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RETROFIT SEISMIC STRATEGIES OF MULTI-SPAN SIMPLY SUPPORTED BRIDGE COMPARISON USING FRAGILITY FUNCTION METHODS

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SUMMARY: *The paper investigates the reliability of retrofitted multi-span simply supported bridges using seismic isolation strategy. For bridges, the seismic isolation aim is to protect the support piers by limiting the seismic shear transferred by the deck. The effectiveness of this retrofit strategy depends on the mass ratios and overall dynamic interaction between the decks and the piers. To this end, a portfolio of Italian bridges, built in the 60s with box girder and different retrofit isolation strategies, have been considered. The effectiveness of the examined seismic protection strategies has been investigated through fragility curves within the PBEE method. In particular, performance indexes, evaluated by nonlinear dynamic analyses, have been considered to calculate fragility functions using MSA method approach. The achieved results lead to assess the effectiveness of the seismic isolation retrofitting of simply supported span bridge, highlighting the influence of the piers on dynamic behaviour.*

KEYWORDS: *Bridge Seismic Isolation, PBEE, Fragility curves,*

1 Introduction

From the end of the 1960s, there have been studies done on the correlation between the economic growth of an area and its infrastructures. An example can be found in the classification proposed by Hansen (1956) that correlates the infrastructures with their socioeconomic function.

Given the age and the vulnerability to medium intensity earthquakes shown by bridges, viaducts and tunnels, the management and maintenance costs have been increased. This is especially true in Italy, where the infrastructures were mostly realised between the 70s and 80s without earthquake engineering criteria, and, since maintenance policies are lacking, it is possible to witness an increasing deterioration of structural integrity.

Figure 1 shows the exponential growth of the Italian road infrastructure from 1928 until today; in 1928 the road network was 21000 km long that has grown, according to the data provided by ANAS and the Ministry of Infrastructure and Transport in the annual report [Ufficio di Statistica 2014], to 180175 km by the end of 2012. The 15% [Autostrade per l'Italia S.p.A. 2015] of said network are viaducts or bridge structures. [Casarotti 2004] proposed a suitable structural classification of multi-span simple supported bridges built in the 60s and 70s.

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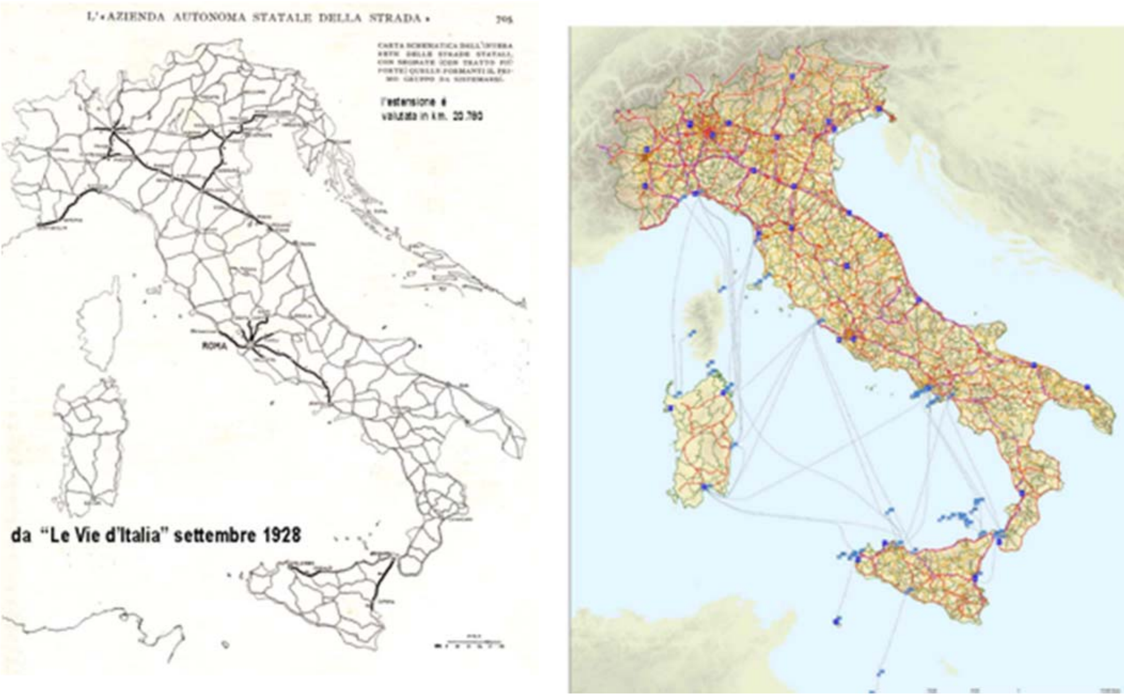


	1930	1990	2000	2012
Road [km]	20780	161938	167725	180175

Figure 1 - *The evolution of road infrastructure in Italy*

The inadequacy of the old design philosophies has been observed in recent decades, given the high vulnerability of these structures to the several seismic events occurred in different parts of the world, even if the infrastructures were designed with earthquake resistant criteria [Priestley 1996]. Therefore, it is necessary to introduce maintenance policies with high-cost efficiency that can improve or upgrade the high number of multi-span simply supported bridges. For this purpose, the seismic isolation devices have shown promising results in improving the performance of existing bridges against stronger earthquakes [Buckle et al. 1990, Skinner et al. 1993, Naeim et al. 1999, Imbsen 2001].

The paper investigates the effectiveness of seismic isolation strategy to increase the overall seismic performance and the behaviour of some typologies of the existing multi-span bridges built in the 70s and 80s. To this end, the Friction Pendulum System (FPS) is investigated for the low cost-performance ratio which characterises this type of devices, as well as high displacement and bearing capacity, force decoupling capacity between decks and piers and geometrical compatibility. During the past 20 years, the FPS devices have been used many times for the seismic retrofitting of existing bridges. The effect of this seismic isolation strategy is different for building and bridges; in fact, while in a building is obtained a filtering action to protect the upper structure, in the bridges a shear cut-off effect is observed that protect the piers. Given this difference, more research should be carried out to evaluate the reliability of bridges retrofitted with isolators.

The paper presents the results of an extensive probabilistic analysis by using some structural reference models with the aim of evaluating the seismic vulnerability of bridges. To this end, the Multiply Stripe Analysis (MSA) is considered to estimate the fragility functions by non-linear dynamic analysis [Mander et al. 1999, Shinozuka et al. 2000, Mackie et al. 2004, Choi et al. 2004].

2 Fragility Function Methodology

Fragility functions are useful tools for assessing the seismic vulnerability of highway bridges in choosing retrofit techniques, pre-earthquake planning and post-earthquake loss estimation.
Fragility functions define the conditional probability of achieving or exceeding a specified damage state for a given set of inputs with variable intensity. They can be derived from different approaches such as damage observations and/or static structural analysis [Kennedy and Ravindra 1984, Kim and Shinozuka 2004, Calvi et al. 2006, Villaverde 2007, Porter et al. 2007, Shafei et al. 2011].
In this research have been considered analytical fragility functions developed through dynamic structural analysis. The analytic approach allows for the collected data to be defined by selecting the Intensity Measure (IM) levels used for the analysis, as well as the number of analyses to be done at each IM level.
Said functions are calculated with data obtained by the seismic response of bridges obtained from non-linear Time History analysis and are widely used, both in academic research and in practical application.
The lognormal Cumulative Distribution Function Equation (1) (CDF) is usually used to define the fragility function:

$$P(C|IM = x) = \Phi\left(\frac{\ln(x/\theta)}{\beta}\right) \tag{1}$$

where $P(C|IM = x)$ is the probability that a ground motion with $IM = x$ will cause the structure to collapse, $\Phi(\,)$ is the standard normal CDF, θ is the median of the fragility function (the IM level with 50% probability of collapse) and β is the standard deviation of ln(IM) (sometimes referred to as the dispersion of IM). In this paper, the Probabilistic Seismic Demand Model (PSDM) is used to calculate analytic fragility functions using the non-linear analysis. The PSDM can be developed using the "cloud" approach or the "stripe" approach to correlate engineering demand parameters (EDP) with earthquake intensity measures (IM). With the "scaled" approach, all the considered seismograms are scaled to pre-defined intensity levels corresponding to a seismic risk level set by performing incremental dynamic analysis (IDA) at different levels of risk.
In the analysed case, structural analyses have been performed on a discrete array of IM levels using different earthquakes for each IM level. This method is the Multiple Stripe Analysis (MSA), in which the Conditional Spectrum approach has been used. Said approach provides a set of seismic events for each investigated limit state, scaled according to the variation of the IM described by the pseudo-spectral acceleration (SPA), evaluated in correspondence of the fundamental vibration period of the bridge [Baker 2010, Iervolino et al. 2010, Lin et al. 2013].

In this regard, the maximum likelihood method [Shinozuka 2000, Baker and Cornell 2005, Baker 2014] has been used. In particular, the probability $P(z_j)$ of exceeding the limit state for each level of IM_J considered is given by the binomial distribution:

$$P(z_j) = \binom{n_j}{z_j} p_j^{z_j} (1 - p_j)^{n_j - z_j} \tag{2}$$

Where n_j describes the number of considered seismic events, z_j the number of events for which the state limit is not fulfilled and p_j the probability that it has an intensity IM_J. The fragility function is derived using the maximum likelihood approach. To this function corresponds the highest probability of correlation with the results obtained from all the analyses carried out by varying the IM. To that end, by describing the limit state assuming a log-normal probability distribution law, it is possible to estimate the average (θ) and variance (β):

$$\{\hat{\vartheta}, \hat{\beta}\} = \arg\max_{\vartheta,\beta} \sum_{j=1}^{m} \left\{ \ln\binom{n_j}{z_j} + z_j ln\Phi\left(\frac{\ln\left(\frac{x_j}{\vartheta}\right)}{\beta}\right) + (n_j - z_j)\ln\left(1 - \Phi\left(\frac{ln\left(\frac{x_j}{\vartheta}\right)}{\beta}\right)\right)\right\} \tag{3}$$

3 Seismic events selection

Both PSDA and MSA methods rely on a large number of non-linear time history analysis to derive fragility functions. Therefore, to obtain an acceptable prediction of the bridge response, it is necessary to define a significant number of earthquake records. The MSA approach is used in combination with the Conditional Spectrum to select earthquakes that represent a specific site and IM level [Bradley 2010, Iervolino et al. 2010, Lin et al. 2013].
According to the Italian Technical Regulations for Construction [Ministero delle Infrastutture e dei Trasporti 2008], the seismic actions that have to be considered for design purposes are defined from the "seismic hazard" of the construction site. The seismic actions are used to evaluate the structural performance compared to the considered limit states. The construction site chosen for the analysis is located in Campania Region (Italy) and situated at the following geographical coordinates: Longitude 14.975 - Latitude 41.0264 Figure 2.

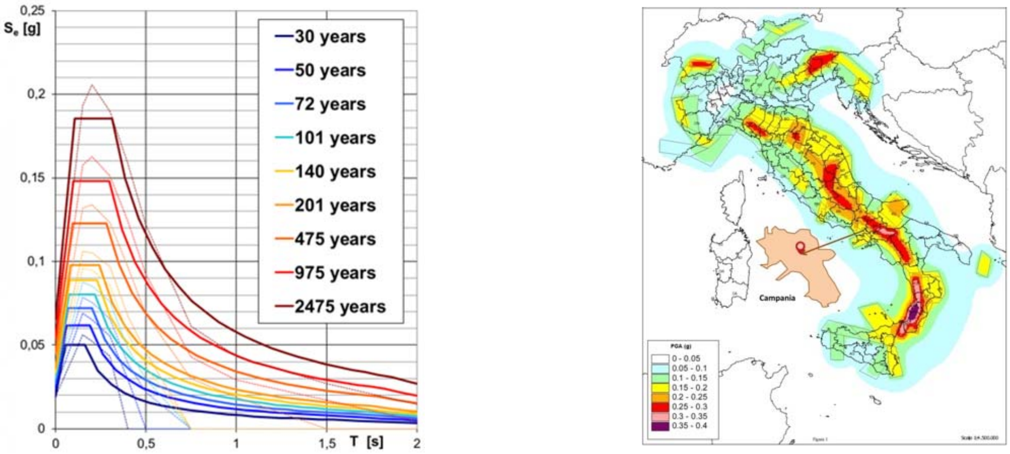
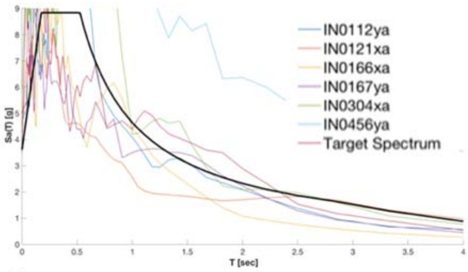
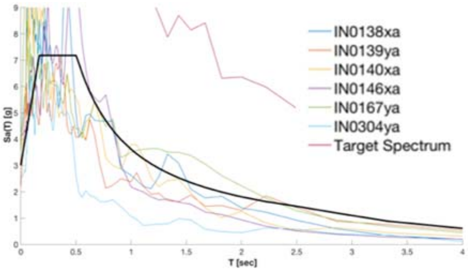


Figure 2 - *Elastic response spectra for different reference return period and location*

In particular, 21 earthquakes have been selected through the software REXEL [Iervolino et al. 2009], which allowed to obtain combinations of accelerograms compatible with the design spectrum given by the Italian regulation in the appropriate interval of vibration periods.
Figure 3 describes a summary of the considered earthquake records and the Elastic Demand Spectra for the Damage Limit State SLD, for the Life-saving Limit State SLV and the Collapse Limit State SLC. Moreover, for each Limit State, the considered seismic events were scaled by changing the PGA in the range 0.0-1.0g with a step of 0.1g.

Nominal Life V_n (years)	50
Use Class	III
SLC (events)	7
SLD (events)	7
SLV (events)	7





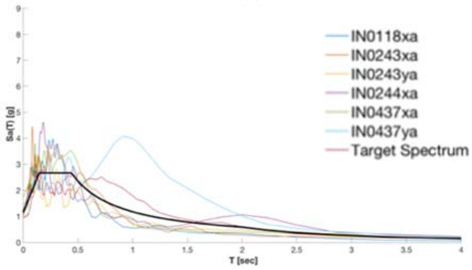


Figure 3 - *Elastic demand spectra considered for SLC, SLD and SLV for a bridge of class III and V_n=50 years*

4 Seismic events selection

Earthquakes are characterised by one of the following IM: PGA, PGV, Sa (T) or Arias Intensity (AI) [Padgett 2007]. The IM's choice plays a crucial role both in the fragility analysis and in interpreting the results of the simulation. In this research, the chosen IM is the PGA because, as stated by Padgett 2007, it is the optimal choice between the various IMs regarding convenience, sufficiency and risk computability. From the analyses, it is possible therefore to calculate the number of earthquakes that produce the exceedance of the limit state considered for each IM level [Ebrahimian 2015]. The capacity model is needed to measure the damage of both structural components and the entire system, and it is described here in terms of damage index (DI) as a function of the EDP. Damage models are formulated by experimental analyses where the observed damage and measured capacity are related to the applied demand level. Damage states (DS) are identified by the associated limit values (LS) of the DI adopted for the various damage stages. Note that some uncertainties could be introduced into the capacity model and contribute to the overall structural fragility. In this study, by evaluated the shear and plastic rotations at the base of the pier, the damage of structural elements is represented by damage indexes (DI) in terms of exceedance of the considered Limit States. The values of resistant shear shown in Table 1 have been derived by using the Priestley formulation [Priestley 1996]. The rotations limit has been set by using the criteria set out in section 8 of the Italian NTC 2008 [Ministero delle Infrastutture e dei Trasporti 2008]. The Table 1 and Table 2 summarise the considered EDPs limits.

Table 1 - *Resisting shear V_R according to the Priestley formulation*

	V_R [kN]								
	SLC			SLV			SLD		
Piers	Not retrofitted	R=2.5 m	R=3.1 m	Not retrofitted	R=2.5 m	R=3.1 m	Not retrofitted	R=2.5 m	R=3.1 m
1	1256	1270	1271	1289	1299	1308	1364	1364	1364
2	1397	1419	1421	1449	1465	1479	1568	1568	1568

Table 2 - *Base hinge limit rotations according to NTC 2008*

Pier	ϑSLD (rad)	ϑSLV (rad)	ϑSLC (rad)
1	0.0068	0.0053	0.0071
2	0.0148	0.0159	0.0212

5 Fragility analysis of highway bridges

The methodology described in the previous paragraph is used to derive the fragility curves of a typical highway bridge constructed in Italy between the 1960s and 1970s with and without retrofitting using seismic isolation devices [Imbsen 2001, Petti et al. 2015].

5.1 Reference bridge description and modelling

The research was done on typical highways and motorways bridges built in the 70s and 80s in the form of a simply supported beam and high piers as shown in Figure 4.
The bridge considered has three spans made of reinforced concrete slabs of length 41.00 m and thickness of 0.20 m. The slabs are supported by eight longitudinal prestressed reinforced concrete beams supported by two box-coupled piers, of which 1 of height 21.90 m and 1 of height 41.22 m, pile foundation with pile cap in reinforced concrete and containment abutments. The bridge has a straight and horizontal axis. In the as-built configuration elastomeric bearings are supporting the deck. As already mentioned the piers are connected in the crosswise direction, thus forming a frame.
The bridge was modelled with the software SAP2000 as a plane finite element numerical model (FEM) for both the as-built and retrofitted configurations. The as-built configuration is characterised by neoprene bearings, the retrofitted one by Friction Pendulum System (FPS). Moreover, for both the support conditions, two different type of span configuration have been investigated: decks not longitudinally connected (as-built), and decks longitudinally connected (retrofitted). In the case of isolated bridges, two radii of curvature (R=2.50m and R=3.10m) have been considered for the FPS devices.
The FEM has been constructed by using frame elements to describe the decks and the coupled columns, divided in sub-frame of length 3.00 m. Auxiliary nodes and body constraints that represent the geometry of the pier cap have been used to connect the decks to the piers. Given the high stiffness of the pile foundation in respect to the pier, it was decided to use a fully-fixed support condition.

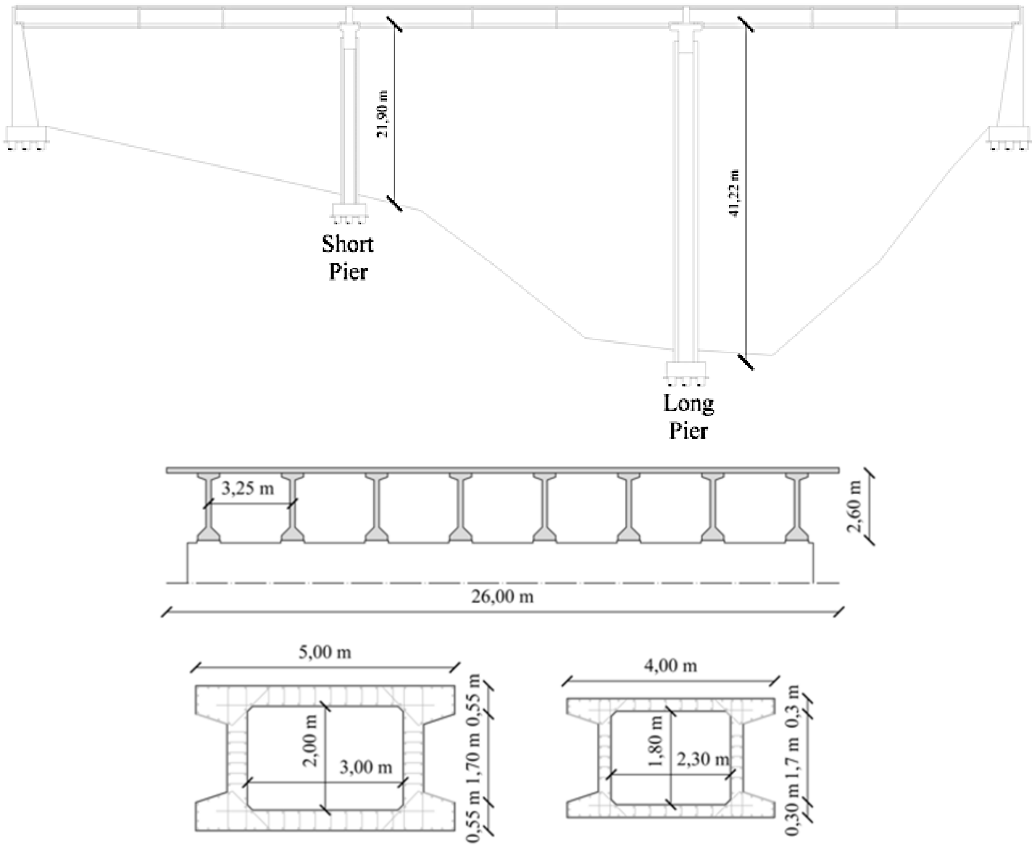


Figure 4 - *Geometrical characteristics of the chosen bridge*

The frame element sections, representative of the decks and piers, were modelled by using the geometry of an existing bridge. The non-linear properties of the pier section were modelled with multilinear plastic hinges characterised by constructing the relation between bending and rotation, shown in Figure 5, evaluated by considering a fibre section analysis using the software SAP2000 [Computer and Structures, Inc. 2016].

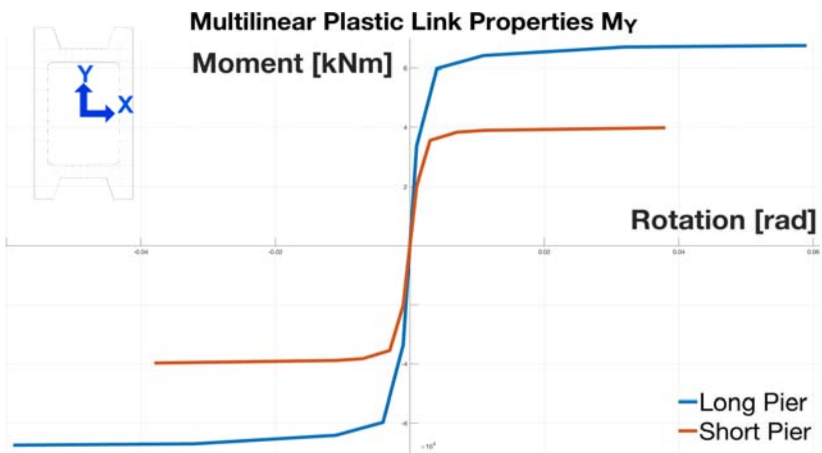


Figure 5 - *Mechanical characteristics of the Multilinear Plastic Link Properties*

The bearings of both the original and retrofitted configuration are realized with double joint link gap. The FPS has been modelled with the Friction Isolator link [Computers and Structures, Inc. 2016, Petti 2013] to better describe the characteristic of the device (radius of curvature R, friction properties, axial stress F_{Rd}, axial stiffness K_{axial}, lateral stiffness K, effective stiffness K_{eff}). In Table 3, Table 4 and Table 5 are described the mechanical properties of the bearings used in the model. The bridge deck has been modelled with an equivalent section in terms of area and inertia, with a frame element with mass distributed on the barycentric axis.

Table 3 - *Current state - Mechanical characteristics of elastomeric bearings*

F_{Rd} (kN)	K_H (kN/mm)	K_V (kN/mm)
1250	3.43	1114

Table 4 - *Design State - Mechanical characteristics of FPS bearings with R=2,50m*

R (m)	K_{eff} (kN/mm)	K (kN/mm)	K_{axial} (kN/mm)
2.50	7189.35	3921.47	10105499

Table 5 - *Design State - Mechanical characteristics of FPS bearings with R=3,10m*

R (m)	K_{eff} (kN/mm)	K (kN/mm)	K_{axial} (kN/mm)
3.10	5123.21	3162.47	10105499

5.2 Alternative Strategies for retrofitting

For the bridge retrofitting were considered ten different configurations that have the aim to improve the overall deck behaviour by using FPS and by connecting the decks. In this way, it is possible to obtain uniform displacements along the longitudinal direction Table 6.

Table 6 - *Bridge Configurations*

Retrofit Option	Description
ROD	The simply-supported decks are connected with longitudinal chains
R=2.5 2%	Friction Pendulum isolator with effective radius of concave sliding surface equal to 2.5 m and Coulomb friction 2%
R=2.5 2% - ROD	Combination between R=2.5 2% and ROD
R=2.5 5%	Friction Pendulum isolator with effective radius of concave sliding surface equal to 2.5 m and Coulomb friction 5%
R=2.5 5% - ROD	Combination between R=2.5 5% and ROD
R=3.1 2%	Friction Pendulum isolator with effective radius of concave sliding surface equal to 3.1 m and Coulomb friction 2%
R=3.1 2% - ROD	Combination between R=3.1 2% and ROD
R=3.1 5%	Friction Pendulum isolator with effective radius of concave sliding surface equal to 3.1 m and Coulomb friction 5%
R=3.1 5% - ROD	Combination between R=3.1 5% and ROD

6 Fragility function of the bridges

By using the methodology presented above, fragility curves were evaluated for the different components of bridges in the as-built and retrofitted configurations.

Short Pier *Long Pier*

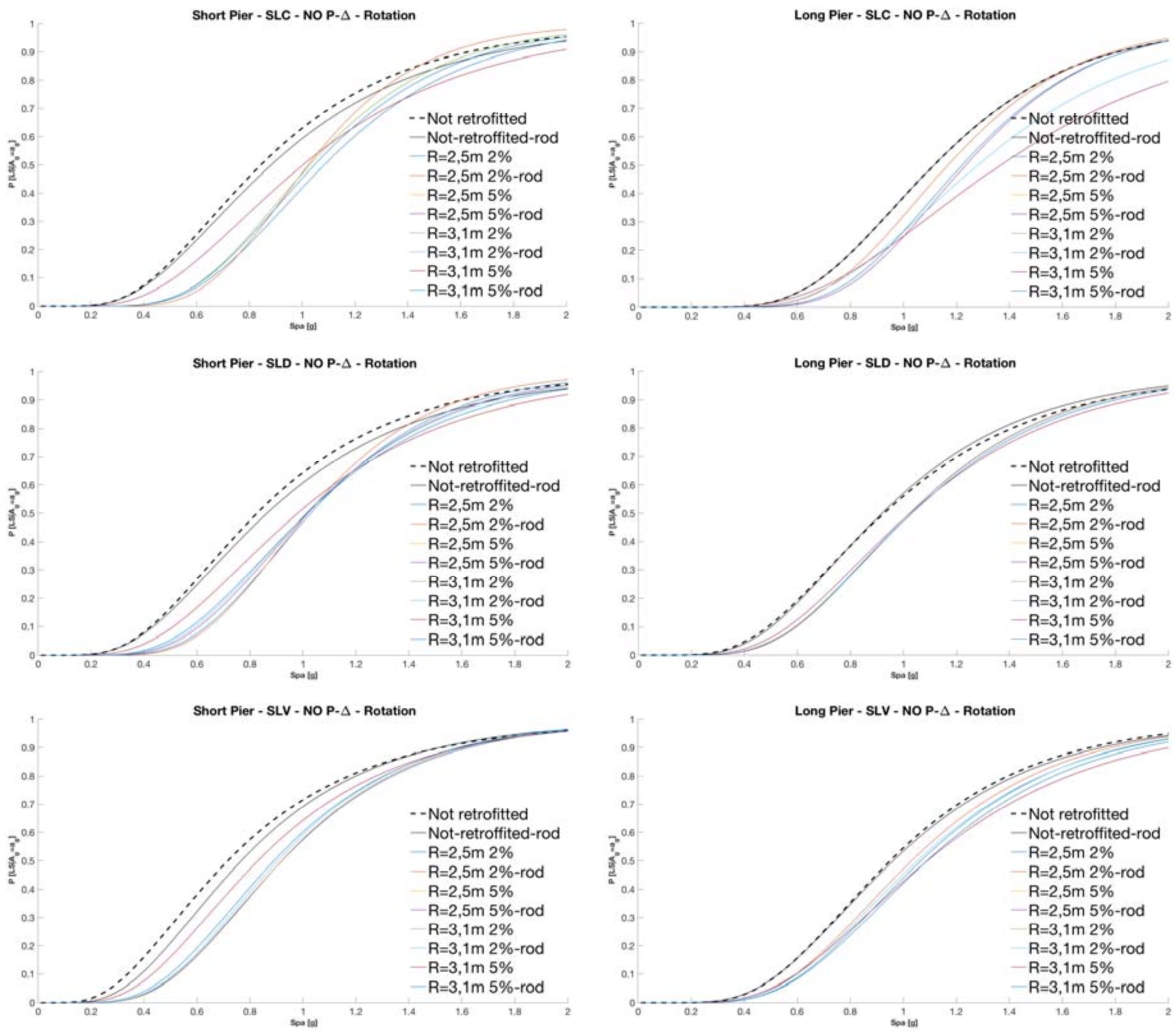


Figure 6 - *Fragility Curves of Plastic Rotation SLC-SLD-SLV*

Figure 6 describes the comparison between the fragility curves of the not retrofitted model and of the retrofitted model for the SLV, SLC and SLD limit states, respectively. The plastic rotation and shear at the base of the piers are used as the EDP to graph the fragility functions. Furthermore, Figure 7 presents fragility curves for the short and long pier by choosing the shear at the base as EDP, highlighting the efficiency of seismic isolators to reduce the fragility of this component. The use of seismic isolators has reduced the fragility, as well as vulnerability and damage states, of both piers.

By considering the probability of collapse for the different cases for a value of SPA equal to 1.0 g, it is possible to see that there is a reduction of the plastic rotations that can reach the mean value of 20% in respect to the not retrofitted case. The same trend is observed for the shear at the base of the piers with a reduction that can reach the mean value of 15% as shown in Table 7. This result is obtained only for the SLC limit state.

The replacement of conventional elastomeric bearings with seismic isolators could reduce the risk of uncoupling or slippage of the supports due to the connection between the superstructure and the infrastructure by using the aforementioned seismic isolators.

Short Pier *Long Pier*

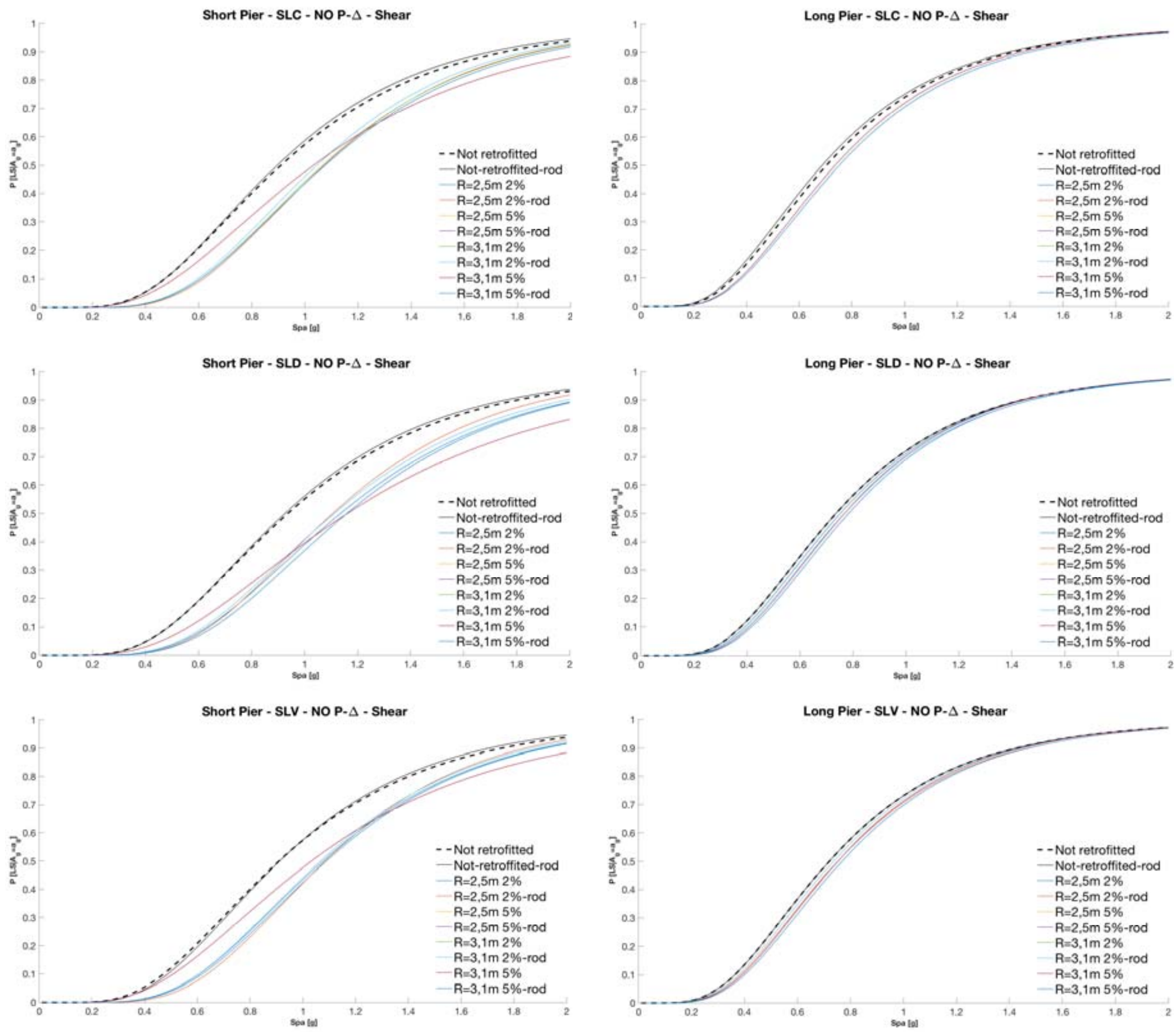


Figure 7 - *Fragility Curves of Base Shear SLC-SLD-SLV*

Table 7 - *Probability of damage to the Piers at SLC State Limit at SPA equal to 1.0 g*

	Percentage of collapse %			
	Plastic Hinge Rotation		Base Shear	
Model	Short Pier	Long Pier	Short Pier	Long Pier
Not Retrofitted	63	39	59	74
ROD	59	39	44	75
R=2.5 2%	42	27	44	72
R=2.5 2% - ROD	47	32	44	72
R=2.5 5%	45	25	52	71
R=2.5 5% - ROD	59	24	44	65
R=3.1 2%	47	27	46	72
R=3.1 2% - ROD	45	27	48	72
R=3.1 5%	50	25	44	72
R=3.1 5% - ROD	45	27	42	71

In general, a small reduction of the vulnerability is observed in the event of moderate intensity. This small decrement can be explained by the fact that the mass of the superstructure, for this type of bridge, is smaller than that of the piers. During an extreme event, less inertial loads would be transferred to these structures producing less damage.

Furthermore, the P-Δ effects were introduced to investigate the influence of the vertical loads on the behaviour of the piers. Considering the P-Δ effects, it is registered an increase of the piers seismic response. This behaviour depends on several factors, mostly imputable to the dynamic coupling effects between pears and decks and the different behaviour of the deck supports (rubber bearing or FPS devices). For the rubber bearing case, there is an increase of the shear with the relative displacement of the decks produced by P-Δ effects, while for the FPS the shear is limited by the friction force. For both cases, the P-Δ effects impact the effective stiffness of the piers changing the overall dynamic behaviour: for the higher displacements the overall system is better described by the lower frequencies. Hence, these aspects need to be considered in combination with the characteristics of the seismic demand.

Short Pier *Long Pier*

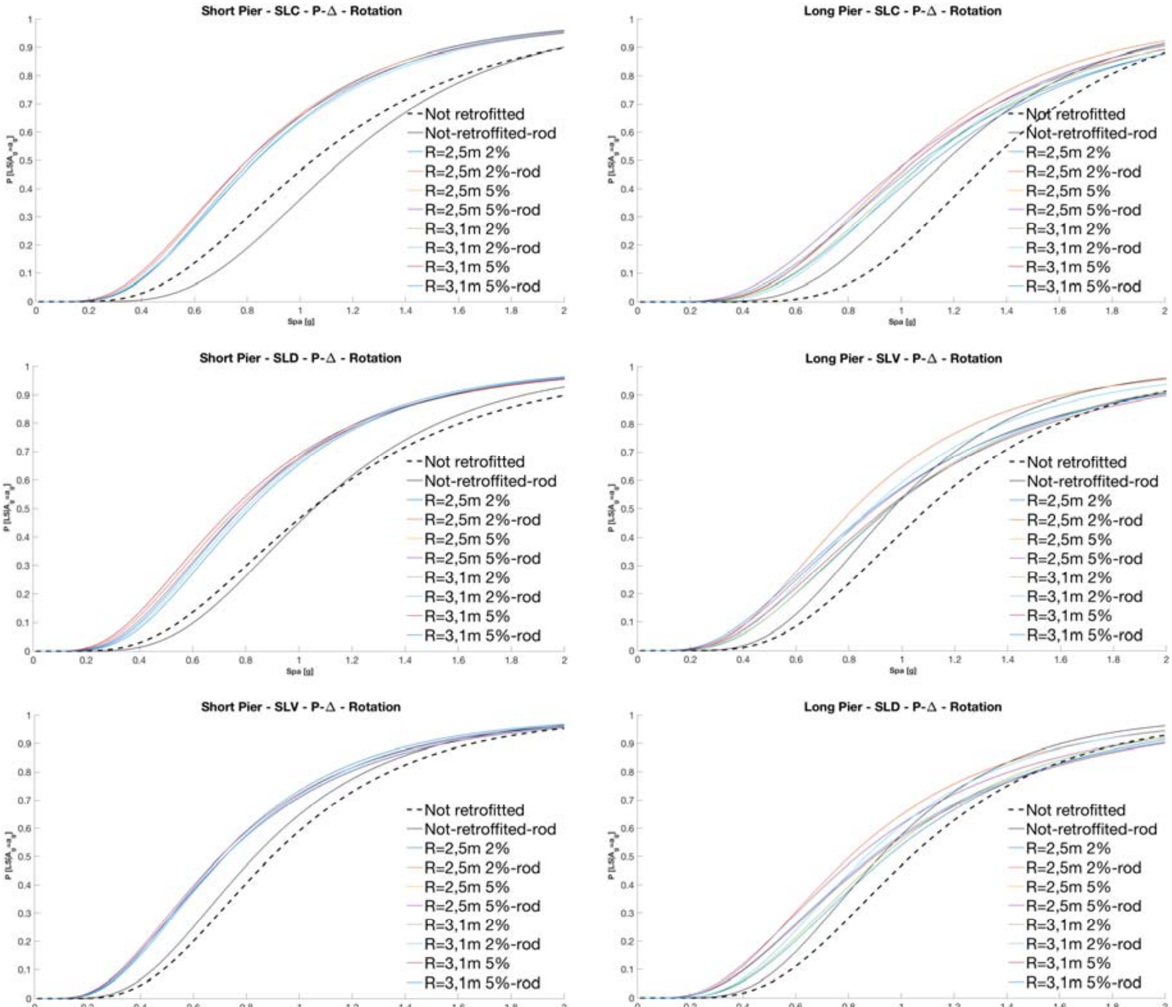


Figure 8 - *Fragility Curves of Plastic Rotation SLC-SLD-SLV with P-Delta effect*

Short Pier *Long Pier*

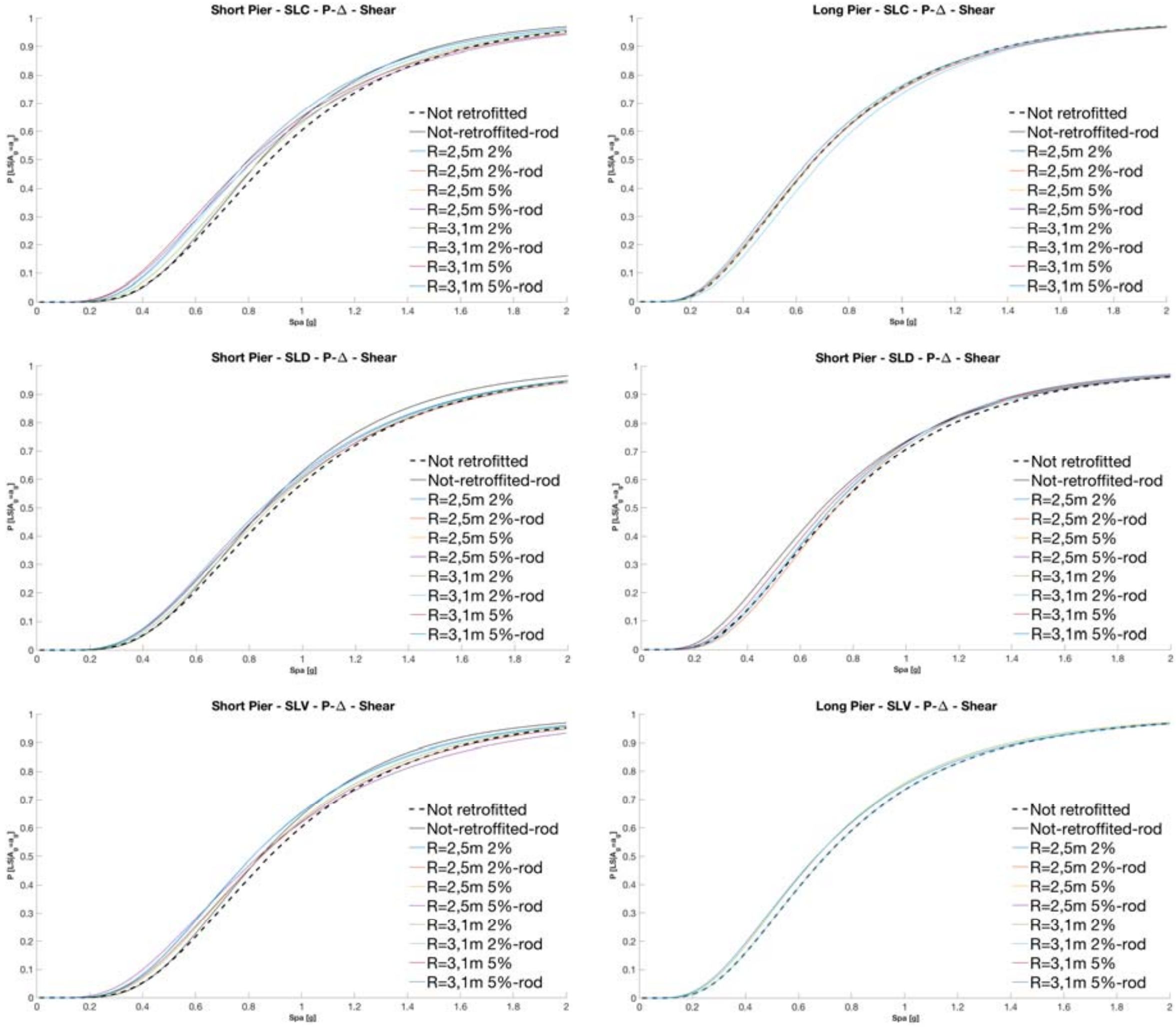


Figure 9 - *Fragility Curves of Base Shear SLC-SLD-SLV with P-Delta effect*

Figures 10 shows the comparison between the overall behaviour in the case of the presence or not of the P-Δ effects on the SLC threshold. The figures show that not all retrofit solutions have the same degree of functionality by considering P-Δ effects.

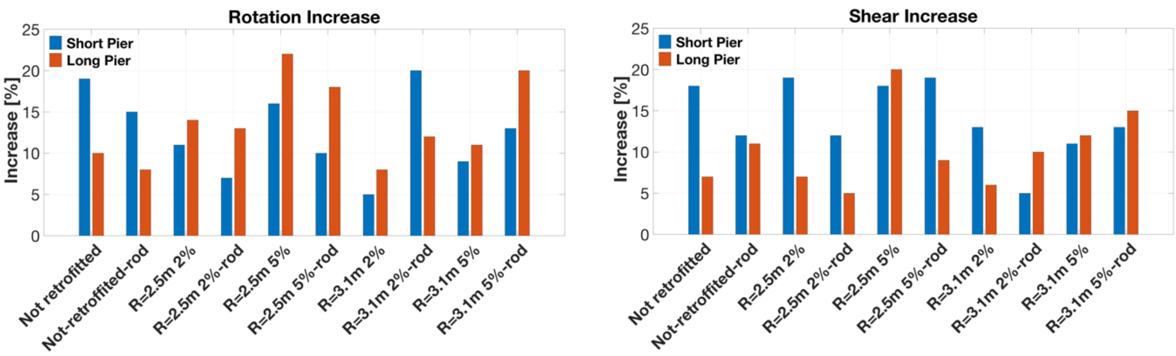


Figure 10 - *Increment of the probability of collapse obtained by considering the P-Δ effects in the case of SLC*

In particular, it is possible to see how the structural components behave differently: with regard to the rotations at the base of the piers, and therefore the possibility of forming plastic hinges, the use of FPS causes an increase in stress due to non-linearity, mainly on long stacks and when the bays are connected. Moreover, in the right figure, it is possible to observe that the maximum increment of the shear happens in the case with FPS R=2.5 and when the spans are connected.

7 Conclusion

The paper presents the numerical evaluation of fragility curves for the typical classes of bridges built in Italy between the 70s and 80s both in the retrofitted and not-retrofitted case. The methodology used for assessing the fragility curves included the use of 2-D analytical models subjected to a set of accelerograms representative of southern Italy.

Experimental results and finite element analysis were used to define the mechanical characteristics of the bridge elements in terms of shear and rotation. Several mechanical components were used to estimate the fragility of the seismically isolated system.

For the considered bridge configuration, the use of FPS isolation techniques alone seems to not be able to reduce the overall seismic response of the pears, with an increase of the fragility up to 20% of the maximum rotation at the base of the pier and up to 15% of the shear at the base of the pier. Better results have been obtained by combining the studied isolation devices and the linked decks. For all investigated cases, as expected, the P-Δ effects worsen the response of the system.

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ANALISI COMPARATA TRA STRATEGIE DI MIGLIORAMENTO SISMICO DI PONTI A TRAVATA SEMPLICEMENTE APPOGGIATA MEDIANTE CURVE DI FRAGILITÀ

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SUMMARY: *Il lavoro indaga l'affidabilità delle tecniche di isolamento sismico di ponti a più campate semplicemente appoggiati. L'obiettivo dell'isolamento sismico, per i ponti, è proteggere le pile limitando il taglio sismico trasferito all'impalcato. L'efficacia di questa strategia di adeguamento e/o miglioramento dipende dai rapporti di massa e dall'interazione dinamica complessiva tra l'impalcato e le pile. A tal fine, è stato considerato un portafoglio di ponti italiani, costruito negli anni '60 con impalcato a cassone e diverse strategie di isolamento. L'efficacia delle strategie di protezione sismica esaminate è stata studiata attraverso le curve di fragilità all'interno del metodo PBEE. In particolare, gli indici delle prestazioni, valutati da analisi dinamiche non lineari, sono stati considerati per calcolare le funzioni di fragilità utilizzando l'approccio al metodo MSA. I risultati ottenuti portano a valutare l'efficacia delle strategie di isolamento sismico di ponti a travata semplicemente, evidenziando l'influenza delle pile sul comportamento dinamico.*

KEYWORDS: *Isolamento sismico ponti, PBEE, Curve di Fragilità,*

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