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SEISMIC ASSESSMENT OF THE DAMAGE LIMIT STATES OF EXISTING HIGHWAY BRIDGES

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Abstract

The majority of existing bridges originate from various periods of design and construction. Due to their age, they have been exposed to environmental influences, often exacerbated by increased traffic loads and different hazards. For bridges located in areas prone to seismic activity, including our region, it is particularly important whether the structure was designed and built according to codes that do not meet modern seismic performance requirements. Environmental effects contribute to gradual deterioration, but de-icing salts and the aggressiveness of the environment lead to corrosion, which can often result in faster deterioration. Therefore, assessing the load-bearing capacity and seismic performance of beam bridges, which are the most common type of bridge, is crucial. For this purpose, it is necessary to apply appropriate assessment methods, especially for structures that were not built according to modern code requirements and for older bridges with a higher degree of deterioration and thus reduced load-bearing capacity.

This paper presents a review of relevant literature, codes, and recommendations related to the pre-earthquake evaluation of bridges from countries frequently affected by strong earthquakes, including the USA, Japan, India, New Zealand, Turkey, and others. Based on this and the analysis of fragility curves (for substructures), this paper lists some recommendations for bridge evaluation as a case study, serving as a basis for making decisions on the type of structural intervention on the bridge or its demolition and removal.

Considering that for bridges, unlike reinforced concrete buildings, the condition is set that the deck structure does not exhibit yielding, which dissipates the energy of an earthquake, it is recommended that yielding occurs first in the elements of the lower structure (columns, abutments, and even foundations). Therefore, numerical analyses are focused on the sub-structure.

A comparative numerical analysis of six bridge mid-column models, featuring identical superstructures but differing substructures was performed in the paper. The prestressed concrete bridges, spanning 88 meters with configurations of 24.0+40.0+24.0m, were designed according to both old JUS design standard (three models) and new EN standards (three models). The substructures consist of RC columns with foundations and a prestressed beam of variable height (1.40m to 2.60m). Models type 1 has massive rectangular mid-columns, while types 2 and 3 consist of two columns with a crossbeam—type 2 features rectangular columns, and type 3 has circular columns. Using nonlinear analysis, the study evaluates how these design choices impact seismic damage limit states and column responses for different damage limit states. The results show significant differences in structural resilience and vulnerability, emphasizing the importance of design factors in enhancing seismic performance and ensuring safety.

Keywords: Concrete road bridges, pre-earthquake evaluation, deterioration, damage state, performance, seismic fragility

1. Introduction

Seismic assessment analysis of an existing bridge is to determine the level of risk associated with loss of serviceability, severe damage, or collapse. It depends on the age of the bridge, soil conditions, structural type, and site seismicity. Seismicity includes site-specific information on the expected ground motion. Ideally, it should consist of a basic response spectrum shape, or shapes, with peak ground acceleration related to annual probability of exceedance, and should be appropriate to the soil conditions of the site. Vulnerability represents the susceptibility to damage or collapse and is related primarily to

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structural aspects. The vulnerability will depend on the structural form and topography. Importance relates to the consequences of damage or failure and typically includes consideration of traffic volume, type of crossing, length of detour resulting from bridge closure, and the significance of the bridge to lifeline operations such as the passage of emergency services vehicles to hospitals [1].

In the book [1], chapter 7 is devoted to the seismic assessment of existing buildings. A detailed methodology for the evaluation of bridges and their elements is presented in [2]. For the assessment and design of bridges in Europe, EN 1998 [3] is used. Additionally, the methodology for the design of concrete bridges is discussed in [1], [4], and [5], and retrofitting of bridges is analyzed in detail in [1]. In assessing the condition of bridges, evaluating their damage is crucial, as covered in works [6], [7]. Post-earthquake rapid assessment of bridges (Case study Glina County) is discussed in [8].

In most studies, it is shown that inspections of structures form the basis for assessing the performance of existing bridges. Inspection methods according to different codes are analyzed in [9], generally, and in [10] for the assessment and retrofitting using state-of-the-art approaches. For assessing the condition and durability of deteriorated bridges, the work [10] can be used, and for bridges exposed to chloride corrosion [11]. Corrosion of steel reinforcements is a major factor causing seismic performance deterioration of reinforced concrete (RC) bridges. This phenomenon is considered regarding the seismic performance assessment of aging RC bridges [12].

Corrosion of reinforcing steel is one of the primary sources of durability problems in reinforced concrete (RC) bridges. Authors of the paper [13] quoted that most structures designed and constructed after 1990 have performed well. However, there are a significant number of older major infrastructure artifacts that are located in an aggressive environment and are suffering from material aging and deterioration. This will increase the probability of failure of deteriorated structures in big earthquakes. It can also result in significant permanent damage in smaller earthquakes.

All bridges become more vulnerable to seismicity due to corrosion deterioration. Therefore, neglecting the effects of corrosion deterioration may lead to unconservative estimates of seismic damage risk [14].

Most existing bridges were designed according to outdated codes that did not address current requirements, including seismic resistance. In this regard, it is necessary to emphasize that bridges perform a strategic function in civil protection plans for emergency management and must remain functional in the aftermath of an extreme event. Therefore, the assessment of their potential performance in the case of ground motions is of great importance in earthquake-prone countries [15].

For more accurate assessments, various numerical and analytical procedures of different complexity and accuracy are used. Assistance in choosing analysis methods can be obtained using methods for fragility analysis. Fragility functions for assessing seismic risk of typical concrete bridges by means of nonlinear static analysis (NSA) and nonlinear dynamic analysis (NDA) are described in [16]. In the study [15], linear and nonlinear performance analyses of multi-span bridges are compared. Seismic resilience assessment of bridges considering both maximum and residual displacements is the subject of [16], and displacement-based seismic performance of RC bridge piers is discussed in [17].

Examples from the application of fragility curve development for seismic evaluation of RC bridges in Algeria are presented in [18]. Seismic fragility analysis of continuous girder bridges with varying pier heights is the subject of [2], and a bridge exposed to corrosion is discussed in [19]. A special case of seismic fragility and resilience improvement of conventional bridges supported by double-column piers is analyzed in [20], and seismic performance assessment of deteriorated two-span reinforced concrete bridges in [21]. Seismic vulnerability assessment of deteriorated bridges is analyzed in [22].

This paper presents a comparative numerical analysis of three bridge models which have identical super-structures but different sub-structures, specifically the supporting mid-columns. The analyzed prestressed concrete bridges span 88 meters and they are located on the same site. The design solutions for the frame structural system were selected based on terrain characteristics and project requirements, adhering to European standards EN1990 [23], EN1991 [24], EN1992 [25], and EN1998-2 [3]. Each bridge has two lanes (3.50 m width) and two pedestrian paths (1.5 m width) (Fig. 1), with spans of

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24.0+40.0+24.0m (Fig. 2). The substructure includes RC columns with foundations and a prestressed beam with a variable-height box cross-section (1.40m to 2.60m), cast in situ.

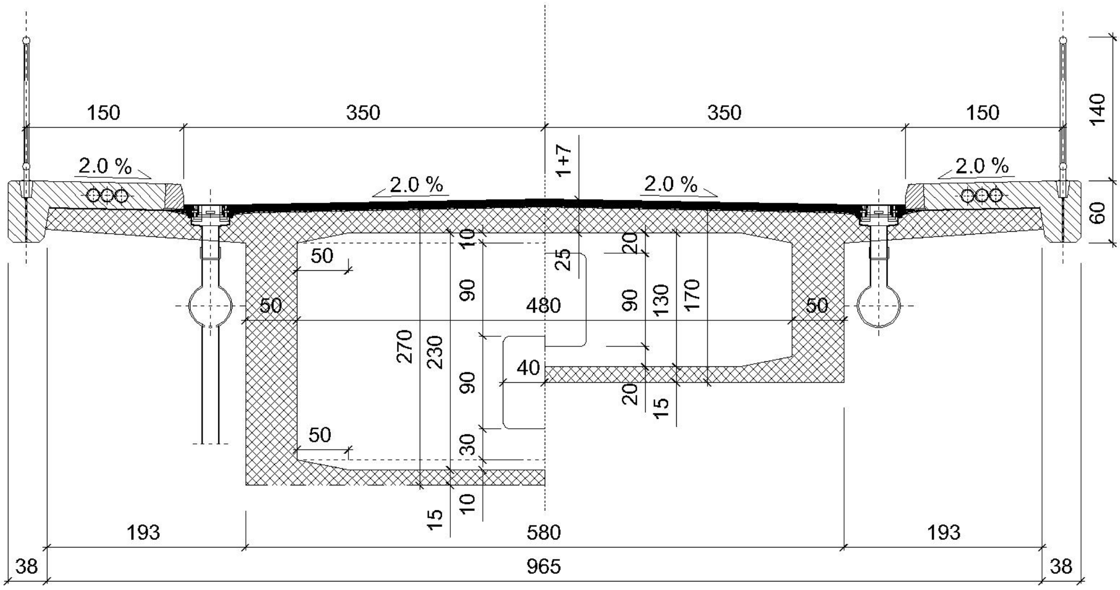

Figure 1. Upper deck cross-section of bridge models (M1, M2 and M3)

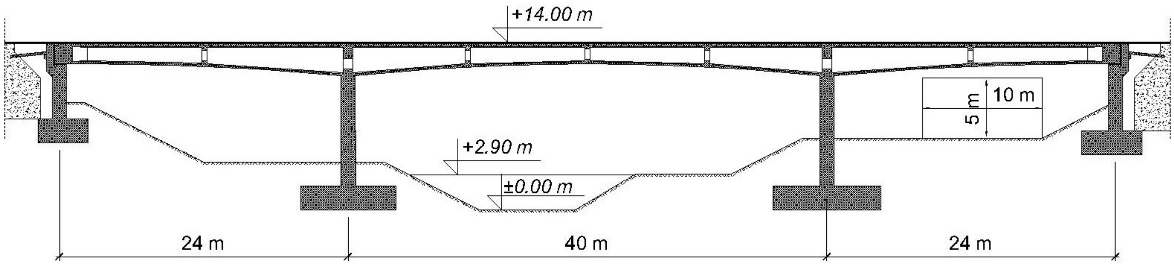

Figure 2. Bridge model 1 (M1) – longitudinal view

The paper analyzes how these design choices according to different (EN [3], [23-25] and JUS [26-28]) standards impact seismic fragility and bridge column responses using nonlinear analysis methods. Structural damage can lead to significant material loss, compromising the bridge's functions and posing risks to nearby areas. Therefore, damage limit states were calculated using NSA, which was conducted and compared to NDA results in the paper [29] and it gave a valid match of the calculated values.

Fragility curves were calculated using NDA results and earthquake time-history (TH) data. The structural responses of bridge columns were compared across different damage states, contributing to the assessment of seismic damage and resilience. The findings enhance understanding of differences between (EN [3], [23-25] and JUS [26-28]) standards and various design factors affect the seismic response and fragility of the bridge structural system.

2. Methodology of the analysis

2.1.Geometric and material properties of the structures

The analyzed concrete bridges, have an 88 meters span, with configurations of 24.0+40.0+24.0m (Fig. 1). In Model 1 (M1) in Fig. 3, the mid-columns are massive and designed as DCL and in models 2 (Fig. 4) and 3 (Fig. 5) (M2 and M3), the mid-columns are designed as DCM elements. For M2 and M3, the mid-columns construction comprises three elements. These consist of two piers and a crossbeam, which

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are rigidly connected to the main deck. In M2, the mid-columns have a rectangular cross-section, while in M3, the mid-columns have a circular cross-section. The crossbeams have a rectangular cross-section.

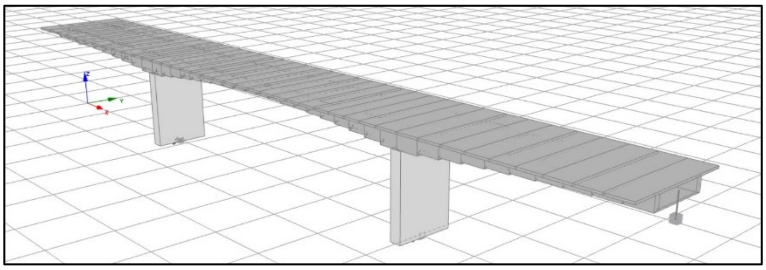

Figure 3. Bridge model 1 (M1), [32]

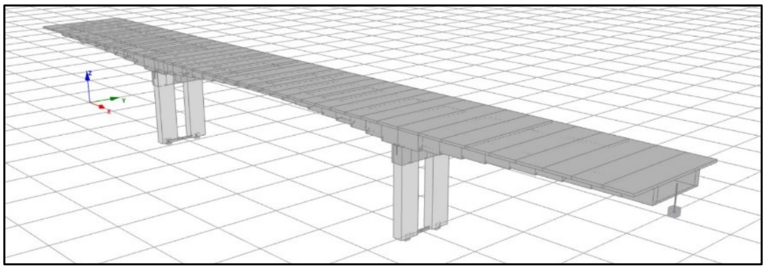

Figure 4. Bridge model 2 (M2), [32]

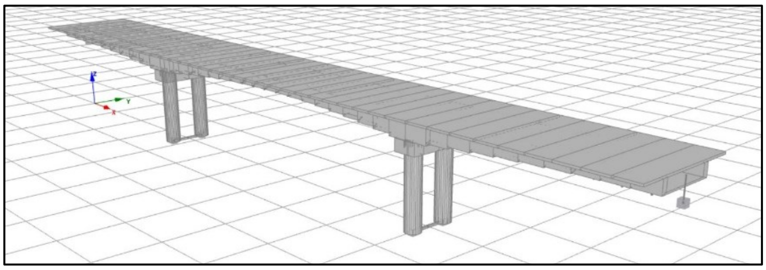

Figure 5. Bridge model 3 (M3), [32]

The bridge structures are of the rigid frame type, restrained perpendicular to the bridge alignment at the abutments. The mid-columns are fixed at the bottom and they all have a height of $H_C = 8.45$ m. Concrete of grade C30/37 and steel B500 for reinforcement were used in calculation according to EN standards [3], [23-25]. Concrete of grade MB40 (which has similar properties to C30/37) and steel RA400/500 for reinforcement were used in calculation according to JUS standards [26-28]. The bridge mid-columns in M1, M2, and M3 were designed in accordance with the both standards (EN [3], [23-25] and JUS [26-28]).

The calculations were carried out using the software packages *Radimpex Tower* [30] (linear analysis), *CSi SAP2000* [31] (damage limit state thresholds calculation), and and *Seismosoft Seismostruct* [32] (nonlinear analysis). For the calculation of seismic forces, 20% of traffic load was included in the seismic combination and modal analysis of the structures. The structures were designed to design spectrum type 1, soil type E, and horizontal design acceleration of 0.1g according to [3] and for soil of category III and seismic zone VIII, according to [26].

The assessment of damage limit states in the mid-columns was conducted on three different models to compare and analyze the values at which these damage limit states (LS) occur. The current modeling approach of analyzed structures (Table 1) provides a balanced framework for the assessment of bridge performance, with nonlinear analysis of critical components (mid-columns) and simplified modeling of other structural elements. Table 2 provides a detailed comparison of the geometric and material properties of the mid-columns in each model.

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Table 1. Structural element type used in the models

Element	Structural element type in the models
Mid-column	Nonlinear element with hinges
Mid-column foundation	Restraints
Cross-beam	Linear element
Bridge deck	Linear element
Abutments	Restraints

Table 2. Geometric and material properties of the mid-columns in each model

Cross-section properties	M1	M2	M3
	EN | JUS		
Ductility class:	DCL (q = 1.5) (EN)	DCM (q = 3.0) (EN)	DCM (q = 3.0) (EN)
Crossbeam:	/	d/b/l = 1.2/1.6/6.2m	d/b/l = 1.2/2.4/6.2m
Cross-section height: d [m]	6.0	2.0	Diameter D = 1.6m
Cross-section width: b [m]	1.2	1.0	
Longitudinal rebars in columns (EN | JUS):	132 Ø25 | 120 Ø22 As = 648.1 cm^2 | 456.0 cm^2	44 Ø25 | 44 Ø22 As = 216.0 cm^2 | 167.2 cm^2	36 Ø28 | 36 Ø22 As = 221.4 cm^2 | 136.8 cm^2
Stirrups in column:	Ø12/15 | Ø10/20, Confinement bars: 4 x 7 | 2 x 7	Ø12/15 | Ø10/20 Confinement bars: 5 x 8 | 1 x 5	Ø12/15 | Ø10/20 Confinement bars: 1 hoop

2.2. Damage limit state threshold values

Table 3 defines different levels of damage and their threshold values for the mid-columns of the bridge models and how these thresholds are used to evaluate the structural performance. To determine the damage limit LS values, NSA was performed. By analysing the response of the structures, the damage states for various conditions were assessed. Similar approach to the definition of damage or limit state values is used in paper [33], [29].

Table 3. Description of the damage state threshold values

Nr.	Limit state (LS)	Description	M1	M2	M3
	EN | JUS				
1	**Small damage (SD)**	Depends on steel and concrete values. SD occurs either in the first step, when reaching the reinforcement yield limit ($\varepsilon_s \geq \varepsilon_{sy}$) or the stress limit of concrete with maximum strength in the protective layer of concrete ($\varepsilon_c \geq \varepsilon_{c1}$).		$\varepsilon_s \geq \varepsilon_{sy} = 0.0025$ or $\varepsilon_c \geq \varepsilon_{c1} \approx 0.002$	
2	**Moderate damage (MD)**	This level of damage is assumed to occur when reaching the stress limit in the protective layer of concrete ($\varepsilon_c \geq \varepsilon_{cu,1}$) or the maximum stress of the confined concrete core ($\varepsilon_{c,c} \geq \varepsilon_{cc,1}$)	$\varepsilon_c \geq \varepsilon_{cu,1} = 0.0035$ or $\varepsilon_{c,c} \geq \varepsilon_{cc,1} = 0.0030\|0.0023$	$\varepsilon_c \geq \varepsilon_{cu,1} = 0.0035$ or $\varepsilon_{c,c} \geq \varepsilon_{cc,1} = 0.0037\|0.0022$	$\varepsilon_c \geq \varepsilon_{cu,1} = 0.0035$ or $\varepsilon_{c,c} \geq \varepsilon_{cc,1} = 0.0024\|0.0024$
3	**Extensive damage (ED)**	Occurs in the first step, when the ultimate stress is reached in the confined concrete core ($\varepsilon_{c,c} \geq \varepsilon_{cc,u}$).	$\varepsilon_{c,c} \geq \varepsilon_{cc,u} = 0.0069\|0.0049$	$\varepsilon_{c,c} \geq \varepsilon_{cc,u} = 0.0142\|0.0106$	$\varepsilon_{c,c} \geq \varepsilon_{cc,u} = 0.0077\|0.0065$
4	**Complete damage (CD)**	This state results in the case of tensile fracture of longitudinal reinforcement bars ($\varepsilon_s \geq \varepsilon_{s,u}$), the mid-column drift reaches ($\theta \geq \theta_u$) and the loss of system balance.	$\varepsilon_s \geq \varepsilon_{s,u} = 0.05$ or $\theta \geq \theta u$	$\varepsilon_s \geq \varepsilon_{s,u} = 0.05$ or $\theta \geq \theta u$	$\varepsilon_s \geq \varepsilon_{s,u} = 0.05$ or $\theta \geq \theta u$

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2.3. Incremental NDA

Nonlinear time-history analysis (THA) is conducted by using ten particular accelerograms [34-35] (Fig. 6 and Table 4) in only one direction, perpendicular to the bridge deck alignment, because of its simplicity and the wide application of this method of analysis.

The main criterion for selecting earthquake records was that the mean of the selected and scaled records should match the elastic response spectrum EN1998-1 [3] used in the analysis.

THA data were scaled with a common scale factor $F_S = 0.8$, which was obtained using the least-squares method. Scaled accelerograms are used for NDA, with the increment of $\Delta PGA = 0.1g$, in a total scaling factor range of 0.1–1.0g.

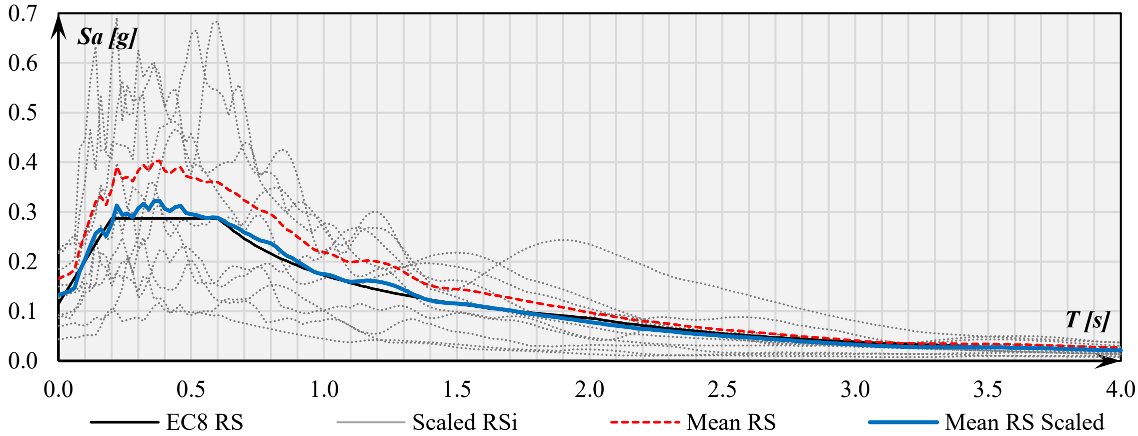

Figure 6. Response spectra used in the analysis (scaled RS_i, mean RS and mean scaled RS)

Table 4. Main properties of the earthquakes that were used in NDA

ID	Earthquake Location	Earthquake ID (Component/Orientation)	Station ID/Code	Date/Time	Mw	Original PHA [cm/s²]
EQ01	*Spitak, Armenia*	213 (Y)	173	07.12.1988. 07:41:24	6.7	179.580
EQ02	*Manjil, Western Iran*	230 (Y)	189	20.6.1990. 21:00:08	7.4	87.045
EQ03	*Umbria Marche, Central Italy*	286 (Y)	221	26.9.1997. 09:40:30	6.0	218.340
EQ04	*Umbria Marche, Central Italy*	286 (Y)	224	26.9.1997. 09:40:30	6.0	106.660
EQ05	*Alkion, Greece*	559 (X)	214	15.6.1995. 00:15:51	6.5	55.501
EQ06	*Düzce, Turkey*	497 (Y)	3139	12.11.1999. 16:57:20	7.2	112.320
EQ07	*Umbria, Central Italy*	EMSC-20161030_0000029 (N-S)	CNE	30.10.2016. 06:40:18	6.5	288.280
EQ08	*Emilia-Romagna, Italy*	IT-2012-0011 (N-S)	MOG0	29.5.2012. 07:00:02	6.0	167.075
EQ09	*Adana, Turkey*	TK-1998-0063 (E-W)	0105	27.6.1998. 13:55:53	6.2	271.955
EQ10	*Emilia-Romagna, Italy*	IT-2012-0011 (N-S)	MIR08	29.5.2012. 07:00:02	6.0	242.970

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3. Discussion of the results

3.1. Damage state values

The NSA results are shown in Fig. 7. Mid-column drift values for the analysed models for different damage limit states (LS) provide the insights into the seismic performance of the bridge structures designed according to EN [3], [23-25] and JUS [26-28].

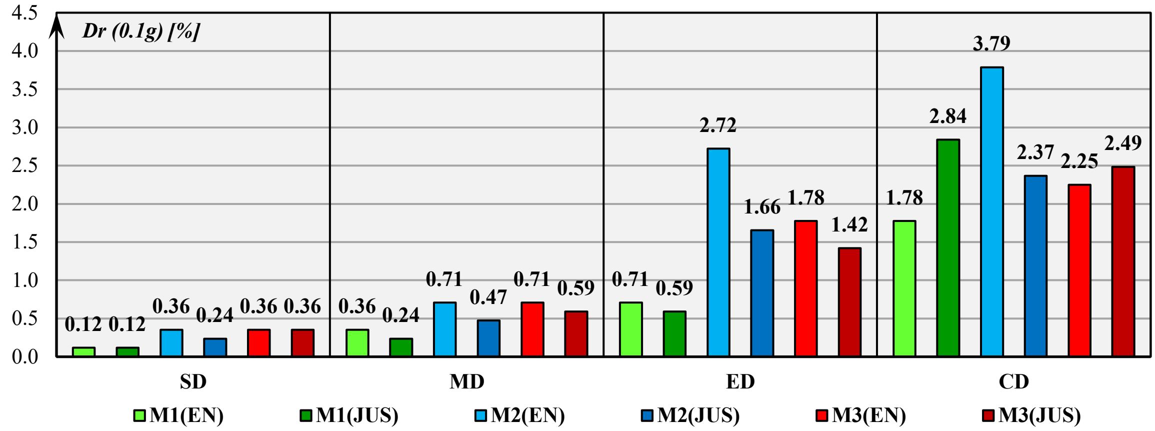

Figure 7. Comparison of drift values of mid-column for M1, M2 and M3, derived from NSA

At the Small Damage (SD) state, both M1 (EN) and M1 (JUS) have the same drift value of 0.118%, indicating similar behaviour in the massive rectangular columns. For M2, the drift values are higher, with 0.355% for M2 (EN) and 0.237% for M2 (JUS), while M3 has an equal drift value of 0.355% for both standards, suggesting that circular columns are more flexible.

In the Moderate Damage (MD) state, M1 (EN) allows a higher drift (0.355%) compared to M1 (JUS) at 0.237%. M2 (EN) also shows a higher drift (0.710%) than M2 (JUS) (0.473%), while M3 (EN) at 0.710% is greater than M3 (JUS) at 0.592%, indicating that the columns designed according to EN standard will achieve higher deformation value before moderate damage state is reached.

In the Extensive Damage (ED) state, the difference between the two standards becomes more pronounced. M1 (EN) has a drift of 0.710%, while M1 (JUS) is at 0.592%. M2 (EN) shows a much larger difference at 2.722% compared to 1.657% for M2 (JUS). M3 (EN) is at 1.775%, while M3 (JUS) is at 1.420%, that the columns designed according to EN standard will achieve higher deformation before reaching extensive damage state.

At the Complete Damage (CD) state, M1 (JUS) has a drift of 2.840%, compared to 1.775% for M1 (EN). M2 shows an even larger gap, with 3.787% for M2 (EN) and 2.367% for M2 (JUS). M3 shows slightly different values for each standard, with 2.249% for M3 (EN) and 2.485% for M3 (JUS).

Overall, the EN standards [3], [23-25] allow higher drift values across all models, indicating that these structures are more ductile before reaching the damage states. This suggests a greater deformation capacity when the structure is designed according to EN, while JUS standards [26-28] impose more conservative drift values, meaning that damage occurs at lower drift levels.

3.2. Fragility analysis

Fragility curves are calculated as the empirical fragility curves. An empirical fragility function is one that is created by fitting a function to approximate observational data. The observational data in this case represents number of assets exposed to some level of excitation, and the number of those that failed when subjected to the environmental excitation (i.e., reaching or exceeding the specified limit state). Fragility curves give insights into the probability of exceeding certain damage limit states (SD, MD, ED, CD) under varying ground motion peak ground acceleration (PGA).

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In the case of the calculation of fragility curves, an engineering demand parameter (EDP) is used as a referent value in order to calculate the probability of the occurrence of a defined damage state (DS) at a particular intensity measure (IM) value $\left(P_{DS_i|IM_j}\right)$, using the expression:

$$P_{DS_i|IM_j}\left(\mu_{LN,IM_j}^{EDP_i}, \sigma_{LN|IM_j}^{EDP_i}\right) = 1 - \Phi\left(\frac{\ln EDP - \mu_{LN,IM_j}^{EDP_i}}{\sigma_{LN|IM_j}^{EDP_i}}\right) \tag{1}$$

where $\mu_{LN,IM_j}^{EDP_i}$ and $\sigma_{LN|IM_j}^{EDP_i}$ are the mean and standard deviation in log-log space of the probability density function (PDF) of the variable $\ln EDP$ for a particular $\ln IM_j$ value. $\ln EDP_i$ is the lognormal value of a DS threshold. These fragility curves are calculated using methods of statistical analysis and probability described in [36], [37]. The results of the analysis are shown in Fig. 8 – 12.

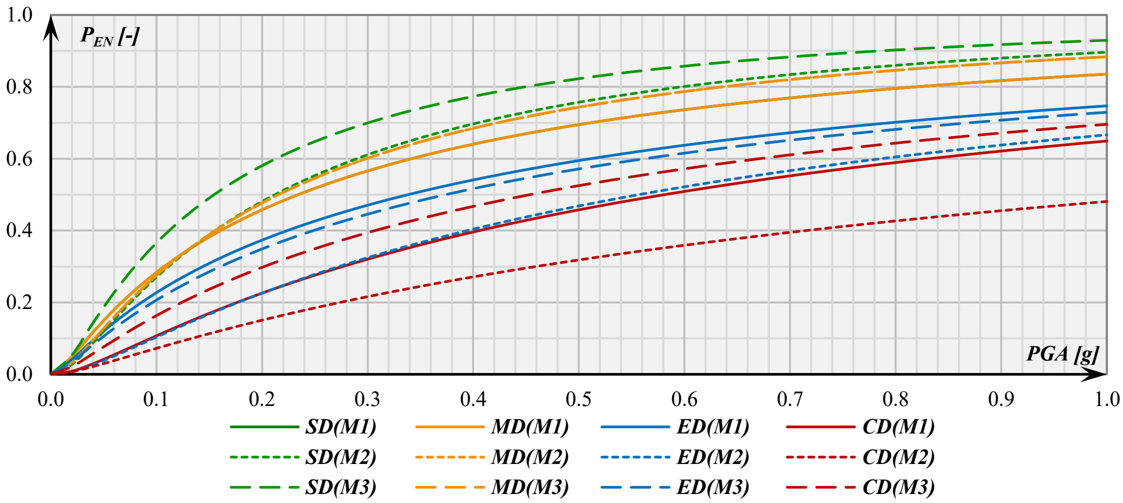

Figure 8. Fragility curves for M1, M2 and M3, according to EN

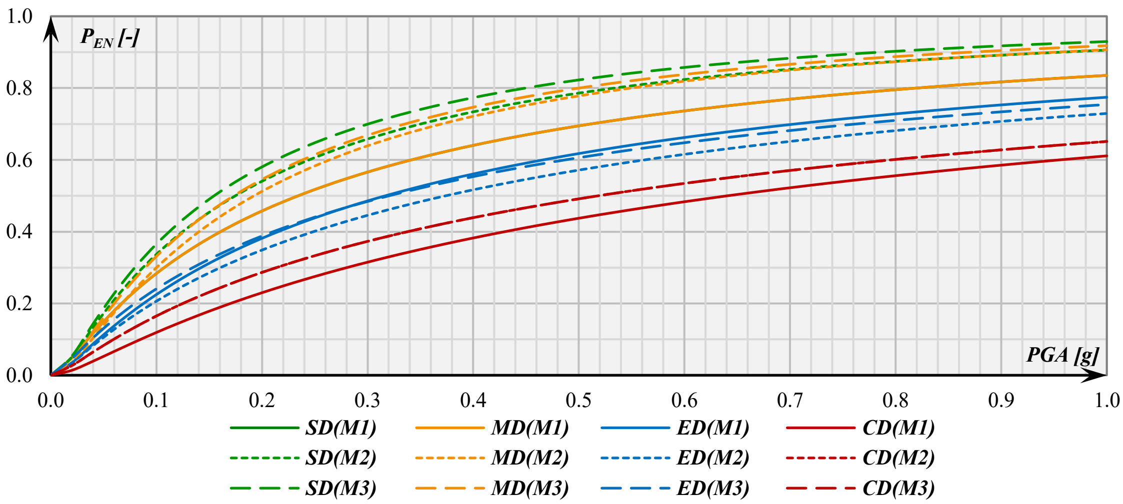

Figure 9. Fragility curves for M1, M2 and M3, according to JUS

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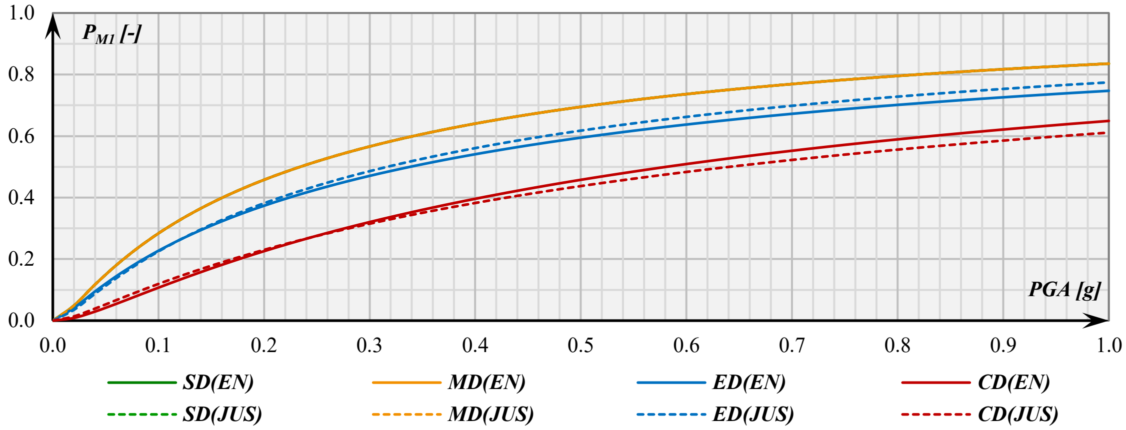

Figure 10. Fragility curves for M1(EN) and M1(JUS), for damage LS: SD, MD, ED, CD

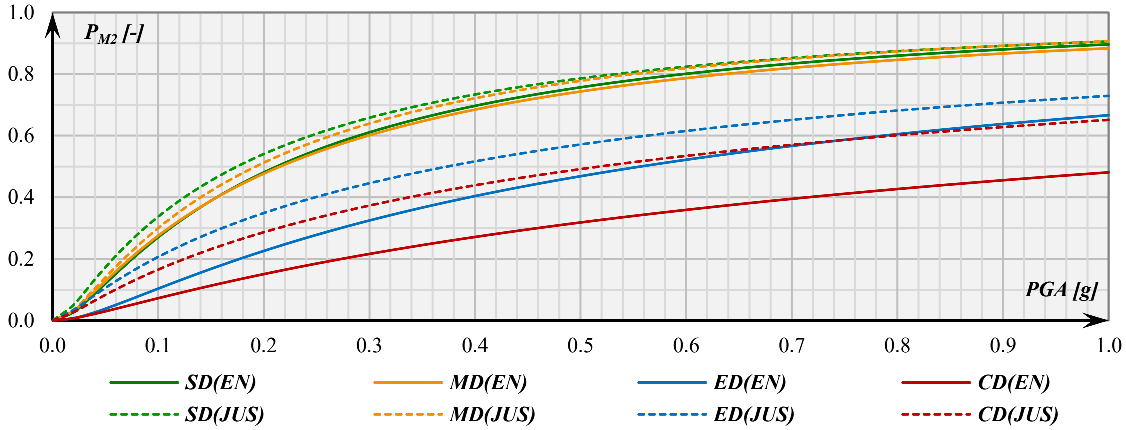

Figure 11. Fragility curves for M2(EN) and M2(JUS), for damage LS: SD, MD, ED, CD

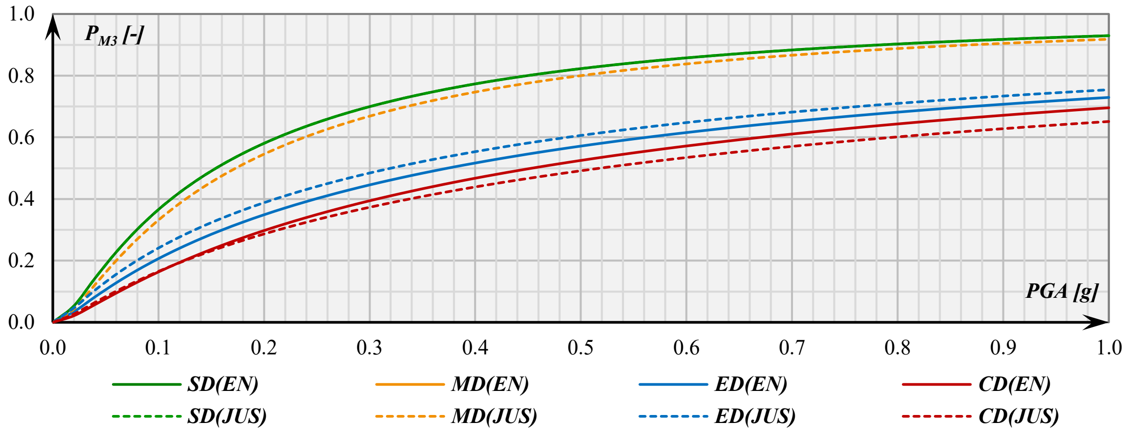

Figure 12. Fragility curves for M3(EN) and M3(JUS), for damage LS: SD, MD, ED, CD

The results provided represent fragility curves for three models (M1, M2, and M3) under two different design codes (EN and JUS). The fragility analysis typically evaluates the probability of a system or structure reaching different damage states (SD - Slight Damage, MD - Moderate Damage, ED - Extensive Damage, CD - Complete Damage) at various ground accelerations (PGA, in g).

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These results allow for a comparison of how each model behaves under varying intensities of seismic loading, as well as how the two design codes (EN and JUS) influence the fragility curves.

3.3. Comparison between M1, M2, and M3

M1 shows a higher level of resistance to damage at lower PGA values, with a gradual increase in the probability of reaching MD and ED as PGA increases. At higher PGAs, the probability of reaching ED and CD rises significantly. M2 and M3 follow a similar pattern but with lower resistance at lower PGAs compared to M1.

M2 shows a higher probability of MD and ED damage, while M3 exhibits a slightly higher probability of damage in the Moderate to Complete Damage states compared to M2, particularly at higher PGAs. The progression toward severe damage states is more pronounced in M2 and M3, with the probability of reaching CD being higher for both models compared to M1. However, the transition from MD to ED to CD in M3 occurs more gradually than in M2, suggesting greater vulnerability in the higher PGA range for M3.

M1 shows a gradual increase in probability of damage as PGA increases, but it maintains a higher overall level of resilience across all damage states. This suggests that M1 is more robust against seismic actions. M2 and M3 show similar trends, with a higher sensitivity to seismic excitation. This is reflected in a faster rapid progression from SD to MD and ED as PGA increases.

3.4. Comparison between EN and JUS

EN appears to define a more resilient structural design, particularly at lower PGAs, as seen in the lower probabilities of damage across all models. This could indicate that the EN code may impose stricter or more robust safety requirements for building design and performance under seismic events.

On the other hand, JUS appears to be more sensitive to higher seismic accelerations, as the damage states progress more quickly with increasing PGA, indicating either a less strict design for moderate shaking or a faster progression to failure in structures designed under JUS.

The fragility results suggest that structures designed according to EN may have a greater margin of safety in terms of resilience to seismic events, especially for lower-magnitude earthquakes. However, structures designed under JUS may face more significant damage at higher PGA values, necessitating more robust reinforcement for earthquake-resilient designs. EN design standards seem to offer better performance in terms of damage mitigation at both low and high seismic intensities.

The analysis reveals that the models exhibit different behaviours under seismic loading, with M1 being more resilient than M2 and M3. Additionally, structures designed according to EN standards demonstrate better resistance to damage at both low and high PGAs compared to those designed according to JUS. The results indicate a more conservative approach in EN, particularly in mitigating damage under low to moderate seismic loads, while JUS standards predict higher damage probabilities at higher PGAs, especially for the more sensitive models.

4. Conclusions

This study shows the importance of the assessment of the seismic performance and lateral load-bearing capacity of existing bridges, especially those not designed to modern seismic standards. The research focused on comparison of the bridge models with different substructures (massive rectangular mid-columns, and two-column frame mid-column) designed under old (JUS [26-28]) and modern (EN [3], [23-25]) standards. The results from nonlinear static and dynamic analyses show that bridges designed according to the EN standard exhibited greater seismic resilience, allowing higher deformation before reaching critical damage states compared to those designed using JUS standards.

Fragility analysis results show the significance of design choices in order to enhance the seismic performance of beam bridges, particularly in earthquake-prone regions. The results show that bridges with circular columns and dual-column frame mid-columns (M2 and M3) are more flexible and

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experience larger drift values before reaching damage limit states, than massive rectangular mid-columns (M1) which can influence seismic resilience. Additionally, the columns designed according to the EN standard consistently show the greater deformation capacity before critical damage states, suggesting a more ductile behaviour under seismic actions.

In general, the results suggest that bridges designed with modern seismic codes exhibit improved resilience, which is crucial for maintaining functionality and safety during seismic events. Moreover, this study underscores the need for comprehensive evaluations and retrofitting of older bridges to meet current seismic performance standards, ensuring their long-term safety and serviceability.

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