Computers and Concrete, *Vol. 20, No. 4 (2017) 419-427*
DOI: https://doi.org/10.12989/cac.2017.20.4.419

Seismic evaluation of isolated skewed bridges using fragility function methodology

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(Received January 20, 2017, Revised May 10, 2017, Accepted May 13, 2017)

Abstract. A methodology, based on fragility functions, is proposed to evaluate the seismic performance of seismic isolated 45° skewed concrete bridge: 1) twelve types of seismic isolation devices are considered based on two different design parameters 2) fragility functions of a three-span bridge with and without seismic isolation devices are analytically evaluated based on 3D nonlinear incremental dynamic analyses which seismic input consists of 20 selected ground motions. The optimum combinations of isolation device design parameters are identified comparing, for different limit states, the performance of 1) the Seismic Isolated Bridges (SIB) and 2) Not Seismic Isolated Bridge (NSIB) designed according to the AASHTO standards.

Keywords: skewed bridge; seismic isolation device; fragility function methodology; incremental dynamic analysis (IDA); isolation device design parameters

1. Introduction

A variety of bridge damages under major earthquake indicates a high degree of their vulnerability and, as far as skewed bridges are concerned, San Fernando earthquake (1971) outlined that highway bridges are damage-prone: increasing the skew degree of bridge the behavior becomes more complex and the effects of coupling becomes more significant. Even if in the last few decades many researchers have studied the seismic responses of skewed highway bridges, the research findings have not been comprehensive enough to address global system response. The design codes and guidelines have improved significantly for dynamic and static analyses of straight highway bridges, but the lack of detailed procedure is still evident for the responses of skewed highway bridges. Seismic isolation based strategy, has been applied effectively to retrofit existing bridges, or design new ones, in seismically active zone: it is well known that thanks to the implementation of seismic isolation devices, a strong structural damage reduction can be achieved. Despite the many numerical, experimental and empirical studies (e.g., Dicleli and Buddaram (2006), Constantinou *et al.* 1992 Bessason and Haflidason 2004, Shinozuka *et al.* 2000, Lu *et al.* 2005) carried out for Seismic Isolated Bridges (SIB), in the case of skewed bridges, there is still a lack of research considering, however, the main common concepts that concern 1) the natural period and damping increasing 2) structural inelastic deformation reduction or prevention depending on the considered limit state 3) seismic isolators allocation that, differently from buildings where generally base isolation is adopted, is performed between the superstructure and substructure reducing the seismic effect on piers (see Fig. 1) and foundations as well as thermal effects.

Therefore, by making small changes in abutment systems and expansion joints, seismic isolation systems for bridge can be utilized with very little costs.

Recognizing the advantages of the seismic isolation strategy for bridges, the literature studies (Dicleli and Buddaram 2006, Shen *et al.* 2004) stressed on the need to properly optimize the mechanical properties of the adopted isolators and to check their effectiveness considering an appropriate set of ground motions. It is worth noticing that in Priestley *et al*. (1996) the authors state that the possibility of the wide set of the design parameters available make a simple design principle hard to be optimized.

In this framework, many studies have investigated the effectiveness and optimum design of isolation device components in bridges using deterministic methods (Diclel and Buddaram 2006, Park *et al*. 2002, Jangid 2005).

Since the effectiveness of the seismic isolation system largely depends on not deterministic variables such as the frequencies of both structures and ground motions, recent studies are based on probabilistic method which results are presented in terms of fragility curves obtained 1) empirically by statistical analyses of damaged bridges in the

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http://www.techno-press.org/?journal=cac&subpage=8
ISSN: 1598-8198 (Print), 1598-818X (Online)

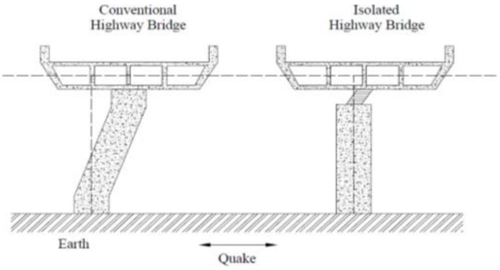


Fig. 1 Seismic isolated bridges (ISB): Scheme of the effects (Nielson 2005)

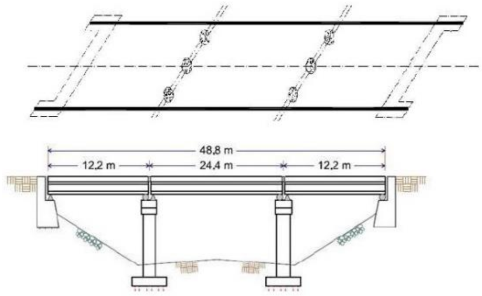
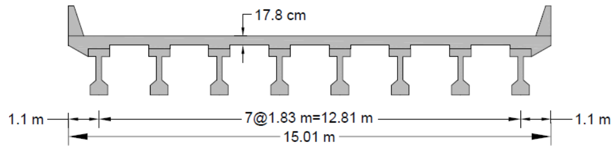
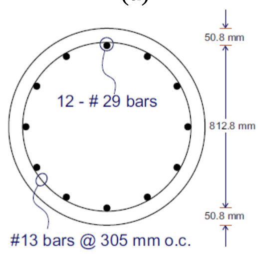


Fig. 2 Bridge plan view and elevation



(a)



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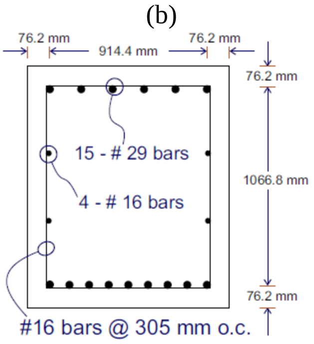

(c)

Fig. 3 The scheme of deck (a) Column section (b) and Cap beam section (c) (Nielson 2005)

past earthquakes, 2) expertly by expert's analysis and 3) analytically which complexity of the simulation depends on the adopted mechanical properties as well as the considered uncertainties.

Karim and Yamazaki (2001, 2007) developed a simple method to obtain fragility functions of the SIBs, determining the contribution of isolators on reducing damage probability of bridge columns, not considering the failure of isolation devices. Nielson and DesRoches (2007) demonstrated that bridges are a brittle systems with respect to its components individually and, for that reason, seismic evaluation of SIBs should be based on the system level fragility rather than the component fragility level. There are many other studies in the field of seismic isolation devices application for bridges and among them the following can be mentioned (Ghobarah and Ali 1988, Choi *et al*. 2004, Kunde and Jangid 2006, Padgett and DesRoches 2008, Zhang and Huo 2009, Alam *et al*. 2012, Shafieezadeh *et al*. 2012, Billah and Alam 2014, Cardone 2014). However, few studies are available for Siemic Isolated Skewed Bridges SISBand as far as the knowledge of the authors is concerned, Robson *et al*. (2001) and Kim *et al*. (2014) can be mentioned.

In this framework, the here presented study is focused on fragility function based methodology used to evaluate the seismic performance of SISBs and to examine the effect of different isolation device design parameters on the seismic performance of the bridge in order to select the optimal set: 1) a literature (Nielson 2005) is considered imposing a 45° skewness 2) proposing for the same bridge 12 seismic isolation configurations which design parameters are evaluated based on the simplified method proposed in (AASHTO 2010) 3) an opportune set 20 ground motions has been selected based on the FEMA P695 (2009) far field ground motion records and 4) having defined both limit states (namely slight, moderate and extensive) and damage index, the fragility curves of the considered 45° skewed bridge are evaluated 1) performing incremental non-linear dynamic analysis 2) combining for each considered limit states the 20 selected ground motions and the 13 bridge configuration that include the Not Seismic Isolated Skewed Bridge (NSISB included).

2. Bridge configuration and modeling

The skewed (45°) bridge typology, used in this study, is derived by modifying a non-skewed model developed by Nielson (2005),which characteristics are based on data obtained from a survey of numerous bridge plans: 1) it is a three spans (12.2/24.4/12.2 m) bridges (see Fig. 2) having a width of almost 15 meters 2) the deck (see Fig. 3(a) consists of eight AASHTO (2006) (type I,III) pre-stressed girders: the dimensions (height, upper-bottom width of flanges, thickness of web) of the I-section beam that concerns the end spans are 71.1/30.5-40.6/15.2 cm, while the dimensions of the I-section beam type III (concerning central span) are 114.3/40.6-55.9/11.4 cm 3) the 3+3 piers (Hwang *et al*. 2001), which height is 5.75 m, having the concrete (strength equal to 20.7 MPa) circular section reinforced with steel bars (yield strength equal to 414 MPa) as reported in Fig 3(b), are transversally connected through the cap beam reported in Fig 3(c). 4) the initial stiffness of the adopted elastomeric pads, for the NSIB are 3.4 kN/mm and 6.2 kN/mm respectively for the end and central spans while the stiffness for the adopted seismic isolators is

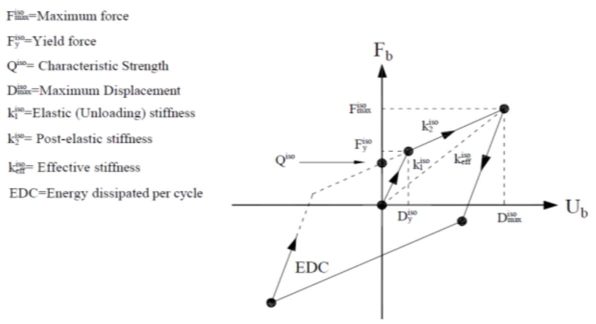

Fig. 4 The bilinear force-displacement relationship for the seismic isolation devices

reported in Table 1, as discussed in the following chapter 3.

Concerning the numerical model, elastic shell elements have been adopted for the deck and bilinear plastic model have been adopted for the plastic hinges of the bridge piers: 1) the plastic hinge length is obtained using recommendation given in (Paulay and Priestley 1992). 2) Mander *et al*. (1988) model is adopted for confined and unconfined concrete, whilethe elasto-plastic steel stress-strain model with no hardening is adopted for reinforcement.

The abutments are modelled using beam elements supported on springs. A rigid bar is used to connect the nodes between girders and bearings, bearings and cap beams, and cap beams and tops of the columns. The bridge has two different types of bearings: Fixed and expansion, which alternate along the length of the bridge and are noted as triangles and circles respectively in Fig. 2. The model has been implemented in the software SAP2000 (2012).

Concerning the NSIB, studied in, having a fundamental period (in longitudinal direction) of 0.48 sec, different directions of earthquake are considered and applied, concluding (Bayat *et al.* 2017) that the critical direction is the longitudinal one, so that this is the one selected, in this study, to evaluated the bridge fragility.

3. Characterization and modeling of seismic isolation devices

Seismic isolators may generally be classified in the following two categories: those that use elastomeric components (elastomeric-based) and those that use sliding components (sliding seismic isolation systems). These two categories include an energy dissipation mechanism that can be increased combining them with hysteretic device obtaining: For example, the Lead Rubber Bearings (LRB) the typical bilinear force-displacement relationship for the most seismic isolators that includes hysteretic dissipation is shown on Fig. 4.

In this paper, a general case of seismic isolation devices is analyzed which may include both mentioned categories, considering that the number of seismic isolation devices will be depend to the mechanical properties and dimensions of the adopted isolation device type.

The simplified analysis method, defined in (AASHTO 2010), has been adopted for the design, idealizing the

Table 1 The characteristic seismic isolation design parameters for one-span devices set

N	10	10	10	20	20	20	30	30	30	50	50	50
T_{eff} (s)	2	2.5	3	2	2.5	3	2	2.5	3	2	2.5	3
k_1^{iso}(kN/mm)	14.88	9.60	6.56	29.92	19.20	13.28	45.12	28.80	20.00	75.20	48.16	33.44
k_2^{iso}(kN/mm)	1.49	0.96	0.66	1.50	0.96	0.66	1.50	0.96	0.67	1.50	0.96	0.67
Q^{iso} (*kN*)	73.44	46.88	32.64	73.76	47.20	32.80	74.08	47.52	32.96	75.68	48.32	33.60

isolated bridge span as a single degree of freedom system in which the 1) effective mass is equal to the mass of the deck, 2) stiffness and damping are a combination of stiffness and damping of the isolation system and substructure. The typical adopted bilinear force-displacement relationship is reported in Fig. 4.

The considered seismic isolation system consists of seismic isolation devices positioned under the girders between the superstructure and substructure: 12 combination of seismic isolation device shave been considered combining two parameters: 1) the ratio N between the elastic stiffness k_1^{iso} and post-elastic stiffness $k_2^{iso}$and 2) the effective period of isolation system T_{eff}. The effective damping has been fixed to 0.15.

The values of N parameter has been assumed equal to 10, 20, 30, and 50 (Zhang and Huo 2009) and T_{eff} value has been assumed equal to 2,2.5 and 3s. Based on these two selected design parameters, other required modeling and design parameters are determined using equations defined in AASHTO (2010) (Table 1); they are: the elastic and post-elastic stiffness of the isolator ($k_1^{iso}$and k_2^{iso}) and characteristic strength of the isolator (force in isolator at zero displacement) Q^{iso}.

4. Fragility function methodology

Fragility functions, in this context, are defined as the conditional probability of exceedance of particular damage or limit state for a given ground motion intensity measure. They can be written in mathematical form as follows

$$P[DM \geq dm^{DS}/IM] = \phi\left(\frac{\ln(IM) - \mu}{\sigma}\right) \tag{1}$$

where 1) DM is a demand measure such as an Engineering Demand Parameter (EDP) that is generally assumed to be either the ductility or the drift 2) dm^{DS} is a limit value for DM when a particular damage state DS is considered 3) ϕ is the standard normal cumulative distribution function, 4) μ andσ are respectively, based on the log-normal distribution, the median and the standard deviation values of the intensity measure at which bridge reaches the threshold dm^{DS}.

They can be classified in: 1) empirical if obtained by statistical analyses of damaged bridges in the occurred earthquakes, 2) expert if based partially or completely on expert's opinion, 3) analytical if derived using numerical models to simulate the behaviour of systems.

Due to the deficiency of observed damage data from the past earthquake and subjectivity in expert's opinion, the

Table 2 *DIs* and corresponding *DSs* for concrete columns and bearings

Component/Damage	Slight (DS=1)	Moderate (DS=2)	Extensive (DS=3)	Collapse (DS=4)
Column	Cracking and spalling	Moderate cracking and spalling	Degradation w/o collapse	Failure leading to collapse
	$\mu>1$	$\mu>2$	$\mu>4$	$\mu>7$
	$\theta>0.007$	$\theta>0.015$	$\theta>0.025$	$\theta>0.050$
Bearing	$\gamma>100\%$	$\gamma>150\%$	$\gamma>200\%$	$\gamma>250\%$

only way to obtain vulnerability of bridges is by using analytical methods, here adopted considering PGA as intensity measure that according Padgett *et al*. (2008) is the optimum choice based on efficiency, practicality, sufficiency and hazard computability.

For each level of the considered intensity measure (IM = PGA), considering the all set of selected ground motions 1) the DMs are obtained by conducting nonlinear time-history, 2) the probability of exceedance of a certain damage state (defined by dm^{DS}) is calculated, 3) the dotted fragility function (the pairs of intensity measure IM and corresponding probability of exceedance) is derived and then 4) fitted with a log-normal cumulative distribution function.

4.1 Damage index and limit states

Realistic and comprehensive limit and damage states determination is one of the important steps in the process of obtaining the fragility functions because of their direct influence on derived fragility functions (Erberik and Elnashai 2004). The damage states are represented by discrete points on a continuous damage scale: they correspond to given values of Damage Index (DI) that measures Engineering Demand Parameters (EDPs) such as the attained section ductility (μ_k) or drift ratio (θ) of the columns which are the most critical components of conventional highway bridges with either continuous decks or simply supported spans. For a given *EDP* the damage should be measured based on prescriptive and descriptive indicators characterized by a given threshold for each damage states. The four damage states (slight, moderate, extensive and collapse) defined by HAZUS (2003) have been considered as reported in Table 2 where:

1) concerning the columns two different damage index (*DI*), section ductility (μ_k) and drift ratio (θ), have been reported considered the limitations respectively reported in Choi et al. (2004) and Yi *et al*. (2007). In this study, column drift ratio (θ) is adopted evaluating it considering the combination of transverse and longitudinal displacements;

2) concerning the seismic isolators the shear strain has been assumed as DI since it can better characterize the bearing behavior, depending on the rubber damping and bearing size (Zhang and Huo 2009): experimental studies showed that material behavior of the rubber bearings remains almost linear up to a shear strain (γ) of 100% and that failure strain can reach the value of 400% (Naeim and Kelly 1999), even if complete damage is considered when the shear stain exceeds 250%.

Table 3 Characteristics of the selected ground motion records (FEMA P695 2009)

Earthquake						Recording station	
ID No	M	PGA (g)	PGV (cm/s)	Year	Name	Name	owner
1	7.0	0.55	44	1992	Cape Mendocino	Rio Dell Overpass	USGS
2	7.6	0.44	115	1999	Chi-Chi, Taiwan	CHY101	CWB
3	7.1	0.82	62	1999	Duzce,Turkey	Bolu	ERD
4	6.5	0.35	31	1976	Friuli, Italy	Tolmezzo	------------
5	7.1	0.34	42	1999	Hector Mine	Hector	SCSN
6	6.5	0.35	33	1979	Imperial Valley	Delt	UNAMUCSD
7	6.5	0.38	42	1979	Imperial Valley	El Centro Array#1	USGS
8	6.9	0.51	37	1995	Kobe, Japan	Nishi-Akashi	CUE
9	6.9	0.24	38	1995	Kobe,Japan	Shin-Osaka	CUE
10	7.5	0.22	40	1999	Kokaeli,Turkey	Duzce	ERD
11	7.3	0.24	52	1992	Landers	Yemo Fire Station	CDMG
12	7.3	0.42	42	1992	Landers	Coolwater	SCE
13	6.9	0.53	35	1989	Loma Prieta	Capitola	CDMG
14	6.9	0.56	45	1989	Loma Prieta	GiloryArrey#3	CDMG
15	7.4	0.51	54	1990	Manjil	Abbar	BHRC
16	6.7	0.52	63	1994	Northridge	Beverly Hills-Mulhol	USC
17	6.7	0.48	45	1994	Northridge	Canyon Country	USC
18	6.6	0.21	19	1971	San Fernando	LA-Hollywood Stor	CDMG
19	6.5	0.45	46	1987	Superstition Hills	El Centro Imp.Co	CDMG
20	6.5	0.52	36	1987	Superstition Hills	Poe Road (temp)	USGS

The damage state for a SIB under earthquake has to be described by the combination of damage state of both components, pier and bearing, so that it is assumed that the damage state of the system is dictated by the largest damage state at component level

$$DS_{system} = max\left(DS_{pier}, DS_{bearing}\right) \tag{2}$$

4.2 Seismic ground motion records selection

Developing probabilistic models, based on nonlinear dynamic analyses, requires an appropriate selection of ground motion records. As general rules mentioned in FEMA P695 (2009), the ground motion bin should be unbiased to any site-specific seismological characteristic of a probable future earthquake event to keep its generality and versatility. Each selected ground motion record should be structural type independent so that it is also applicable to a variety of structures located at different sites. Furthermore, the number of records should be enough to cover the earthquake variability in a justified way. According to the mentioned objectives, a set of far-field ground motions (Table 3) is selected, based on FEMA P695 (2009) far field ground motion records. The twenty ground motion records are selected considering that previous studies have shown that 10 to 20 records are usually enough to provide sufficient accuracy in the estimation of seismic demands (Shome and Cornell 1999, Bayat *et al.* 2015a,b,c, 2017).

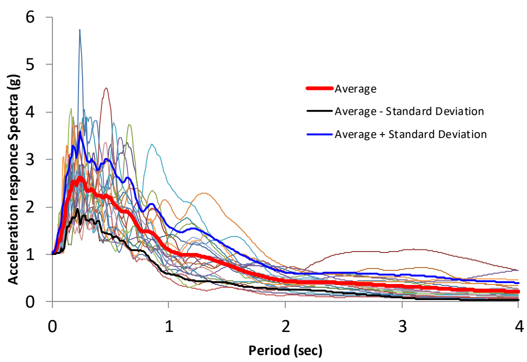

Fig. 5 Response spectra of the selected ground motions:1) mean values and 2) mean values +/- standard deviation

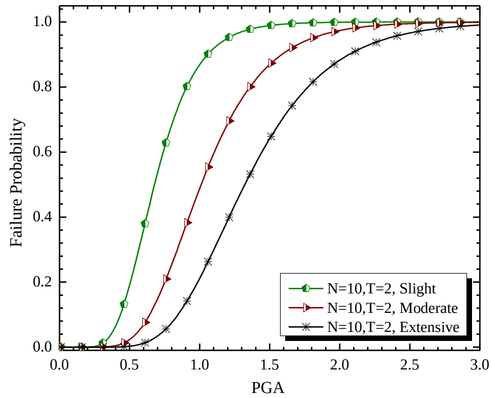

Fig. 6 Fragility curves for the SISB (N=10,T=2)

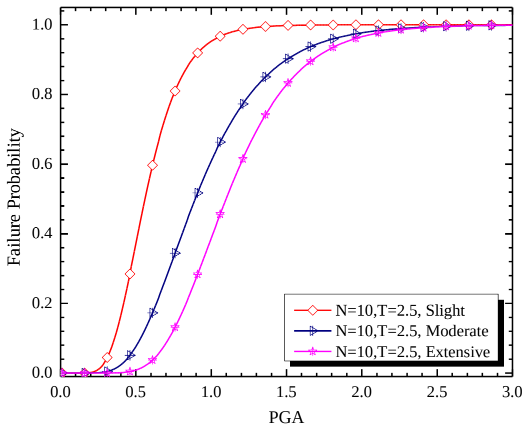

Fig. 7 Fragility curves for the SISB (N=10,T=2.5)

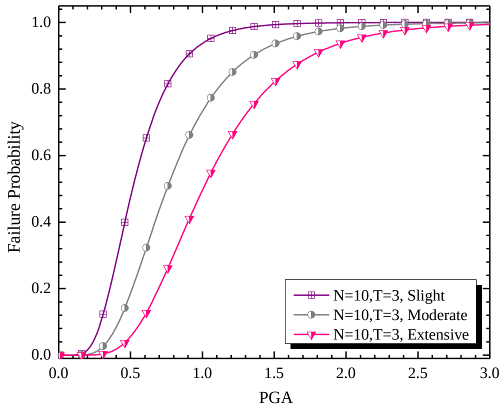

Fig. 8 Fragility curves for the SISB (N=10,T=3)

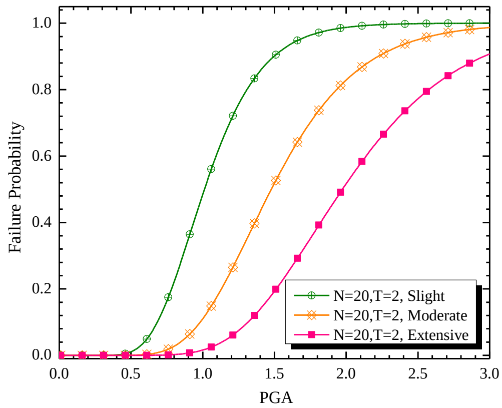

Fig. 9 Fragility curves for the SISB (N=20,T=2)

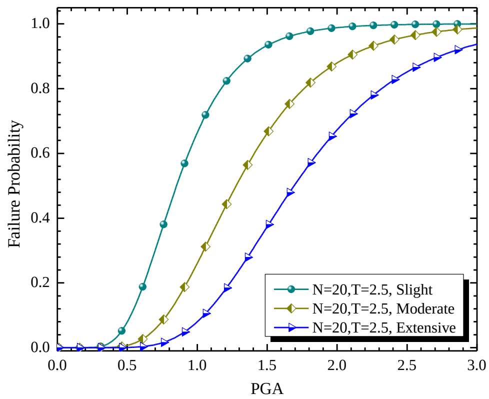

Fig. 10 Fragility curves for the SISB (N=20,T=2.5)

The selected FEMAP695(2009) ground motions records satisfy the following criteria:

(a) Magnitude > 6.5; (b) Distance from source to site > 10 km (average of Joyner-Boore and Campbell distances) (Joyner and Boore 1993); (c) Peak Ground Acceleration (PGA) > 0.2 g and Peak Ground Velocity (PGV) > 15 cm/sec; (d) Soil shear wave velocity, in the upper 30 m, greater than 180 m/s; (e) Lowest useable frequency < 0.25 Hz, to ensure that the low frequency content was not removed by the ground motion filtering process; (f) Strike-slip and thrust faults (consistent with California); (g) No consideration of spectral shape; (h) No consideration of station housing, but PEER-NGA records were selected to be "free-field".

In the Fig. 5 response acceleration spectra of the selected ground motions are reported, together with 1) the mean value and 2) mean value +/- standard deviation.

5. Analysis results and discussions

The methodology described in the previous section is used to obtain the fragility curves of the considered non-isolated (NSISB) and isolated skewed bridges (SISB).

The selected 20 ground motion records are applied to the bridges in longitudinal direction, that resulted to be the critical one for skewed (45°) not isolated bridge (Bayat *et al.* 2017). A complete full incremental dynamic analysis has

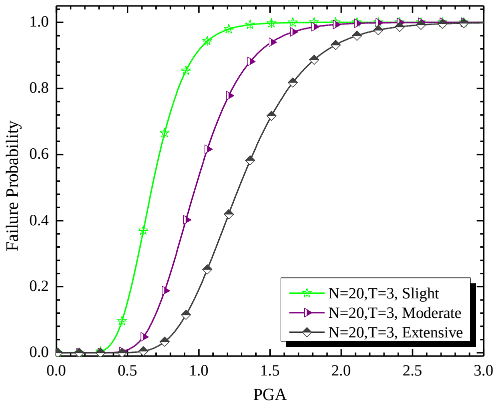

Fig. 11 Fragility curves for the SISB (N=20,T=3)

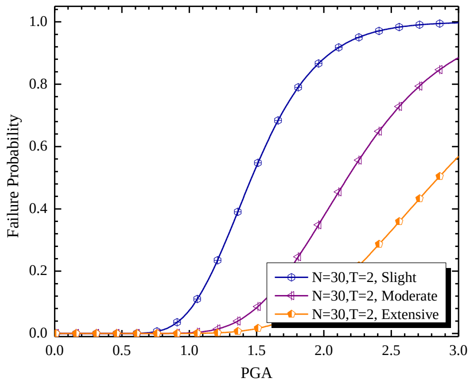

Fig. 12 Fragility curves for the SISB (N=30,T=2)

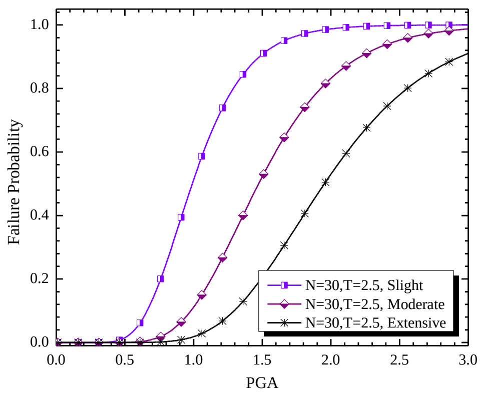

Fig. 13 Fragility curve for the SISB (N=30,T=2.5)

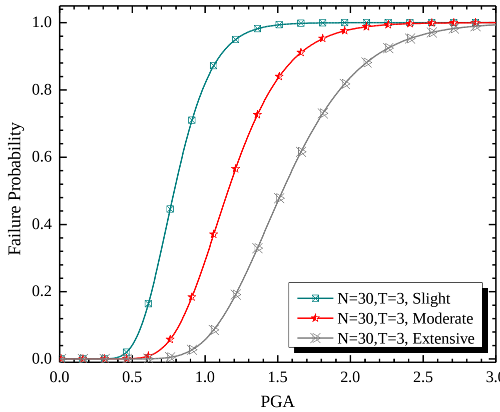

Fig. 14 Fragility curves for the SISB (N=30,T=3)

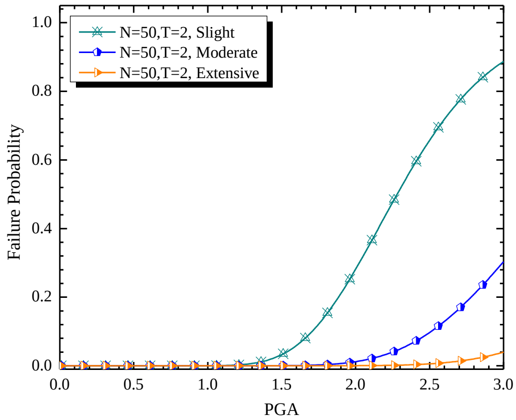

Fig. 15 Fragility curve for the SISB (N=50,T=2)

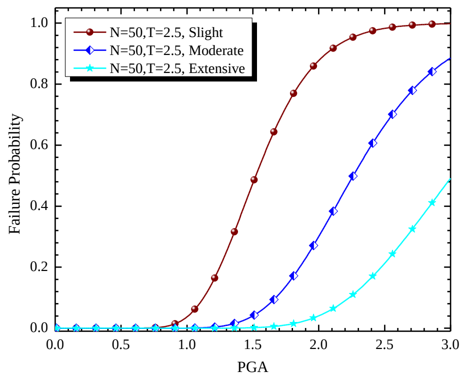

Fig. 16 Fragility curves for the SISB (N=50,T=2.5)

been done 1) scaling each ground motions with increments of 0.1 g, up to the maximum considered PGA which value is 3.0 g 2) stopping the scaling if the extensive limit state is reached.

The slight, moderate and extensive damage levels have been considered and for each of them the threshold values reported in Table 2, have been adopted for both column sand seismic isolators.

The derived fragility curves for the 12 considered types of the SISB(12 different combination of the ratio N and the effective period of isolation system T_{eff}) are shown in Figs. 6 to 17.

In order to define a criteria to select the optimal set of the isolators parameter, the all set of fragility curves have been collected as reported in Figs. 18, 19, 20 where the curves for the slight, moderate and extensive limit states have been respectively reported: it can be observed that the fragility of the NSISB can be assumed as a threshold between two set of performance, respectively less performing (if lies on the left) or more performing (if lies on the right) than the NSISB.

It is worth noticing that 1) some fragility curves intersect the curve of the NSISB, that means they are less performing of it , for low value of the PGA 2) generally if a typology is more performing for a PGA corresponding to a low level of exceedance probability (eg 15%), it will be more performing for greater PGAs too 3) for each limit state the best solution can be selected either considering the Most Performing Selection (MPS) or, if the MPS implies a

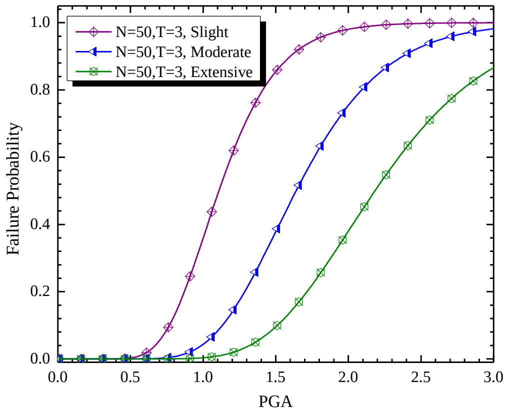

Fig. 17 Fragility curves for the SISB (N=50,T=3)

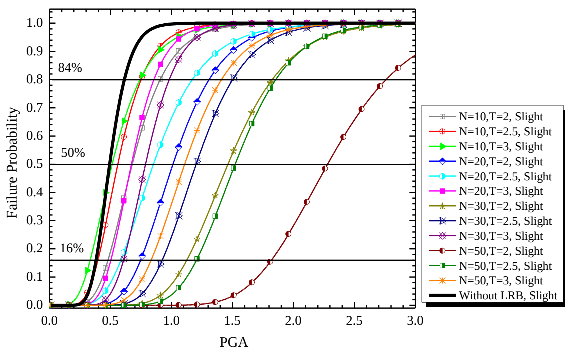

Fig. 18 Comparsion of fragility curves for the SISB for slight damage state

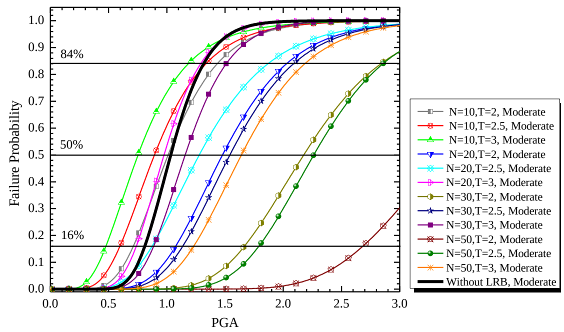

Fig. 19 Comparsion of fragility curves for the SISB for moderate damage state

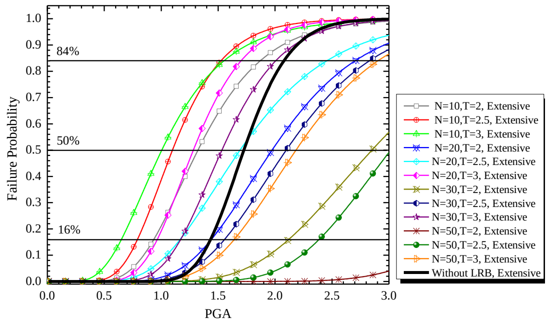

Fig. 20 Comparsion of fragility curves for the SISB for extensive damage state

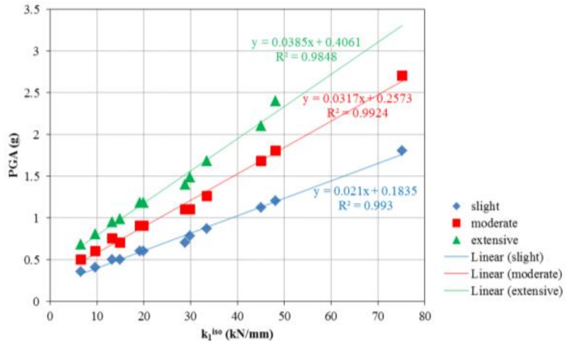

Fig. 21(a) PGA values of 16% failure probability for the considered values of k_1^{iso}

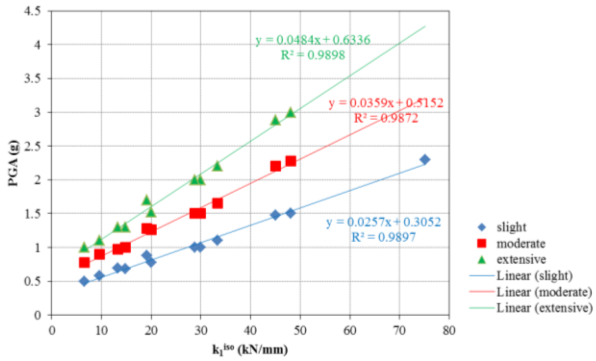

Fig. 21(b) PGA values of 50% failure probability for the considered values of k_1^{iso}

cost increasing, the selection can be done selecting the typology having a curve closer to the curve of the NSIB, but always on the right of it: this procedure can be identified as Most Performing and Economic Selection (MPES).

If the MPES is adopted, it can be depicted that the typology with a N value of 30 and a period of 3 sec are valid for slight and moderate limit state but not valid for the extensive limit state for which N = 50 and T = 3.0 sec is the optimal solution. It is worth noticing that passing from the slight to extensive limit state the number of more performing typologies decrease, so that a criteria to be adopted is to select one or more typologies more performing, than the NSIB, for all the considered limit states.

As far as the here presented results are considered, only three typologies satisfy this criteria and all of them are characterized by a N value equal to 50 and a period of 2.0,2.5 and 3 sec so that the optimal solution is 1) N=50 T = 2 sec concerning the MPS 2) N = 50 T = 3 sec concerning MPES.

Further on, it has to be pointed out that for each of the considered DSs there is a strong correlation between the bridge performance and the elastic stiffness (k_1^{iso}) of the seismic isolators; this can be deduced from Fig. 21(a), (b) where, for different values of the considered k_1^{iso}, the PGAs have been reported for two different levels of failure probabilities that are 16% (Fig. 21(a)) and 50% (Fig. 21(b)). On the other hand, the correlation with N and Q^{iso} is not so evident as can be deduced through Figs. 22(a), (b) where the points reported in Fig. 21(b) have been differently marked depending on the values of N (Fig. 22(a)) and Q^{iso} (Fig. 22(b)): points with the same values of k_1^{iso}can have the same PGA values even if they have different values of N and Q^{iso}.

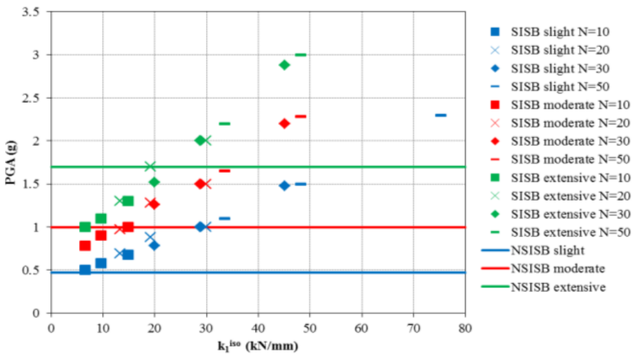

Fig. 22(a) PGA values of 50% failure probability for the considered values ofk_1^{iso}: marker differentiation for different values of the parameter N

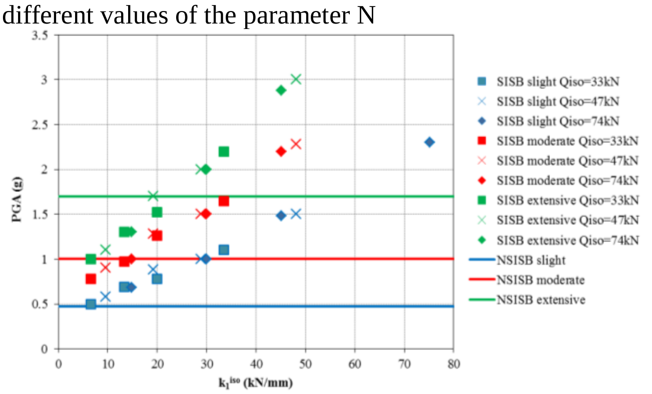

Fig. 22(b) PGA values of 50% failure probability for the considered values of k_1^{iso} marker differentiation for different values of the Q^{iso} parameter

Preliminary conclusions, based on the previous outlined results, are that in the case here considered a strategy to be adopted is, as it could have been expected, 1) to reduce the Q^{iso} far enough to avoid damage in the columns 2) define the initial stiffness big enough to have acceptable shear strain values for all the considered limit state.

6. Conclusions

In this paper, fragility function methodology is used to investigate the effect of different isolation device design parameters on the seismic performance of the skewed bridge in order to obtain the optimal design parameters. Twelve different types of seismic isolation devices are defined based on two isolation device design parameters: the ratio of the elastic stiffness to the post-elastic stiffness and the effective period of isolation system. The fragility curves for three-span concrete girder 45° skewed bridge with and without seismic isolation devices are analytically evaluated performing non-linear time history analyses for 20 selected ground motions. In order to identify the optimal isolation device design parameters, the fragility curves of isolated skewed bridges (SISB) and non-isolated skewed bridge (NSISB) have been compared: the significant differences in the fragility curves for each of the considered isolation device parameters combination is noticed, indicating that initial stiffness, k_1^{iso}, of the isolation device has a significant influence on the damage probability of the considered isolated bridges. The obtained results show that seismic isolation generally reduces, but not always, the bridge damage probability and confirm the effectiveness of the application of seismic isolation to the skewed bridges.

Further on two criteria have been identified for the selection of the optimal solution: they are based on the concept of Most Performing Selection (MPS) and Most Performing and Economic Selection. For both criteria is assumed that the selection of the optimal solution has to satisfy all the considered limit states and it has to be always more performing than the solution without seismic isolators.

As far as the here presented results are considered, only two typologies satisfy this criteria and all of them are characterized by a N value equal to 50 and a period of 2.0, 2.5 and 3 sec so the optimal solutions are 1) N=50 T = 2 sec concerning the MPS 2) N = 50 T = 3 sec concerning MPES.

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