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DEVELOPMENT OF K QUOTIENT METHODOLOGY FOR INTER-CODE LATERAL FORCE COMPARISON

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Abstract

The purpose of this paper is to provide clarification to individuals attempting to create models for seismic exposure as well as seismic vulnerability and risk, based on the seismic lateral loads established by the Yugoslavian seismic design codes. The particular focus of the research are the JUS 39/64 and JUS 31/81 seismic design codes, since they were active codes for a significant period of time, in which a lot of the structures still existing today in Macedonia as well as Yugoslavia were built. The main focus of this paper is the development of the K-Quotient method, which aims to provide a direct comparison between the lateral load levels obtained by analysing structures using the mentioned seismic codes.

Keywords: Building codes, Seismic risk, Seismic exposure.

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 authority to regulate building construction is delegated to local jurisdictions. In others, building codes are developed by governmental agencies and are enforced nationwide. Former Yugoslavia, of which North Macedonia was a part of, used a combined model by enforcing seismic design codes nationwide but also delegating authority to regulate building construction to Federal units.

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

Works along this line are already ongoing on the European scale. The European Commission's Horizon 2020 SERA is the first concerted effort among the research community to produce a uniform European risk model (ESRM20). The European seismic risk assessment exposure model is presented in Crowley et al. (2020a and b)[1,2]. In support of Crowley et al., 2020b, Crowley et al., 2021[3] presents the "Model of seismic design lateral force levels for the existing reinforced concrete European building stock," Republic of North Macedonia included.

An understanding of the level of design of a given building class is fundamental for the development of vulnerability models that can represent the features of each design level (Borzi et al. 2008[4]; Verderame et al. 2010[5]; Romao et al. 2020[6]; Crowley et al. 2021) in national seismic risk assessment models.

Yugoslavia enforced the first earthquake design regulations JUS61/48[7] in 1948, Table 1. The second (JUS39/64) was introduced in 1963/1964 following the 1963 Skopje earthquake, at the time when IAEE recorded 25 to 27 seismic design codes worldwide. The third, JUS31/81, was introduced in 1981 (in effect from August 1982). Out of 63 codes currently recorded by IAEE, it is the 31st. By Official Gazette of RNM No. 211 of 02.09.2020, Macedonia adopted the Eurocode Design System MKC EN 1990 –

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MKC EN 1999[10], allowing the parallel use of JUS31/81 and all other related standards for a period of 5 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 in Zamolo, 2021 (in Croatian)[11]. However, the subject is not treated in such technical details as this paper does.

Table 1. Evolution of Yugoslavian technical regulations for earthquake resistant design

Regulation	**Abbreviated as**	**Level**
Provisional Technical Regulations for Building Loading (Official Gazette of FNRT 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
Regulations for Design and Construction of Structures in Seismic Regions (Official Gazette of SRM No. 33/63 of 24 September 1963) Translation of SI18/63	MK33/63	Republic, Macedonia
Provisional 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 (Not discussed in this contribution)	National SFRY
Regulations on Technical Norms for Design and Calculation of Civil Engineering Facilities in Seismic Regions; (1986, Draft)	JUSxx/86 (Not discussed in this contribution)	National SFRY
Eurocode Design System MKC EN 1990 – MKC EN 1999 (OGoRNM No. 211 of 02.09.2020 Eurocode 8: Design of structures for earthquake resistance - Part 1: General rules, seismic actions and rules for buildings	MKC EN 1998-1	National NRM

The lateral force coefficient K, Eq. 1, follows Crowley et al., 2021[3] formulation that coincides with the JUS31/81 definition.

$$K = K_O \cdot K_S \cdot K_P \cdot K_D \qquad (1)$$

K0 is a coefficient of building importance, Ks is a coefficient of seismic intensity, Kp 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 Kd 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., 2021.

The contribution also tends to resolve the “memory hole” effect that generations trained in ongoing Code have inappropriate experience 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

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models. Accordingly, the second primary objective of this contribution is to clarify and save specific evolutionary facts for seismic design in RNM (Republic of North Macedonia) and SFRY (SFR Yugoslavia) from the memory hole.

2. K – Quotient method

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>20m, or n>5 stories) cannot be made. The comparisons of computer-aided results can be made; however, the insight in behaviour and contribution of some coefficients constituting the seismic coefficient will be lost. For stiff buildings (H≤20m, or n≤5 stories), both codes prescribe the use of a simplified method. JUS39/64 calculates the story shears F_k^{64} according to Eq. 2 and then by sums them as the base shear F_b^{64} using Eq.3. The period of n≤5 story buildings, calculated by T=0.1n, is T≤0.5s and fully corresponds to the JUS39/64 definition of "stiff structure".

$$S_{ik}^{64} = K_i^{64} \cdot w_k = K_S^{64} \cdot S^{64} \cdot S_{ie}^{64} \cdot K_{i\eta}^{64} \cdot w_k \tag{2}$$

$$F_b^{64} = \sum_{k=1}^{n} F_k^{64} \tag{3}$$

Where K_S^{64} is a factor which relates to the site seismicity as well as the importance factor of the structure, S^{64} is a soil amplification factor, S_{ie}^{64} is a dynamic coefficient and $K_{i\eta}^{64}$ is a coefficient which takes into account the modal shape. A more detailed explanation of the JUS39/64[8] and JUS31/81 seismic design codes can be found in Milutinovic et. al. (2022)[12].

The most efficient way to compare or scale the values of base shears defined by two or more codes is taking the appropriate ratio. The ratio, RK_b, defined by Eq. 4 expresses the F_b^{81} base shear in terms of F_b^{64}.

$$RK_b = \frac{F_b^{81}}{F_b^{64}} = \frac{K^{81}}{K^{64}} = \frac{K_O^{81}}{K_O^{64}} \frac{K_P^{81}}{K_P^{64}} \frac{S^{81}}{S^{64}} \frac{S_e^{81}}{S_e^{64}} \frac{K_S^{81}}{K_S^{64}} \frac{K_\eta^{81}}{K_\eta^{64}} = RK_O \, RK_P \, RS \, RS_e \, RK_S \, RK_\eta \tag{4}$$

If, for a particular Seismic Zone (K_s), Building Importance Category (K_0), site Ground Conditions (S), Building Type (K_p) and Number of Stories (n), the RK_b is:

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

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

The RK_b of Eq. 4 is a product of six quotients (ratios). Accordingly, this contribution terms it K – Quotient method.

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

2.1. Stiff Buildings

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

$$F_b^{81} = K^{81} \cdot W = K_0^{81} \cdot K_P^{81} \cdot K_S^{81} \cdot S^{81} \cdot S_e^{81} \cdot K_{i\eta}^{81} \cdot W \tag{5}$$

The period of n≤5 story buildings is in the range 0-0.5s. It is comprised of all three $S\ e81$ plateaus and the $S\ e64$ plateau. Thus $S_e^{81} = S_e^{64} = 1$. As discussed above, for using a simplified method JUS39/64 (Article 2.6) request that S_e^{64} spectral shape is multiplied by a factor of 1.5. Thus,

$$RS_e = \frac{S_e^{81}}{S_e^{64}} = \frac{1}{1.5} = \frac{2}{3} \tag{6}$$

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

$$RK_S = \frac{K_S^{81}}{K_S^{64}} = \frac{PGA^{81}}{PGA^{64}} = 1 \tag{7}$$

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

$$K_\eta^{64} = \sum_{k=1}^{n} K_{k\eta}^{64} = 1.5\,\frac{n\,(n+1)}{2n+1} \tag{8}$$

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

$$RK_\eta = \frac{K_\eta^{81}}{K_\eta^{64}} = \frac{2}{3}\,\frac{2n+1}{n+1} \tag{9}$$

Defined RK_0, RS, RK_p, and RK_η ratios are displayed in Tables 13 to 16, respectively. Based on Eq. 7 and 8, Eq. 4 reduces to:

$$RK_b = \frac{2}{3}\,RK_O\,RK_P\,RS\,RK_\eta \tag{10}$$

Table 2 JUS39/64-JUS31/81 RK_O ratios

Building Importance Category		I_{MCS} **VII**	**VIII**	**IX**
1	RK01	0.75	0.75	1.00
2	RK02	1.00	1.00	1.00
3	RK03	0.75	1.50	1.50

Table 3 JUS39/64-JUS31/81 RS ratios

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

Table 4 JUS39/64-JUS31/81 RK_P ratios

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

Table 5 JUS39/64-JUS31/81 RK_η ratio

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

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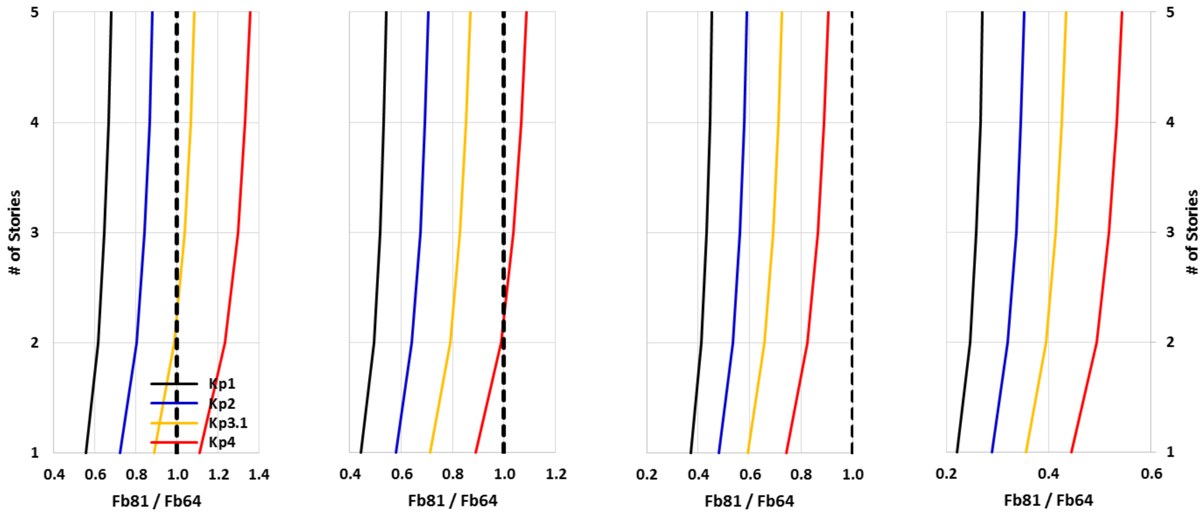
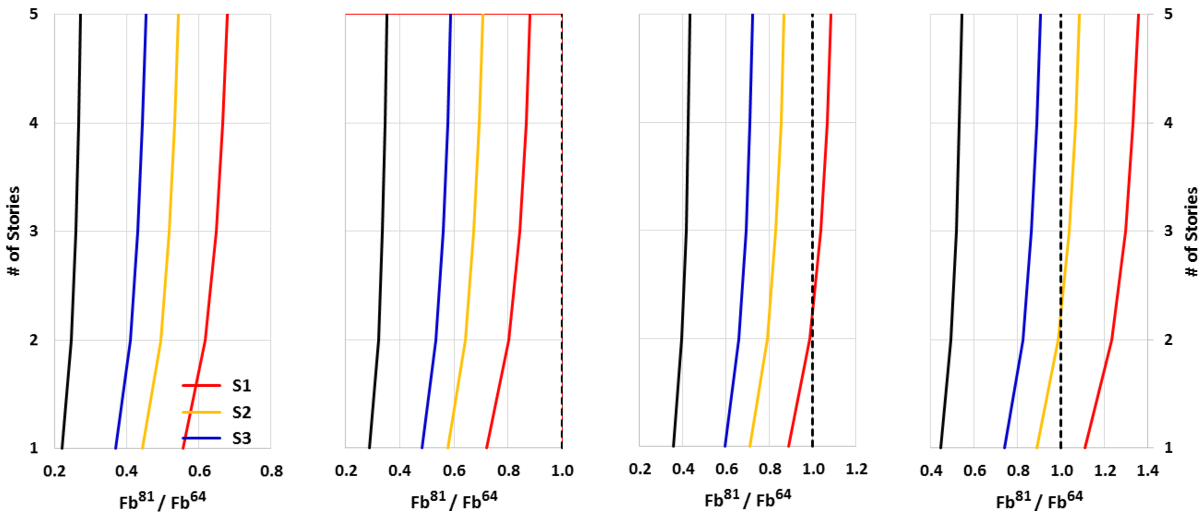


Ground type $GT_1(S_1)$ Ground type $GT_2(S_2)$ Ground type $GT_3(S_3)$ Ground type $GT_4(S_4)$

Fig. 1 Behaviour of K_b by number of stories, ground types, and structural typology;
Plots by Ground types



Structural Type K_{p1} Structural Type K_{p2} Structural Type $K_{p3.1}$ Structural Type K_{p4}

Fig. 2 Behaviour of K_b by number of stories, Ground Categories, and structural typology
Plots by structural typology

From Tables 2 to 5, it is observable that RK_P and RK_η ratios, both ≥1 tend to increase RK_b. On the other hand, RS and RS_e (Eq. 6) ratios tend to decrease it. RKO ratio, Table 2, for Importance Class I buildings decrease RK_b in $IMCS$ VII and VIII seismic zones whereas stipulate it for Importance Class III buildings in $IMCS$ (Importance class seismicity) VIII and IX zones.

Behaviour of RK_b by ground categories are presented in Figs. 1 and by structural typology in Figs. 2. Data used to generate Figs. 1 and 2 are tabulated in Table 6.

From Figs. 1 and 2, it is evident that JUS31/81 is superior only for all K_{p4} structures on $GT1$ ground category and n=3-5 stories $K_{p3}.1$ structures on $GT2$ ground category.

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Table 6 K conversion coefficient values

VIII	Soil Factor S				Structural Type K_P			
# of Stories	S_1	S_2	S_3	S_4	K_{P1}	K_{P2}	$K_{P3.1}$	K_{P4}
	$RK_{P1} = 1.0$				$S_1 = 0.8333$			
5	0.6790	0.5432	0.4527	0.2713	0.6790	0.8827	1.0864	1.3580
4	0.6666	0.5334	0.4445	0.2664	0.6666	0.8666	1.0666	1.3333
3	0.6481	0.5185	0.4321	0.2590	0.6481	0.8426	1.0370	1.2962
2	0.6173	0.4939	0.4116	0.2467	0.6173	0.8024	0.9876	1.2345
1	0.5555	0.4445	0.3704	0.2220	0.5555	0.7222	0.8889	1.1111
	$RK_{P2} = 1.3$				$S_2 = 0.6667$			
5	0.8827	0.7062	0.5885	0.3527	0.5432	0.7062	0.8692	1.0865
4	0.8666	0.6934	0.5778	0.3463	0.5334	0.6934	0.8534	1.0667
3	0.8426	0.6741	0.5618	0.3367	0.5185	0.6741	0.8297	1.0371
2	0.8024	0.6420	0.5350	0.3207	0.4939	0.6420	0.7902	0.9877
1	0.8827	0.7062	0.5885	0.3527	0.4445	0.5778	0.7111	0.8889
	$RK_{P3.1} = 1.6$				$S_3 = 0.5556$			
5	1.0864	0.8692	0.7243	0.4341	0.4527	0.5885	0.7243	0.9054
4	1.0666	0.8534	0.7112	0.4262	0.4445	0.5778	0.7112	0.8890
3	1.0370	0.8297	0.6914	0.4144	0.4321	0.5618	0.6914	0.8643
2	0.9876	0.7902	0.6585	0.3947	0.4116	0.5350	0.6585	0.8231
1	0.8889	0.7111	0.5926	0.3552	0.3704	0.4815	0.5926	0.7408
	$RK_{P4} = 2.0$				$S_4 = 0.3333$			
5	1.3580	1.0865	0.9054	0.5427	0.2713	0.3527	0.4341	0.5427
4	1.3333	1.0667	0.8890	0.5328	0.2664	0.3463	0.4262	0.5328
3	1.2962	1.0371	0.8643	0.5180	0.2590	0.3367	0.4144	0.5180
2	1.2345	0.9877	0.8231	0.4933	0.2467	0.3207	0.3947	0.4933
1	1.1111	0.8889	0.7408	0.4440	0.2220	0.2886	0.3552	0.4440

JUS31/81 base shear prevail JUS39/64

Cases calculated below

The proposed K-Quotient method is easily applicable for comparing ELF(Equivalent Lateral Force)-based base shears. Fig. 3 is a tabular presentation of the procedure proposed by Eqs. 4 and 10.

RK_b =

RS_e
2/3

(Eq. 25)

SZ	RK_S
VII	1.0
VIII	1.0
IX	1.0

(Eq. 26)

RK_O	VII	VIII	IX
RK01	0.75	0.75	1.00
RK02	1.00	1.00	1.00
RK03	0.75	1.50	1.50

(Table 13)

S	RS
S1	0.8333
S2	0.6667
S3	0.5556
S4	0.3333

(Table 14)

K_P	RK_P
KP1	1.0
KP2	1.3
KP3.1	1.6
KP3.2	1.0
KP4	2.0

(Table 15

$K\eta$	RK_η
η5	1.2222
η4	1.2000
η3	1.1167
η2	1.1111
η1	1.0000

(Table 16)

Fig. 3 Break down on RS_e, RK_S, RK_O, RS, RK_P and RK_η ratios constituting K transformation coefficient

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For example, for Category II ($RKO1$ = 1.00, Table 2), 5 story (n = 5, $\eta5$ = 1.2222, Table 5), RC structure ($KP1$ = 1.00, Table 4), located in the seismic zone of VIII degree intensity (RKS = 1.00), sited on medium ground conditions ($S2$ = 0.6667, Table 3), RKb calculated by order as presented in Fig. 3, is:

RKb = (2/3) · 1.00 · 1.00 · 0.667 · 1.00 · 1.2222 = 0.5432, and $\boldsymbol{Fb81}$= 0.5432 $\boldsymbol{Fb}64$

If the building is with stiffness discontinuity (e.g. soft story, $KP4$ = 2.00), the RKb would be:

RKb = (2/3) · 1.00 · 1.00 · 0.667 · 2.00 · 1.2222 = 1.0865, and $\boldsymbol{Fb81}$= 1.0865 $\boldsymbol{Fb}64$

For four story (n=4, $\eta4$ = 1.2000) masonry building strengthened by tie-columns (K$P3.1$ = 1.6) and all other coefficients as in the previous case, RKb is

RKb = (2/3) · 1.00 · 1.00 · 0.667 · 1.60 · 1.2000 = 0.8534, and $\boldsymbol{Fb81}$ =0.8534 $\boldsymbol{Fb}64$

Relative to JUS31/81, the JUS39/64 simplified method is over-conservative for stiff building. The largest conservativism is associated with lateral force coefficient associated to GT3 and GT4 ground conditions (0.5556 and 0.3333, respectively), then to GT2 (0.6667), and finally to GT1 (0.8333).

2.2. Flexible Buildings

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

DTB (Tomic, 2018) [13] building is 43 metres in height GF+ME+12 story tower with T_x=T_y=1.848s. JTB (Trajcevski, 2018) [14] building is a GF+ME+4 story block type building 19.3 metres in height with T_x =1.108s and T_y=1.048s. ME is the mezzanine story. The floor plan of both buildings is presented in Figs. 4 and 5. Both were designed and executed according to JUS39/64 at the beginning of the 1970s'. In addition, analysed is 10 storey 5 bay (in longitudinal) and 3 bay (in transverse) direction. The building (B10) is a dual RC frame/wall system with a shear wall placed in the middle span of two internal frames.

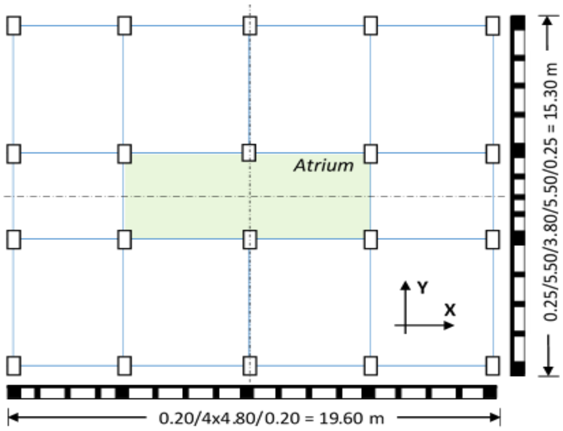

Fig. 4 JTB Building, characteristic floor plan

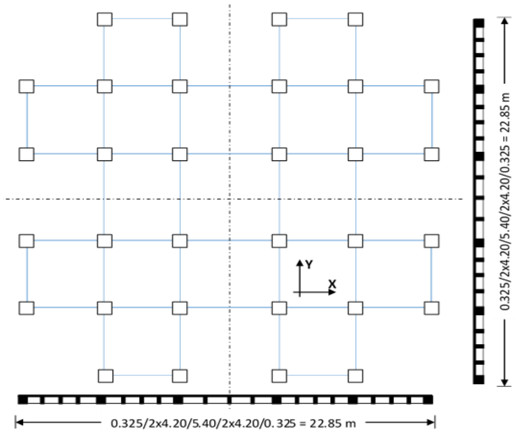

Fig. 5 DTB Building, characteristic floor plan

The analyses are conducted for all three ground types. The results are presented in Table 7. RK_b values are of the same order, irrespective of the fact that JUS39/64 are derived from 3 mode MRS (Modal Response Spectrum) and JUS31/81 by ELF method. Common to both JTB and DTB buildings is the inverse trending of ground type dependent RK_s relative to trends outlined by the simplified method. For $GT1$ ground type, both RK_{bs} are ≤1, hence JUS38/64 did prescribe the higher base shear than JUS31/81. The RK_b≥1.0 are attributed to $GT3$ ground typology demonstrating that JUS31/81 overwhelms the JUS39/64 base shear.

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Comparatively JUS39/64 and JUS31/81 base shears are about the same for $GT2$. However, the results for B10 building, although following the same pattern of GT dependency, are all above the RK_b≥1.0.

Table 7 Lateral force ratio (RK_b) for JTB and DTB buildings

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

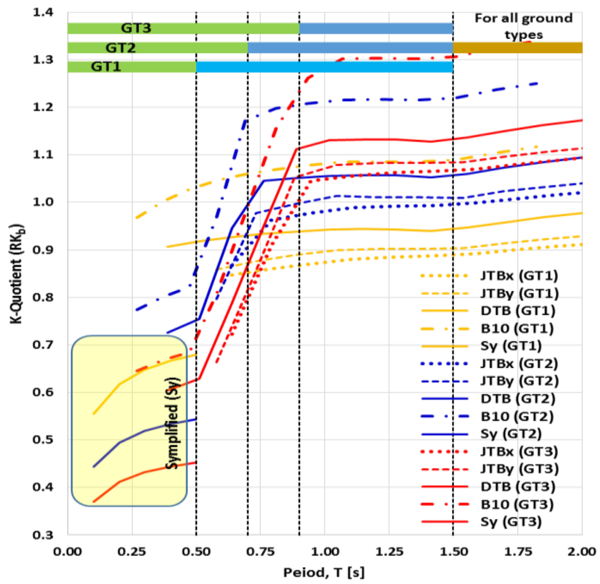

Fig. 6 Spectral behaviour of lateral force ratio (RK_b) for stiff and flexible Buildings

The analyses are conducted for all three ground types. The results are presented in Table 7. RK values are of the same order, irrespective of the fact that JUS39/64 are derived from 3 mode MRS and JUS31/81 by ELF method. Common to both JTB and DTB buildings is the inverse trending of ground type-dependent RKbs relative to trends outlined by the simplified method. For $GT1$ ground type, both RK_{bs} are ≤1, hence JUS38/64 did prescribe the higher base shear than JUS31/81. The RK_b≥1.0 are attributed to $GT3$ ground typology demonstrating that JUS31/81 overwhelms the JUS39/64 base shear. Comparatively JUS39/64 and JUS31/81 base shears are about the same for $GT2$. However, the results for B10 building, although following the same GT dependency patterns, are all above the RK_b≥1.0.

To further elucidate the behaviour of RK_b over the velocity and displacement spectral regions, a numerical triage has been performed on JTB, RTB, and B10 buildings. For all three buildings,

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formulated are a number of computational models by simply adding or cancelling out stories. The JTB building from six was upgraded up to 14, DTB from 14 to 20, and B10 to 15 stories by replicating the top story mass and stiffness characteristics of the basic 6, 14, and 10 story computational models. Downscaling of computational models is realized by simply removing stories down to the 3rd story. In such a way, formulated are 43 models in total, 18 for DTB building, 13 for B10, and two times 12 (for each direction separately) computational models that have been analysed for all three ground types. All computations, 165 in total, were conducted by in-house developed Matlab script CodeSDB. The Authors are fully aware that formulation of such computational models influence their Eigenvalue characteristics in general and the behaviour of $K_{\eta 64}$ coefficient in particular. Apart from indicated deficiency of models used, the analyses, Fig. 6:

- Confirmed that in the period range 0.9 - 2.0s+ JUS31/81 overwhelms the JUS39/64 base shear for GT3 ground type. Hence over the entire velocity and displacement spectral regions of GT3 spectral shape the RKb > 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 TC,($GT2$/$GT3$) - 2.0+ seconds range JUS39/64 overwhelms the JUS81/81 base shear. For DTB building with the first period being in the displacement spectral region (T_1=1.848s), RKb converge to 1, whereas for JTB building (T_{1x}//T_{1y}=1.108/1.048s) it converges to value close to 0.95s. The exception is the B10 building for which RK_b > 1 over the entire $TC,GT2 - 2.0s$ period range;
- The order of RKb relations over the TC,($GT2$/$GT3$) - 2.0s+ period range is $\boldsymbol{RK_b,GT3}$ > $RK_b,GT2$ > $RK_b,GT1$;
- Over the 0.5-TC,($GT2$/$GT3$), the RKb relations reorder themselves so that $\boldsymbol{RKb,GT1}$ > $RKb,GT2$ > $RKb,GT3$;
- The simplified calculation of base shear for "stiff" buildings is over conservative relative to calculating them by MRS method; and
- The sloping of RKb,GTi,in post TCi velocity and over the entire displacement (TD≥1.5s) regions is influenced by $K\eta 81 = n$ coefficient. The effect is particularly pronounced in TD≥1.5s range where RKb increases monotonically and almost linearly.

Table 8 Computed $RK_{b,GTi,}$ quotients for JTB, DTB, and B10 buildings for closest period larger than 0.5s

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

All discussed values are relevant only for $KP1$ structural typology ($KP1$ = 1, $RP1$ = 4), which was and still is dominantly implemented RC typology in RNM and Yugoslavia since the 1964 year. The minimum calculated $K - Quotients$ for all ground types at periods slightly over 0.5s are presented in Table 8. $minGTi$ values for $KP1$ structural typology ($KP1$ = 1) are all <1 causing the full prevalence of JUS39/64 over JUS31/81 as well as for three cases of $KP2$ structural typology ($RP2$ ≈ 3.1). Irrespective of the ground typology, for one $KP2$ case and all $KP3$ ($RP3$ = 2.5) and $KP4$ ($RP4$ = 2) the JUS31/81 will prevail JUS39/64 base shear.

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As shown, the behaviour of RKb is quite irregular over the analysed spectral range of 0.5-2.0s and does not provide a clear picture to decide on the seismic design level of both codes. RKb is a method of analysis and ground type dependent. For $GT2$ located buildings JUS31/81 consistently overwhelms the JUS39/64 base shear. For other ground types the RKb behaviour is variable. For some of analysed buildings $RKb > 1$ whereas for other $RKb < 1$. The result of this irregularity is that RKb cannot discriminate which code is superior to the other. Hence the irregular behaviour of $K - Quotient$ over the entire analysed spectral range does not allow its exclusive use as a metric for scaling JUS31/81 and JUS39/64 seismic design levels.

3. Challenges for Vulnerability Assessors and Exposure Modellers

Due to discussed irregular behaviour the $Kb - Quotient$ and resulting base shears are not suitable parameters for deciding mutual seismic design levels of JUS39/64 and JUS31/81 codes. The novelty of JUS31/81 relative to JUS39/64 code was that it:

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

On this ground, rather than on the value of lateral force coefficients or $K - Quotient$ behaviour, Milutinovic and Trendafiloski, 2003 adopted and assigned four design levels to seismic design codes historically used in Macedonia and Yugoslavia, Table 9.

Table 9 Design and construction eras based on enforcement time of JUS, SZM, & CRC standards

<table>
<tr><th colspan="2">Era</th><th>Enforcement span</th><th>JUS</th><th>SZM</th><th>CRC</th></tr>
<tr><td></td><td>Pre-code</td><td>< 1948</td><td colspan="3">n/a</td></tr>
<tr><td>CDL</td><td>Low code</td><td>1948-1964</td><td>JUS61/48</td><td>SZM1948</td><td rowspan="2">CRC46/47</td></tr>
<tr><td rowspan="2">CDM</td><td rowspan="2">Moderate-code</td><td rowspan="2">1965-1982</td><td rowspan="2">JUS39/64</td><td rowspan="2">SZM1950</td></tr>
<tr><td rowspan="2">CRC51/71</td></tr>
<tr><td rowspan="3">CDH</td><td rowspan="3">High code</td><td rowspan="3">1983-2025</td><td rowspan="3">JUS31/81</td><td rowspan="2">SZM1982</td></tr>
<tr><td rowspan="2">CRC11/87</td></tr>
<tr><td>SZM1990</td></tr>
<tr><td></td><td>Eurocode system</td><td>≥ 2020</td><td>MKS EN 1998-1</td><td>MKS EN 1998-1/NA</td><td>MKS EN 1992-1-1</td></tr>
</table>

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 9. The implication of such non harmonized enforcement timing is that the JUS39/64and JUS31/81 are both associated with two CRC codes, hence with potentially two vulnerability levels dictated solely by their provisions.

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4. Conclusions

The most relevant findings are as summarized below:

- Eq. 5 is not suitable for analysing and comparing Eurocodes wise the lateral force coefficients (K), although for comparing JUS39/64 and JUS31/81, it may be;
- The appropriate K comparison format is the R-code format. In particular, since EN 1998-1 (Eurocode 8) uses it, all lateral force coefficients and its constituents of previous codes can efficiently be compared with it

$$K = \gamma_I \cdot a_{gR} \cdot S \cdot \frac{1}{q} \cdot S_e(T) \tag{11}$$

  where γI is building importance factor, agR reference peak ground acceleration on type A ground, S soil factor, q behaviour factor and, Se (T) elastic horizontal ground acceleration response spectrum shape.

- $K - Quotient$ due to its irregular and ground-type-dependent behaviour over the entire spectral range does not allow its exclusive use as a metric for scaling JUS31/81 and JUS39/64 seismic design levels.
- Proposes and documents K-Quotient method that efficiently compares and intrinsically screens the formulation of all K constituents;
- 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 $KP1$ structural typology it prescribes from 1.2 to 3 times greater base shear;
- K-Quotient behaviour of "flexible" buildings varies with ground typology. For $GT2$ JUS31/81 base shear is consistently greater than of JUS39/64. For other ground conditions, K-Quotient varies in the range of 0.8 to 1.2. The discussed values are relevant only for $KP1$ structural types. For other structural types, JUS31/81 and JUS39/64 base shear prevalence is alternative;
- The minimum calculated $K - Quotients$ for periods slightly over 0.5s (Table 8) and ground types, for $KP1$ structural typology ($KP1$ = 1) are all <1 causing the full prevalence of JUS39/64 over the JUS31/81as well as for three cases of $KP2$ structural typology ($RP2 \approx 3.1$). Irrespective of the ground typology, for one $KP2$ case and all $KP3$ ($RP3 = 2.5$) and $KP4$ ($RP4 = 2$) cases the JUS31/81 prevails JUS39/64 base shear.
- $KP1$ structural types are mostly implemented systems in RNM and Yugoslavia since the 1964 year, hence the irregularity in the behaviour of $K - Quotient$ over the 0.5-2.5s+ period range prevent its exclusive use as a metrics for scaling JUS31/81 and JUS39/64 seismic design levels;
- The development of $K - Quotient$ values is based on actual data from only three buildings and related numerical simulations. The influence of CRC codes on vulnerability and resistance of the exposure is practically unknown. Further research along these two lines is indispensable.

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

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

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