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Effectiveness analysis of deck isolation retrofit for simply supported span bridge by means of combined fragility functions

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ABSTRACT
Italian road infrastructures built in the 60s and the 80s show high fragilities for both static and seismic load conditions as sadly highlighted in recent events. The design criteria used at the time with generally poor maintenance processes led to high vulnerability of such relevant constructions. Due to the international relevance of the matter, several researches have been carried out in the last decades to assess the safety of existing bridges and retrofit strategies.
This study focuses on the effectiveness of isolation strategies to retrofit the multi-span simply supported bridges by using fragility curves within the PBEE method. To this aim, specifically probabilistic seismic demand models are implemented to develop fragility curves for single damage mechanism and entire bridge behaviour by considering material and geometric nonlinear effects. In particular, the considered method envelope the behaviour of single components to develop overall fragility curves in the case of general seismic demands.

1 INTRODUCTION

The Italian road infrastructural heritage is composed of the national network, road (20773 km) and highway (6668 km), from the regional and provincial networks (151583 km) and a countless of municipal road networks (72081 km), according to data provided by ANAS and the Ministry of Infrastructure and Transport in the annual report (Office of Statistics2014). In particular, it consists of a large number of bridges, viaducts and overpasses necessary for the orography of the territory and the high urban density.

Table 1: Report od Autostrade per l'Italia.

Highway	Tunnels	Bridge	International Tunnels
68 %	14 %	17 %	1%

14% of this network are viaducts or bridge structures (Autostrade per l'Italia SpA 2015). (Casarotti, 2004) proposed an adequate structural classification of simple multi-span bridges built in the '60s and' 80s.

Table 2: Bearing device type from Casarotti Studies.

Neoprene Disc	Friction	Others
61 %	35 %	4 %

From this study, it was possible to understand that in about 61% are used neoprene bearing devices. Moreover, road infrastructures are very vulnerable, as they are affected by design (structural and technological) deficiencies related mainly to the construction period, to the applied technologies and the materials used, together, especially in the last few years, to a lack of adequate maintenance.

The paper investigates an innovative methods for the evaluation of the seismic vulnerability of existing reinforced concrete bridges in a probabilistic way, by using fragility curves. Fragility curves, as can be seen in the scientific literature, have highlighted the complexity of the bridge system, making it difficult to use in daily practice. This difficulty depends on being able to consider the bridge in its entirety, the use of a new combination between the engineering parameters would give the possibility both to understand the behaviour of the single structural element that of the entire bridge. This choice often influences the results especially for complex structures such as the bridges, to make the fragility curves immediately readable, it is herein proposed an innovative combined use of the EDPs to understand both the behaviour of the single bridge elements and global behaviour.

The paper discusses, in the end, the results of the innovative combined use of the fragility curves through an extensive probabilistic analysis by using a reference model 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 nonlinear dynamic analysis (Mander et al. 1999, Shinozuka et al. 2000).

2 FRAGILITY FUNCTION METHOD

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 (Villaverde 2007, Porter et al. 2007, Shafei et al. 2011, Padgett, J. E., et al. 2008).

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 "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 this study 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 2015, Iervolino et al. 2010, Lin et al. 2013). In this regard, the maximum likelihood method (Shinozuka 2000, Baker and Cornell 2005) 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):

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

The variability of single EDPs and the mutual influence of single damage limits leads to a difficult understanding of the overall behaviour of the bridge system. The studies done highlighted the need to understand the global damages, not the local damages of each element. The combination of all the fragility curves hitherto derived would give the possibility of comprehensively comprehending global behaviour. This combination would make it possible to identify the element with the highest probability of collapse, and not only the probability of total collapse.

The discussed method, through the envelope of the fragility curves of each EDPs considered, would give the possibility of intuitively understanding the overall behaviour of the bridge system and in addition, would allow to asses of the probability of collapse of the structural element.

$$\max_{i=1}[P(F_i)] \leq P(F_{sys}) \leq 1 - \prod_{i=1}^{m}[1 - P(F_i)] \qquad (4)$$

These first order limits are valid for a series-type system, in which a failure of one of the components constitutes a system error (Melchers RE. 1999). When a bridge is modelled in the longitudinal direction as in this study, in fact, it behaves like a series system. The lower limit represents the probability of failure for a system whose components are all entirely dependent on the stochastic point of view. The upper limit assumes that the components are all statistically independent and provide a conservative approach to estimate the overall fragility of the bridge.

3 SEISMIC DEMAND

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).

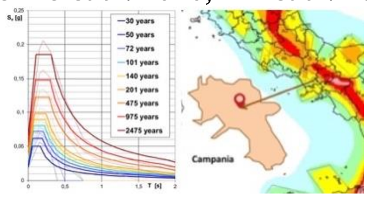
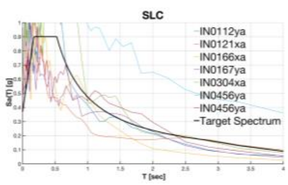
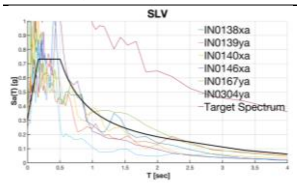


Figure 1. Elastic response spectra for different reference return period and location

According to the Italian Technical Regulations for Construction (NTC 2018), the seismic actions that have to be considered for design purposes are defined from the "seismic hazard" of the construction site Figure 1.

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





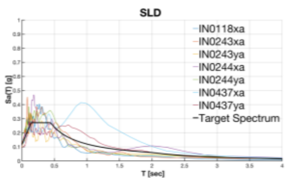


Figure 2 - Elastic demand spectra considered for SLC, SLD and SLV for a bridge of class III and Vn=50 years

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 2 describes a summary of the number 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, the considered seismic events were scaled by changing the PGA in the range 0.0-1.0g with a step of 0.1g to implement the MSA analysis.

4 FRAGILITY ANALYSIS OF HIGHWAY BRIDGE

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. 2016, Petti et al.2018).

4.1 Reference bridge description and modelling

The research was done on typical highways and motorways bridges built in the 60s and 80s in the form of a simply supported beam and high piers as shown in Figure 4.

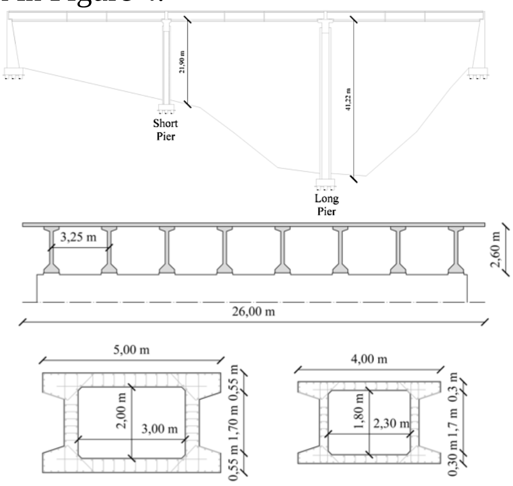


Figure 3 - Geometrical characteristics of the refernce bridge

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.

The bridge was modelled with the software SAP2000 (Computer and Structures, Inc. 2016) 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). Two radii of curvature (R=2.50m and R=3.10m) have been considered for the FPS devices.

Moreover to simplify the computational cost, an automated procedure has been implemented that uses Matlab scripts to generate the FEM model and extrapolate the results from it.

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. Given the high stiffness of the pile foundation in respect to the pier, it was decided to use a fully-fixed support condition. 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 4, evaluated by considering a fibre section analysis using the software SAP2000.

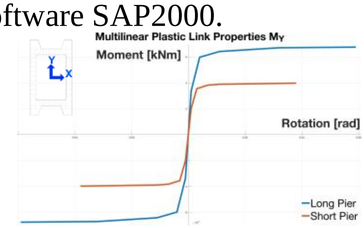


Figure 4. Mechanical characteristics of the Multilinear Plastic Link Properties

The bearings of the reference 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. In Table 3, Table 4 and Table 5 are described the mechanical properties of the bearings used in the model.

Table 3 Mechanical characteristics of elastomeric bearings

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

Table 4. 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. 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

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.

4.2 *Alternative Strategies for retrofitting*

Two different configurations for the bridge retrofitting were considered, by using two different values of the radius of curvature of 2.5 and 3.1 m. Table 6.

Table 6. Bridge Configurations

Retrofit Option	Description
R=2.5 2%	Friction Pendulum isolator with effective radius of concave sliding surface equal to 2.5 m and Coulomb friction 2%
R=3.1 2%	Friction Pendulum isolator with effective radius of concave sliding surface equal to 3.1 m and Coulomb friction 2%

5 BRIDGE CAPACITY AND LIMIT STATES THRESHOLDS

The analysis of the damage reported by the bridges during the recent earthquakes, highlights the presence of structural deficiencies due to old design techniques. The main damages that are identified are the failure of the piers for shear and ductility mechanisms for the substructure, while the span pounding for the superstructures. Most studies on fragility analysis of bridges use column ductility as the primary damage measure. Park and Ang 1985 suggested a damage index based on energy dissipation, and Hwang et al. 2000, used the capacity/demand ratio of the bridge piers to develop fragility curves. In this study, damage states are defined for piers ductility demand, piers shear demand and span pounding. The discrete conditions of damage were defined based on the response of the structures obtained from the performed nonlinear static analyses. Nonlinear static analyses were conducted on the bridge piers, based on the obtained results, the comparison between the maximum δ_{max} and ultimate δ_u deformation or stress were used to define the model of damage.

$$DI = \delta_{max} \geq \delta_u \tag{5}$$

Recent studies on bridge infrastructures in Italy (Cardone et al. 2011, Borzi et al. 2014), consider two limit damage state (DS) or performance levels: Limit State Damage (LSG) and Limit State Collapse (LSC). The damage state LSG defines the condition of limited structural damages in which it would be careful to implement structural repairs. The damage state LSC describes the condition in which the bridge is severely damaged, and it is near to collapse. This implies that significant degradation has occurred in the stiffness and strength of the piers, and large displacements occur which might cause span pounding. Given that the objective of the study is to evaluate the seismic vulnerability of the entire bridge system, it will be considered only the LSC damage index Table 7.

Table7: Definition of Limit States.

Damage State	Failure mechanism	Description	
Collapse (LSC)	Pier flexural capacity	Pier chord rotation exceeds pier chord rotation at collapse	$\theta \geq \theta_u$
	Pier shear capacity	Pier shear force exceeds pier shear resistance	$V \geq V_r(\theta)$
	Span pounding	Impact between adjacent spans	$\delta \geq \delta_u$
	Unseating of the deck	Deck displacement in the longitudinal direction is greater than the seat length	

The capacity model is needed to measure the damage of structural components and the entire system, and it is described here concerning 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. The values of resistant shear shown in Table 8 have been derived by using the Priestley formulation (Priestley 1996). The rotations Table 8 show the limit has been set by using the criteria set out in section 8 of the Italian NTC 2018.

Table 8: Resisting shear V_R according to the Priestley formulation and the base Plastic hinge limit rotations according to NTC 2018.

V_R [kN]			
SLC			
Piers	Not retrofitted	R=2.5 m	R=3.1 m
S	1256	1270	1271
L	1397	1419	1421

Piers	ϑ_{SLC} (rad)
S (short)	0.0071
L (long)	0.0212

For the geometry of the reference bridge used for the analyses and the retrofit techniques proposed, the span pounding is a dominant phenomenon of collapse compared to the unseating of the deck. From the original drawings, it has been found that the length between the two spans is equal to about 60 cm, it is considered the maximum allowable displacement before hammering for each span is equal to 30cm.

5.1 *Fragility function of the bridges*

By using the methodology presented above, fragility curves were evaluated

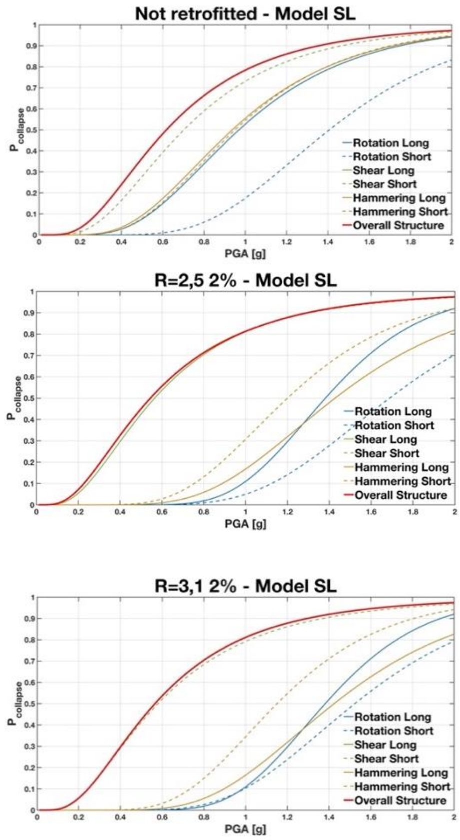


Figure 5. Envelope of the fragility curves

Table 9. Typologies of mechanism collapsed and Collapse Probability at 0,5 g.

Model	Type of mechanism collapsed	Collapsed Probability
Not Retrofitted	SL	77 %
R=2,5 2%	SS	80 %
R=3,1 2%	SL	80 %

SL = Shear Pier Long; SS = Shear Pier Short

The envelope of the fragility curves it's shown in the figure 5, of the individual EDPs gives the possibility to understand the mechanism that causes the structural collapse. From the Table 9 it is clear that the geometric configuration of the bridge is the first variable that influences the type of collapse mechanism. Although improvements have been found for individual EDPs in many cases, the general behaviour of the bridge system has not undergone a lowering of the probability of collapse as can be seen in Table 9 where the probability of collapse is reported to a PGA of 0.5g.This highlights the complexity of the bridge structural system, where the improvement of a single element may not lead to the improvement of the overall behaviour. These fragility curves can be used in determining the potential losses resulting from earthquakes and can be used to assign prioritization for retrofitting.In this case, in fact, the piers are elements endowed with high seismic mass characterized by a relevant dynamic behaviour, moreover, they were designed in the 60s and 70s when the design criteria were noticeably different from the modern ones, and therefore characterized by a fragile behaviour towards shearing actions. For this reason, what has emerged from the results obtained is the difficulty to improve the structural performance of the entire bridge with only the FPS isolators. Therefore, in order to reduce the vulnerability of the piers elements, it might be appropriate to adopt combined retrofit strategies that envisage the use of both retrofitting techniques, aimed at improving flexural and shear strength, as well as seismic isolation.

6 CONCLUSION

The paper presents an innovative method for deriving fragility curves, useful for understanding the overall behaviour of the structure.

The results show that the application of seismic isolation using FPS systems may not be effective in the improvement or seismic adaptation of particular complex structures such as bridges and viaducts. In cases where the substructure (piers) is caracterized by high seismic mass, the local seismic response can overcome the overal response. In this cases, the desk isolation do not improve the schear beaviour of the piers.

The presented method can be improved by considering further types of descriptive EDP of the behaviour, i.e. the abutments and foundations to more thoroughly investigate the overall response of the structure.

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