

METHODOLOGIES FOR ASSESSING BRIDGE PERFORMANCE UNDER SEISMIC ACTION

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Abstract

This paper analyses the seismic response of four continuous precast prestressed concrete girder bridges, with total lengths ranging from 80 m to 200 m, representative of Romanian road infrastructure. The structures were initially designed with neoprene elastomeric bearings, in compliance with the Romanian code for seismic design P100-1/2006 [1]. The seismic performance of the bridges was assessed through nonlinear static (Pushover) analyses using the FEMA 440 [2] equivalent linearization procedure, for five seismic hazard levels (return periods of 100, 225, 475, 970 and 2475 years). Results highlighted a significant vulnerability of the conventional neoprene system at moderate seismic intensities, with displacement demands exceeding the nominal bearing capacity from a return period of 225 years onwards. As an alternative, a pot bearing system was investigated, allowing larger displacements and providing more controlled seismic behavior. The study demonstrates that pot bearings reduce spectral accelerations by up to 54%, but increase the behavior factors, while maintaining bearing integrity at higher seismic hazard levels.

Keywords: neoprene bearings, pot bearings, Pushover, behavior factor, seismic isolation.

1. INTRODUCTION

Prestressed precast concrete girder bridges represent one of the most widespread structural typologies in Romanian road infrastructure, owing to their economic efficiency and rapid construction. Their seismic behavior is significantly influenced by the bearing system adopted, which governs the transfer of forces between the superstructure and the substructure.

Current seismic code (P100-1/2013) and also former P100-1/2006, regulate the design of such structures for a limited seismic hazard level, without providing

a comprehensive picture of structural behavior under earthquakes more severe than the design event [3]. In the context of recent normative proposals (P100-1/2025), which prescribe significant increases in design seismic actions, the vulnerability assessment of existing bridges equipped with neoprene bearings becomes a priority.

This paper presents a comparative seismic performance analysis of four representative bridges, initially equipped with neoprene elastomeric bearings and subsequently re-analysed with pot bearings. The assessment is carried out through nonlinear static Pushover analyses at five seismic hazard levels, using the FEMA 440 equivalent linearization methodology [2].

2. ANALYSED STRUCTURES

Four continuous prestressed precast concrete girder bridges were analysed, with static schemes ranging from two to five spans of 40.00 m each. The four configurations, designated S2, S3, S4 and S5, were selected to cover a representative spectrum of situations encountered in current bridge design practice, in Romania.

2.1. Common characteristics

All bridges share the same geometric and material characteristics for both the superstructure and substructure levels. The superstructure consists of five prestressed precast girders of $L = 40.00\text{m}$ length and $H = 2.10\text{m}$ height, made of C50/60 concrete, connected by a 20cm cast-in-place slab (C35/45) and monolithic crossbeams that ensure continuity over piers and solidarization of the beams. The carriageway width is 12.00m, bordered by H4b safety barriers, with a total superstructure width of 13.70m.

The substructure consists of massive abutments and piers founded on deep drilled piles with a diameter $\phi 1.20\text{m}$. The pier elevation consists of two circular columns with diameter $D = 2.00\text{m}$, connected at the top by a $2.00\text{m} \times 1.50\text{m}$ rectangular cap beam, made of C30/37 concrete.

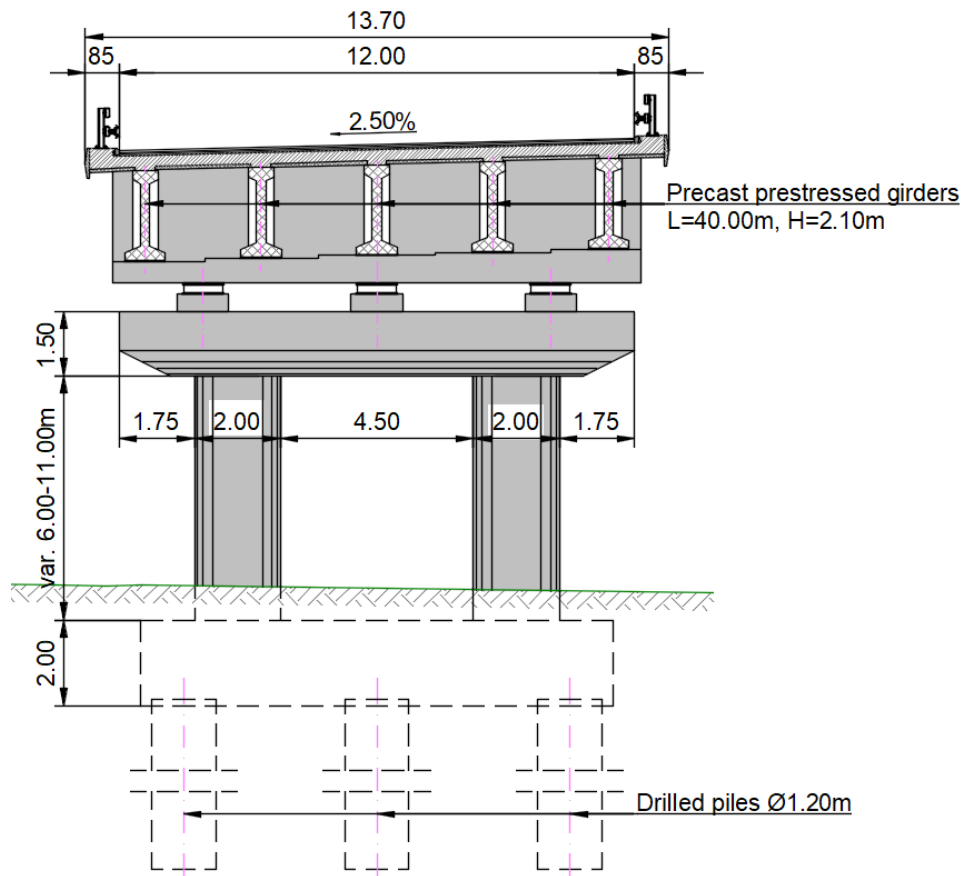


Figure 1. Cross section of the analysed bridges

2.2. Structural configurations (S2 – S5)

The bridges differ in the number of spans and pier heights, as summarised in Table 1. The non-uniformity of pier heights generates an uneven lateral stiffness distribution, with direct consequences on how seismic forces are attracted and distributed throughout the structure. This variability is most pronounced in configurations S4 and S5, where the height difference between piers reaches 3.00 m and 5.00 m respectively.

Table 1. Structural configurations of the four analysed bridges

Bridge	Spans	Pier P1 (m)	Pier P2 (m)	Pier P3 (m)	Pier P4 (m)
S2	2	8.00	-	-	-
S3	3	8.00	10.00	-	-
S4	4	7.00	11.00	9.00	-
S5	5	8.00	11.00	9.00	6.00

3. SEISMIC ACTION AND NUMERICAL MODELS

3.1. Elastic response spectrum and Design spectrum

The analysed bridges were considered located in a seismic zone characterised by a peak ground acceleration of $a_g = 0.20g$ and a corner period of $T_c = 0.70s$. The initial design was carried out in accordance with P100-1/2006, using a behavior factor of $q = 1.50$, corresponding to limited energy dissipation capacity.

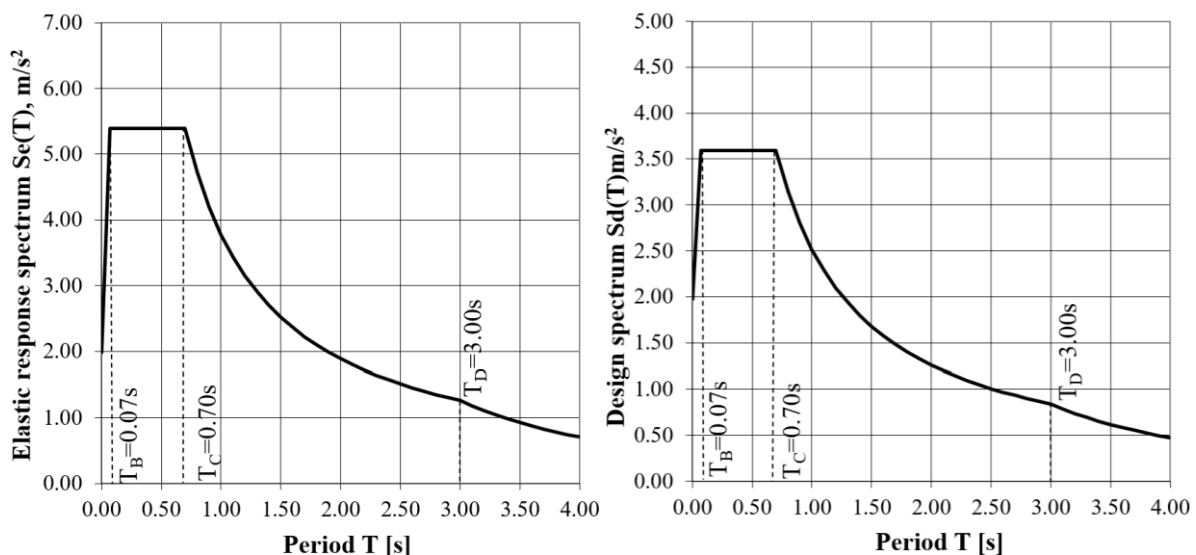


Figure 2. Elastic response spectrum (left) and design spectrum $q=1.50$ (right)

3.2. Finite element models

The finite element models (Figure 3) were developed to achieve the most faithful representation possible of the actual structural behavior. The prestressed precast girders were modelled using frame elements, capable of capturing axial and bending stiffness as well as shear and torsional effects. The cast-in-place slab was modelled with shell elements, connected to the precast girders through rigid links ensuring full composite action.

At the substructure level, both the pier elevations and the bored piles were modelled with frame elements, while the pile caps were represented by shell elements. The pile-soil interaction was modelled with springs spaced at 1.00m intervals, with stiffness determined from a representative borehole at the site. This approach simplifies the actual variability of the geotechnical profile, but is justified in the context of this comparative study, where the primary interest lies

in the relative response between structures rather than in the evaluation of foundation forces.

The bearings were introduced through links with mechanical properties corresponding to each bearing type.

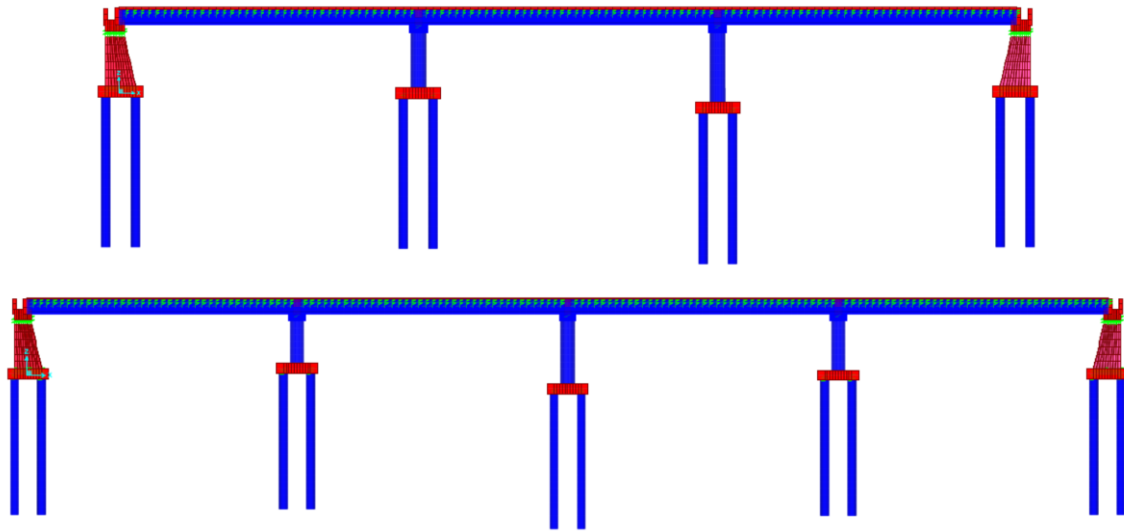


Figure 3. Finite element models (structures S3 and S4)

3.3. Bearing systems analyzed

3.3.1. Neoprene elastomeric bearings

In the reference configuration of the structures, the bearing system is represented by conventional elastomeric bearings, consisting of alternating layers of neoprene and vulcanised steel plates. Their function relies on the shear deformability of the elastomer, which allows relative displacements between the deck and the substructure in both the longitudinal and transverse directions, while simultaneously transferring the vertical loads from the deck.

The four bridges feature differentiated bearing schemes depending on the number of spans, as follows:

- for bridge S2, the deck is supported by three fixed bearings on pier P1 and three movable bearings at both abutments;
- for bridge S3, the fixed bearings are maintained at P1, while movable bearings are placed at the abutments and pier P2; also, antiseismic stoppers are installed on pier P2, which are engaged once the displacement capacity of the bearings is exceeded;
- for bridge S4, both P1 and P2 are equipped with fixed bearings, distributing the horizontal forces to two piers, while the bearings at pier

- P3 and the abutments remain movable; antiseismic stoppers are installed at P3;
- for bridge S5, the fixed bearings are located at P2 and P3, while P1, P4 and the abutments have movable bearings; antiseismic stoppers are installed at P1 and P4.



Figure 4. Movable guided bearing (left) and fixed bearing (right) [4]

In the adopted configurations, the movable bearings allow horizontal displacements of up to 10.00cm, but are not entirely free in reality. Due to the lateral stiffness of the elastomer, they develop a horizontal force proportional to the displacement, quantified through a lateral stiffness of $k_h = 5000 \text{ kN/m}$.

3.3.2. Pot bearings

As an alternative, pot bearings from [5] were analysed, featuring a significantly larger displacement capacity ($\Delta_{\max} = 25.00\text{cm}$). Movable bearings were modelled as free-sliding or guided devices, while fixed bearings were modelled using a mixed configuration.

To ensure a relevant comparison between the two solutions, the static scheme of the bridges remains identical to that used for the neoprene configurations. Accordingly, the positioning of the fixed and movable bearings is maintained strictly in the same locations.

The bearing system configuration was implemented as follows:

- Movable bearings: modelled as free sliding or guided devices, allowing displacements resulting from thermal variations or seismic actions in the designated directions;
- Fixed bearings (on piers): a mixed configuration was adopted, consisting of one fully fixed bearing (blocked in both directions) and two bearings fixed in the longitudinal direction and guided in the transverse direction.

4. MODAL ANALYSIS AND INITIAL RESULTS

The linear seismic response of the structures was determined using the response spectrum method. As the number of spans increases, the fundamental period increases progressively from $T_1 = 0.75\text{s}$ (S2-N) to $T_1 = 1.05\text{s}$ (S5-N), while the seismic coefficient $V_{\text{base}}/G_{\text{tot}}$ decreases, reflecting the distribution of seismic forces based on the number of spans and height of piers (Table 2).

Table 2. Dynamic response of the four structures (longitudinal direction)

Bridge	Spans	T_1 (s)	Participating mass [%]	V_{base} [kN]	$V_{\text{base}} / G_{\text{tot}}$ [-]
S2-N	2	0.75	93.00	12265.00	0.60
S3-N	3	0.99	92.00	13950.00	0.44
S4-N	4	1.01	91.00	18600.00	0.42
S5-N	5	1.05	91.00	23000.00	0.35

The maximum sectional forces at the base of the piers and the deck displacements are summarised in Table 3. The obtained displacements fall within the admissible limits of the neoprene bearings ($\Delta_{\text{adm}} = 10.00\text{cm}$), confirming the validity of the structural design for the design earthquake.

Table 3. Maximum forces and deck displacements (IMR = 100 years)

Bridge	M_{max} [kN·m]	V_{max} [kN]	Δ_{max} [cm]	Δ_{adm} [cm]
S2-N	18540.00 (P1)	3070.00	7.30	10.00
S3-N	19500.00 (P1)	2760.00	9.00	
S4-N	19600.00 (P1)	2790.00	9.30	
S5-N	20500.00 (P3)	2900.00	9.70	

5. SEISMIC PERFORMANCE ASSESSMENT – PUSHOVER ANALYSIS

5.1. Methodology

The seismic performance assessment was carried out through nonlinear static Pushover analyses, following the MADRS equivalent linearization procedure from FEMA 440. This represents an improvement over the Capacity Spectrum Method (CSM) from ATC-40, correcting limitations of earlier versions in estimating nonlinear seismic response.

The main steps of the methodology are: obtaining the capacity curve through Pushover analysis [6], converting it into Acceleration–Displacement Response Spectrum (ADRS) format, determining the equivalent linear parameters (effective period T_{eff} and effective damping β_{eff}), generating the Modified ADRS (MADRS) and identifying the performance point at the intersection of the capacity spectrum with the MADRS. The total behavior factor was determined as the product of structural overstrength (q_R) and ductility (q_μ): $q = q_R \cdot q_\mu$. The design overstrength factor (q_{sd}) was neglected from the analysis.

Plastic hinges are located at the base of the piers. Their nonlinear behavior was defined through moment–curvature ($M-\Phi$) diagrams, accounting for biaxial bending interaction M_2-M_3 . Admissible rotation values for the performance levels are: IO = 0.003 rad; LS = 0.012 rad; CP = 0.015 rad [7].

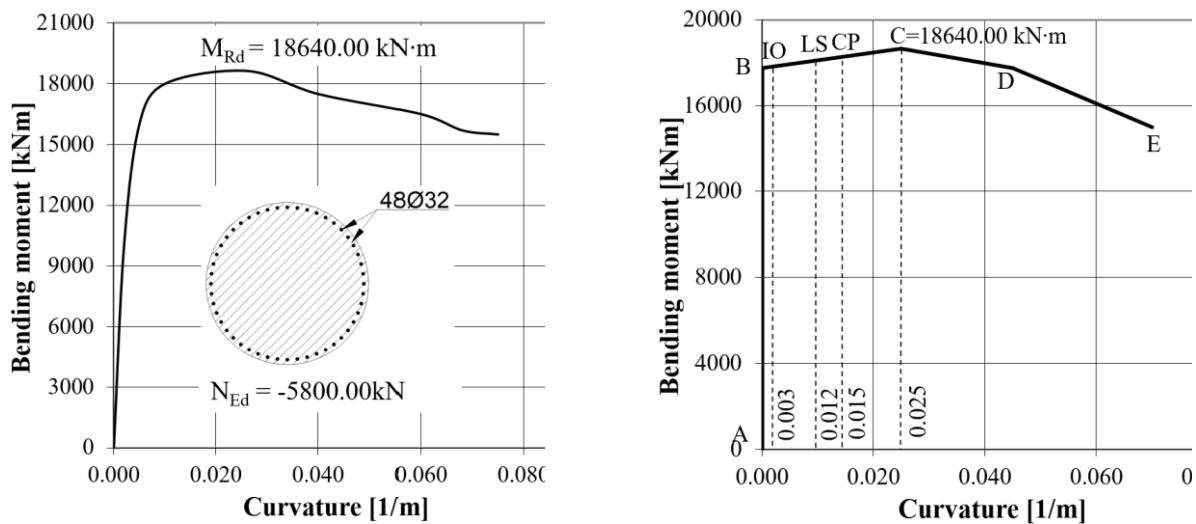


Figure 5. Moment-curvature diagram and plastic hinge definition – Pier P1, Bridge S2-N

5.2. Capacity curves – neoprene system (S2-N – S5-N)

The capacity curves obtained for the neoprene configurations reveal two distinct sources of nonlinearity: the inelastic behavior of the piers (plastic hinge formation at the base) and the response of the bearings. On the capacity curves, both pier yielding and the displacement limit of the movable bearings ($\Delta_r = 10.00\text{cm}$) are marked. Once this limit is reached, the static scheme changes and results are affected by significant uncertainties.

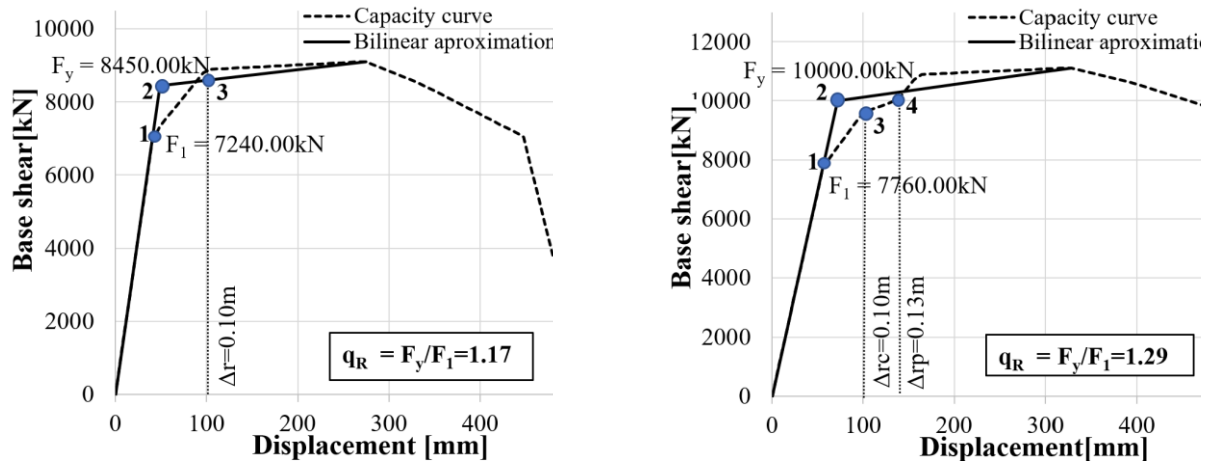


Figure 6. Capacity curves for S2-N (left) and S3-N (right)

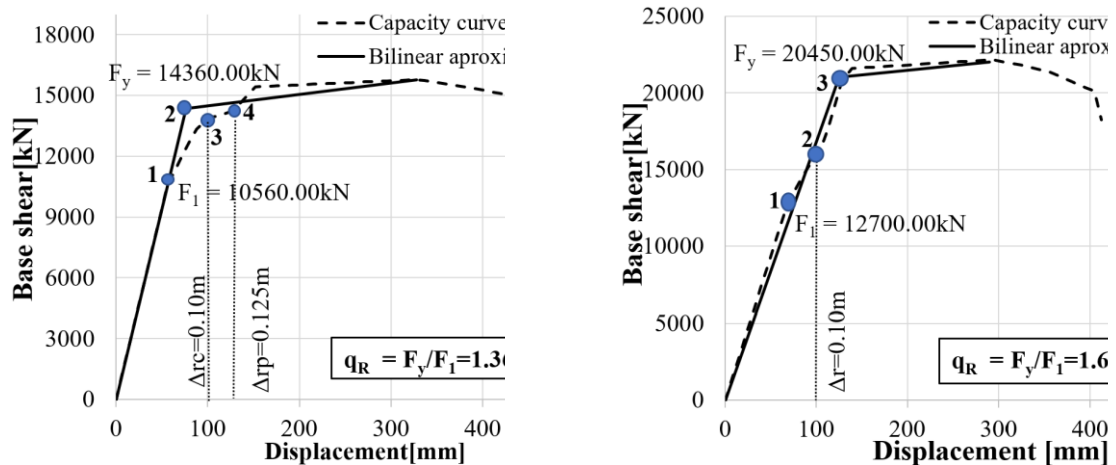


Figure 7. Capacity curves for S4-N (left) and S5-N (right)

The progression toward the system yield force (F_y) highlights the critical role of structural redundancy:

- For structure S2-N, redundancy is limited ($q_R=1.17$), with the yield force F_y being primarily influenced by the stiffness contribution of the abutment bearings rather than a distributed plastic mechanism;
- In structures S3-N and S4-N, a progressive increase in redundancy is observed (q_R ranging from 1.29 to 1.36), reflecting the successive involvement of multiple piers in the plastic mechanism;
- For structure S5-N, a critical particularity arises: the bearing displacement capacity is exceeded before the system yield force is reached. Consequently, F_y is determined within a range where the initial

static scheme is no longer valid, necessitating caution in the interpretation of behavior factors q .

Based on the capacity curves, a distinct behavior is observed for structures S3 and S4, where the bearings on the piers (Δ_{rp}) reach their displacement capacity later than those on the abutments (Δ_{rc}), at which point the antiseismic stoppers engage, leading to a recovery in global stiffness. This contrasts with the response of structures S2 and S5, where the displacement capacity is exceeded simultaneously at the same point for both bearing types. Also for S5-N, this transition is more abrupt and occurs before the system can fully develop its plastic capacity, introducing significant uncertainties into the determination of the behavior factor q .

5.3. Performance points and behavior factors – neoprene system

Performance points were determined for five seismic hazard levels (return periods of 100, 225, 475, 970 and 2475 years). The comparative analysis reveals a progressive increase of the secant period T_{sec} and structural ductility with increasing return period.

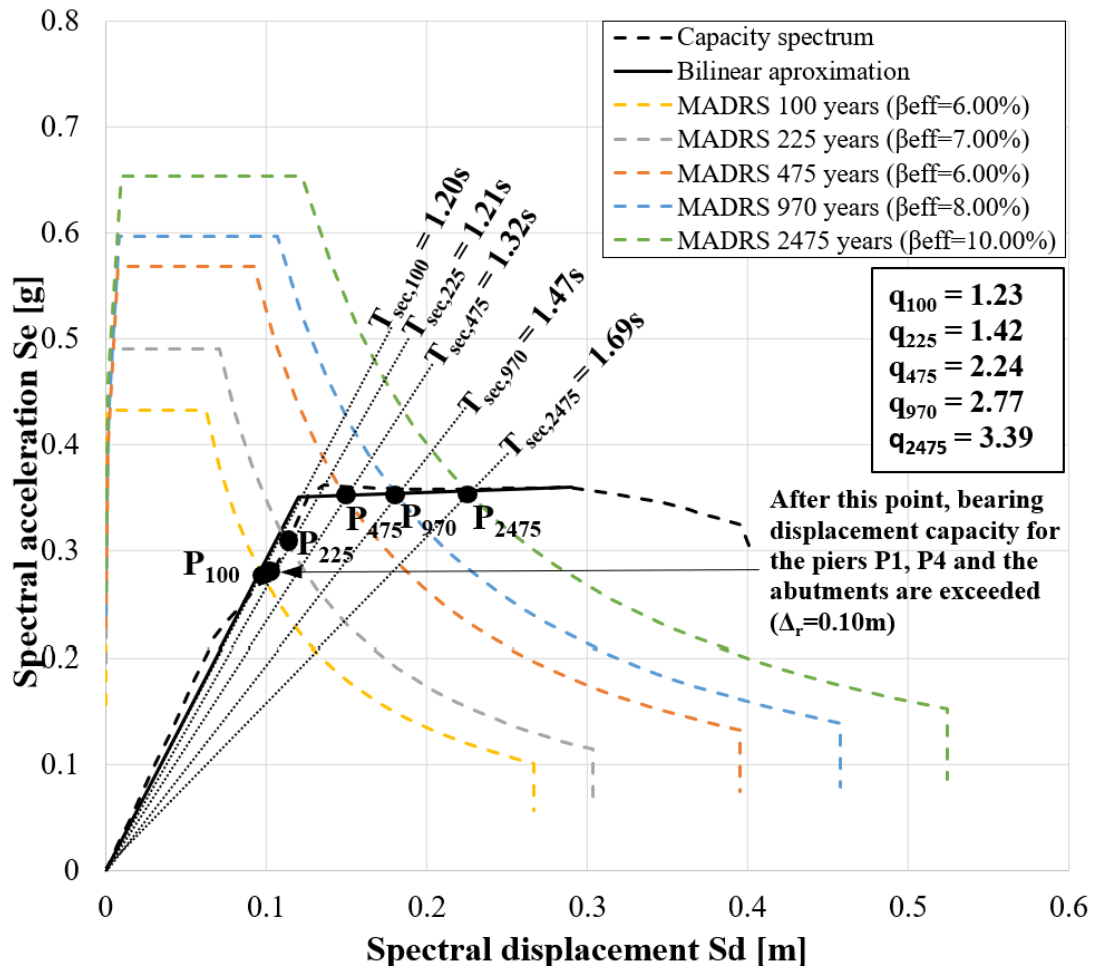


Figure 8. MADRS equivalent linearization method for determining the performance points (Example S5-N)

The results are summarised in Tables 4 and 5. All structures respond satisfactorily to the design earthquake (IMR = 100 years), with performance points within the bearing displacement limits. However, for structures S3, S4 and S5, from IMR = 225 years onwards, the seismic displacement demand exceeds the nominal bearing capacity, triggering seismic stopper engagement and modifying the static scheme. Structure S2 is the only one maintaining predictable behavior also at IMR = 225 and 475 years.

Table 4. Structural redundancy q_R and ductility q_μ – neoprene system

Bridge	q_R	q_μ (100yr)	q_μ (225yr)	q_μ (475yr)	q_μ (970yr)	q_μ (2475yr)
S2-N	1.17	1.39	1.58	1.86	2.27	2.89
S3-N	1.29	1.32	1.49	1.81	2.20	2.77
S4-N	1.36	1.27	1.40	1.71	2.07	2.63
S5-N	1.23-1.60*	1.00	1.00	1.40	1.73	2.12

* $q_R = 1.23$ for 100yr, 1.42 for 225yr and 1.60 for 475, 970 and 2475yr

Table 5. Total behavior factor $q = q_R \cdot q_\mu$ – neoprene system

Bridge	100yr	225yr	475yr	970yr	2475yr
S2-N	1.63	1.85	2.18	2.65*	3.38*
S3-N	1.70	1.92*	2.33*	2.83*	3.57*
S4-N	1.73	1.91*	2.32*	2.81*	3.57*
S5-N	1.23	1.42*	2.24*	2.77*	3.39*

* Values exceeding the nominal displacement capacity of the movable neoprene bearings – results are affected by significant uncertainties

6. POT BEARING SYSTEM

The performance analysis of the neoprene-equipped structures demonstrates that neoprene bearings limit the seismic capacity of the analyzed bridges, with displacement demands exceeding the nominal bearing capacity even at moderate seismic intensities ($IMR > 100$ years). This vulnerability leads to an uncertain structural response, characterised by the activation of seismic stoppers and the modification of the static scheme through impact.

The transition to pot bearings aims to evaluate the structural response under conditions of larger permitted displacements, without altering the bearing static

6.1. Capacity curves – pot bearing system (S2-P – S5-P)

The capacity curves for the pot bearing configurations show a significant reduction in base shear force compared to the neoprene system. The increased flexibility of the system leads to larger natural periods and consequently lower spectral accelerations. Structures S2-P and S3-P exhibit no structural redundancy ($q_R = 1.00$), while S4-P and S5-P reach values of $q_R = 1.16$ and $q_R = 1.15$ respectively.

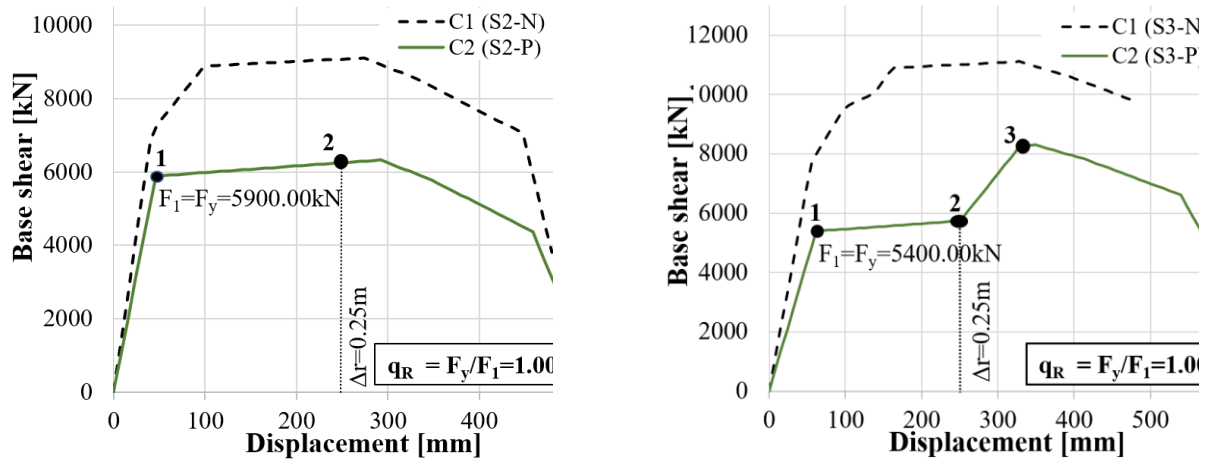


Figure 9. Capacity curves for S2-N (left) and S3-N (right)

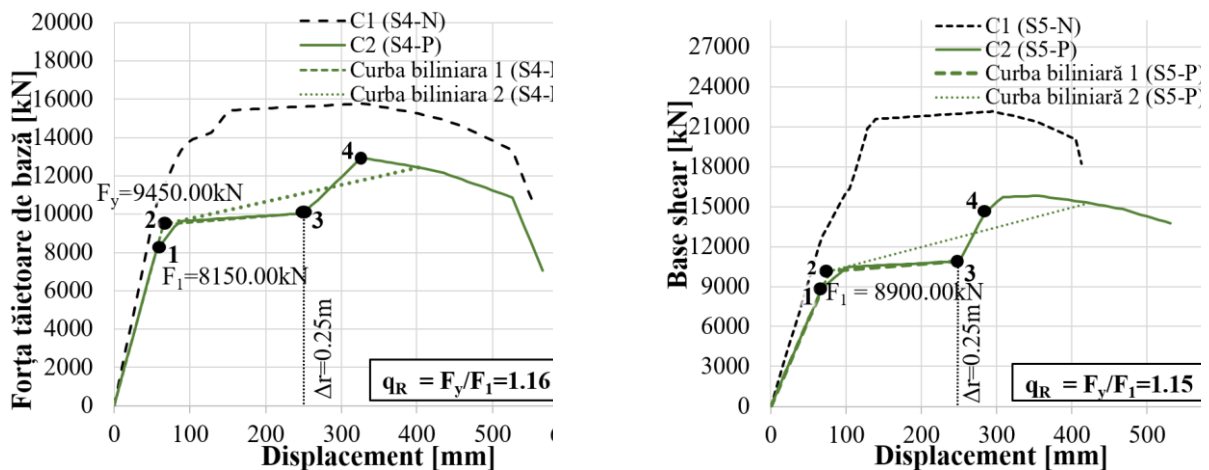


Figure 10. Capacity curves for S4-N (left) and S5-N (right)

6.2. Performance points – pot bearing system

Regardless of the structure analysed and the seismic hazard level considered, the introduction of pot bearings leads to a consistent reduction in spectral acceleration compared to the neoprene variants, through increased flexibility and consequently larger secant periods (Figure 11). The reduction in spectral accelerations ranges between 28% and 54%, depending on the structure and seismic intensity. The maximum reduction ($\Delta\%_{\max} = 54\%$) was recorded for structure S5 at IMR = 475 years, while the minimum ($\Delta\%_{\min} = 28\%$) corresponds to structure S4 at IMR = 2475 years (Figure 12).

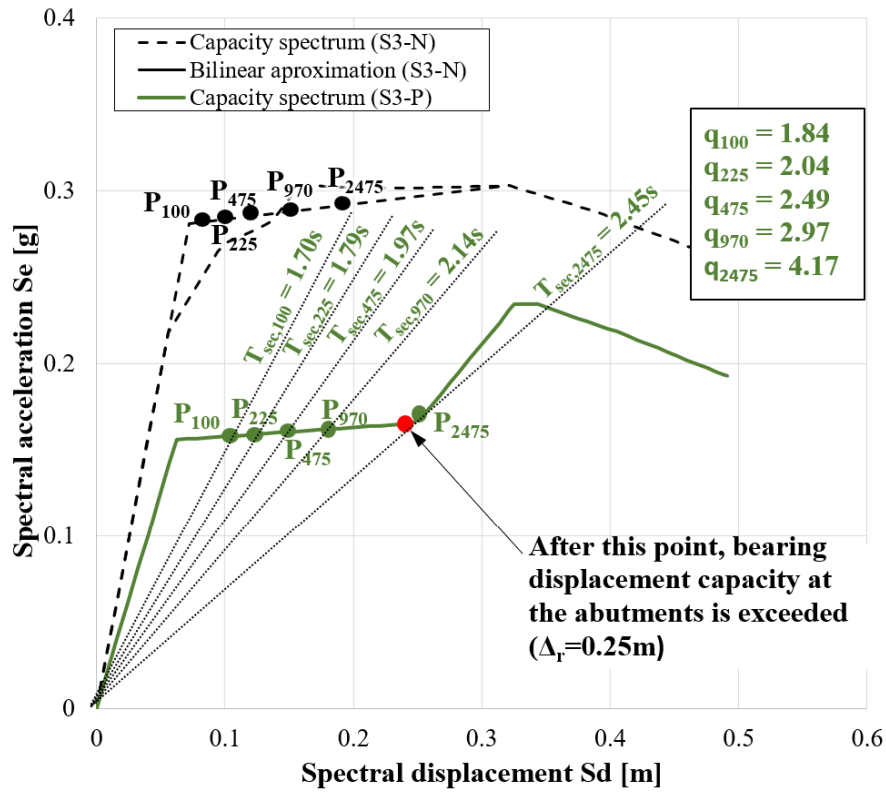


Figure 11. Comparison between performance points (Example S3-N and S3-P)

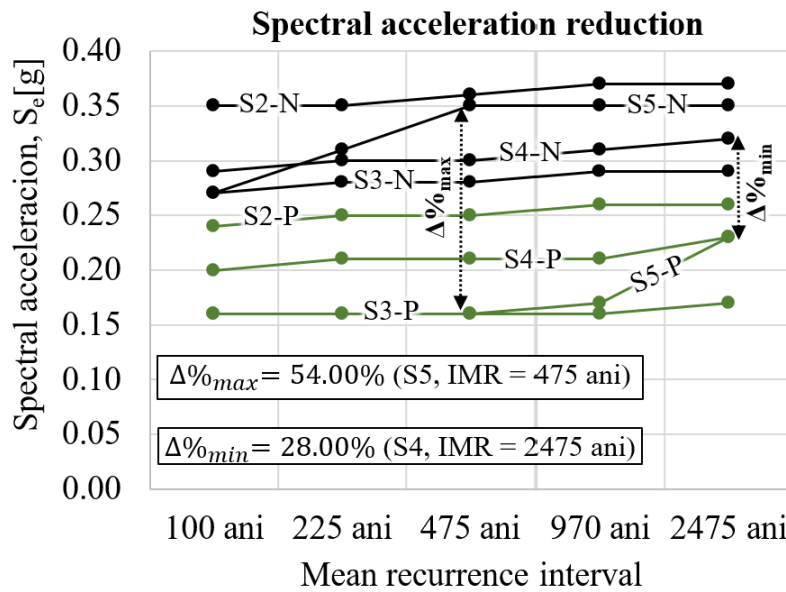


Figure 12. Spectral acceleration reduction between neoprene and pot bearings

Table 6. Total behavior factor $q = q_R \cdot q_u$ – pot bearing system

Bridge	100yr	225yr	475yr	970yr	2475yr
S2-P	1.73	2.05	2.38	3.17	4.55
S3-P	1.84	2.04	2.49	3.97	4.17*
S4-P	1.86	2.12	2.57	3.10	4.07
S5-P	1.74	1.97	2.48	2.77	4.03*

* Values exceeding the nominal displacement capacity of the movable neoprene bearings – results are affected by significant uncertainties

Although higher behavior factors reflect larger post-elastic deformations in the substructure, pot bearings tend to remain intact at significantly higher seismic hazard levels compared to neoprene-equipped structures. Bearing degradation in pot bearing configurations was only recorded for structures S3-P and S5-P at the extreme hazard level of IMR = 2475 years.

7. COMPARATIVE ANALYSIS AND DISCUSSION

A direct comparison between the two bearing systems is presented in Figure 13. For all analysed structures and at all seismic hazard levels, pot bearings yield higher behavior factors than the neoprene system. This difference becomes more pronounced as seismic intensity increases.

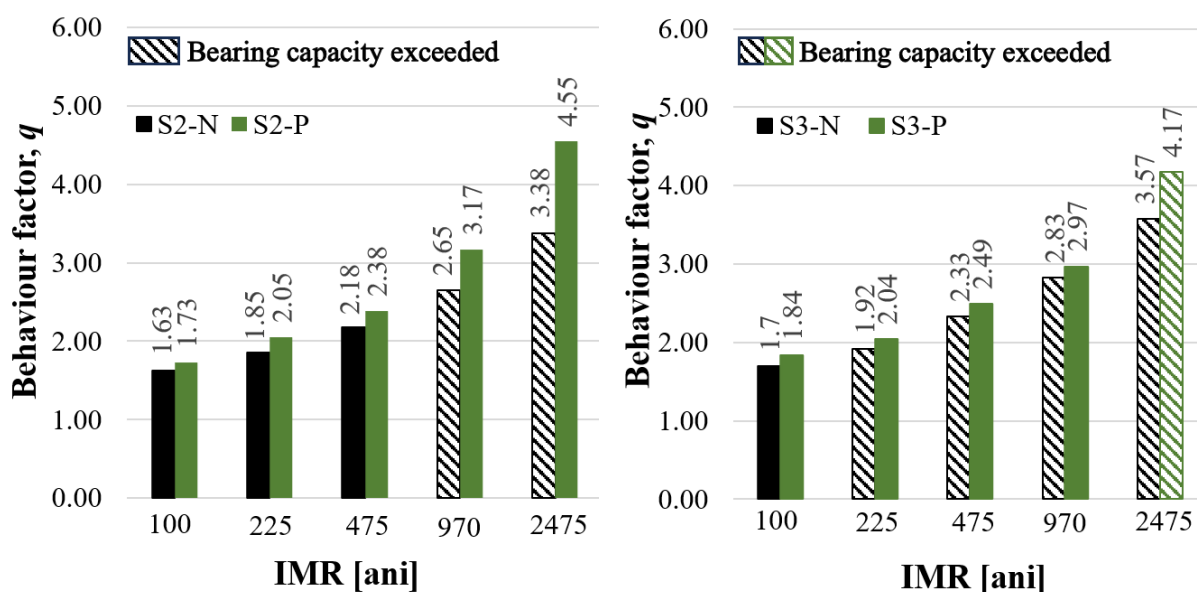


Figure 13. Evaluation of behavior factors: Neoprene vs. Pot Bearings (S2 and S3 configurations)

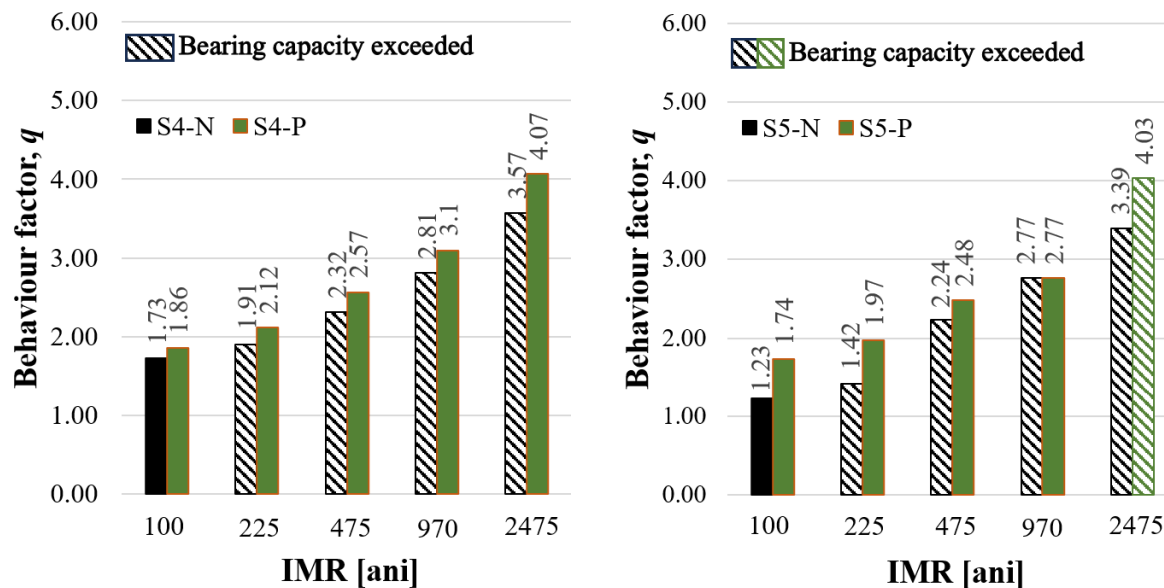


Figure 14. Evaluation of behavior factors: Neoprene vs. Pot Bearings (S4 and S5 configurations)

The main conclusions drawn from the comparative analysis are:

- At the design earthquake (IMR = 100 years), both bearing systems provide satisfactory seismic performance, with performance points falling within the bearings displacement limits;
- The neoprene system becomes vulnerable from IMR = 225 years onwards (for S3, S4 and S5), when the displacement demand exceeds the nominal bearing capacity, modifying the static scheme and introducing significant uncertainties in the evaluation of the behavior factor;
- The pot bearing system maintains bearing integrity at considerably higher seismic hazard levels, offering a more predictable and controlled structural response;
- The spectral acceleration reductions achieved by switching to pot bearings range from 28% to 54%, clearly demonstrating the benefits of increased bearing system flexibility;
- An increase in the number of spans leads to greater structural redundancy and lower ductility factors, regardless of the bearing type. Bridges with more spans rely more on redundancy than on the ductility of individual elements.

8. CONCLUSIONS

This study has highlighted the limitations of the conventional neoprene bearing system when subjected to moderate and severe seismic actions, and has clearly demonstrated the benefits of adopting pot bearings as an alternative for prestressed precast concrete girder bridges.

The Pushover analyses carried out for five seismic hazard levels showed that neoprene bearings reach their admissible displacement limit even at moderate seismic intensities ($IMR = 225$ years for S3, S4 and S5), leading to a modification of the static scheme and to an uncertain evaluation of the behavior factor, q . By contrast, pot bearings maintain the integrity of the bearing system at higher hazard levels, ensuring a more reliable and predictable structural response.

The spectral acceleration reductions obtained by switching to pot bearings (28–54%) demonstrate that replacing the bearing type alone, without modifying the static scheme, can significantly improve the seismic performance of the bridge. It is important to note, however, that the higher behavior factors recorded for pot bearing configurations reflect larger post-elastic deformations of the substructure. In other words, while the bearing system itself performs better and remains intact for longer, the piers are subjected to greater inelastic demands. This is a clear indicator that pot bearings improve the overall seismic performance of the bridge, but do not fundamentally change the energy dissipation mechanism, which still relies on plastic deformations of the structural elements.

From a broader perspective, although pot bearings represent a clear improvement over neoprene bearings in terms of seismic behavior, they remain a non-isolated solution. The most promising and forward-looking approach for the seismic protection of bridges is full seismic isolation, achievable through seismic isolators like Lead Rubber Bearings (LRB) or Friction Pendulum Systems (FPS). These devices decouple the deck motion from the substructure, drastically reducing the seismic forces transmitted to the piers and keeping the structural elements within the elastic range even under extreme seismic events. Unlike pot bearings, seismic isolators shift the fundamental period of the structure well beyond the dominant frequency content of the earthquake, simultaneously providing energy dissipation without relying on structural damage.

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