a product of $S_{63}^{GT}$ soil factor and the 5% damped elastic spectral shape $S_{/e1}^{63}$, where $0.5 \leq S_{/e1}^{63} \leq 1.5$ (5)

$$S_{/e1}^{63} = K_i^{63} \cdot w_k = K_{DS}^{63} \cdot K_{Di}^{63} \cdot K_{/u1D702}^{63} \cdot w_k$$ (4)

$$S_{/e}^{63} = 0.75 / T_i \leq 1.5$$ (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*: Simplified—requesting that if the period of the structure is not calculated according to empirical formulas, it should be set to $T = 0.5$ s and coefficient $K_{/u1D702}^{63} = 1.0$ used;
  - *more accurate*: MRS—requiring a maximum of three modes but does not provide mode combination criteria.

### Table 5 SI18/63 and MK33/63 design seismic coefficients $K_s^{63}$ and $S_{63}^{GT}$ soil factors

| Ground type (GT) | Description                                                                 | I_MCS Seismic zone $S_{63}^{GT}$ | Diff (%) |
|-----------------|------------------------------------------------------------------------------|----------------------------------|----------|
| 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             | ±0       |
| Weak PGA $GT_3$ | Heterogeneous deposits; soft marls and clays                                | 0.025 0.05 0.10 1.25 0.8 0.16 0.32 | +25      |

Diff Percent difference of soil factors relative to $S_{/e1}^{GT_2}$ $[Diff = 100 \cdot (S_{/e1}^{GT_1} - S_{/e1}^{GT_2}) / S_{/e1}^{GT_2}]$
However, the distinct feature of SI18/63 and MK33/63 is the definition of the coefficient of seismic intensity, $K_{DS}$, as a function of mapped seismicity ($IMAP$) and the ground type dependent soil coefficient, $S$; i.e., $K_{DS} = f(IMAP, S)$. Hence, the influence of local soil conditions upon amplitude and frequency content of regional motions is implicitly considered in coefficients defined in Table 5.

The importance class $KO$, and the energy dissipation $K_{63}$ coefficients, while explicitly prescribed by certain SI18/63 and MK33/63 provisions, do not appear in Eq. (4). Similarly, $S_{63}$ soil factor is assimilated by $K_{DS}^{63}$ seismic intensity coefficient defined as tabulated relation between the mapped seismic intensity $I_{MSC}$ and the ground typology dependent soil factor ($S_{63}$), i.e. $K_{DS}^{63} = f(I_{MSC}^{63}, S_{63})$ Table 5.

Equation 4, rewritten in expanded form to include discussed coefficients, is:

$$S_{ik}^{63} = K_i^{63} \cdot w_k = K_{O}^{63} \cdot K_P^{63} \cdot K_S^{63} \cdot S_{ei}^{63} \cdot K_{n}^{63} \cdot w_k$$ (6)

$K_S^{63}$ seismic coefficient is a result of disaggregation of $K_{DS}^{63}$ into $K_{O}^{63}$ and $S_{63}^{63}$, Table 5. More details on the disaggregation process of $K_{DS}^{63}$ are provided in Sect. 3.6.2.

SI18/63, its translation in Macedonian MK33/63 and JUS39/64 are derivatives of SN-8–57 and SNiP II-A.II 12.62 USSR codes. Their concepts and structure are practically identical, hence, the entire discussion related to JUS39/64 presented in Sect. 3 is relevant to these two codes also with the following two exceptions:

- definition of the seismic weight – segment formulation of the imposed loads. This difference is presented later, in Sect. 3.2 and Table 6; and

| 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 based | ELF, RSM (3modes) | $q + 0.5p^{(3)}$ |
| JUS39/64 | Working stress | Ductility based | ELF, RSM (3modes), RTH allowed | $q + 0.5p^{(4)} + s^{(5)} + e^{(6)} + c^{(7)}$ |
| JUS31/81 | ULS* allowed  | Ductility based | ELF, RTH           | $q + 0.5p^{(8)} + s + e$ |

*ULS, Ultimate limit 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 at the 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
JUS39/64 (Table 11) increases the SI18/63 and MK33/63 $K_S^{63}$ values, Table 5, for 33.3%, 25%, and 20% for ground types GT1, GT2 and GT3, respectively.

PGA$^{63}$, the last row of Table 5, is calculated and presented according to the elaborations presented in Sects. 3.3 and 3.6.1 for parameters jointly prescribed by SI18/63 and MK33/63 codes.

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.

3 Detail comparative analysis of JUS39/64 and JUS31/81 seismic coefficients

JUS39/64 and JUS31/81 controlled seismic resistant design in FNRY/SFRY last 57 years. JUS39/64 sixteen and JUS31/81 forty-one years. According to these two codes, most of the construction was designed and constructed in SFRY/RNM. JUS39/64 as a code, although termed provisional, was valid for all structures (buildings, civil engineering, /bridges/, and infrastructure /water supply and sewerage/ systems). JUS31/81 is limited only to building and building-like structures.

JUS39/64 and JUS31/81 codes are conceptually different. The first is based on USSR’s SN-8–57 and SNiP II-A.II 12.62 Codes, while the second is on USA’s UBC76 and UBC79 Codes. The other distinct feature of JUS31/81 over the JUS39/64 is that specification of some constituents of the lateral force (or seismicity) coefficient K has evolved from an implicit to a more explicit form.

Both codes define the seismic action similarly but also differently. JUS39/64 (Eq. 7) defines the modal seismic action $S_{ik}^{64}$ as a product of three coefficients and tributary story weights $<w>$. JUS31/81 (Eq. 8) defines the seismic base shear $F_b^{81}$ as a product of four coefficients and the structure’s total weight $<W>$.

Abundant literature on the topic demonstrates that the lateral force coefficient (K), in its most general form, is a product of six independent, physically sustainable coefficients that aggregate the: (1) influence the importance class of the building ($K_O$); (2) energy dissipation ($K_P$); (3) site seismic intensity ($K_S$); (4) site ground type ($S$); (5) 5%-damped elastic response spectral shape $S_c$; and (6) a geometrical form of a building’s linear standing waves (herein termed as modal shape coefficient and denoted by $K_u$).

JUS39/64 reference method for determining the seismic action effects is the modal response spectrum analysis (MRS) based on a linear-elastic model of the structure and the design spectrum. This type of analysis shall be applied to $H \geq 20$ m buildings (nominated
herein as “flexible buildings”). For $H < 20$ m (nominated herein as “stiff buildings”), JUS39/64 and JUS31/81 allow alternate simplified methods.

For flexible structures JUS39/64 the original definition (OGoSFRY No. 39/64 of 30 September 1964) of $<i$th$>$ mode seismic action at $<k$th$>$ story, $S_{64}^{ik}$, is:

$$S_{64}^{ik} = K_S \cdot \beta_i \cdot \eta_{ik} \cdot w_k = K_{64}^{DS} \cdot K_{Di}^{64} \cdot K_{\eta_{ik}}^{64} \cdot w_k$$

(7)

JUS31/81 reference method is the equivalent lateral force (ELF) analysis method. For determining the base shear force $F_b$ the original definition (OGoSFRY No. 31/81 of 5 June 1981) is:

$$F_{81}^b = K \cdot W = K_O \cdot K_S \cdot K_P \cdot K_D \cdot W = K_{81}^O \cdot K_{81}^S \cdot K_{81}^P \cdot K_{81}^D \cdot W$$

(8)

The major problem in comparing JUS39/64 and JUS31/81 codes is the structure of the JUS39/64 code, Eq. (7). It defines the lateral force coefficient as a product between two independent and physically sustainable coefficients ($K_{Di}^{64}$ and $K_{\eta_{ik}}^{64}$) and a complex $K_{DS}^{64}$ coefficient termed design seismic coefficient. The $K_{DS}^{64}$ is a function of all other coefficients, i.e., $K_{DS} = f(K_O, K_S, K_P, S)$.

JUS39/64 design procedure is simple and straightforward. For a given building category (BC) and mapped seismicity (IMSC), the designer defines the design seismicity (IDSG) from Table 8. Then for defined IDSG and ground type, reads in $K_{DS}$ from Table 11 and combine it in modal $K_{DS} \cdot K_{Di}^{64} \cdot K_{\eta_{ik}}^{64}$ lateral force coefficient.

The distinct feature of JUS39/64 is an implicit definition of the following coefficients:

- under the term “design seismicity, IDSG” establishes a relation between the mapped seismicity (IMAP) and the importance class of the building (K_O), i.e., $I_{DSG} = f(I_{MAP}, K_O)$; and,
- links $I_{DSG}$ with the ground type-dependent soil coefficient (S) to formulate the design seismic intensity coefficient (K_DS), i.e., $K_{DS} = f(I_{DSG}, S)$.

JUS31/81, templated largely by UBC 1976 and 1979, is even more pragmatic. It defines each coefficient individually by Eq. (8) and determines the lateral force coefficient as a product between $K_O, K_S, K_P and K_D$.

To assure direct and effective comparison of constituents of lateral force coefficients of both codes, it is necessary to disaggregate all implicit relations into independent physically sustainable ingredients, i.e.:

$$K = K_O \cdot K_P \cdot K_S \cdot S \cdot S_e \cdot K_{\eta}$$

(9)

The subsequent sections, in all detail, present and discuss the disaggregation of coefficients of Eqs. (7) and (8) into their physically backed constituents (Eq. 9) and compare the values of prescribed JUS39/64 and JUS31/81 disaggregates.

### 3.1 Definition of base shear

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

$$S_{64}^{ik} = K_i^{64} \cdot w_k = K_O^{64} \cdot K_P^{64} \cdot K_S^{64} \cdot S_e^{64} \cdot S_{64}^{ik} \cdot w_k$$

(10)
where

\[ K_{64}^{11} = x_{ik} \left( \sum_{j=1}^{n} w_j x_{ij} \right) / \left( \sum_{j=1}^{n} w_j x_{ij}^2 \right) \]  

(11)

and combine the modal responses by

\[ E_{64}^{k} = \sqrt{0.5 \left[ \left( E_{64,1}^{k, \text{max}} \right)^2 + \sum_{i=1}^{3} E_{64}^{i} \right]} \]  

(12)

for defining the seismic action effect under consideration (force, displacement, etc.). \( x_{ik}, x_{ij} \) of Eq. (11) 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

\[ K_{64}^{k} = h_k \left( \sum_{j=1}^{n} w_j h_{ij} \right) / \left( \sum_{j=1}^{n} w_j h_{ij}^2 \right) \]  

(13)

Irrespective of the method used, the base shear is a sum of story shears \( F_k \)

\[ F_{64}^{b} = \begin{cases} \sum_{k=1}^{n} F_{64}^{k} & \text{For stiff (H < 20m) structures} \\ \sum_{k=1}^{n} F_{64}^{b_k} & \text{For flexible structures (H ≥ 20m) structures} \end{cases} \]  

(14)

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

\[ F_{81}^{b} = K_{81} \cdot W = K_{0}^{81} \cdot K_{p}^{81} \cdot K_{s}^{81} \cdot S_{e}^{81} \cdot S_{T}^{81} \cdot K_{n}^{81} \cdot W \]  

(15)

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

\[ S_{k}^{81} = \left( F_{81}^{b} - S_{T} \right) \frac{w_j h_j}{\sum_{j=1}^{n} w_j h_j} \begin{cases} N < 5 & S_{T} = 0 \\ N \geq 5 & S_{T} = 0.15 F_{81}^{b} \end{cases} \]  

(16)

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., warehouses, silos, libraries, archives, and similar), then the seismic loads are determined for the most unfavourable case of maximum or minimum actual loading. JUS U.C7.121 is identical to 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’s seismic weight is presented in Table 6.

Besides the genesis of seismic weights, Table 6 clearly and comparatively presents some other general features (design concept, code type, and methods of analysis) of all JUSs, irrespective of the fact that this section focuses only on JUS39/64 and JUS31/81.

### 3.3 The seismic intensity coefficient \( K_s^{64}, K_s^{81} \)

JUS39/64 quantifies the design seismic intensity coefficient, \( K_{SD}^{64} \), in a tabular form, Table 11. For a given “design seismicity, \( I_{DSG} \)”, and the ground type (GT). \( K_{DS}^{64} \) table prescribes the single design value of the coefficient of seismic intensity. The \( I_{DSG} \) dependent values corresponding to medium (GT2) ground type are equal to JUS31/81 \( I_{MCS} \) dependent \( K_s^{81} \) values. As such, in the following, it is treated as JUS39/64’s nominal coefficient of seismic intensity \( K_s^{64} \). JUS31/81 straightforwardly prescribes the nominal \( I_{MSC} \) dependent coefficient of seismic intensity \( K_s^{81} \). In summary, both codes prescribe the same nominal seismic intensity coefficients \( K_s \) for mapped (\( I_{MCS} \)) or design \( I_{DSG} \) seismicity of \( I_{MCS/DSG} = VII, VIII \) and IX degrees, Table 7.

\( K_s \) is a dimensionless coefficient that, combined in a product with other \( K \) constituents, defines the seismic action in the \( <w, W> \) unit system.

The most important intrinsic property of \( K_s \) is that in JUS and SNiP codes, it is a quotient between the two physical quantities, the peak ground acceleration (PGA) and the energy dissipation capacity \( (D_L) \), i.e.:

\[
K_s = \frac{1}{D_L} \frac{PGA}{g}
\]  

For nominal values of \( K_s \) (0.025, 0.05, and 0.1) and Importance class-II building typology \( (D_L = 4) \) the calculated PGA values are 0.1, 0.2 and 0.4 g, \( (g = 9.81 \text{ m/s}^2) \). Table 7. It is evident that the \( K_s \) value doubles for an increase in seismic intensity for one grade. Corresponding PGA values pattern themselves in the same way. Accordingly, from 1964 onwards, the nominal \( K_s - \text{PGA} \) relation has not changed, thus, so far, it is constant. More details are presented and discussed in Sect. 3.5.

| Seismicity zone | \( I_{MCS}, I_{DSG} \) | \( K_s^{64} \) | \( K_s^{81} \) |
|----------------|--------------------------|-------------|-------------|
| VII            | 0.025                    | 0.05        | 0.1         |
| VIII           | 0.1                      | 0.2         | 0.4         |

Table 7 \( K_s^{64}, K_s^{81} - \text{PGA relation} \) from 1963 onwards
3.4 Building importance factor $K_{O}^{64}, K_{O}^{81}$

JUS39/81 does not explicitly define the Importance factor $K_{O}^{64}$.

However, it defines and relates the so-called design seismicity, $I_{DSG}$, to the site-specific (or mapped) seismicity, $I_{MCS}$, and the building category (BC). This relation $I_{DSG} = f(I_{MAP}, BC)$, ending to $I_{DSG} = f(I_{MAP}, K_{O})$, (Table 8, or JUS39/64, Article 2.1, Table 1) is a specific pattern of early SNiPs (USSR norms for construction in seismic regions) overtaken by SI18/63, MK33/63, and JUS39/64 codes. The presented format SNiP maintained, including SNiP II-7-1981 edition, when it was abandoned with the transition from $K_{S}$ to PGA (SNiP II-7-1981*) format.

For importance class II structures $I_{DSG} = I_{MCS}$, Table 8. $I_{DSG}$ is generally increased for one intensity grade for importance class I and decreased for one intensity grade for importance class III buildings.

The exceptions are JUS39/64, Article 2.1, Table 1 (or Table 8 in this manuscript), for which JUS39/64 prescribes:

- Importance category I, $I_{MCS} = IX$, $I_{DSG} = IX$, +; seismic action computed for $I_{MCS} = IX$ shall be increased by a $K_{O}$ factor of 1.5; and
- Importance category III, $I_{MCS} = VII$, $I_{DSG} = VII$: $I_{DSG} = I_{MCS} = VII$.

Table 8 presents the mutual links between the following key parameters:

- Building category (BC) and prescribed $I_{DSG}$ seismicity for given $I_{MCS}$ seismicity;
- $I_{DSG}$ compatible $K_{S}^{64}$ values presented in standard round parentheses; and
- Derived $K_{O}^{64}$ coefficients presented in curly parentheses.

For each prescribed $I_{DSG}$, the values of $K_{O}^{64}$ coefficients (Tables 8 and 9) are calculated as quotients between the values of seismic intensity coefficients (Table 7) that would correspond to the prescribed design seismicity $I_{DSG} (K_{S}^{64}, I_{DSG})$ and the values of seismic intensity coefficients that correspond to the mapped seismicity $K_{S}^{64}, I_{MCS}$, as presented in Eq. (18).

Worked example $K_{O}^{64}$ for BC1 category structure located in $I_{MCS}^{64}$=VIII zone is presented in blue in Table 8.

$$K_{O}^{64} = \frac{K_{S}^{64}, I_{DSG}}{K_{S}^{64}, I_{MCS}}$$

(18)

$K_{O}^{64}, K_{O}^{81}$ 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 $K_{O}^{81}$ is a function of a building category only.

Referring to the presented values, $K_{O}^{64}$ relative to $K_{O}^{81}$ for 33.3% stipulated the value of importance coefficient for Category-I buildings located in $I_{MCS}$ zones of VII and VIII degrees. Also, it is 33.3% lower for Category-III buildings in $I_{MCS}$ zones of VIII and IX degrees.
| Building category (BC) | Type of building or structure                                                                                                                                                                                                 | Mapped seismicity (I_{MCS}) | Design seismicity (I_{DIC}) |
|----------------------|--------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|-----------------------------|-----------------------------|
|                      |                                                                                                                                                                                                                          | VII                         | VIII                        | IX                          |
|                      |                                                                                                                                                                                                                          | (0.025)                     | (0.025)                     | (0.05)                      |
|                      |                                                                                                                                                                                                                          |                            |                             | (0.5)                       |
| I BC_{I}             | Buildings that contain rooms for assembly use (schools, cinemas, theatres, gymnasia, etc.); buildings with functions 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 (0.05)                | (K_{DIC}^{II} = 0.10)      | (0.10)                      |
|                      |                                                                                                                                                                                                                          |                             | K_{O} = PGA_{IX}/PGA_{VIII} | (1.5)                       |
|                      |                                                                                                                                                                                                                          |                             |                             | PGAVIII=0.1/0.05 = (2.0)    |
| II BC_{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 (0.025)                 | VIII (0.05)                 | IX (0.10)                   |
|                      |                                                                                                                                                                                                                          |                             |                             |                             |
| III BC_{III}         | Auxiliary industrial buildings on which the primary production does not depend; energy facilities of local significance; stables for cattle on community farms                                                                                                                                 | VII (0.025)                 | VII (0.025)                 | VIII (0.05)                 |
|                      |                                                                                                                                                                                                                          |                             |                             |                             |
| IV BC_{IV}           | Buildings whose collapse cannot endanger human lives and cause damage to expensive equipment; temporary structures                                                                                                                                                              |                             |                             | Seismic influences shall not be taken into account |
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 did JUS39/64 discuss the energy dissipation capacity of structures. On the other side, due to the presence of ductility and damping $K_p$ coefficient in Eq. (15), JUS31/81 is considered as ductility code.

Both JUSs are classical $K_S$-format codes assigning the non-dimensional coefficient $K_S$ and corresponding PGAs 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 transiting from non-dimensional $K_S$ into PGA-format (R-format) code assigned PGA values (in g, g = 9.81 m/s$^2$) to zoned territories. For that, SNiP II-7–81 introduced coefficient $K_1$ which in this manuscript 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 $D_L = 4$ 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.

In that light, the $K_p$ should be treated as a structural typology dependent modifier of the nominal ductility and damping coefficient $D_L$. 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, Eq. (19).

### 3.5 Ductility and damping coefficients $K_p$ and $R_p$ Factor

Table 9 JUS39/64 and JUS31/81 importance factors

| Building category | $K_64$ Importance factor matrix | $K_81$ Importance factor vector |
|-------------------|--------------------------------|--------------------------------|
|                   | VII | VIII | IX | | VII | VIII | IX |
| BC$_I$            | 2.0 | 2.0  | 1.5 | 1.5 | |
| BC$_{II}$         | 1.0 | 1.0  | 1.0 | 1.0 |
| BC$_{III}$        | 1.0 | 0.5  | 0.5 | 0.75 |

For buildings and structures in which permanent deformations and local damage are not allowed, and. $D_L = 1.0$ and for 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).
\[ K = K_O \cdot K_P \cdot K_S \cdot S \cdot S_e \cdot K_\eta = K_O \cdot K_P \cdot \frac{PGA}{D_L} \cdot S \cdot S_e \cdot K_\eta = K_O \cdot \frac{PGA}{R_p} \cdot S \cdot S_e \cdot K_\eta \]  

(19)

where

\[ R_p = \frac{D_L}{K_P} \]  

(20)

The advantage of transforming JUS’s standard seismic coefficient expression (Eq. 15) in R-format (Eq. 19) is that its constituents are coefficients with clear physical meaning. Hence they could be directly related, studied, and compared with their equivalents from other R-codes. It is noted that EN 1998–1: Eurocode 8 seismic coefficient is formulated in R-Code format, so JUS39/64 and JUS31/81 \( R_p \) and Eurocode 8 <\( q\) values are directly comparable.

The disadvantage of Eq. (15) is that it shall first be disaggregated for defining PGA and the \( D_L \) coefficient, which academic and research communities in RNM and SFRY are barely aware of, and then via \( K_p \) multiplier (Eq. 20) link it in physically recognizable \( R_p \) coefficient of Eq. (19).

Values of JUS39/64 and JUS31/81 ductility and damping coefficient \( (K_p) \) and corresponding \( R_p \) values are displayed comparatively in Table 10. Table 10 presents JUS31/81 structural typology but splits the \( K_{p3} \) typology into two subcategories \( K_{p3,1} \) and \( K_{p3,2} \). The \( K_{p3,2} \) typology (shaded in grey in Table 10) is common to JUS39/64 and JUS31/81 codes.

JUS39/64 Article 2.5.3 for structural typology \( K_{p3,2} \) prescribes \( K_{p3,2} = 1.6 \). However, it does not recognise other structural typologies, which implies that their \( K_p = 1.0 \).

### 3.6 Design response spectra \( K_{D64} \), \( K_{D81} \)

JUS39/64 and JUS31/81 define the shape of horizontal elastic design spectra \( K_D \), Eq. (11), as a product between the soil coefficient \( S \) and the 5%-damped elastic 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.

\[ K_D(T) = S \cdot S_e(T) \]  

(21)

Such disaggregation is necessary and allows discussing both coefficients separately. In Eqs. 14 and 18 \( K_D \) of Eq. (1) is represented in its disaggregated form defined by Eq. (21).

#### 3.6.1 5%-damped elastic response spectral shapes \( S_{e64} \), \( S_{e81} \)

The amplitude and frequency modification of regional motions both codes achieve with different, ground type dependent \( S_{e64} \) and \( S_{e81} \) soil factors.

Characteristic to the JUS39/64 \( S_{e64} \) (Article 2.5.3) is the use of one spectral shape defined by Eq. (22) and a single constant acceleration region plateau extending over the \( T = 0–0.5 \) s period range.

\[ 0.50 \leq S_{e64} = \frac{0.75}{T_i} \leq 1.5 \]  

(22)
| Structural typology by JUS31/81 | 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 $K_{P2}$, and all modern wooden structures, except those listed under $K_{P3.1}$ and $K_{P3.2}$ 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 period of vibration $T \geq 2.0$ s | 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 |

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

Structural typology common to both JUS39/64 and JUS31/81 codes
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 IDSG Zone | \( S^{64} \) factor |
|------------------|-------------|-------------------------------|-------------------|
| Good | GT₁ | Hard rock, homogeneous gravel deposits | V₁I₂I₃ | \( S₁ \) 0.8 |
| Medium | GT₂ | Homogeneous sand deposits; overconsolidated clay and marl deposits; moderately heterogeneous clayey and sandy-gravel deposits | V₁I₂I₃ | \( S₂ \) 1.0 |
| Week | GT₃ | Heterogeneous deposits; soft marls and clays | V₁I₂I₃ | \( S₃ \) 1.2 |
| Very weak | GT₄ | Soft deposits of considerable thickness | V₁I₂I₃ | \( S₄ \) 2.0 |

\( K_{S}^{64}, K_{S}^{51} \) Seismic coefficients before transformation to PGA, Table 7
Using a single plateau formulation makes it evident that JUS39/64 focuses on the amplitude but completely neglects the frequency modification of regional ground motions.

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

$$\frac{2T_{C,GT_i}^{81}}{3} \leq S_{e,GT_i}^{81} = \frac{T_{C,GT_i}^{81}}{T_i} \leq 1$$

(23)

where $T_{C,GT_i}^{81}$ is the upper, ground type-dependent corner period of the constant spectral acceleration branch denoted by subscript $<C>$. The basis for the development of $S_{e,GT_1}^{81}$ elastic spectral shapes are $K_D^{64} = S_{e,GT_1}^{64} \cdot S_{e,GT_1}^{64}$ elastic design spectral shapes:

- $S_{e,GT_1}^{81}$ is constructed by normalizing the $K_{D,GT_1}^{64}$ with its constant acceleration plateau value (1.2);
- $S_{e,GT_1/3}^{81} = S_{e,GT_1/81}^{81}$ for $GT_2$ and $GT_3$ ground types are constructed by intersecting $K_{D,GT_2/3}^{64}$ with $S_{e,GT_1/3}^{81}$ ordinate;
- The $GT_2$ intersection point (the upper corner period of the constant spectral acceleration branch, $T_{C,GT_2}$) is moved left for 0.05 s 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 (0.2 s) plateau extensions by a decrease of ground stiffness.

In addition, JUS31/81 extracted the $GT_4$ ground category from the Code provisions. Due to such construction of $S_{e,GT_1}^{81}$, the $S_{e,GT_1}^{81}$ and $K_{D,GT_1}^{81}$ elastic response spectral shape sets completely overlap over all three spectral ranges.

One of the problems of such construction of $S_{e,GT_1}^{81}/K_{D,GT_1}^{81}$ elastic response spectral shape sets is the extraction (due to $S_{e,GT_1}^{81} = 1$) of GT-dependent amplitude influence from the modification of regional motions over the entire constant acceleration region plateaus. However, due to adopted $T_{C,GT_i}^{81}$ upper corner period values of the constant acceleration region (0.5 s, 0.7 s, and 0.9 s for $GT_1$, $GT_2$, and $GT_3$, respectively), the $S_{e,GT_1}^{81}/K_{D,GT_1}^{81}$ elastic response spectral shape sets maintain their amplitude and frequency patterns over the entire constant velocity spectral range. From $T = 1.5$ s (upper corner period of all three velocity spectral regions) over the entire constant displacement spectral range, the frequency modification is again neglected and $S_{e,GT_1}^{81}$ values are limited to 0.33, 0.47 and 0.60 for $GT_1$, $GT_2$, and $GT_3$, respectively.

Values of corner periods of the constant spectral acceleration ranges $T_{C,GT_i}$ were estimated as too high (Poceski 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 s, justifying it by the fact that such a definition of $S_{e,GT_1}^{81}/K_{D,GT_1}^{81}$ 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 in constructing flexible buildings.

Table 11 presents ground typology (GT) distinguished by JUS39/64. The GT-dependent design response spectral sets for both codes are presented comparatively in Fig. 2. Solid and dashed lines denote $K_{D,GT_1/4}^{64}$ and $K_{D,GT_1/3}^{81}$ design response spectral shapes, respectively.
3.6.2 Soil factor $S^{64}, S^{81}$

Table 2 of JUS39/64 defines the "Coefficient of seismicity" $K_{SD}^{64}$ as a function of mapped seismicity $I_{MCS}$ and the ground type (GT). It recognizes three ground types (good, medium and weak) 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 masks 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 m from the surface EB73, 1973. The EB87, 1987 renamed it in "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."

$$K_{SD}^{64} = S^{64} [K_S^{64}]^T$$  \hspace{1cm} (24)

To disaggregate $K_{SD}^{64} = f(S^{64}, K_S^{64})$ into $S^{64}$ and $K_S^{64}$ coefficients, the $K_{SD}^{64}$ table, Table 11, is normalized to medium ground conditions ($GT_2$). The results of normalization are $I_{MCS}$ and $I_{DSC}$ independent $S^{64}$ soil factor (4 $\times$ 1 column) vector and seismic intensity coefficient $K_S^{64}$ (1 $\times$ 3 row vector, Table 7). The matrix product defined by Eq. (24) fully reconstructs the $K_{SD}^{64}$ table presented in Table 11.
JUS31/81, as discussed above, defines three different ground-type dependent design response spectral shapes. Each spectral shape is characterised by its own $T_{C,GT}$, plateau, Fig. 2. The ground type-dependent corner periods of the constant spectral acceleration branch $T_{C,GT}$ are presented in Table 12. The plateau for good ground type extends up to 0.5 s, whereas for medium and weak ground typology is extended by an equidistant increment of 0.2 s, to 0.7 s and 0.9 s, 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 $S_{GT}$ shapes. The $S_{GT}/S_{GT2}$ normalization of design response spectral shapes defines the spectral behaviour of $S_{GT}$ soil factors, which are presented in Fig. 3 and tabulated in Table 12.

$S_{GT}$ over all three spectral regions (acceleration, velocity, and displacement) equals 1.0. $S_{1}$ is equal to 1.0 over its acceleration region (0–0.5 s), from where nonlinearly decrease to a value of 0.714 ($= T_{C,GT1}/T_{C,GT2}$) until the $T_{C,GT2}$ corner period of 0.7 s and maintain it over the rest of $T_{C,GT2}$ velocity and the entire $T_D$+ displacement region. $S_{3}$ equals to 1.0 over the entire $T_{C,GT3}$ acceleration region. From $T_{C,GT2}$ to $T_{C,GT3}$ it linearly increases to a value of 1.286 ($= T_{C,GT3}/T_{C,GT2}$) and maintains it over the rest of $T_{C,GT3}$ velocity and the entire $T_D$+ displacement region.

Relative to $GT_2$ ground type, the maximum $S_{GT}$ and $S_{GT3}$ variations (Fig. 3, Table 12) are ±28.6% and are slightly greater than $S_{GT1}$ and $S_{GT2}$ ones that are ±25.0%, Table 5.

The $K_{D,GTi}$ expressed in $S_{GT2}$ and $S_{GTi}$ terms is

### Table 12  Behaviour of $S_{GT}$ soil factor over the 0-$T_D$ + period range

| Ground type | $S_{GT}$ | $T_C$(s) | $T_D$(s) | $T_{C,GT1} - T_{C,GT2}$ | $T_{C,GT2} - T_{C,GT3}$ | $T_{C,GT3} - T_D$ |
|-------------|----------|----------|----------|--------------------------|--------------------------|-------------------|
| Good $GT_1$ | $S_1$    | 0.5      | 1.5      | 1.0                      | Fig. 3 (N1)              | 0.714 (− 28.6%)   |
| Medium $GT_2$ | $S_2$    | 0.7      | 1.5      | 1.0                      |                         |                   |
| Weak $GT_3$  | $S_3$    | 0.9      | 1.5      | 1.0                      | Fig. 3 (L2)              | 1.286 (+ 28.6%)   |
| Very weak $GT_4$ | $S_4$    | Extracted |          |                          |                         |                   |

---

**Fig. 3** Spectral behaviour of $S_{GT}$ soil factors
where

\[
K_{D,GT}^{81} = \frac{T_{GT2}^{81}}{T} \begin{cases} 
S_{1}^{81} (\text{Eq.22}) & 0 \leq T \leq T_{C,GT1} \\
1 & T_{C,GT1} \leq T \leq T_{C,GT2} \\
S_{3}^{81} (\text{Eq.23}) & T_{C,GT2} \leq T \leq \infty 
\end{cases}
\]  

(25)

4 Comparative analysis of \( K^{64} \) and \( K^{81} \) lateral force (Seismic) coefficients

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

For stiff buildings (\( H \leq 20 \) m, or \( n \leq 5 \) stories), both codes prescribe the use of a simplified method. JUS39/64 calculate story shears according to Eq. (10) and then by Eq. (14) sums them in \( F_{64}^{b} \). The period of \( n \leq 5 \) story buildings, calculated by \( T = 0.1n \), is \( T \leq 0.5 \) s and fully corresponds to the JUS39/64 definition of “stiff structure.”

4.1 K-Quotient (KQ) method

The most efficient way to compare or scale the values of base shears defined by two or more codes is by taking the appropriate ratio. The ratio, \( R_{Kb} \), defined by Eq. (28) expresses the \( F_{b}^{81} \) base shear in terms of \( F_{b}^{64} \)

\[
R_{Kb} = \frac{F_{b}^{81}}{F_{b}^{64}} = K_{Kb} = \frac{K_{O}^{81}}{K_{O}^{64}} \frac{K_{p}^{81}}{K_{p}^{64}} \frac{S_{c}^{81}}{S_{c}^{64}} \frac{S_{e}^{81}}{S_{e}^{64}} \frac{K_{S}^{81}}{K_{S}^{64}} \frac{K_{\eta}^{81}}{K_{\eta}^{64}} = R_{Kb}RK_{K}RSRS_{b}RK_{S}RK_{\eta}
\]  

(28)

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 \( R_{Kb} \) is:

- \( R_{Kb} > 1 \), JUS31/81 base shear force is larger than that prescribed by JUS39/64; and,
- \( R_{Kb} < 1 \), JUS31/81 base shear force is smaller than that prescribed by JUS39/64.

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It is also true for all $RK_i$ 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. (28) is a product of six quotients (ratios). Accordingly, this manuscript terms it. $K - Quotient (KQ)$ method.

KQ 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 ≤ 20 m, 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. (13). For the same structural category (n < 5 storeys), JUS31/81 calculates the $F_{81}^b$ by Eq. (15).

The period of n < 5 story buildings is in the range 0–0.5 s. It is comprised of all three $S_{81}e$ plateaus and the $S_{64}e$ plateau. Thus $S_{81}e = S_{64}e = 1$. As diussed above, for using a simplified method JUS39/64 (Article 2.6) requests that $S_{64}e$ spectral shape is multiplied by a factor of 1.5. Thus,

$$RS_e = \frac{S_{81}e}{S_{64}e} = \frac{1}{1.5} = \frac{2}{3}$$ (29)

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

$$RK_s = \frac{K_{81}^s}{K_{64}^s} = \frac{PGA_{81}}{PGA_{64}} = 1$$ (30)

$RK_s$ ratio is defined by Eq. (31). Details on its derivation are enclosed in Appendix C.

$$RK_s = \frac{K_{81}^s}{K_{64}^s} = \frac{2n + 1}{3}$$ (31)

Defined $RK_O$, $RS$, $RK_P$, and $RK_s$ ratios are displayed in Tables 13, 14, 15, 16, respectively. Based on Eqs. 28 and 29, Eq. (28) reduces to:
Table 14 JUS39/64-JUS31/81 RS ratios

| Ground category | $S_{64}$ | $S_{81}$ | RS   |
|-----------------|---------|---------|------|
| I               | 1.2     | 1.0     | 0.8333 |
| II              | 1.0     | 1.0     | 0.6667 |
| III             | 1.2     | 1.0     | 0.5556 |
| IV              | 2.0     | 1.0     | 0.3333 |

Table 15 JUS39/64-JUS31/81 RKp ratios

| Building | Typology | $K_{64}$ | $K_{81}$ | RKp |
|----------|----------|----------|----------|-----|
| (1)      | RKp1     | 1.0      | 1.0      | 1.0 |
| (2)      | RKp2     | 1.0      | 1.3      | 1.3 |
| (3.1)    | RKp3.1   | 1.0      | 1.6      | 1.6 |
| (3.2)    | RKp3.2   | 1.6      | 1.6      | 1.0 |
| (4)      | RKp6     | 1.0      | 2.0      | 2.0 |

Table 16 JUS39/64-JUS31/81 RKq ratio

| # of Stories | $K_{q6}$ | $K_{q5}$ | $K_{q4}$ | $K_{q3}$ | $K_{q2}$ | $K_{q1}$ | $K_{q6}$ | $K_{q5}$ | $K_{q4}$ | $K_{q3}$ | $K_{q2}$ | $K_{q1}$ | RKq |
|--------------|---------|---------|---------|---------|---------|---------|---------|---------|---------|---------|---------|---------|------|
| 6            |         |         |         |         |         |         |         | 0.8077  |         |         |         |         | 1.2381 |
| 5            |         |         |         |         |         |         |         | 0.8182  |         |         |         |         | 1.2222 |
| 4            |         |         |         |         |         |         |         | 0.8333  |         |         |         |         | 1.2000 |
| 3            |         |         |         |         |         |         |         | 0.8571  |         |         |         |         | 1.1667 |
| 2            |         |         |         |         |         |         |         | 0.9000  |         |         |         |         | 1.1111 |
| 1            |         |         |         |         |         |         |         | 1.0000  |         |         |         |         | 1.0000 |

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

Table 17 $K$ conversion coefficient values

| # of Stories | Soil factor $S$ | Structural type $K_P$ |
|--------------|----------------|-----------------------|
|              | $S_1$ | $S_2$ | $S_3$ | $S_4$ | $K_{P1}$ | $K_{P2}$ | $K_{P3.1}$ | $K_{P4}$ |
| $RK_{p1} = 1.0$ |       |       |       |       |         |         |           |           |
| 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   |
| $RK_{p2} = 1.3$ |       |       |       |       |         |         |           |           |
| 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   |
| $RK_{p3.1} = 1.6$ |       |       |       |       |         |         |           |           |
| 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   |
| $RK_{p4} = 2.0$ |       |       |       |       |         |         |           |           |
| 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
From Tables 13, 14, 15, 16, it is observable that $R_{K_P}$ and $R_{K/u_2}$ ratios, both $\geq 1$ tend to increase $R_{K_b}$. On the other hand, $R_S$ and $R_{S_e}$ (Eq. 28) ratios tend to decrease it. $R_{K_O}$ ratio, Table 13, for Importance Class I buildings decrease $R_{K_b}$ 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 $R_{K_b}$ by ground categories are presented in Fig. 4 and by structural typology in Fig. 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.

The proposed KQ method is easily applicable for comparing ELF-based base shears. Figure 6 is a tabular presentation of the procedure proposed by Eqs. (27) and (31).

For example, for Category II ($R_{K_{OZ}} = 1.00$, 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.

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

$$R_{K_b} = \frac{2}{3} R_{K_O} R_{K_P} R_{S} R_{K\eta}$$  (32)

If the building is with stiffness discontinuity (e.g., flexible or soft story, $K_{p4} = 2.00$), the $R_{K_b}$ would be:

$$R_{K_b} = \frac{2}{3} \cdot 1.00 \cdot 1.00 \cdot 0.667 \cdot 1.00 \cdot 1.2222 = 0.5432$$ and $F_{b}^{64} = 0.5432F_{b}^{64}$

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

$$R_{K_b} = \frac{2}{3} \cdot 1.00 \cdot 1.00 \cdot 0.667 \cdot 1.60 \cdot 1.2000 = 0.8534$$

Relative to JUS31/81, the JUS39/64 simplified method is over-conservative for stiff building. The largest conservatism is associated with the lateral force coefficient associated with $GT_3$ 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 $R_{K_b}$ ratio (or $R_{K_b}$-quotient) is calculated for three selected flexible buildings located in $I_{MCS}$ zone of IX degrees with $K_S = 0.10$. All three buildings are reinforced concrete space moment-resisting frame (skeleton) buildings ($K_P = 1$; $R_P = 4$) symmetric in both orthogonal horizontal directions. They are residential buildings with the ground floor
(GF) hosting commercial uses, hence Importance category II ($K_0 = 1$) buildings. The floor plan of B10 buildings is presented in Fig. 11.

DTB (Tomic, 2018) building is 43 m in height GF + ME + 12 story tower with $T_x = T_y = 1.848$ s. JTB (Trajcevski, 2018) building is a GF + ME + 4 story block type building 19.3 m in height with $T_x = 1.108$ s and $T_y = 1.048$ s. 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 B10 building is a dual RC frame/RC wall system with a shear wall placed in the middle span of two internal frames.

Both buildings were designed in 1967 by JUS39/64 and erected by 1972. They are part of the so-called “City Wall,” an architectural landmark of Skopje that was designed and constructed following the 1963 earthquake to solve the housing needs (in total 1,814 condominiums for the accommodation of cca. 7,800 inhabitants) in the City centre. They are both representatives of the dominant ($K_{p1}$) building typology implemented during the reconstruction of the City of Skopje. The City Wall, as a prominent urban element of the City of Skopje, circumferences the City centre on the right riverbank of the Vardar River. It consists of 24 GF + ME + 12 story towers (DTB building) and 9 GF + ME + 6 story “blocks,” ending with orthogonally placed GF + ME + 4 Annexes (JTB building). GF is Ground Floor and ME is the Mezzanine. B10 building is a pre-earthquake RC structure designed and erected according to JUS61/48, structurally damaged (grade 3) in the earthquake, and repaired and strengthened by 1972 according to JUS39/64 (Fig. 11).

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 $RK_b$ s relative to trends outlined by the simplified method. For $GT_1$ ground type, both $RK_b$ s are $\leq 1$, hence JUS38/64 did prescribed the higher base shear than JUS31/81. The $RK_{b,1.0}$ are attributed to $GT_3$ ground typology demonstrating that JUS31/81 exceeds 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 $RK_{b,1.0}$. 

**Fig. 7** JTB Building, characteristic floor plan
While not defining the criteria, for structures with a flexible ground or any other story or structures with dramatic or sudden changes in stiffness (structural type $K_{P4}$ in Table 10) JUS31/81 requests. 

$K_p = R_p = 2$. The values of $K_{P4}$ based $RK_b$ quotients presented in Table 18 will double; hence for all GTs JUS31/81 will exceed JUS39/64.

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, a number of computational models are formulated 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 the formulation of such computational models influences their Eigenvalue characteristics in general and the behaviour of $K_{64}$ coefficient in particular.

The results of the analyses are summarized in Fig. 9. The horizontal, bar-diagram-like solid lines at the top of the figure define JUS39/64 and JUS31/81 ground-type-dependent extensions of the constant acceleration (greenish colour), velocity (bluish), and displacement (brownish) spectral ranges. The greyish shaded rectangle at the bottom left corner present $R_{K_b}$’s calculated by “approximate” (simplified) for stiff and “more accurate” (MRS for JUS39/64 and ELF for JUS31/81) methods of analysis for flexible buildings. Orange, blue, and red colours are used to distinguish better the results derived for GT1, GT2, and GT3 ground conditions, respectively. Dotted and dashed lines denote results from x/y-directional analyses of JTB building, solid lines the results of DTB, and dot-dash lines denote results derived for B10 building.
| # of stories | GT | W [t] | Longitudinal Direction LN (X) | Transverse Direction TR (Y) |
|--------------|----|-------|-------------------------------|-----------------------------|
|              |    |       | T [s] | JUS39/64 | JUS31/81 | RKb | T [s] | JUS39/64 | JUS31/81 | RKb |
|              |    |       | Fb [kN] | %W | Fb [kN] | %W |     | Fb [kN] | %W | Fb [kN] | %W |     |
| JTB 6        | 1  | 1,802 | 92.37 | 5.10 | 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 | 238.20| 3.44 | 230.67 | 3.33 | 0.968 | 1.048 | 95.66 | 5.31 | 85.97 | 4.77 | 0.899 |
|              | 2  | 297.75| 4.3  | 322.93 | 4.7 | 1.085 | 142.55 | 7.9 | 154.76 | 8.6 | 1.086 |
|              | 3  | 357.30| 5.2  | 415.20 | 6.0 | 1.162 | 154.76 | 8.6 | 154.76 | 8.6 | 1.086 |
| B10 10       | 1  | 2,880 | 124.16| 4.3  | 134.70 | 4.7 | 1.085 | 1.048 | 95.66 | 5.31 | 85.97 | 4.77 | 0.899 |
|              | 2  | 155.20| 5.4  | 188.58 | 6.5 | 1.215 | 1.048 | 95.66 | 5.31 | 85.97 | 4.77 | 0.899 |
|              | 3  | 186.24| 6.5  | 242.45 | 8.4 | 1.302 | 1.048 | 95.66 | 5.31 | 85.97 | 4.77 | 0.899 |

Direction not analysed, Dual system ($K_p$) symmetric like the JTB building
Apart from indicated deficiency of computational models used, the analyses, Fig. 9:

- Confirmed that in the period range 0.9—2.0 s JUS31/81 exceeds the JUS39/64 base shear for GT3 ground type. Hence over the entire velocity and displacement spectral regions of GT3 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 GT1 ground type, the trends are inverse. Over the entire $T_{C,2}/T_{C,3} > 2.0$ + seconds range JUS39/64 exceeds the JUS31/81 base shear. For DTB building, with the first period being in the displacement spectral region ($T_1 = 1.848$ s), $R K_b$ converge to 1, whereas for JTB building ($T_{1x}/T_{1y} = 1.108/1.048$ s) it converges to value close to 0.95 s. The exception is the B10 building, for which $R K_b > 1$ over the entire $T_{C,GT2} > 2.0$ s period range;
- The order of $R K_b$ relations over the $T_{C,(GT2)/(GT3)} > 2.0$ s + period range is $R K_{b,GT3} > R K_{b,GT2} > R K_{b,GT1}$;
- Over the 0.5-$T_{C,(GT2)/(GT3)}$ the $R K_b$ relations reorder (inverse) themselves so that $R K_{b,GT1} > R K_{b,GT2} > R K_{b,GT3}$;
- The simplified calculation of base shear for “stiff” buildings is over conservative relative to calculating them by MRS or ELF method; and,

![Image](image-url)
The sloping of $RK_{b,GTi}$ in post $T_{ci}$ velocity and over the entire displacement ($T_D \geq 1.5$ s) regions is influenced by $K_{b1} = n$ coefficient. The effect is particularly pronounced in $T_D \geq 1.5$ s range where $RK_b$ increases monotonically and almost linearly.

All values discussed presented in Table 19 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 0.5 s 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 shown, the behaviour of $RK_b$ is quite irregular over the analysed spectral range of 0.5–2.0 s and does not provide a clear picture to decide on the seismic design level of both codes. $RK_b$ is a method of structural analysis and ground category dependent.

For $GT_3$ located buildings JUS31/81 consistently exceeds 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 uniformly differentiate 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.

For $R_p = 2/K_p = 2$ (structures with a flexible ground or any other story or structures with dramatic or sudden changes in stiffness) numerical triage has not been performed. From Table 19, it is evident that the minimum $R_p$ values range from 0.629 to 0.809. Multiplied by $K_{p1}/K_{p4} = 2$ for all ground types $RK_b$ minimums would be in the range of 1.258 to 1.618, i.e. $> 1$. Hence, over the entire spectral range of 0.5–2.0 + s, JUS31/81 will unconditionally exceed JUS39/64 from 25.8 to 61.8%.

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 the 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’s 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: $\leq 0.35$ (for columns, Article 61) and $\leq 0.20$ (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.

Based on data, arguments, and discussion presented in Sect. 2.2.1 and Appendix B of this manuscript, we partially revise Table 20 for Skopje and Macedonia. The particular focus is on buildings constructed in the 1948–1963 period.

Till the 1963 Skopje earthquake, JUS61/48 was in implementation for 15 years. However, a number of international and national post-earthquake assessments and other studies documented that local design practice in Macedonia generally ignored JUS61/48 seismic provisions and that newer (post-1950) mid-rise buildings had been designed only for wind loads (Berg 1964 and 1965, Kapsarov 1972).

This factography classifies all pre-1963 construction of Table 20 in the pre-code (CDN) category. The exception is the $\geq GF + 6$) RC frame construction that, due to wind design, had minimal protection against lateral forces. As such, they might be considered low-code (CDL) construction. The revised segment of Table 20 is presented in Table 21.

The number of 1948–1964 buildings in SFRY/Macedonia is not negligible and calls for particular attention. The most essential is a clarification of whether the local design practice used JUS61/48 seismic design provisions or not and a concept of lateral load systems against wind loading.

In that regard, we again emphasise that the above discussion, including the entire discussion in Appendix B, is only relevant for Skopje and Macedonia. The characteristics of masonry and RC frame building typology in other Federal units of FNRY are unknown to the Authors. They could be modified by local interpretation and implementation of JUS61/48 seismic and wind provisions.

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 the evolution of Yugoslavian seismic resistant design requirements, 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 ($K_0, K_P, K_S, S, \frac{K}{u_1}$) and two sets of 5%-damped elastic spectral shapes ($S_\alpha, S_\beta$). Each section is appropriately concluded, so a number of conclusions will not be repeated herein.

The most relevant findings of this study are summarized below:

- Equation 15 is not suitable for analysing and comparing Europe-wise 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. 19). In particular, since EN 1998–1 (Eurocode 8) uses it. All lateral force coefficients and their constituents of previous codes can efficiently be compared within it.
\[ K = \gamma_I \cdot a_{gR} \cdot S \cdot \frac{1}{q} \cdot S_e(T) \]  

(33)

where \( \gamma_I \) 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 \((RK_b)\) 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 the KQ 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 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 0.5 s (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;
- \( K_{P1} \) structural types are mostly implemented systems in RNM and Yugoslavia since 1964 year, hence the irregularity in the behaviour of K-Quotient over the 0.5–2.5 s + prevent its exclusive use as a metric 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 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 by Crowley et al., 2021a, b, 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 a continent-wise model 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 this contribution provides sufficiently detailed, compact, and valuable technical information on the temporal evolution of seismic design codes used in Macedonia and Yugoslavia from 1948 until they were replaced by Eurocode Design System EN 1990–EN 1999.
Appendix A Design codes referred to in this contribution

This appendix provides the full administrative 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: CH-8-57 „Normy 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: CHиП 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).

SNiP II-7-81: CHиП II-7-81 „Строительные нормы и правила, Часть II, Глава 7: Строительство в сейсмических районах (The Construction Norms and Rules, Part II, Chapter II: Construction in Seismic regions)“. ГОССТРОЙ СССР, Москва 1981 (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)

GSI18/63: “Dimenzioniranje Gradbenih objektov v potresnih območjih (Dimensioning of buildings in seismic areas) “Slovenian association of structural engineers and technicians, Ljublana, 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 SFRY 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 Buildings Damaged by Earthquakes, and for Reconstruction and Revitalization of Buildings; (Official Gazette of SFRY No. 52 of 4 October 1985, in Serbo-Croatian)

JUSxx/86: Regulations on Technical Norms for Design and Calculation of Civil Engineering Facilities in Seismic Regions; (1986, Draft, in Serbo-Croatian)

RNM 211/20 The Rulebook for Amending the Rulebook for Design Standards and Norms for Design (Official Gazette of RNM No. 211 of 02.09.2020, in Macedonian)

EB87: Explanatory Booklet for Seismological Map of SFR Yugoslavia, Seismological Association of SFR Yugoslavia, Belgrade, April 1987

MKC EN 1998-1: Eurocode 8: Design of structures for earthquake resistance - Part 1: General rules, seismic actions and rules for buildings (Official Gazette of RNM No. 211 of 02.09.2020, in Macedonian)

JUS U.C7.121: Bases for design of structures-Loads due to use and occupancy in residential and public buildings (Official Gazette of SFRY No. 49/88 of 12.08.1988, in Serbo-Croatian)
Appendix B Characteristics and damage patterns of pre-1963 Skopje’s building typology and JUS61/48 wind design of pre-earthquake midrise RC frame structures

Characteristics and damage patterns of pre-1963 Skopje’s building typology

The pre-earthquake building stock of Skopje contained all possible types of systems concerning the construction materials and the building heights.

Until the earthquake, the housing stock of the city amounted to about 40,000 apartments, built in the following periods: (1) 20% until the First World War /WWI/; (2) 30% between the two world wars; and (3) 50% from the Second World War /WWII/ to the 1963 earthquake, Kapsarov (1972).

The post-WWII period is the era of extensive construction of classic (≤ GF + 5 story, GF denoting the ground floor) masonry buildings characterized by massive solid brick structural walls thick 25 to 38 cm and the following three floor systems:

- Cast-in-place RC joist and slab system;
- Cast-in-place concrete slab on precast RC joists.

Both RC joist and slab systems are stiffened with perimeter and internal reinforced concrete belts cast over the structural walls; and,

- timber floor constructed over the solid brick structural walls.

RC frame systems entered construction practice in the mid-1950s. In Macedonia, the following two RC frame systems were favoured:

- complete moment-resisting RC frame designed to withstand both gravitational and lateral wind loads, unassisted by walls, stairwells, or elevator shafts;

In the following, these RC frame systems we refer to as “skeleton^5” frame systems in order to differentiate this type of structure from “frame system” as defined by JUS31/81 and EN 1998-1; and

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^5 Similar to EN 1998-1 “frame system” – structural system in which, irrespectively of the height of the building, both the vertical and lateral loads are exclusively (100%) resisted by spatial frames. All post-1963 studies refer this system to as “complete moment-resisting RC frame” or “frames unassisted by shear walls (stairwells or elevator shafts) to resist all of the lateral loads.
With internal shear walls\textsuperscript{6} located dominantly at stairwells and elevator shafts. The frames were not designed to carry any lateral loading, thus only to transport gravitational loads.

In the following, these RC frame systems assisted by the shear walls, we refer to as “quasi-dual\textsuperscript{7}” frame systems. The term “quasi-dual” is used to differentiate this type of structure from “dual” RC systems defined by JUS31/81 and EN 1998-1.

RC frame structures used the same RC floor systems as masonry buildings but fixed in peripheral and internal beams and, in addition, continuous, cast-in-place, 12-13 cm thick reinforced concrete slabs (Berg 1964, 1965).

Sozen (1964) estimated that over 90% of construction in Skopje was in URM. RC frame structures were the latest building typology that began to be applied in the last years preceding the Skopje earthquake. Accordingly, they were presented in the lowest percentage of Skopje’s building stock.

According to the Authors’ experience and knowledge, for low-rise (≤ GF + 5 stories, ~18 to 20 m in height) buildings, irrespectively whether they were of masonry or RC frame typology, the local design and construction practice completely disregarded lateral wind loading since buildings of these heights were treated as semi-protected. The midrise (≥ GF + 6) buildings were designed for wind only, except three GF + 13 RC quasi-dual towers in Karpos II settlement that were designed according to JUS61/48 seismic provisions.

In support, we cite Berg (1964, 1965) “Information obtained from engineers in Skopje after the 1963 earthquake disclosed that the seismic code provisions had been generally ignored, but the newer buildings had been designed for wind loads in accordance with the code,” referring to the Chapter 2 of JUS61/48. Term “newer buildings,” refer to RC frame structural typology, with frames generally heavy in one direction, usually the longer plan dimension of the building, and connected with relatively light beams in the opposite direction.

Kapsarov (1972), based on a 1964 UYI\textsuperscript{8} study on 255 damaged buildings (completed March 1964), confirmed Muto et al. (1963), Sozen (1964), and Berg’s (1964, 1965) observations and findings.

Although the lateral building strength of the entire pre-1963 RC building stock was based either on ignorance of seismic provisions or the wind load design, with minor

\textsuperscript{6} The share walls were dominantly in plain concrete with 10 or 20 MPa strength, a combination of plane concrete (lower half of the building) and solid brick walls in lime, cement, or lime-cement mortar (upper part of the building). Other internal partition and façade walls were made from perforated brick or hollow block masonry.

\textsuperscript{7} Similar to EN 1998-1 “wall-equivalent dual system” – dual system in which the shear resistance of the walls at the building base accommodate 100% of the total lateral load.

\textsuperscript{8} By mid-1964, the Union of five Yugoslav Institutes (UYI) completed technical documentation and a detailed damage assessment study on 255 randomly selected houses/buildings (65% masonry and 35% RC). The results are presented in 37 volumes. The following institutes conducted the study that was intended to assist in the development of the JUS39/64 seismic code: (1) Institute for Testing Materials and Structures (IMK) – Sarajevo, BA (6 volumes/29 buildings); (2) Institute of Civil Engineering of Croatia (IGH) – Zagreb, HR (8 volumes/50 buildings); (3) Materials Testing Institute (ZIM) at the Technical Faculty in Skopje – Skopje, MK (7 volumes/75 buildings); (4) Institute for Testing Materials and Structures (IIM) – Belgrade, RS (4 volumes/40 buildings); and, (5) Institute for Materials and Construction Research (ZRMK) – Ljubljana, SI (12 volumes/51 buildings). The completion of the project was announced at the “International seminar on earthquake engineering” held under the auspices of the Federal Government of Yugoslavia and UNESCO, Skopje, 29 September to 2 October 1964 (https://unesdoc.unesco.org/ark:/48223/pf0000014176), later on, recognised as the first European Conference on Earthquake Engineering (1ECEE).
exceptions, this building typology survived very well the 1963 earthquake. Only two build-
ings collapsed out of ~ 500 RC frame buildings in Skopje, but two-thirds of this group had
survived the earthquake with light damage, Sozen (1964).

One of these collapses was the roof shell (6 cm thick) of Trade Fair Hall I. Apart
from significant efforts to track the other RC collapsed building, it is still unknown to
the Authors. The damage of all others was within the JUS61/48 live-safety requirements.
A number of buildings were damaged beyond repair and demolished; however, the vast
majority had survived the earthquake with light or very light damage.

The above discussion is strictly valid for Skopje and Macedonia. The Authors are not
familiar with the local interpretation and implementation aspects of JUS61/48 in other Fed-
eral units, i.e.:

- whether the JUS61/48 seismic provisions were respected for ≤ GF + 5 buildings or not;
- on the favoured structural system (skeleton or the quasi dual) for providing lateral
resistance to wind loading of ≥ GF + 6 buildings?

According to the assembled ≥ GF + 6 building portfolio, Skopje’s design practice
favoured using the quasi-dual lateral resistance RC frame systems. Out of 23 buildings
identified, 22 were of the quasi-dual type and survived the earthquake with light damage.
The only one GF + 6 building was of skeleton type. Compared with quasi-dual buildings, it
showed very poor behaviour.

The post-WWII period in FNRY is the era of extensive construction of classic (≤ GF + 5
story) masonry and RC frame buildings of various heights. In Skopje, all damaged build-
ings were repaired and strengthened to comply with JUS39/64 seismic code. Even some
lightly damaged buildings were upgraded to JUS39/64 requirements. This was not the case
in other SFRY Federal units where post-WWII – 1964 construction still exists in a form as
they were designed and constructed. Particularly in Belgrade (RS), Zagreb (HR), Ljubljana
(SI), other Republic’s capitals, and larger cities. As a curiosity, we mention the GF + 12
masonry tower building constructed in Ljubljana before 1964.

JUS61/48 wind design of pre-earthquake midrise RC frame structures

The characteristic wind pressure (w) JUS61/48 relates to the geographical zone, the height
of the building, and the degree of protection, Table 22. JUS61/48 maps the division of the
FNRY territory into three geographical zones of wind speed: (I) zone of moderately strong
winds; (II) zone of strong Kosava and Vardarac winds; and (III) zone of strong Bora winds, Fig. 10.

The practice in Skopje has been to design for tWc = 1.2 times the wind force, w, tabu-
lated in Table 22. The factor of 1.2 comes from the wind provisions requiring 0.8w for
pressure on the windward side (We) and 0.4w suction on the leeward side (Ws). Total

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9 The portfolio is assembled according to detailed descriptions of damaged buildings presented in Muto
et al. (1963), UYI (1964), Sozen (1964), Berg (1964 and 1965), Kapsarov (1972), and Kodelja (1974).
10 https://glossary.ametsoc.org/wiki/Kossava.
11 https://glossary.ametsoc.org/wiki/Vardar.
12 https://glossary.ametsoc.org/wiki/Bora.
wind base shear, $F_{bw}$, is a product of the total wind coefficient, $tWc$, and the total area of exposed building surface, $Aw$ (Fig. 11).

For the flexible buildings discussed in Sect. 4.1.2, Table 23 summarizes the geometry of windward and leeward surfaces (TL, H, and Aw), buildings’ exposure characteristics (We, Table 22), calculates directional wind base shears, $F_{bw}$, and compares them with seismic base shears (Table 18, Sect. 4.1.2) calculated by JUS39/64 and JUS31/81 seismic design codes for medium (GT2) ground type. All discussed buildings are flat roof structures designed for medium (GT2) ground type. All symbols are detailed in Table 23.

Comparatively, the ratios between JUS39/64 and JUS31/81 seismic base shears relative to JUS61/48 wind base shear are 2.40 to 3.62 and 2.63 to 3.8 times larger, respectively.

Although designed by very low, wind-based base shear, Skopje’s midrise (6–14 stories) RC frame buildings performed well in the 1963 earthquake. A number of buildings were moderately damaged; however, the vast majority had survived the earthquake with light or very light damage.

### Appendix C Derivation of $RK_\eta$ quotient

**Table 19** Computed $RK_{\eta GTi}$ quotients for JTB, DTB, and B10 buildings for closest period larger than 0.5s

| Selected Building | T [s] | $R_{p1} = 4 (K_{p1} = 1)$ | $GT_1$ | $GT_2$ | $GT_3$ | $\min GT_i$ |
|-------------------|-------|--------------------------|--------|--------|--------|--------------|
| JTB, x            | 0.640 | 0.847                    | 0.867  | 0.723  | 0.723  |
| JTB, y            | 0.580 | 0.858                    | 0.796  | 0.664  | 0.664  |
| DTB               | 0.514 | 0.918                    | 0.755  | 0.629  | 0.629  |
| B10               | 0.580 | 1.045                    | 0.970  | 0.809  | 0.809  |

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

| Era            | Enforcement span | JUS | SZM  | CRC       |
|----------------|------------------|-----|------|-----------|
| CDN            | Pre-code         | <1948| n/a  | n/a       |
| CDL            | Low code         | 1948–1964| JUS61/48| SZM1948  | CRC46/47  |
| CDM            | Moderate-code    | 1965–1982| JUS39/64| SZM1950  | CRC51/71  |
| CDH            | High code        | 1983–2023| JUS31/81| SZM1982  | CRC11/87  |
|                |                  |     | SZM1987|           |           |
| Eurocode system|                  | ≥2020| MKS EN 1998–1| MKS EN 1998-1/NA | MKS EN 1992-1-1 |

JUS39/64 defines the $<t^{th}>$ mode seismic action at story $<k>$
The base shear, $F_b$, for fundamental mode shape ($i = 1$) approximated by horizontal displacements increasing linearly along with the height of the building (Eq. 13)

$$S_{i,k} = K_s \beta_i K_{\eta_i} w_k$$  \hspace{1cm} (C.1)

The base shear, $F_b$, for fundamental mode shape ($i = 1$) approximated by horizontal displacements increasing linearly along with the height of the building (Eq. 13)

$$F_b = \sum_{k=1}^{n} F_k = K_s \cdot \beta \cdot \sum_{k=1}^{n} h_k \sum_{j=1}^{n} w_j h_j^2 \cdot w_k$$ \hspace{1cm} (C.2)

and $w = w_1 = w_2 = \ldots = w_N$, $W = nw$, $h = h_1 = h_2 = h_3 \ldots h_N$, and $h_k = kh$

$$F_b = K_s \cdot \beta \cdot w \sum_{k=1}^{n} k h \sum_{j=1}^{n} j w_j h_j^2 \cdot w_k = K_s \cdot \beta \cdot w \sum_{k=1}^{n} K_s \cdot \beta \cdot w \sum_{j=1}^{n} j w_j^2 \cdot w_k$$ \hspace{1cm} (C.3)

$$F_b = K_s \cdot \beta \cdot \frac{3}{2n+1} \sum_{k=1}^{n} k = K_s \cdot \beta \cdot w \cdot \frac{3 (n+1)}{2 (2n+1)} = K_s \beta K_{\eta_i} W$$

where

| Table 21 | Pre-1964 Design and construction in Skopje and Macedonia, including enforcement times of JUS, SZM, and CRC standards |
|---|---|---|---|
| Era | Enforcement span | JUS | SZM | CRC |
| ≤ GF+5 Masonry and RC Frame buildings | | | | |
| CDN | Pre-code | <1948 1948–1964 | JUS61/48 | SZM1948 | CRC46/47 |
| ≥ GF+6 RC Frame buildings | | | | |
| CDL | Low code | 1948–1964 | JUS61/48 | SZM1948 | CRC46/47 |

| Table 23 | Geometry of JTB, DTB, and B10 midrises and comparison between seismic and wind base shears |
|---|---|---|---|---|---|---|---|
| Bldg | We | D | W | TL | H | Aw | w | tWc | Fbw | % W (Ratio) |
| JUS61/48 | JUS39/64 | JUS31/81 |
| JTB | SP | T | 1,802 | 19.60 | 19.3 | 378 | 90 | 1.2 | 40.85 | 2.27 | 6.6 (2.91) | 6.7 (2.95) |
| JTB | SP | L | 1,802 | 15.30 | 19.3 | 295 | 90 | 1.2 | 31.89 | 1.77 | 6.4 (3.62) | 6.3 (3.56) |
| DTB | NP | L/T | 6,920 | 22.85 | 43.0 | 983 | 105 | 1.2 | 123.80 | 1.79 | 4.3 (2.40) | 4.7 (2.63) |
| B10 | NP | T | 2,880 | 21.40 | 30.0 | 642 | 105 | 1.2 | 80.89 | 2.81 | 5.4 (3.16) | 6.5 (3.80) |
| B10 | NP | L | 2,880 | 13.00 | 30.0 | 390 | 105 | 1.2 | 49.14 | 1.71 | Not analysed |

We, Wind exposure type, Table 22; D, Direction of assessment; (L-Longitudinal, T-Transversal); W, Building mass, in tons; TL, Total length, in metres; H, Maximum height, in metres; Aw, Wind exposed area, in square metres; w, Characteristic wind pressure, in kN/m²; tWc, Total wind coefficient; tWc = We +Ws; We = 0.8 (External pressure coefficient); Ws = 0.4 (Internal pressure /suction/ coefficient); Fbw, Wind base shear, in kN;
Fig. 10 JUS61/48 Wind speed map

\[ K_{64}^\eta = \frac{3(n + 1)}{2^{2n + 1}} \]  \hspace{1cm} (C.4)

Table 22 Characteristic wind pressure by geographical zones, the height of the building, and the degree of protection

| Building height above ground (m) | Type of wind exposure (We) | Characteristic wind pressure w [kN/m²] for geographic zone |
|----------------------------------|---------------------------|----------------------------------------------------------|
| < 10                             | Protected                 | I  30  40  55                                            |
|                                  | Semi protected            | II 40  55  80                                           |
|                                  | Not protected             | III 45  70  110                                         |
| 10–30                            | Semi protected            | I  50  75  110                                         |
|                                  | Not protected             | II 60  90  130                                         |
| 30–60                            | Not protected             | III 70  105  150                                      |
| > 60                             | Not protected             | I  80  120  170                                        |

Protected (P): Buildings up to 10 m in height being erected in settlements between tall buildings or walls, in dense, high forests, or in another location that is entirely and permanently protected from strong winds

Semi protected (SP): Buildings 10–30 m in high in settlements, forests, or valleys protected from the strongest wind effects

Not protected (NP): Stand-alone buildings in unprotected locations (on a hill or in a plain) exposed to full wind from any side; all not protected mountain places of altitude > 800 m
Under the already introduced assumptions, JUS31/81 defines directly base shear

\[ F_b = KW = K_O \cdot K_s \cdot K_P \cdot K_D \cdot W = K_O \cdot K_s \cdot K_P \cdot K_D \cdot K_{\eta} \cdot W \]  

(C.5)

For the JUS31/81 ELF method the \( K_{\eta} \) is a dummy variable with a value of < 1>

\[ K_{\eta}^{81} = 1 \]  

(C.6)

Cancellation of the influence of other coefficients, \( RK_{\eta} \) is obtained as a quotient between \( K_{\eta}^{81} \) and \( K_{\eta}^{64} \) modal shape coefficients, hence

\[ RK_{\eta} = \frac{K_{\eta}^{81}}{K_{\eta}^{64}} = \frac{1}{\frac{1}{2} \frac{(n+1)}{2n+1}} = \frac{2}{3} \frac{2n+1}{n+1} \]  

(C.7)

Closed-form solutions of the following summations:

\[ \sum_{j=1}^{n} k = \frac{n(n+1)}{2} \quad \sum_{j=1}^{n} k^2 = \frac{n(n+1)(2n+1)}{6} \]

are available from an abundance of Internet sites; here is noted the only one: https://brilliant.org/wiki/sum-of-n-n2-or-n3/

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