SEISMIC RELIABILITY OF BRIDGES EQUIPPED WITH FPS

AFFIDABILITA' SISMICA DI PONTI ISOLATI CON DISPOSITIVI FPS

SEISMIC RELIABILITY OF BRIDGES EQUIPPED WITH FPS

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ABSTRACT

The present analysis deals with the seismic reliability of isolated multi-span continuous deck bridges considering as the main aleatory uncertainties relevant to the problem the sliding friction coefficient of the friction pendulum (FP) isolators together with the seismic records characteristics. A six-degree-of-freedom model is defined to simulate the elastic response of the reinforced concrete pier, the infinitely rigid response of the deck supported by the seismic devices and the non-linear velocity-dependent behavior of the FPS bearings. The reinforced concrete abutment is modelled as a rigid support above which a FPS device is located. A set of natural records with different characteristics is properly selected and scaled to increasing intensity levels. The randomness on the friction coefficient is described by an appropriate probability density function to sample. For different system and isolator properties, fragility curves of both the reinforced concrete pier and FP devices supporting the deck are estimated. In accordance with the hazard curve of the design site, the seismic reliability curves are derived by means of the convolution integral.

SOMMARIO

La presente analisi valuta l'affidabilità sismica di ponti a più campate isolati considerando tra le principali incertezze aleatorie, il coefficiente di attrito degli isolatori attritvi a pendolo (FP) insieme alle caratteristiche delle registrazioni sismiche. E' stato definito un modello a sei gradi di libertà per simulare la risposta elastica della pila, la risposta infinitamente rigida dell'impalcato supportato dai dispositivi sismici e il comportamento non lineare dipendente dalla velocità degli isolatori FPS. La spalla è modellata come un supporto rigido sopra il quale si trova un dispositivo FPS. Un insieme di registrazioni accelerometriche naturali con diverse caratteristiche è stato selezionato. L'aleatorietà del coefficiente di attrito è descritta attraverso un'appropriata funzione di densità di probabilità da campionare. Per diverse proprietà del sistema strutturale, vengono stimate le curve di fragilità sia della pila che dei dispositivi FPS. In accordo con la curva di pericolosità del sito di progetto, le curve di affidabilità sismica vengono ricavate mediante l'integrale di convoluzione.

1 INTRODUCTION

The passive control techniques as seismic isolation has been widely used over the last decades for its capability of enhancing the protection of structure and infrastructure [1]. With particular reference to bridges, their safety assessment is one of the main topics for engineers and Authorities [2]. Seismic isolation allows to uncouple the deck from the horizontal components of the seismic motion reducing significantly the deck acceleration and forces transmitted to the pier with respect to non-isolated bridges [3]-[4]. In this framework, the seismic reliability-based design (SRBD) approach has been proposed in [5] to provide tools useful to design isolation devices considering the main uncertainties. The present analysis deals with the seismic reliability of isolated multi-span continuous deck bridges considering as the main aleatory uncertainties relevant to the problem the sliding friction coefficient of the friction pendulum (FP) isolators together with the seismic records characteristics. A six-degree-of-freedom model is defined to simulate the elastic response of the reinforced concrete pier, the infinitely rigid response of the deck supported by the seismic devices and the non-linear velocity-dependent behavior of the FPS bearings [6]. The reinforced concrete abutment is modelled as a rigid support above which a FPS device is located [8]. The FPS device behaviour has been modelled as suggested by [9]. Adopting the friction coefficient as the main random variable, it has been described by means of normal distribution adopting the Latin hypercube Sampling Method (LHS) [10] to perform sampling for probabilistic analysis. Furthermore, a set of 30 natural seismic records with different spectral characteristics has been used to take also into account the seismic motion uncertainty. The considered spectra are scaled to increasing intensity levels according with the seismic hazard of the reference site (i.e., L'Aquila (Italy)). Then, incremental dynamic analyses (IDAs) [11] have been performed to characterize the seismic demand and the capacity of the considered bridge. The estimates of the response statistics (i.e., peak deck displacement with respect to the pier and to the abutment and peak pier displacement with respect to the ground) have been adopted to assess the seismic fragility curves [12] of the isolation devices (and of the deck) and of the RC pier The mentioned above fragility curves have been useful to determine the seismic reliability of the bridge equipped with FPS in line to [13], assuming the hazard curves of the site and specific reference period.

2 MODELLING DYNAMIC RESPONSE OF BRIDGES ISOLATED WITH FPS

In line to [3],[8] , the structural behavior of an isolated three-span continuous deck bridge (Figure 1), is herein reproduced adopting the following modelling strategy: 5 dof relates to the lumped masses of the RC pier and 1 dof to the rigid RC deck mass. The reinforced concrete abutment is assumed rigid.

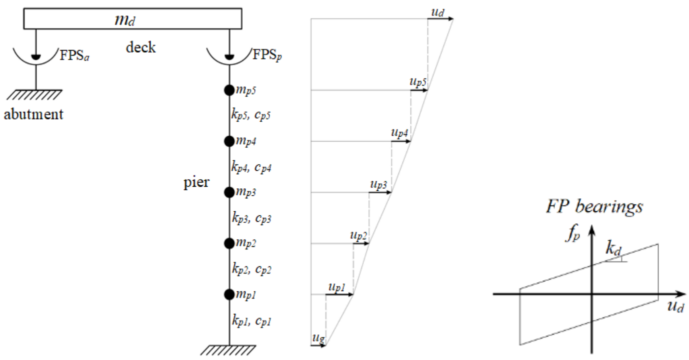

Fig. 1. 6 dof model of a bridge isolated by FPS devices and response of the FPS on the pier.

The equations of motion governing the response of a bridge equipped with FPS devices subjected to horizontal seismic input according to configuration of Figure 1 can be expressed as:

$$\begin{aligned}
&m_d\ddot{u}_d(t)+m_d\ddot{u}_{p5}(t)+m_d\ddot{u}_{p4}(t)+m_d\ddot{u}_{p3}(t)+m_d\ddot{u}_{p2}(t)+m_d\ddot{u}_{p1}(t)+c_d\dot{u}_d(t)+f_b(t)+f_a(t)=-m_d\ddot{u}_g(t)\\
&m_{p5}\ddot{u}_{p5}(t)+m_{p5}\ddot{u}_{p4}(t)+m_{p5}\ddot{u}_{p3}(t)+m_{p5}\ddot{u}_{p2}(t)+m_{p5}\ddot{u}_{p1}(t)-c_d\dot{u}_d(t)+c_{p5}\dot{u}_{p5}(t)+k_{p5}u_{p5}(t)-f_b(t)=-m_{p5}\ddot{u}_g(t)\\
&m_{p4}\ddot{u}_{p4}(t)+m_{p4}\ddot{u}_{p3}(t)+m_{p4}\ddot{u}_{p2}(t)+m_{p4}\ddot{u}_{p1}(t)-c_{p5}\dot{u}_{p5}(t)-k_{p5}u_{p5}(t)+c_{p4}\dot{u}_{p4}(t)+k_{p4}u_{p4}(t)=-m_{p4}\ddot{u}_g(t)\\
&m_{p3}\ddot{u}_{p3}(t)+m_{p3}\ddot{u}_{p2}(t)+m_{p3}\ddot{u}_{p1}(t)-c_{p4}\dot{u}_{p4}(t)-k_{p4}u_{p4}(t)+c_{p3}\dot{u}_{p3}(t)+k_{p3}u_{p3}(t)=-m_{p3}\ddot{u}_g(t)\\
&m_{p2}\ddot{u}_{p2}(t)+m_{p2}\ddot{u}_{p1}(t)-c_{p3}\dot{u}_{p3}(t)-k_{p3}u_{p3}(t)+c_{p2}\dot{u}_{p2}(t)+k_{p2}u_{p2}(t)=-m_{p2}\ddot{u}_g(t)\\
&m_{p1}\ddot{u}_{p1}(t)-c_{p2}\dot{u}_{p2}(t)-k_{p2}u_{p2}(t)+c_{p1}\dot{u}_{p1}(t)+k_{p1}u_{p1}(t)=-m_{p1}\ddot{u}_g(t)
\end{aligned} \tag{1}$$

where u_d denotes the horizontal displacement of the superstructure relative to the pier, $u_{p,i}$ is the displacement of the *i*-th (i:1-5) pier mass relative to the *i*-th-1 dof, m_d and m_{pi} respectively the mass of the deck and of the *i*-th (i:1-5) lumped mass of the pier, c_d is the viscous damping constant of the isolated deck, k_{pi} and c_{pi} , respectively, the pier stiffness and inherent viscous damping constant of the *i*-th (i:1-5) dof of the pier, *t* the time, the dot differentiation over time, $f_a(t)$ and $f_p(t)$ denote the reactions of the FP bearings on the abutment and on the pier, respectively, equal to:

$$f_a(t)=\frac{m_d g}{2R}\left(u_d(t)+\sum_{i=1}^{5}u_{pi}\right)+\frac{1}{2}\left(\mu_a\left(\dot{u}_d(t)+\sum_{i=1}^{5}\dot{u}_{pi}\right)\right)m_d g\left(sgn\left(\dot{u}_d(t)+\sum_{i=1}^{5}\dot{u}_{pi}\right)\right);$$
$$f_p(t)=\frac{m_d g u_d(t)}{2R}+\frac{\mu_p(\dot{u}_d)m_d g\, sgn(\dot{u}_d)}{2} \tag{2}$$

where g is the gravity acceleration, R is the radius of curvature of the both FPS devices assumed equal, $\mu(\dot{u}(t))$ the sliding friction coefficient of the isolator on the abutment (a) or of the isolator on the pier (p), which depends on the bearing slip velocity and $sgn(\cdot)$ denotes the sign function. The variation of $\mu(\dot{u}(t))$ can be reproduced according to the results of [14]-[15] as also performed by [6]. Dividing all the Eq. (1)-(2) by the deck mass m_d, dimensionless equations derive with the following dimensionless parameters: mass ratio of the i-th dof of the pier $\lambda_{pi}=m_{pi}/m_d$; damping ratio of the isolated deck and of the i-th lumped mass of the pier, respectively $\xi_d=\frac{c_d}{2m_d\omega_d}$, $\xi_{pi}=\frac{c_{pi}}{2m_{pi}\omega_{pi}}$; circular frequency of the isolated deck and of the *i*-th dof of the pier , respectively, $\omega_d=\frac{2\pi}{T_d}=\sqrt{\frac{g}{R}}$, $\omega_{pi}=\sqrt{\frac{k_{pi}}{m_{pi}}}$.

3 UNCERTAINTIES CONSIDERED FOR RELIABILITY ANALYSIS

In line with the performance-based earthquake engineering (PBEE) approach [16], this study separates the uncertainties related to the seismic input intensity from those related to the characteristics of the record. With this approach, the randomness in the seismic intensity can be described by a hazard curve and the ground motion randomness (related to fixed intensity level) can be represented by a set of ground motions with a different duration and frequency content, and by scaling these records to common intensity measure (IM) value. In particular, 30 natural ground motions, deriving from 19 different seismic events and considering the only horizontal components, have been selected according to [17] as reported by Table 1.

Table 1. Seismic records for reliability analysis.

	Year	Earthquake Name	Recording Station Name	V_{S30} [m/sec]	Source (Fault Type)	M [-]	R [km]	PGA-max [g]
1	1994	Northridge	Beverly Hills - Mulhol	356	Thrust	6.7	13.3	0.52
2	1994	Northridge	Canyon Country-WLC	309	Thrust	6.7	26.5	0.48
3	1994	Northridge	LA – Hollywood Stor	316	Thrust	6.7	22.9	0.36
4	1999	Duzce, Turkey	Bolu	326	Strike-slip	7.1	41.3	0.82
5	1999	Hector Mine	Hector	685	Strike-slip	7.1	26.5	0.34
6	1979	Imperial Valley	Delta	275	Strike-slip	6.5	33.7	0.35
7	1979	Imperial Valley	El Centro Array #11	196	Strike-slip	6.5	29.4	0.38
8	1995	Kobe, Japan	Nishi-Akashi	609	Strike-slip	6.9	8.7	0.51
9	1995	Kobe, Japan	Shin-Osaka	256	Strike-slip	6.9	46	0.24
10	1999	Kocaeli, Turkey	Duzce	276	Strike-slip	7.5	98.2	0.36
11	1999	Kocaeli, Turkey	Arcelik	523	Strike-slip	7.5	53.7	0.22
12	1992	Landers	Yermo Fire Station	354	Strike-slip	7.3	86	0.24
13	1992	Landers	Coolwater	271	Strike-slip	7.3	82.1	0.42
14	1989	Loma Prieta	Capitola	289	Strike-slip	6.9	9.8	0.53
15	1989	Loma Prieta	Gilroy Array #3	350	Strike-slip	6.9	31.4	0.56
16	1990	Manjil, Iran	Abbar	724	Strike-slip	7.4	40.4	0.51
17	1987	Superstition Hills	El Centro Imp. Co.	192	Strike-slip	6.5	35.8	0.36
18	1987	Superstition Hills	Poe Road (temp)	208	Strike-slip	6.5	11.2	0.45
19	1987	Superstition Hills	Westmorland Fire Stat.	194	Strike-slip	6.5	15.1	0.21
20	1992	Cape Mendocino	Rio Dell Overpass	312	Thrust	7.0	22.7	0.55
21	1999	Chi-Chi, Taiwan	CHY101	259	Thrust	7.6	32	0.44
22	1999	Chi-Chi, Taiwan	TCU045	705	Thrust	7.6	77.5	0.51
23	1971	San Fernando	LA - Hollywood Stor	316	Thrust	6.6	39.5	0.21
24	1976	Friuli, Italy	Tolmezzo	425	Thrust	6.5	20.2	0.35
25	1980	Irpinia	Bisaccia	496		6.9	21.3	0.94
26	1979	Montenegro	ST64	1083	Thrust	6.9	21.0	0.18
27	1997	Umbria Marche	ST238	n/a	Normal	6.0	21.5	0.19
28	2000	South Iceland	ST2487	n/a	Strike-slip	6.5	13	0.16
29	2000	South Iceland (a.s.)	ST2557	n/a	Strike-slip	6.5	15.0	0.13
30	2003	Bingol	ST539	806	Strike-slip	6.3	14.0	0.30
Source:	PEER, ITACA, ISESD-Internet Site for European Strong-Motion Data							

The spectral-displacement $S_D(T_d,\ \xi_d)$, in correspondence of the isolated period of the bridge $T_d=2\pi/\omega_d$ and for the damping ratio ξ_d, has been adopted as *IM*. According to [17], ξ_d can be set reasonably equal to zero. Then, the value $S_D(T_d)$ *is* assumed ranging from 0.10m to 0.45m to perform the IDAs. A further random variable have been included in probabilistic analysis: the friction coefficient of sliding surface. In this study, a truncated normal PDF, ranging from 0.5% to 5.5% with an average value equal to 3%, has been used to model the sliding friction coefficient at large velocity f_{max} as a random variable, as also discussed in [5]. According to the model of [15], the values of the friction coefficient at the low velocities, f_{min} have been considered dependent random variable and set equal to $f_{max}/3$. The LHS method [17] has been used to generate the input data samples of the structural models, by sampling 15 values from the PDF. The investigation is developed through an extensive parametric analysis involving different types of isolated bridges. Specifically, the deterministic parameters ξ_d and $\xi_{pi}= \xi_p$ are assumed respectively equal to 0% and 5%;

the isolated superstructure period T_d varies in the range between 1s, 2s, 3s and 4s; the RC pier period T_p in the range between 0.05s, 0.01s, 0.15s and 0.2s and is related to the five modes of the dof used for the pier; λ that represents the overall mass ratio related to the sum of the i-th mass ratios (assumed equal), varies in the range between 0.1, 0.15 and 0.2. The range of variation of the parameters is adopted in line to [18]. With this approach, combining the selected deterministic parameters, 720 different types of isolated bridges are defined.

4 RESULTS FROM THE INCREMENTAL DYNAMIC ANALYSES

Concerning each one of the 720 combinations between the deterministic parameters assumed in the parametric study, the differential equations of motion (Eq (1)) have been solved for the 30 seismic records (Table 1), scaled to the increasing values of $S_D(T_d)$. For each deterministic bridge configuration, a total number of 450 simulations has been carried out by pairing each one of the sampled 15 values of the friction coefficient to each one of the 30 scaled seismic records referred to the specific IM. The IDAs have been interpreted assuming the following engineering demand parameters (EDPs): the pier top response u_p with respect to the ground (determined as the sum of $u_{p,i}$ $i{:}1\text{-}5$), the deck response with respect, respectively, to the pier top u_d and to the abutment $u_{d,abut}$ (determined as the sum of u_d and u_p). For all the engineering demand parameters, the peak values are assessed and then a set of samples is obtained for each EDP at each value of the *IM*. The output set has been probabilistically treated by means of a lognormal distribution. The sample of data, consisting of structural responses, therefore represents the performance request (in terms of displacement) for both the deck and the pier, i.e., the seismic demand. The lognormal PDF can be estimated for each EDP by calculating the sample lognormal mean and the sample lognormal standard deviation, or dispersion, through the maximum likelihood technique [17]. Then, is possible to determine the values of the 50-th, 84-th, 16-th percentiles of each lognormal PDF [17].
The IDA results of the deck (i.e., of the seismic device located on the pier and on the abutment) as well as the IDA curves of the pier can be determined. In **Errore. L'origine riferimento non è stata trovata.** showing the EDPs versus the *IM* for one selected case.

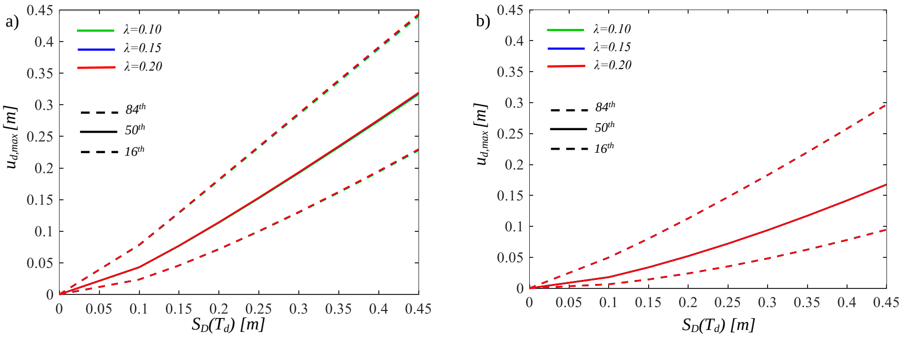

Fig. 2. IDA curves of the deck response with respect to the pier, for T_p=0.05s and T_d =1s (a), T_d =4s (b).

After the evaluation of the EDPs from non-linear IDAs it is possible to determine the probabilities P_f exceeding different limit states (*LSs*) conditional to each level of the *IM* assumed in the previous analyses. For this reason, the *LS* thresholds need to be defined as commented below.
Regarding the *LS* thresholds related to the isolation system, nine different values of the radius in plan of the single concave surface have been assumed ranging between 0.10-0.5m [19]. With reference to the performance of the RC pier, four discrete performance levels or *LSs* (*LS1*, *LS2*, *LS3*

and *LS4*), corresponding respectively to "fully operational", "operational", "life safety" and "collapse prevention", are provided by [20]. Within the displacement-based seismic design, the measurable structural response parameter, pier drift index (*PDI*), is adopted to define the specific *LS* threshold the reference life of a structural system. The PDI is defined as the ratio between the maximum to displacement of the pier and the height of the pier.

5 EVALUATION OF SEISMIC RELIABILITY

The mean annual rates exceeding the *LSs* can be derived performing convolution integral between the seismic fragility and seismic hazard curves [5]. In the present study, the site considered is L'Aquila (Italy) and the related seismic hazard curves are reported in Figure 3.

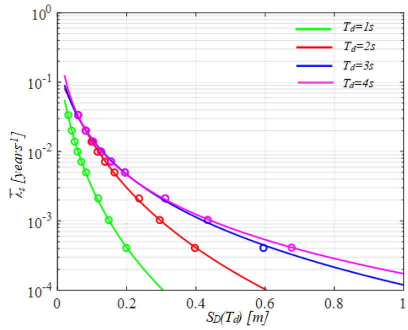

Fig. 3. Seismic hazard in terms of average annual rates of exceeding the IM $S_D(T_d)$. (L'Aquila)

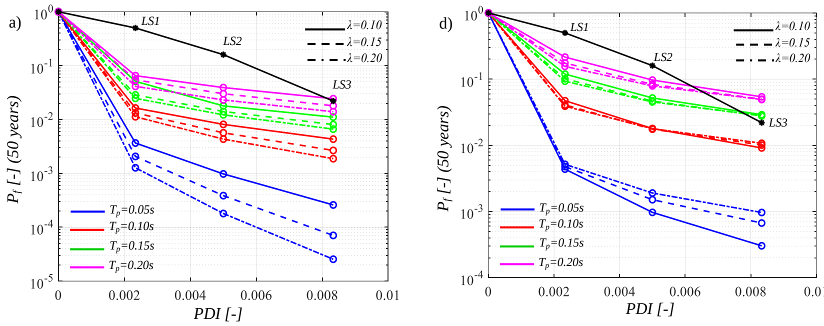

Fig. 4. Seismic reliability curves related to the pier for T_d =1s (a), T_d =4s (b).

The mean annual rate and related probability (concerning 50 years as reference life) of exceeding the corresponding *LS* for isolation devices and the pier, can be determined as follows:

$$\lambda_{LS} = \int P_f(IM) \cdot \left|\frac{d\bar{\lambda}_{IM}(IM)}{d(IM)}\right| d(IM);\; P_f(50years) = 1 - e^{\lambda_{LS}\cdot(50years)} \tag{3}$$

Figure 4 shows an example of the the seismic reliability curves for the RC pier.

6 CONCLUSIONS

This paper relates to the seismic reliability of multi-span continuous deck bridges isolated with single concave friction pendulum (FPS) devices. An extensive parametric study taking into account a wide range of isolator and bridge properties, such as the vibration period of the elastic RC pier, the isolating system period as well as the ratio between the pier and the deck mass (i.e., mass ratio) has been carried out. The seismic reliability assessment leads to the following results:

i. With reference to the pier, the fully operational and operational limit states are always fulfilled demonstrating the effectiveness of the seismic isolation technique.
ii. Regarding the isolation level and then, of the deck, the seismic reliability decreases with the increase of the curvature radius of the isolator due to the high seismic hazard of the site.

With reference to assessment/design of multi-span continuous deck bridges placed in an area with relevant seismic hazard, the proposed results are useful to derive recommendations on the preliminary design of the isolation system with reference to appropriate reliability levels during reference life.

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KEYWORDS

seismic isolation, bridge, friction pendulum isolators, performance-based engineering, seismic reliability.