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RELIABILITY ANALYSIS OF SEISMIC ISOLATION IN RETROFITTING OF SIMPLY SUPPORTED BRIDGES

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Abstract

The paper investigates the reliability of simply supported bridges retrofitted with seismic isolation, using fragility curves, which describe the probability of reaching a certain damage level for an assigned seismic intensity. Taking advantage of the Multi Stripes methodology, nonlinear dynamic analyses of a multi-span bridge, representing the existing ones in Italy built in the 60', have been carried out in order to obtain the fragility functions.

The obtained results allow to assess the isolation retrofit strategies effectiveness to mitigate the seismic risk of simply supported bridges, highlighting the influence of different design strategies on the probability of exceeding the limit states considered.

Keywords: Bridge assessment, Fragility curves, Seismic Isolation

1. Introduction

Since the end of the 1980s, many studies have been carried out analyzing the relationship between infrastructure and economic development of an area; it is recognized that there is a close correlation between the infrastructure and the economic growth of an area. In this context, it is useful to refer to the socioeconomic classification proposed by Hansen (1956) [1] which divides infrastructure in economic, including transport networks of goods, people and energy, social infrastructure, including health and culture infrastructure, innovation, research and development activities and technological and communication, environmental, justice infrastructure, and area structures, including touristic accommodation, trade and monetary intermediation structures. It is therefore clear that the physical elements that build up the transport infrastructure determine and/or affect their ability to achieve the objectives for which they are designed and manufactured.

The problem is particularly evident in transport networks such as bridges, multi span bridge and tunnels which, as highlighted by the seismic events that have stricken Italy in recent decades, have shown a high vulnerability for medium intensity events. The Italian infrastructure network consists of bridges built for the most part in the 60' and 70' in areas generally recognized as seismic only later and, therefore, designed without Earthquake Engineering criteria. Moreover, the lack of ordinary maintenance policies and practices have emphasized the decay of structural performances. This research topic has been studied in an European project called "*Strit*", that stands for "Tools and technologies for the management of the risk of the Transport Infrastructure", in collaboration with other Italian universities.

The issue of existing bridge structural safety has been a focus of attention also at international level due to damage occurred in advanced countries, in the 70s and 80s, to transport infrastructure designed with anti-seismic criteria [2]. In such contexts, it became clear that the old design methods, based on the stress limit elastic method, were inadequate to verify the inelastic structural behavior. The consequences of the elastic design were the underestimation of displacements and deformations under seismic loads, moreover the presence of inelastic actions and concepts of ductility and hierarchy of strength were not taken into account. There is, therefore, the need of maintenance policies aimed at enabling simple improvement or upgrading of existing structures taking into account the economic constraints which do not allow generalized measures of replacement of existing elements. In this context, the new strategies of seismic protection developed in recent decades, including seismic isolation, allow to greatly improve the seismic performance of existing structures even against events of greater magnitude [3,4,5,6].

The research investigates the effectiveness of seismic isolation in the case of retrofitting or seismic upgrading of existing bridge structures, analyzing the behavior of viaducts built in the 60's and 70's belonging to the structural typology of simply supported beams and high caisson piers. In particular, the research aims at investigating the efficiency of different design approaches, applied to a reference structural model, comparing the seismic vulnerability expressed by appropriate fragility functions calculated using nonlinear dynamic analysis [7,8,9,10]. This approach pays attention to the structural behavior of specific parts that constitute the bridge, considering the uncertainties related to the seismic action that, in the case study, is dominant in reference to other kinds of uncertainties.

2. Reference viaduct description

The research considers the study of simply supported beams and high caisson piers bridge Fig.1. The structural type investigated is typical of viaducts built in the 60's and 70's in correspondence of high speed roads.

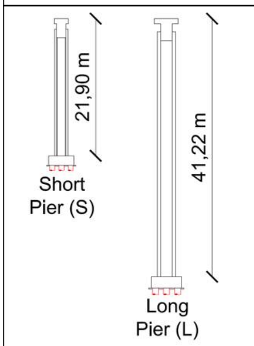
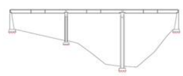
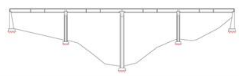
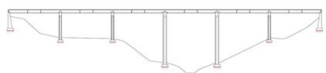
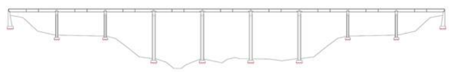
Tipologize of piers	Viaduct configuration	Number of span	Scheme of Viaduct
	1S1L	2	
	1S1L1S	4	
	2S2L1S	6	
	2S4L2S	9	

Fig.1 – Analyzed viaduct configurations

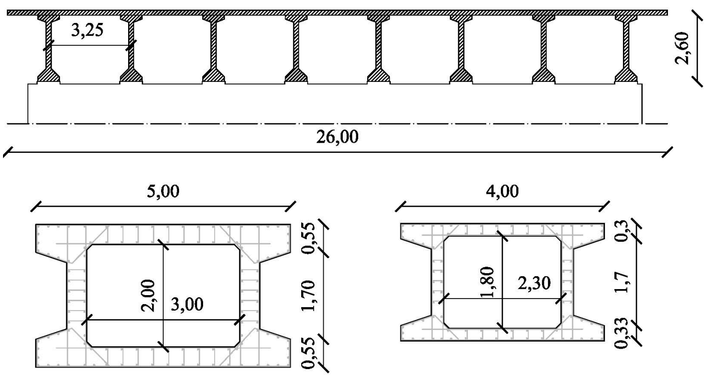


Fig.2 – Geometrical characteristics of the deck and the piers

In particular the bridges taken into account represent some of the possible recurring configuration. The bridge presents a variable number of spans from three to nine of the same length, equal to 41.00 m, consisting of r.c. slab with thickness of 0.20 m and supported by 8 longitudinal prestressed reinforced concrete beams. Furthermore the number of box-coupled piers depend from the number of span. There are two type of piers: the first one of height 21.90 m (Short – S) and the second one of height 41.22 m (Long – L), plinth on piles indirect foundations and containment abutments. The bridge has a straight and horizontal axis. As regards the piers, as mentioned, the same are coupled in the transverse direction, thus realizing a frame.

To the research scope, plan finite element numerical models were built using "Sap2000" program [11]. In particular, the models are representative of the bridge in the present and design state, assuming as the original condition neoprene supports for the prestressed beam, and as design conditions seismic Friction Pendulum System (FPS) bearings,. For the analysis of the effectiveness of the proposed isolation retrofit systems, for both types of support, two different solutions of relative constraint between the spans have also been considered: beams disconnected between the different spans and connected in series.

The finite element numerical model provides coupled columns described by individual frame elements, discretized with sub-frame of length equal to 3.00m, decks described by frame elements, geometrically connected to the columns by means of auxiliary nodes and Constraints (type body) descriptive of the geometry of pulvinus, fixed joints in foundation representative of the relative stiffness of the plinths on piles.

Frame elements, descriptive of decks and piers, were modeled assuming a generic section defining the mechanical parameters according to the actual geometry. The inelastic behavior of piers was modeled using Multinear Plastic hinges, Fig.3, assigning each hinge the bending-rotation relation corresponding to the behavior of a reference single pier investigated assuming a fiber model for the section in the original vertical load assumptions.

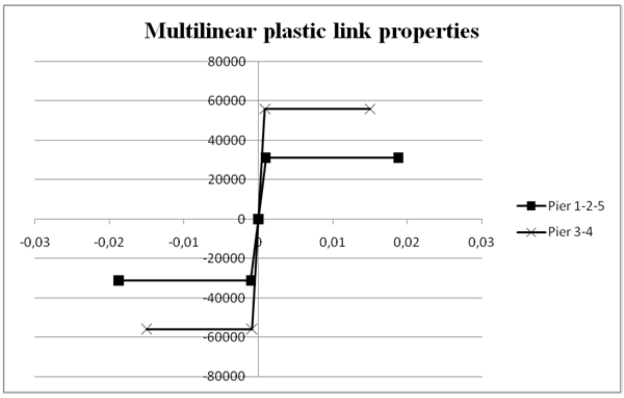


Fig.3 – Plasticity modelling

The supports, descriptive of the present status or the design status, are characterized by double joint link gap. In particular, in the case of retrofit with seismic isolation the link used is Friction Isolator [12,13] that allows to relate the shear response to the frictional properties, the radius of curvature and the axial stress on the device. In the following tables are shown the mechanical properties of the supports considered in the models (Table 1, Table 2, Table 3):

Table 1 – Current state - Mechanical characteristics of elastomeric bearings

F_{zd} *(kN)*	K_o *(kN/mm)*	K_v *(kN/mm)*
1250	3,43	1114

Table 2 – 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 3 – 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

The bridge deck has been modeled, as said, with elastic elements of Frame type described by an equivalent section in terms of area and inertia, the mass is concentrated along the barycentric axis.

3. Vulnerability assessment

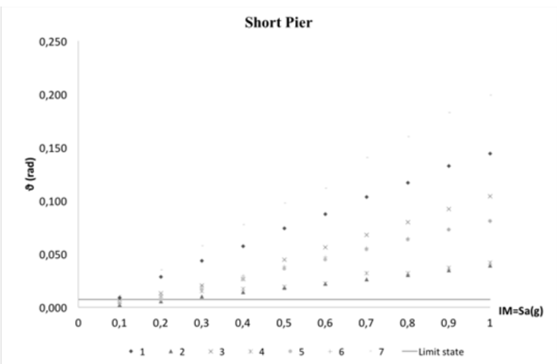
The vulnerability of the considered bridge was investigated by using fragility curves, calculated taking advantage of the Multi-stripes analysis (MSA) method [14] which provides the performance of non-linear time history analysis. In particular, the so-called approach of the Conditional Spectrum has been used, providing for each limit state investigated the use of a set of seismic events recordings scaled according to the variation of seismic intensity (IM - Intensity Measure) described by the spectral pseudo acceleration (SPA), evaluated in correspondence of the fundamental vibration period of the bridge. [15, 16, 17].

For each limit state, the seismic events considered were scaled by changing the SPA in the range 0.0-1.0g with a step of 0.1g. From the analyses it is possible to calculate the fraction of earthquakes that produces the exceedance of the limit state considered for each IM level. In this regard, the maximum likelihood method [18, 19, 20] has been used. In particular, for each level of seismic intensity IMJ considered, the probability P (z_j) of exceeding the limit state is given by the binomial distribution Eq.(1):

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

where n_j describes the number of seismic events considered, z_j the number of events for which the state limit is not fulfilled and p_j the probability that it has an intensity IMJ. By using the maximum likelihood approach, then, the function of fragility is derived, which represents the function which corresponds to the highest probability of correlation with the results obtained from all the analyses carried out by varying the seismic intensity. In this regard, assuming a log-normal law probability distribution to describe the state limit checks, the parameters average (θ) and variance (β) can be estimated as Eq.(2):

$$\{\hat{\vartheta}, \hat{\beta}\} = \arg\max_{\vartheta,\beta} \sum_{j=1}^{m} \left\{ \ln \binom{n_j}{z_j} + z_j \ln\Phi\left(\frac{\ln\left(\frac{x_j}{\vartheta}\right)}{\beta}\right) + (n_j - z_j) \ln\left(1 - \Phi\left(\frac{\ln\left(\frac{x_j}{\vartheta}\right)}{\beta}\right)\right)\right\} \tag{2}$$



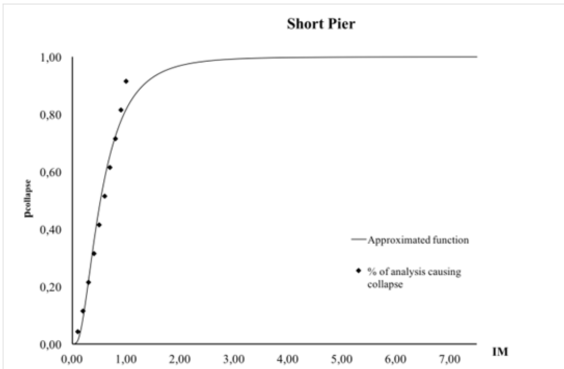


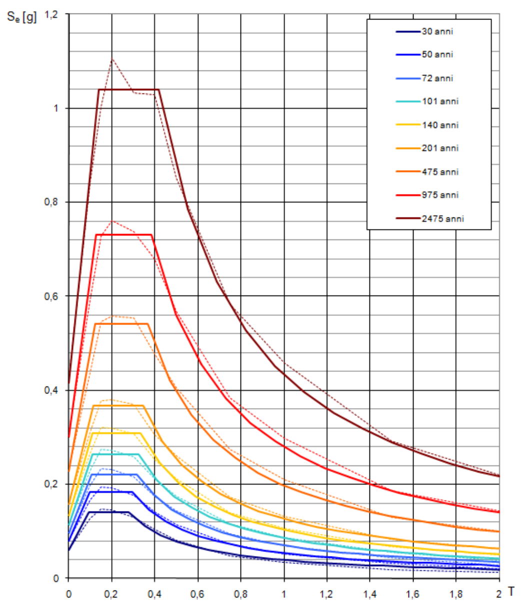
Fig.4 – Number of events causing the exceeding of limit state, left - Example of fragility curve, right.

4. Definition of input ground motion

According to current Italian technical codes for buildings NTC 2008 [21], seismic design actions have been defined taking into account the "basic seismic hazard" of the construction site for each considered limit state. The seismic hazard is defined in terms of maximum horizontal acceleration a_g in free field conditions on rigid reference site with horizontal topographic surface, as well as the ordinates of the elastic response spectrum in acceleration, according to the parameters a_g (maximum horizontal acceleration at the site), F_0 (maximum value of the amplification factor of the horizontal acceleration spectrum) and T_c^* (vibration period at the beginning of the constant velocity tract in the horizontal acceleration spectrum); the construction site is characterized by the following geographical coordinates: 14.975 Longitude - Latitude 41.0264 located in Campania Region (Italy). Table 4 and figure 5 represent respectively the seismic demand parameters and the elastic spectral demand.

Table 4 – Basic seismic hazard parameters

T_r *(years)*	A_g *(g)*	F_o *(-)*	T_c^* *(s)*
30	0,060	2,355	0,279
50	0,080	2,307	0,296
72	0,096	2,297	0,317
101	0,114	2,312	0,327
140	0,133	2,319	0,335
201	0,158	2,327	0,345
475	0,228	2,369	0,366
975	0,302,	2,421	0,383
2475	0,416	2,503	0,417



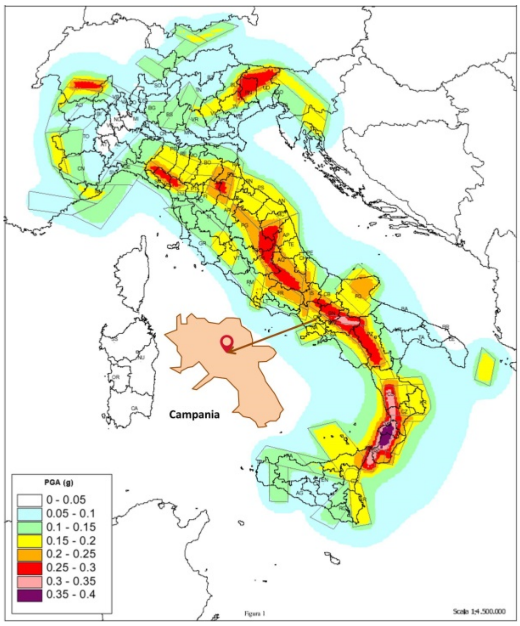


Fig. 5 – Elastic response spectra for different reference return period and Location

The MSA method uses the results obtained by non linear FNA analyses to derive the fragility functions, it has been therefore necessary using a significant number of input earthquakes records to obtain a better statistical prediction of the bridge seismic response. To the scope, 56 earthquake ground motions have been selected Table 5 by using the REXEL software [22], which allowed to obtain combinations of accelerograms compatible with the Italian code spectrum in the range of vibration periods, 0.15s-4T. Table 5 shows a summary of the number of natural event recordings considered, while Figure 6 shows the compatible elastic spectra for the limit states SLD, SLV and SLC.

Table 5 – Number of seismic input considered for different limit states and bridge class

Nominal Life	50		100	
Use class	III	IV	III	IV
SLC	7	7	7	7

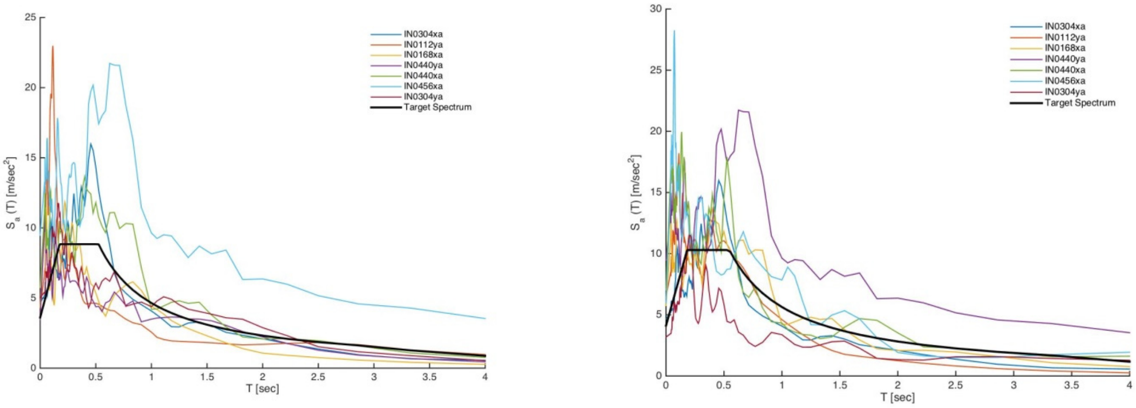


Fig. 6 – Compatible elastic spectra considered for SLC class of bridge III and Vn=50 years on the left and Vn 100 on the right

5. Demage assessment

The response of structures subjected to seismic events are represented and checked by an appropriate choice of Engineering Demand Parameters (EDP_S) in fragility analysis. In the study, the damage of structural elements is represented by damage indexes in terms of exceedance of the considered Limit States, evaluated by means of shear and plastic rotations at the piers base. The values of resistant shear have been derived by using the Priestley formulation [2], the rotations limit by using, instead, the criteria set out in section 8 of the Italian NTC 2008 [21]. The Tables 5 and 6 summarizes the considered EDPs limits:

Table 5 – Resisting shear according to the Priestley formulation

	V_R [kN] – SLC		
Pier	**Not retrofitted**	**R=2,5 m**	**R=3,1 m**
1	1256	1270	1271
2	1245	1259	1260
3	1398	1420	1421
4	1397	1419	1421
5	1250	1264	1265

Table 6 – Base hinge limit rotations according to NTC 2008

Pier	ϑ_{SLC} (rad)
1	0.028
2	0.028
3	0.044
4	0.044
5	0.028

6. Numerical analysis

For the purposes of this study, maximum base rotations of the piers have been investigated according to the considered seismic events by varying IM, described by the spectral acceleration evaluated in correspondence of the fundamental vibration period. In particular, the fragility curves, descriptive of the short and long pier, have been derived considering a spectral acceleration variable from 0.1g to 1.0g, with 0.1g step.

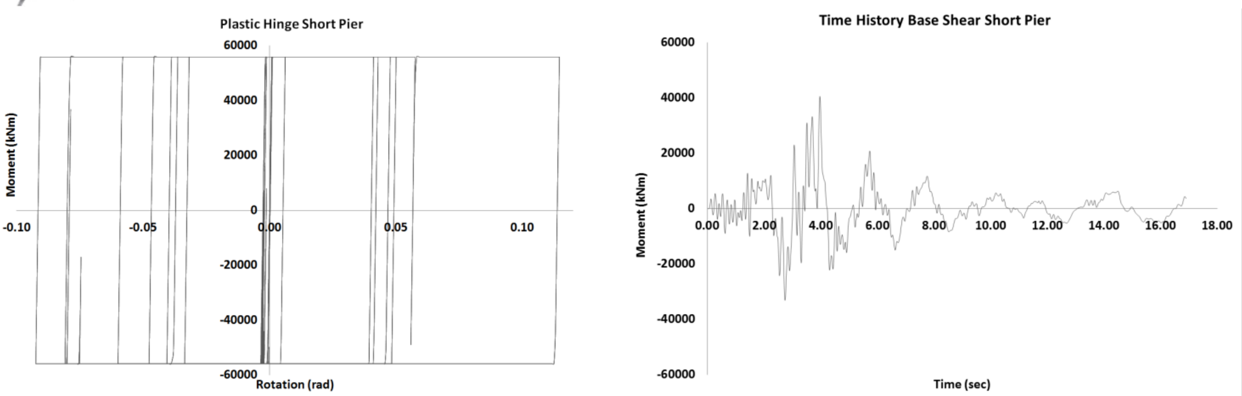
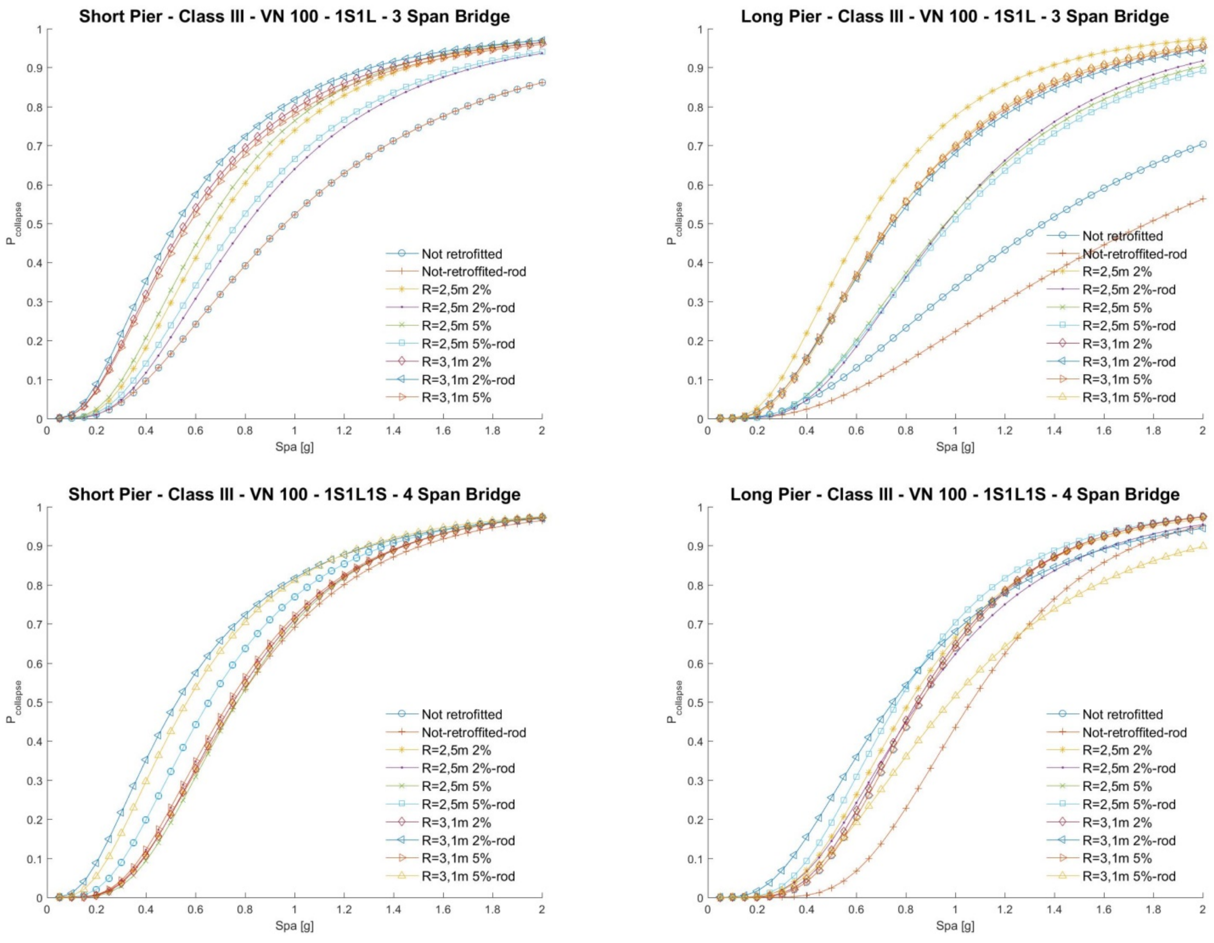


Fig. 7 – Example of moment rotation hysteretic cycle and base shear non-linear time history examples



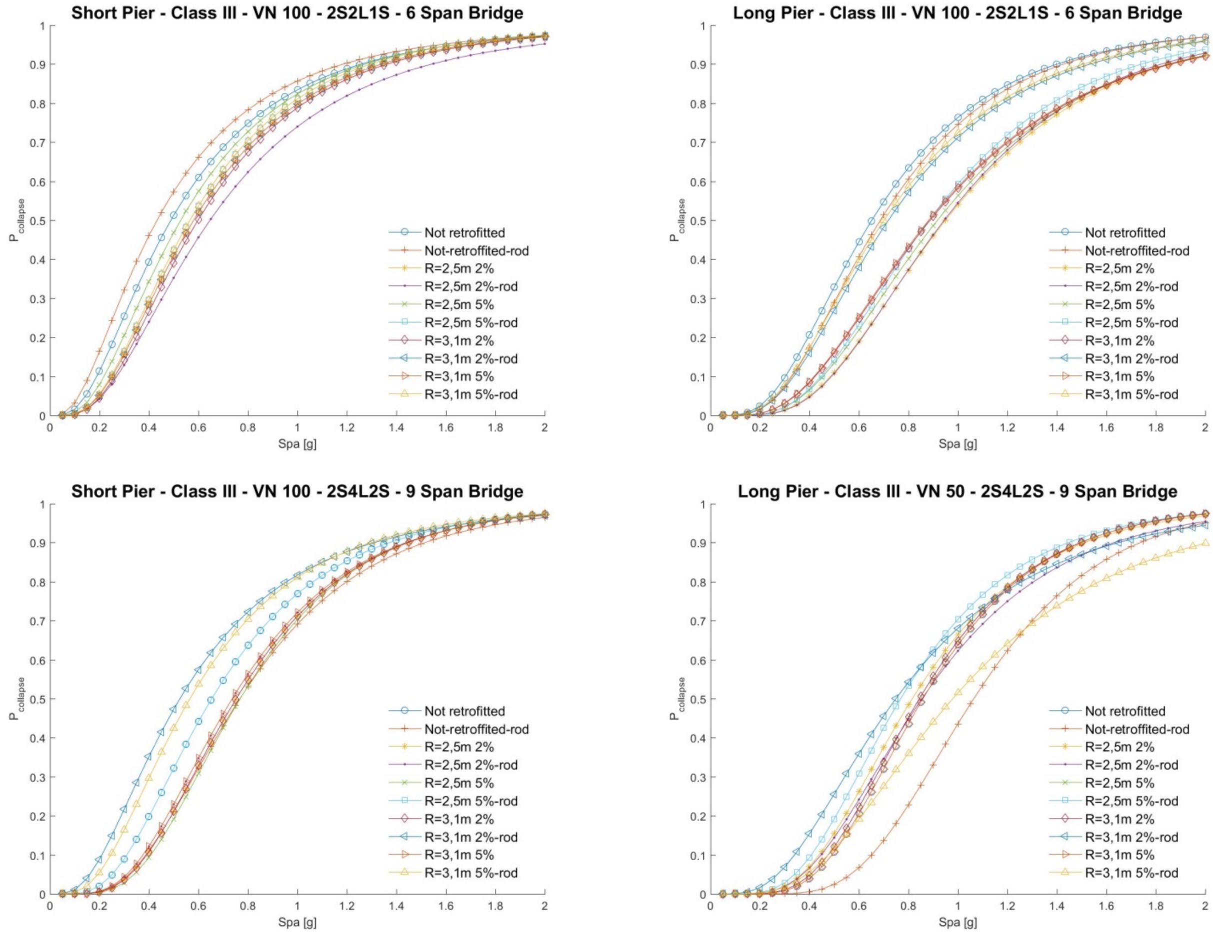


Fig. 8 –Fragility curves for VN=100

Fig. 8 shows the fragility curves of the different bridge considered in this reserch for the short piers and long pier for the SLC limit states, use class III and nominal life equal to 50 years in configurations not retrofitted, not retrofitted with connected spans, isolated with FPS respectively of radius 2.50m and 3.10m and friction coefficient 2% and 5%, retrofitted with connected spans.

The results show the effectiveness of the isolation strategy in the reduction of the probability of exceeding the limit states in the case of the short piers. In the case of long piers, the isolation strategy with FPS isolators of radius 2.5m and coefficient of friction equal to 2% is the most effective in the case of Collapse Limit State; the isolation strategy with FPS isolators of radius 3,1m and friction coefficient of 5% is the most effective in the case of Life Safety Limit State; the isolation strategy with FPS isolators of radius 2.5m, friction coefficient 2% and connected spans is the most effective in the case of Damage Control Limit State.

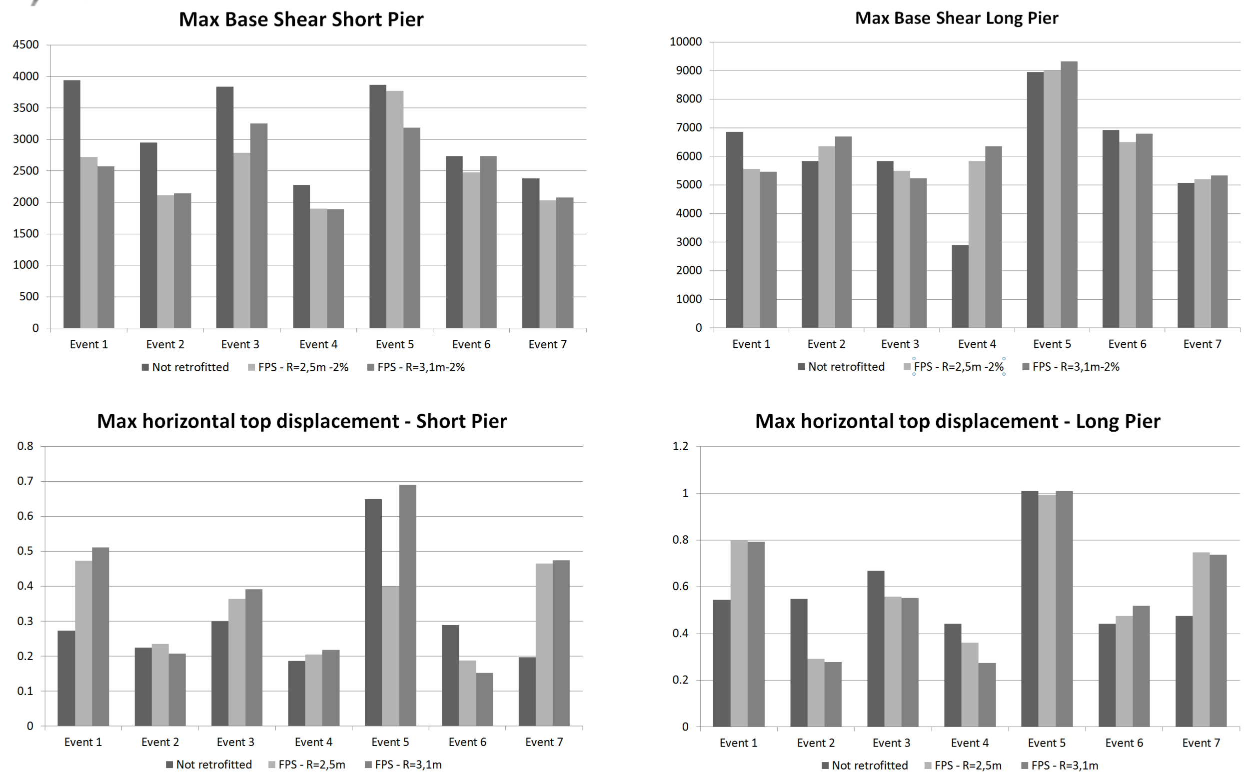
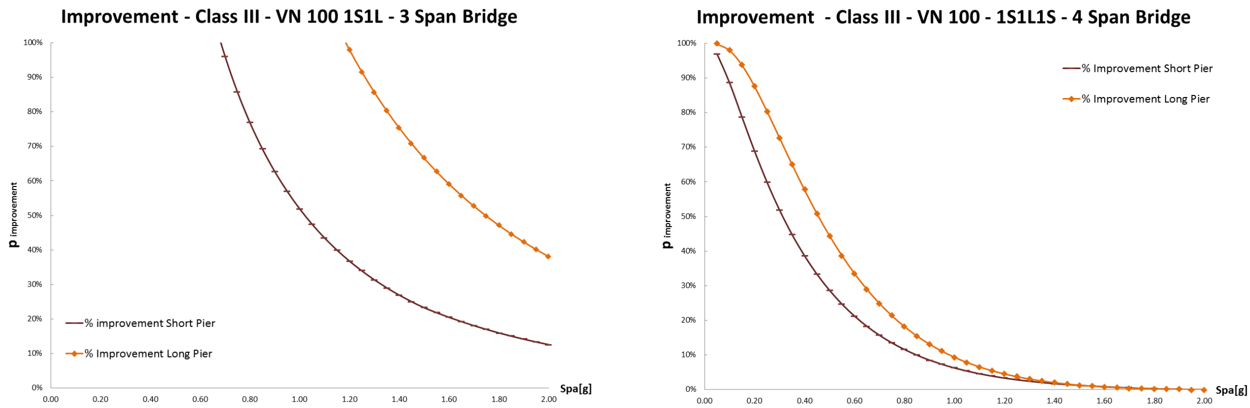


Fig. 9 – Maximum base shear and maximum horizontal top displacement for VN=50 Cu=III

The bar graphs Fig.9 show the maximum base shear and maximum horizontal displacement at the top, respectively of short and long piers, for SLC limit state with use class III and nominal life equal to 50 years in configurations not retrofitted and isolated with FPS respectively of radius 2,5 and 3,1m. Fig. 10 summarize the overall improvement of the seismic response of analyzed viaducts in terms of plastic hinge rotations, describing the percentage decrease of the response of the isolated cases.



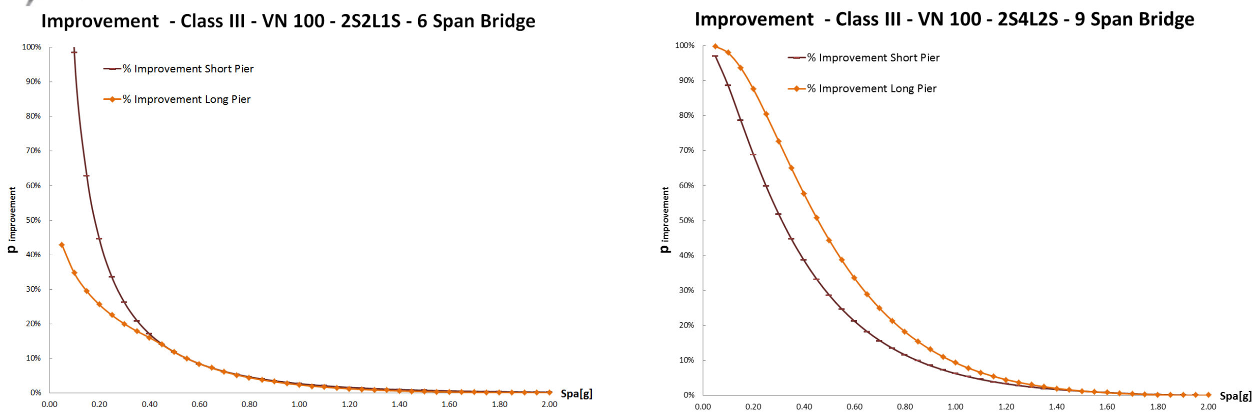


Fig. 10 – Improvement (%) of the analyzed viaduct – retrofitted vs original configuration

Results show, for both types of piers, not always a significant reduction of the base shear in the case of retrofit with seismic isolation. Instead, the isolation strategy leads to generally decrease the plastic demand of the piers. Therefore it is clear that the original shear capacity of the considered bridge piers plays generally an important role on the overall seismic capacity that isolation cannot easily resolve.

In order to reduce the vulnerability of the pier element, would be then opportune to adopt combined retrofit strategies that provide both the use of classical consolidation techniques, aimed at improving shear strength, and seismic isolation strategy.

7. Conclusion

The work has investigated the reliability of simply supported span bridges typical of the 60' and 70' in Italy, retrofitted with seismic isolation. The vulnerability analysis carried out deriving appropriate fragility curves has allowed to evaluate the effectiveness of the seismic isolation strategy, taking into account the uncertainties related to the definition of the seismic action.

The most relevant aspect emerged from the results appears to be that the seismic isolation strategy with friction pendulum bearings is not entirely effective in the improvement of seismic performances of such bridges. In the considered case, the piers are elements with high seismic mass and have a relevant dynamic behavior, moreover it were designed in the 60' when the seismic design criteria were significantly different from the modern ones, leading to shear fragile behavior.

The analyses highlight the role of the original shear strength and the need to combine classical and innovative retrofit techniques, shear strength and isolation.

8. References

[1] HANSEN, N.M., "The Structure and Determinants of Local Public Investment Expenditures", *The Review of Economics and Statistics*, Vol.47, No.2 (1965), pp. 150-162

[2] PRIESTLEY, M., SEIBLE, F., Calvi, G., "Seismic Design and Retrofit of Bridges", Wiley Interscience, New York, N.Y.,1996. 400

[3] BUCKLE, IG, MAYES, RL. "Seismic isolation: History, application and performance_a world view", *Earthquake Spectra*, 6(2), (1990), pp. 161-201.

[4] SKINNER, RI., ROBINSON, W.H., MCVERRY, G.H., "An introduction to seismic isolation", NY: John Wiley & Sons, (1993).

[5] NAEIM, F., KELLY, J.M., "Design of seismic isolated structures: From theory to practice", NY: John Wiley & Sons, (1999).

[6] IMBSEN, R.A., "Use of isolation for seismic retrofitting bridges", *J Bridge Eng ASCE*, 6(6), (2001), pp. 425-438.

[7] MANDER, J.B., BASÖZ, N., "Seismic fragility curves theory for highway bridges", *Proceedings of 5th US conference on lifeline earthquake engineering*, (1999). pp. 31-40.

[8] SHINOZUKA, M., FENG, M.Q., KIM, H.-K., KIM, S.-H., "Nonlinear static procedure for fragility curve development", *J Eng Mech ASCE,* 126(12), (2000), pp. 1287-1295.

[9] MACKIE, K.R., STOJADINOVI¢, B., "Fragility curves for reinforced concrete highway overpass bridges", *Proceedings of 13th world conference on earthquake engineering*, Paper No. 1553, (2004).

[10] CHOI, E., DESROCHES, R., NIELSON, B., "Seismic fragility of typical bridges in moderate seismic zones", *Eng Struct*, 26, (2004), pp. 187-199.

[11] SAP 2000 Computers and Structures, Inc., Berkeley, California, USA.

[12] CSI Analysis Reference Manual, "SAP 2000," Berkeley.

[13] PETTI, L., POLICHETTI, F., LODATO, A., PALAZZO, B., "Modelling and analysis of base isolated structures with friction pendulum system considering near fault events", *Open Journal of Civil Engineering*, 3, (2013), pp. 86-93.

[14] BAKER, J.W., "Efficient analytical fragility function fitting using dynamic structural analysis", *M.EERI, Technical Note,* (2014).

[15] BRADLEY, B. A., "A generalized conditional intensity measure approach and holistic ground - motion selection", *Earthquake Engineering & Structural Dynamics*, 39(12), (2010), pp. 1321–1342.

[16] IERVOLINO, I., GIORGIO, M., GALASSO, C., MANFREDI, G. "Conditional Hazard Maps for Secondary Intensity Measures", *Bulletin of the Seismological Society of America*, 100(6), (2010), pp. 3312–3319.

[17] LIN, T., HASELTON, C. B., BAKER, J. W., "Conditional spectrum-based ground motion selection. Part I: Hazard consistency for risk-based assessments", *Earthquake Engineering & Structural Dynamics*, 42(12), (2013), pp. 1847–1865.

[18] SHINOZUKA, M., FENG, M. Q., LEE, J., NAGANUMA, T.,. "Statistical analysis of fragility curves", *Journal of Engineering Mechanics*, 126(12), (2000), pp. 1224–1231.

[19] BAKER, J. W., CORNELL, C. A., "Vector-valued ground motion intensity measures for probabilistic seismic demand analysis", *Report No. 150, John A. Blume Earthquake Engineering Center, Stanford, CA*, (2005), 321p.

[20] STRAUB, D., DER KIUREGHIAN, A., "Improved seismic fragility modeling from empirical data", *Structural Safety*, 30(4), (2008), pp. 320–336.

[21] NORME TECNICHE PER LE COSTRUZIONI – NTC08 D.M. 14/01/2008

[22] IERVOLINO I., GALASSO C., COSENZA E., "Rexel: computer aided record selection for code-based seismic structural analysis", *Bulletin of Earthquake Engineering*, 8, (2009), pp. 339-362.