Structural Engineering and Mechanics, *Vol. 55, No. 6 (2015) 1177-1202*
DOI: http://dx.doi.org/10.12989/sem.2015.55.6.1177

A novel proficient and sufficient intensity measure for probabilistic analysis of skewed highway bridges

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(Received January 28, 2015, Revised September 3, 2015, Accepted September 7, 2015)

Abstract. In this paper, a new intensity measure of earthquakes for probabilistic seismic analysis is presented for skewed highway bridges. Three different cases of skewed bridges with different skew angles (0^o, 30^o and 45^o) are considered. Well-known intensity measures (e.g., PGA, S_a) are evaluated and critically discussed based on sensitivity analysis: efficiency, practically, proficiency and sufficiency of intensity measures are considered in detail. The analyses demonstrated that the intensity measures have to take into account structural acceleration on a wide range of periods so that a new seismic intensity measure is proposed showing that it has less dispersion compared to others. Since the proposed intensity represents the average value of the S_a (between a lower and upper structural period) it has been called Averaged Spectral Acceleration (ASA). Based on performed incremental dynamic analysis (IDA), the seismic analytical fragility curves of typical skewed highway bridges have been evaluated for different states of damage controlling the low dispersion of the ASA index as well as its proficiency and sufficiency.

Keywords: skewed highway bridges; analytical fragility curve; spectral intensity measures; incremental dynamic analysis (IDA)

1. Introduction

Due to limitations caused by curved roads, skewed bridges frequently allow for the use of various geometrical bridge deck shapes. Despite the recognized utility of this bridge typology_ as discussed in the scientific and technical literature, their performance_ have not been explored in detail_. Therefore the importance of the few studies devoted to understanding their behavior are significant (Meng and Lui 2000, Wakefield *et al.* 2000, Meng and Lui 2002, Meng *et al.* 2004, Dimitrakopoulos 2010, Lou and Zerva 2005, Maleki 2005, Kaviani *et al.* 2012, Alam *et al.* 2012, Sullivan and Nielson 2010, Abdel-Mohti and Pekcan 2013, Hassel and Bennett 2012, Maleki and Bisadi 2006, Kavianijopari 2013). Many of the studies concern modeling or experimental issues and only a few (Kaviani *et al.* 2012) indirectly concerns the issue of the intensity measure: usually the authors, having selected an opportune intensity use it to the fragility curve evaluation without entering in the details of their proficiencies and efficiencies.

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http://www.techno-press.org/?journal=sem&subpage=8 ISSN: 1225-4568 (Print), 1598-6217 (Online)

Recognizing that the fragility curves represents an adequate tool for sensitivity analyses, they have been adopted in these studies to represent the results of the analyses that supported the proposal of a new index named ASA (Average Spectral Acceleration) that takes into account structural acceleration on a wide range of periods: it represents the average value of the S_a (between a lower and upper structural period). So that based on performed incremental dynamic analysis (IDA), the seismic analytical fragility curves of typical skewed highway bridges have been evaluated for different damage states controlling the low dispersion of the ASA index as well as its proficiency and sufficiency.

Given these premises, after a preliminary discussion of studied skewed bridge_ problems and probabilistic adopted frameworks, the proposed ASA index is introduced and discussed. Other chapters contain detail bridge modeling and analyze bridge descriptions together with their Limit State Definition and record selections.

2. Over review of skewed bridges studies

Mang and Luie (2000) used three finite element models to analyze the skewed bridge of Foothill under the effect of 1971 San Fernando earthquake: response spectrum method has been adopted for the seismic analyses. Three different bridge models were derived: elastic deck, rigid deck and stick models. Eigenvalue analysis was performed to determine the natural periods and mode shapes using these three models. The assumptions of this study were similar to Wakefield *et al.* (1991) with the exception of the abutment rotation (along longitudinal axis) disregarded in (Mang and Luie 2000) and included in (Wakefield *et al.* 2000). The results of eigenvalue analyses of three models showed that, fundamental modes are dominated by in-plane rotation due to the skewness, as well as irregular stiffness distribution.

Mang and Luie (2002) proposed a refined stick model for dynamic analysis of skewed bridges. The model utilized a dual-beam stick representation of the bridge deck. The model was presented for preliminary dynamic analysis of skewed highway bridges.

Mang *et al.* (2004) performed a experimental study on a skewed bridge model. Static and dynamic parameters (vibration frequencies, mode shapes, damping) of the model were determined. The experimental model was used to verify the validity of 3 dimension finite element models and stick models, and results showed a good correlation between mentioned models and the experimental model.

The influence of deck abutment pounding on short skewed bridges, was investigated by Dimitrakopoulos (2011) who proposed a novel rigid body approach to study the effect of pounding: bi-directional frictional multi-contact phenomena have been included.

Maleki (2005) considered the seismic behavior of skewed and non-skewed bridges. He considered the effect of gaps for the bearing pad retainers by including the stiffness of end-diaphragms and elastomeric bearings in the modeling.

Kaviyanchi (2012) studied the probabilities of skewed bridge responses. Three types of bridges were studied and fragility curves were evaluated. The engineering parameters of the study are: deck rotation, columns drift and Peak Ground Velocity (PGV) assumed as IM. Forty recorded earthquakes were used for the nonlinear dynamic analysis and collapse fragility curves were evaluated. As mentioned in their study an appropriate choice of intensity index is required to work within the framework of probabilistic seismic response of skewed bridges.

Shahria Alam *et al.* (2012) performed analytical studies on fragility curves of a three span bridge with and without seismic isolators at the base of the continuous deck: 1) incremental dynamic analysis were performed using records with PGA between 0.4g and 1.07g 2) traditional as well as innovative Shape-Memory Allow seats (SMA) were analyzed 3) fragility curves were evaluated for the bridge as well as isolators and columns, showing the better performance of the system with SMA and that when seismic isolation system is considered the damages are localized at the deck joints being the column less damaged if compared with the column of the non-isolated bridges.

Probabilistic seismic analysis has been developed recently and it is well documented in the literatures (Sullivan and Nielson 2010, Abdel-Mohti and Pekcan 2013, Hassel and Bennett 2012, Maleki and Bisadi 2006, Kavianijopari 2013); in the following chapter the used approaches will be described in the framework of a full probabilistic procedure.

3. Over review of probabilistic frame work

3.1 Probabilistic Seismic Demand Analysis (PSDA)

An earthquake is a probabilistic event with high impact and high uncertainty (Wen 2001). By virtue of the random nature of earthquakes and the extensive uncertainty in the prediction of seismic performance of structures, estimated future behaviour of a structure in a probabilistic framework is recommended (Tothong and Luco 2007). Thus, the goals of Performance Earthquake Engineering (PREE) are: 1) to define a structural response parameter (θ) such as the drift ratio and a threshold (x) for it 2) to evaluate the mean annual rate $\lambda_\theta(x)$ of the occurrences characterized by a level of (θ) greater than x 3) to calculate, based on $\lambda_\theta(x)$, the probability to overcome the threshold (x) in a year (Deierlein *et al.*,2003; Cornell and Krawinkler ,2000).

Therefore, the $\lambda_\theta(x)$ based strategy can be adopted to evaluate the seismic performance of a structure. Direct method of calculating $\lambda_\theta(x)$ is to record the structural response in a long time interval obtaining the mean annual rate of exceedance of structure, so that we have (Cornell 2005)

$$\lambda_\theta(x) = \lambda P(\theta > x \,|event) = \lambda \; G_\theta \tag{1}$$

where 1) λ= mean rate of a probable specific earthquake (“M” magnitude) with Poisson periodic distribution (average rate of probable specific earthquake) 2) P=$G_\theta(x)$=conditional probability function of occurrence x in a specific event 3) θ=structural response 4) x=a given level of response.

Since the structure is generally affected by a set of earthquakes with different sources and consequently different characteristics, Eq. (1) can be written as reported in Eq. (2) where 1) M_i and R_i are magnitude and epicenter distance of each considered earthquake 2) $G_{\theta|M_i,R_i}$ is the probability that θ will cross-over a given value of x

$$\lambda_\theta(x) = \sum_i G_{\theta|M_i,R_i}\left(x\,|M_i, R_i\right)\lambda\left(M_i, R_i\right) \tag{2}$$

Based on the previous equation the response rate is the results of different scenarios $\{M, R\}$. A useful tool to assess the probability of a certain limit state or conditional probability is fragility function that can be empirical or analytical, being the analytical fragility function more used in recent years because of their flexibility and their accuracy; on the other hand the empirical fragility functions have some limitations due to the insufficiency of recorded damage data and the subjectivity in defining damage states.

Two are the adopted approaches to obtain analytical fragility functions: the ‘scaling’ (like ‘stripe’) and ‘cloud’ approach. Both methods are based on nonlinear (time history, pushover) and linear analyzes (time history, elastic spectral). In cloud approach a great numbers of accelerograms (different for frequency content and intensity and PGA) are applied to the structure. In the scaling approach, named also IDA (Incremental Dynamic Analysis), a set of selected accelograms is applied incrementing for each of it the PGA. The IDA results are generally presented in terms of Engineering Demand Parameter (EDP) and Intensity Measure (IM), adopting a logarithmic correlation between EDP and IM as reported in Eq. (3): 1) the EDP depend on two coefficients (*a*, *b*) and 2) its standard deviation β is evaluable according Eq. (4)

$$EDP = a\left(IM\right)^{b} \; or \quad \ln(EDP) = \ln(a) + b.\ln(IM) \tag{3}$$

$$\beta_{EDP|IM} \cong \sqrt{\frac{\sum(\ln(EDP_i) - \ln(b.\ln(IM) + \ln(a)))^2}{N-2}} \tag{4}$$

If the standard normal distribution function Φ is adopted, for a given limit state (*x*) and intensity measure (*IM*) the fragility function can be evaluated according Eq. (5)

$$P[EDP \geq x_i \left|IM\right|] = 1 - \Phi[\frac{\ln(x) - \ln(a\, IM^{\,b})}{\beta_{EDP|IM}}] \tag{5}$$

In this work, Eq. (5) will be adopted for the probabilistic seismic demand analysis (PSDA), adopting for (*x*) three different level, evaluated in terms of column drift ratio.

3.2 Seismic Intensity Measures (IM)

The choice of an appropriate intensity measure parameter is important in applying probabilistic seismic analysis. The selection of appropriate parameters can reduce the result dispersion: the concepts of sufficiency, efficiency and practically are recognized to be the attributes of a well selected parameter. Having selected the appropriate intensity, Eq. (2) can be generalized as reported in Eq. (6) if an integral formulation is needed instead of a discrete one (Cornell and Krawinkler 2000), where 1) $G_{\theta|\mathrm{I}Mi}\left(x\left|IM_i\right.\right)$ is the probability that θ will cross-over a given value of *x*, given an earthquake's intensity IM_i 2) $d\,\lambda_{IM_i}\left(IM_i\right)$ is average value of cross over annual rate of IM_i.

$$\lambda_{\theta}\left(x\right) = \int_{IMi} G_{\theta|\mathrm{I}Mi}\left(x\left|IM_i\right.\right)\left|d\,\lambda_{IM_1}\left(IM_i\right)\right| \tag{6}$$

Having defined the probabilistic framework, the selection of the appropriate parameter remains as crucial point. The key challenges are to find the quantitative index for earthquake characterization its introduction (Luco and Cornell 2007, Baker *et al*. 2006). Shome and the colleagues (Shome 1999) have considered that the magnitude is an effective factor in the multi degree response of structures. Some researchers (Shome 1999) proposed the elastic spectrum S_a (*T*) as measure of the earthquake intensity. Alternative Displacement Spectra S_d (*T*) can be adopted (Tothong and Luco 2007) even if not often applied due to the problem related to its uncertainty (Tothong and Luco 2007, Baker and Cornell 2006). Many researchers have stressed that the shape of S_a (T), cannot be considered as unique and deterministic but its variability has to be taken in account. Alternative, instead of S_a, 1) Baker and Cornell (2006) suggested to use the geometric intensity measure 2) Luco and Cornell (2007) proposed, the spectral indices to include the effects of higher modes 3) Baker and Cornell (2005) suggested, for near field, the ratio $R_{T1,T2}$ = S_a (T_1) / S_a (T_2) being T_1 is the fundamental period of structures and T_2 and opportune optimal period to be selected. Other approaches, consider the use of two intensity measures (Baker and Cornell , 2005, 2006), so that Eq. (6) can be rewritten as follows

$$\lambda_\theta(x) = \int_{IM_1} \int_{IM_2} G_{\theta|\mathrm{I}M_1,IM_2}(x\,|IM_1, IM_2) f_{IM_2|IM_1}(IM_2, IM_1) |d\,\lambda_{IM\,1}(IM_1)| \tag{7}$$

In the above equation $f_{IM_2|IM_1}(IM_2, IM_1)$, the conditional probability density function IM_2 on IM_1, is the challenging item in evaluating of the responses: although a process known as "vector probabilistic risk analysis" (Bazzurro and Cornell 1994) have been proposed in the literature for overcoming the problem of the above term calculation, its application, far from the Standard Probabilistic Seismic Hazard Analysis (PSHA), is currently not feasible since it is not generalizable (Bazzurro and Cornell 1994).

The decision about the optimal intensity measures is generally driven by the concepts of efficiency and practically which leads to the evaluation of the proficiency and sufficiency of intensity measures. These criteria are defined in the following section.

(a) Efficient intensity measure

The efficiency is a measure of the dispersion about the median of the results of nonlinear time history analysis: it is inversely proportional to the dispersion, so that lower is the dispersion greater is the efficiency. In this study the dispersion is measured by means of $\beta_{EDP|IM}$ (see Eq. (4)) so that lower values of $\beta_{EDP|IM}$ leads to a more efficient intensity measure as proposed in Padgett et.al (2008). Noticing that $\beta_{EDP|IM}$ affects the structural fragility (see Eq. (5)) as well as the Intensity Measure (see Eq. (6)), it is obvious that different challenges related to the $\beta_{EDP|IM}$ efficiency evaluation have been proposed (Padgett *et al.* 2008).

On passing it has to be noted that, since the efficiency is a measure of the dispersion, greater it is, lower will be the number of running. So that a 50 percent reduction in the dispersion of response leads to a reduction of 75 percent is the minimum required input (Cornell and Krawinkler 2000).

(b) Practical intensity measure

Padgett *et al.* (2008) presented new criteria for selecting an optimal intensity measure in bridges. They introduce the practically of an intensity measure which correlate structural response and seismic hazard. They identified the practically as a coefficient of the regression parameter ***b*** in Eq. (3). The higher value of ***b*** leads to a more practical intensity measure (see Fig. 1).

(c) Composite measure: Proficiency

Padgett *et al.* (2008) composite the measure of efficiency and practically as new criteria to classify the intensity: a "modified" dispersion $\zeta = \beta_{EDP|IM}$ to be introduced in Eq. (5) as reported in

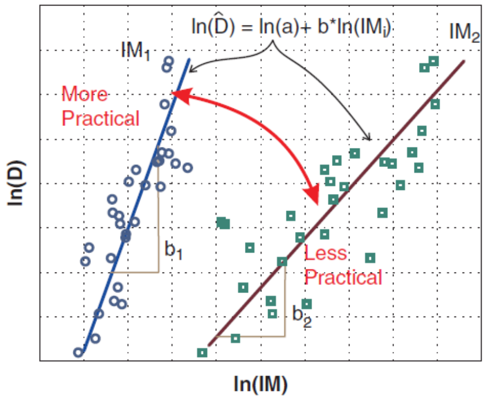

Fig. 1 Comparison of relative practicality of intensity measures (Padgett *et al.* 2008)

the Eq. (8)

$$P[EDP \geq x_i \,|IM|] = 1 - \Phi[\frac{\frac{\ln(x) - \ln(a\,IM^b)}{b}}{\xi}] \tag{8}$$

Noticing that b values greater than 1 will decrease the new dispersion ζ, while b values minor than 1 will increase it, it is obvious that b=1 is the frontier between more and less practical conditions (see Fig. 5) lower values of modified dispersion leads to more *proficient IM* (Padgett *et al*. 2008).

(d) Sufficiency intensity measure

The sufficiency of an IM has also been identified as a viable measure of its appropriateness for use in developing PSDMs (Padgett *et al.* 2008). An intensity measure will be sufficient when its efficiency and practically will have slight changes for different structures belonging to the same typologies (bridges in our cases).

4. Intensity measure definition: Averaged Spectral Acceleration (ASA)

In the next sections a new intensity measure will be validated and compared with the already presented literature proposals that are 1) the PGA 2) the value of S_a at the fundamental period (T_s) of the structure 3) S_a at the different periods in between 0.1T_s and 2.5 T_s.

The proposed intensity measure ($\bar{S}_a$) is the average value (Jeffreys and Jeffreys 1988) of S_a in between T_1 and T_2 (see Eq. (9)) which values will be evaluated based on a parametric analyses

$$\bar{S}_a = \frac{1}{T_2 - T_1}\int_{T_1}^{T_2} S_a(T)\,dT \tag{9}$$

Since the proposed IM is the average value of S_a, it has been named Average Spectral Acceleration (ASA).

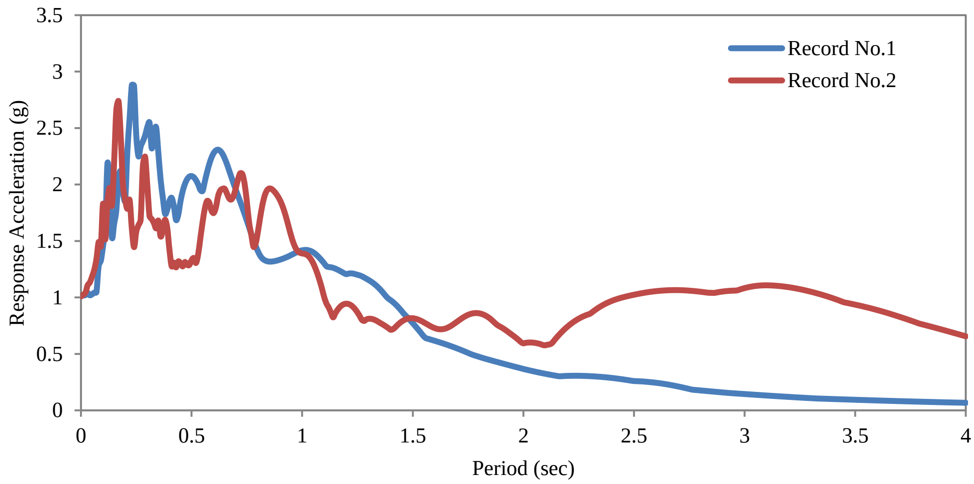

Fig. 2 Comparison of two different pseudo acceleration spectrum

It will be shown that the ASA index will improve proficiency and sufficiency of the classical S_a based index for which as showed in Fig. 2, the evaluation the Spectral Acceleration at a given point (e.g., 1.1 *sec* in the figure) can result non efficient: the S_a values at 1.1 *sec* (considered here representative of the structural period) is equal for both spectra, but slight changing in the definition of referring period can imply great difference between the S_a values of the two considered spectra. So that it seems to be efficient that an index based on S_a evaluation has to take in account the *shape* of the spectrum in between an adequate period range to be evaluated based on the considered structure or typology.

5. Case studies to evaluate Averaged Spectral Acceleration (ASA)

5.1 Bridge descriptions and modeling

The models used in this study are derived from a non-skewed model developed by Nielson (2005), the characteristics of which are based on data obtained from a survey of numerous bridge plans. The common type of the bridge throughout the Central and South-eastern United States is concrete slab on concrete girder highway bridges accounting for approximately 40% of all highway bridges in the region. A typical bridge configuration with standard details is derived through the data collection of concrete girder bridges (Nielson 2005).

Preliminary studies have been carried out referring to the 0^o skewed bridge we studied in (Bayat *et al.* 2015) where a comparison between PGA and S_a have been presented. The new, here presented, preliminary studies regards the validation of the ASA index presented in chapter 4: since the studies presented in (Bayat *et al.* 2015) considered plastic hinges models to simulate pier nonlinear behavior, the same assumption have been here maintained to validate the proposed ASA index for 0^o skewed bridge.

Further studies have been carried out to study the effect of skweness (30^o and 45^o degree) on IMs proficiency and, differently from 0^o skewed bridge; in order to capture the interaction between

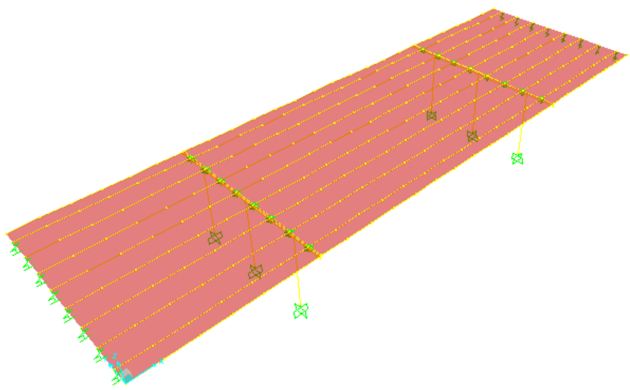

Fig. 3 Three dimensional model of the non- skewed bridge in SAP2000

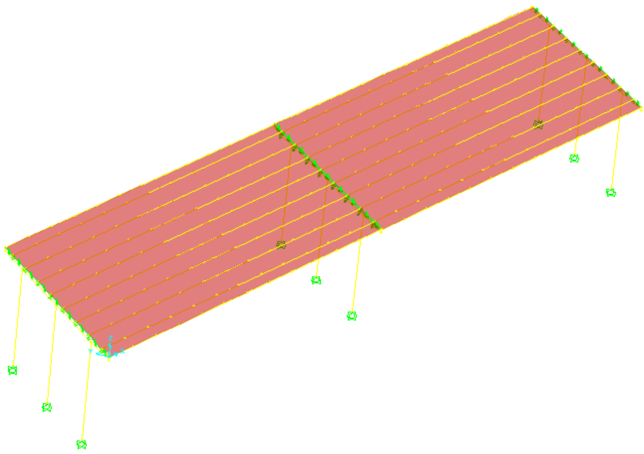

Fig. 4 Three dimensional model of the 30^{o} skewed bridge in SAP2000

N and *M* (in 2 directions) fiber hinges models have been considered: 1) Mander model (1984) has been adopted for confined and un-confined concrete 2) typical steel stress-strain (Elasto-plastic) model with no hardening has been adopted for concrete reinforcement.

Globally, the carried out investigation considers three bridges with different skewness and span length. The non-skewed bridge represents the typology described by Nielson (2005) and the skewed bridges have been defined by modifying the Nielson non-skewed bridge. All the bridges have the same pier (5.75 m in height) while span numbers and the length depend on the skewness: 1) the straight bridge (Fig. 3) has three spans which lengths are 12.2/24.4/12.2 m 2) the 30^{o}skewed bridge (Fig. 4) have two equal length span (24.4 m) 3) the 45^{o}skewed bridge has three spans which lengths are 12.2/24.4/12.2 m.

Each span, which depth is 15.01 m, includes eight AASHTO type pre-stressed concrete girders. The scheme of deck, column and cap beam are presented in Fig. 6(a) to 6(c). The deck is modeled using shell elements. 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. Abutments and the column boundary conditions are considered Free in the longitudinal direction and considered fixed in the

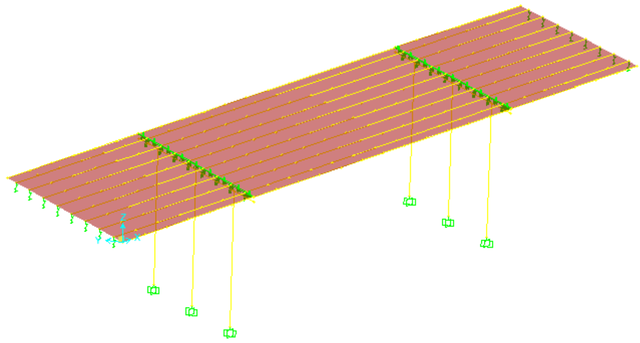

Fig. 5 Three dimensional model of the 45^o skewed bridge in SAP2000

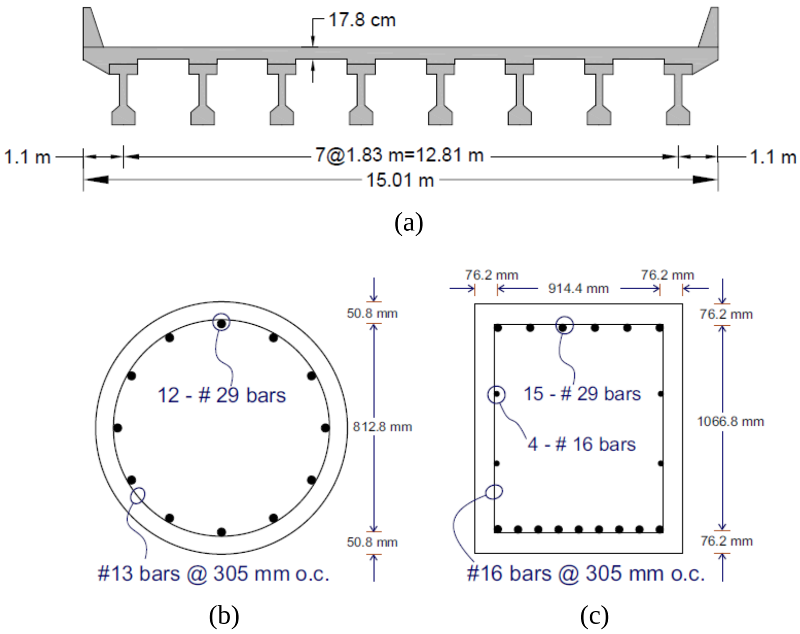

Fig. 6 Concrete member reinforcing layout (a) Deck detail (b) Column (c) Bent beam

transverse direction. We have considered different directions of earthquake and applied it to our bridges. Our results showed that the critical direction is the longitudinal one, therefore the earthquake has been applied to the longitudinal direction of the bridges. The column drift ratio in this study is related to combination of the transverse and longitudinal direction results. The transverse direction is also considered in our study.

The soil-structure interaction is neglected in this study. The fundamental period of the bridges are: 1) straight bridge: 0.55s 2) 30^o skewed bridge: 0.66s 3) 45^o skewed bridge: 0.48s.

5.2 Seismic scenario: earthquake selection

In order to framework the analyses in a seismic scenario well accepted by the international community, the FEMA-P695 (2003) have been pursued for the selection of 20 records (listed in Table 1) which 5% damping acceleration spectra (normalized to a unitary PGA) are reported in Fig. 7 together with the averaged spectra. The earthquake ground motion records cover different

Table 1 Characteristics of the earthquake ground motion histories (FEMA 2003)

			Earthquake		Recording station	
ID No	M	PGA	Year	Name	Name	owner
1	7.0	0.48	1992	Cape Mendocino	Rio Dell Overpass	USGS
2	7.6	0.21	1999	Chi-Chi, Taiwan	CHY101	CWB
3	7.1	0.82	1999	Duzce,Turkey	Bolu	ERD
4	6.5	0.45	1976	Friuli, Italy	Tolmezzo	------------
5	7.1	0.35	1999	Hector Mine	Hector	SCSN
6	6.5	0.34	1979	Imperial Valley	Delt	UNAMUCSD
7	6.5	0.35	1979	Imperial Valley	El Centro Array#1	USGS
8	6.9	0.38	1995	Kobe, Japan	Nishi-Akashi	CUE
9	6.9	0.51	1995	Kobe,Japan	Shin-Osaka	CUE
10	7.5	0.24	1999	Kokaeli,Turkey	Duzce	ERD
11	7.3	0.36	1992	Landers	Yemo Fire Station	CDMG
12	7.3	0.24	1992	Landers	Coolwater	SCE
13	6.9	0.42	1989	Loma Prieta	Capitola	CDMG
14	6.9	0.53	1989	Loma Prieta	Gilory Arrey#3	CDMG
15	7.4	0.56	1990	Manjil	Abbar	BHRC
16	6.7	0.55	1994	Northridge	Beverly Hills-Mulhol	USC
17	6.7	0.44	1994	Northridge	Canyon Country-WLC	USC
18	6.6	0.36	1971	San Ferando	LA-Hollywood Stor	CDMG
19	6.5	0.51	1987	Superstition Hills	El Centro Imp.Co	CDMG
20	6.5	0.52	1987	Superstition Hills	Poe Road (temp)	USGS

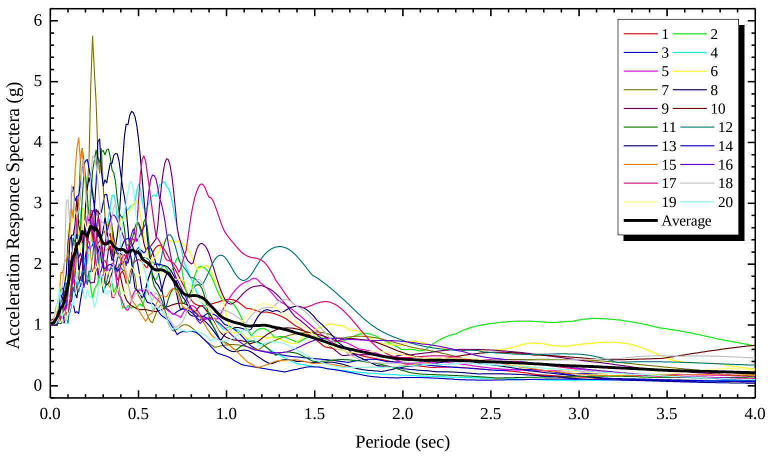

Fig. 7(a) Response acceleration spectra of far field ground motions (b) Percentiles of response acceleration spectra of a suit of 20 far field earthquake ground motion records

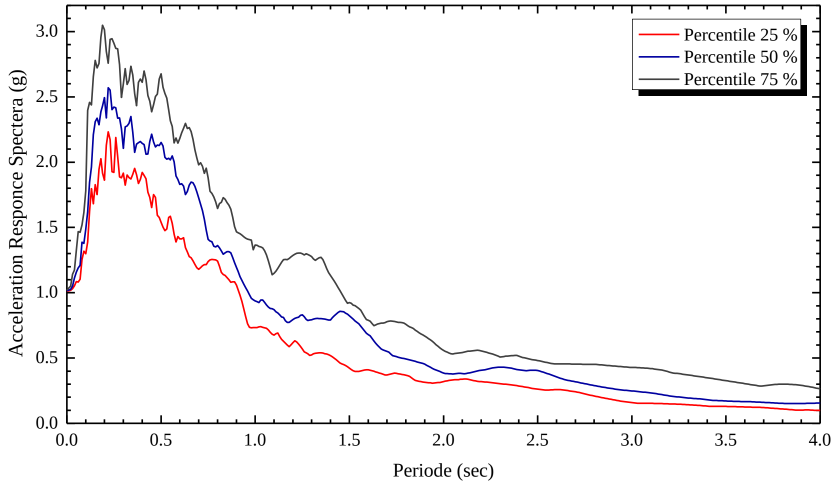

Fig. 7(a) Continued

range of PGA form medium (0.21 g, Earthquake 2 in Table 1) to strong motions (0.8 g, Earthquake 3 in Table 1) which are used to perform incremental dynamic analysis (IDA). The FEMA records satisfy the following criteria: **a)** Magnitude >6.5 **b)** Distance from source to site > 10 km (average of Joyner-Boore and Campbell distances) (Boore *et al.* 1993) **c)** Peak Ground Acceleration (PGA) >0.2 g and Peak Ground Velocity (PGV) >5 cm/sec **d)** Soil shear wave velocity, in upper 30 m of soil, 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".

5.3 Damage index and limit states

In seismic analytical fragility analysis, the response of the structure under earthquake is evaluated in terms of potential damages measured through damage function such as EDPs (FEMA 1999, Choi *et al.* 2004, Yi *et al.* 2007). In highway bridge, the most critical component are pier columns which nonlinear behavior can be measured either in terms of involved section ductility or drift ratio both assumed to be a measure of the Damage Index (DI) that can be classified according four levels (see Table 2) as suggested in (FEMA 1999, Choi *et al.* 2004, Yi *et al.* 2007).

Since the considered range of PGAs, induced a column drift ratio (θ) no greater than 0.05, three levels of damages (Slight, Moderate and Extensive) that respectively correspond to the well

Table 2 Summary of DIs and corresponding LS for concrete columns

DI	Slight	Moderate	Extensive	Collapse
Physical phenomenon (FEMA 1999)	Cracking and spalling	Moderate cracking and spalling	Degradation without collapse	Failure leading to collapse
Section ductility (Choi *et al.* 2004)	μ_k>1	μ_k>2	μ_k>4	μ_k>7
Drift ratio (Yi *et al.* 2007)	θ>0.007	θ>0.015	θ>0.025	θ>0.05

know level proposed in (FEMA 2003) : 1) Immediate Occupancy 2) Life Safety and 3) Collapse prevention.

5.4 Incremental Dynamic Analysis (IDA)

As we have mentioned above, to derive analytical fragility curves, one of the approaches is the incremental dynamic analysis (IDA) that allows to analyze the bridge response from the elastic range till the collapse range. The results of IDA analyses are reported in terms of relation (curve) between the input (earthquake ground motion) and the output (structural response): the crucial aspect in plotting IDA curve is the definition of a parameter representative of the demand, so that as will be discussed in the next section many effort has been devote to define the most appropriate parameter based on the analyses of the most efficient parameters proposed in scientific literature (see chapter 3.2).

6. Results and discussions

Different spectral intensity measures (IM) have been compared against the proposed ASA intensity, evaluating the fragility curves for the considered limit states: 1) slight 2) moderate and 3) extensive.
For each selected accelerogramsthe IDA analysis are run either till 1) the defined limit state is reached or 2) the incremented PGA became greater than 2.5 g (earthquake with PGA >2.5 g are considered to have a very low probability): typical IDA analyses results are reported in Fig. 8(a), (b).

The results of the analyses are reported in Tables 3,4,5 in terms of dispersion coefficients obtained for the three considered skewness (0^o, 30^o, 45^o) and in Figs. 9, 10, 11.The considered IM are the 1) PGA, 2) the S_a evaluated for different structural periods and 3) the ASA index evaluated considering three periods ranges characterized by different lower (T_1) and upper (T_2) periods that are: a) T_1=0, T_2=4*s* b) T_1=0.5T_s, T_2=2T_s c) T_1=0.9T_s, T_2=1.4T_s 4) *b* has been considered as practically measure (see Eq. (3)) 5) $\beta_{D|IM}$ (see Eq. (4)) has been considered to measure the efficiency and 6) $\zeta=\beta_{EDP|IM}/b$ to measure the proficiency 4) the value of *b* has been evaluated by means of regression analyses which results are reported in the Figs. 10, 11, 12 where the correlation between the different considered IM and column drift (EDP) are reported for the PGA and the optimal values of S_a and ASA indexes.

The data (see Tables 1, 2, 3) confirm the results obtained by the authors in (Bayat *et al.* 2015) where the zero skeweness bridge has been studied: S_a is more performing than PGA and, depending on the skweness, the optimal period is in the range T_s or 1.2 T_s.

Further on, the carried out analyses supported the definition of the best performing period range for the evaluation of ASA obtaining that 1) the optimal period range is 0.9-1.4 and 2) ASA evaluated at 0.9-1.4 is the most performing IM for the considered bridge typologies.

The results, in terms of proficiency, are reported in the Figs. 15, 16 where, for each skwness and for each considered *IM*, the proficiency value and error of the IM with respect of the optimized ASA index are given; it can be argued that: 1) the PGA error ranges between ≈80 and 130% 2) the S_a error ranges between ≈10 and 33% .

The results globally state that 1) the PGA is not adequate as IM for this bridge typology having an error to 130% (if compared with the optimized ASA IM) 2) the S_a can be, successfully

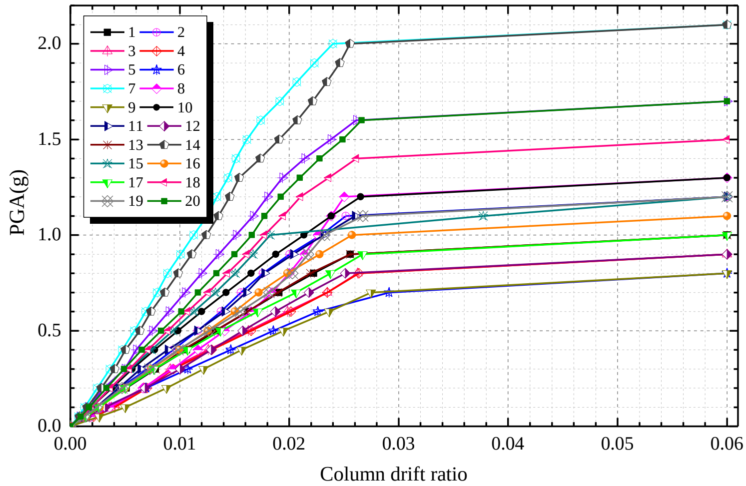

Fig. 8(a) IDA curves: 30 degree skewed highway bridge corresponding to the set of selected accelerogramms

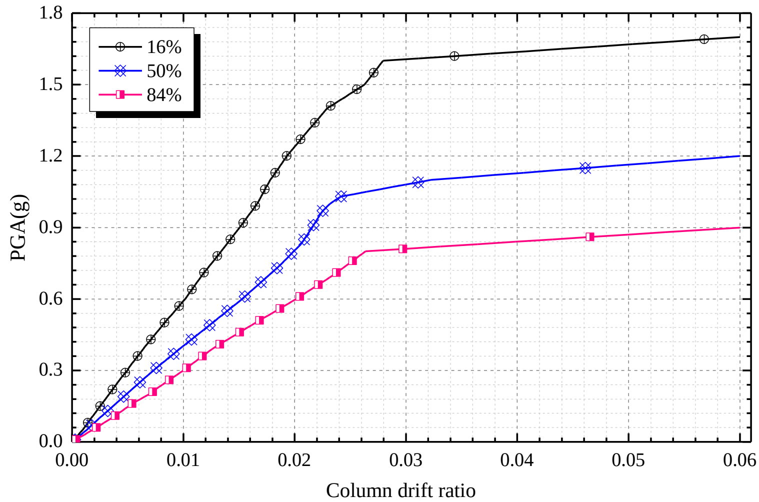

Fig. 8(b) IDA curves corresponding to 16%, 50% and 84% PGA fractal curves: 30 degree skewed highway bridge

optimized, for a specific case, but its proficiency is very sensible to the selected period: for example, as far as 45 skewness is concerned the error can be either ≈10% or ≈100% considering a slight difference of the period at which S_a is evaluated (T_s or $1.2T_s$). The high sensibility has direct effect on the fragility curves (see Fig. 12): it is evident that as far as the 45° skewed bridge is concerned, considering different structural period (T_s or $1.2T_s$) can lead to different failure probability such as in the case of the Extensive Limit State where Failure probability corresponding to S_a=3 can be either 0.2 or 0.4 if the referring periods are respectively T_s or $1.2T_s$.

Based on the previous observations it can be stated that ASA index can be considered an appropriate IM since, choosing an adequate period ranges, overcome the high sensibility of Sa to the selected period: the basic idea of ASA to average the Spectral Acceleration value in an adequate structural period range avoid the high sensibility of Sa when slight change of period are considered.

A sufficient IM is conditionally statistically independent of ground motion characteristics. A proposed intensity measure will be sufficient when it has the less dispersion and more practical in comparison of different intensity measure and also in different structures. If an intensity measure is working well in a single typical structure, it will be sufficient when it is capable to extend to other types of structures. Therefore, we have considered three different types of structures.

The comparison of proficiency values is reported in Fig. 15 from were where the results of the three considered typologies are reported: it can be deduced that the proposed intensity measure (optimized ASA) has the less dispersion for three different bridge typology so that can be argued that it is a proficient and sufficient IM measure.

Table 3 Non-skewed bridge: dispersion index vs IMs

IM	*Ln* (*a*)	*b*	$\beta_{EDP\|IM}$	$\zeta = \frac{\beta_{EDP\|IM}}{b}$	Differences (%)
PGA(g)	**-4.072**	**0.893**	**1.090**	**1.221**	**141.45**
Sa(0.1Ts,5%)	-4.253	0.833	1.221	1.466	189.80
Sa(0.2Ts,5%)	-4.552	0.731	1.695	2.319	358.53
Sa(0.3Ts,5%)	-4.727	0.73	1.840	2.520	398.39
Sa(0.4Ts,5%)	-4.816	0.807	1.675	2.076	310.45
Sa(0.5Ts,5%)	-4.799	0.821	1.508	1.837	263.27
Sa(0.6Ts,5%)	-4.753	0.859	1.515	1.764	248.73
Sa(0.7Ts,5%)	-4.722	0.848	1.273	1.501	196.75
Sa(0.8Ts,5%)	-4.671	0.837	1.277	1.526	201.77
Sa(0.9Ts,5%)	-4.66	0.867	1.288	1.485	193.70
Sa(Ts,5%)	-4.6	0.952	1.156	1.214	140.11
Sa(1.1Ts,5%)	-4.531	0.981	0.682	0.695	37.52
Sa(1.2 Ts,5%)	**-4.465**	**0.969**	**0.654**	**0.675**	**33.54**
Sa(1.3Ts,5%)	-4.392	0.954	0.814	0.853	68.76
Sa(1.4Ts,5%)	-4.294	0.985	0.886	0.899	77.76
Sa(1.5Ts,5%)	-4.3	0.939	0.719	0.766	51.38
Sa(1.6Ts,5%)	-4.237	0.92	0.898	0.976	92.95
Sa(1.7Ts,5%)	-4.123	0.909	1.009	1.109	119.39
Sa(1.8Ts,5%)	-4.028	0.866	1.039	1.200	137.26
Sa(1.9Ts,5%)	-4.001	0.83	1.122	1.352	167.28
Sa(2Ts,5%)	-3.969	0.798	1.237	1.550	206.59
Sa(2.1Ts,5%)	-3.985	0.742	1.309	1.764	248.85
Sa(2.2Ts,5%)	-4.008	0.691	1.444	2.089	313.18
Sa(2.3Ts,5%)	-4	0.673	1.573	2.337	362.18
Sa(2.4Ts,5%)	-3.98	0.671	1.648	2.455	385.54
Sa(2.5Ts,5%)	-3.949	0.686	1.678	2.446	383.70
ASA (0-4)	-3.75	0.952	0.852	0.895	76.89
ASA (0.5Ts-2Ts)	-4.49	1.001	0.553	0.553	9.34
ASA (0.9Ts-1.4Ts)	**-4.63**	**1.003**	**0.507**	**0.506**	**0.00**

Table 4 30 degree skewed bridge: dispersion index vs IMs

IM	*Ln* (*a*)	*b*	$\beta_{EDP\|IM}$	$\zeta = \frac{\beta_{EDP\|IM}}{b}$	Differences (%)
PGA(g)	**-3.95**	**0.864**	**1.240**	**1.435**	**130.3371**
Sa(0.1Ts,5%)	-4.18	0.826	2.107	2.550	309.3098
Sa(0.2Ts,5%)	-4.6	0.778	1.608	2.067	231.7817
Sa(0.3Ts,5%)	-4.73	0.728	1.815	2.493	300.1605
Sa(0.4Ts,5%)	-4.75	0.74	1.780	2.405	286.0353
Sa(0.5Ts,5%)	-4.65	0.846	1.341	1.585	154.4141
Sa(0.6Ts,5%)	-4.6	0.858	1.249	1.456	133.7079
Sa(0.7Ts,5%)	-4.59	0.809	1.412	1.746	180.2568
Sa(0.8Ts,5%)	-4.48	0.888	1.125	1.267	103.3708
Sa(0.9Ts,5%)	-4.39	0.928	1.122	1.210	94.22151
Sa(Ts,5%)	-4.34	0.932	0.769	0.825	32.42376
Sa(1.1Ts,5%)	**-4.22**	**0.944**	**0.714**	**0.757**	**21.50883**
Sa(1.2 Ts,5%)	-4.17	0.92	0.864	0.939	50.72231
Sa(1.3Ts,5%)	-4.14	0.895	1.068	1.193	91.49278
Sa(1.4Ts,5%)	-4	0.902	1.005	1.114	78.8122
Sa(1.5Ts,5%)	-3.9	0.853	1.292	1.515	143.1782
Sa(1.6Ts,5%)	-3.9	0.813	1.470	1.808	190.2087
Sa(1.7Ts,5%)	-3.89	0.772	1.616	2.094	236.1156
Sa(1.8Ts,5%)	-3.89	0.734	1.757	2.394	284.2697
Sa(1.9Ts,5%)	-3.9	0.708	1.854	2.618	320.2247
Sa(2Ts,5%)	-3.89	0.706	1.863	2.639	323.5955
Sa(2.1Ts,5%)	-3.8	0.712	1.862	2.615	319.7432
Sa(2.2Ts,5%)	-3.8	0.729	1.801	2.470	296.4687
Sa(2.3Ts,5%)	-3.7	0.732	1.795	2.452	293.5795
Sa(2.4Ts,5%)	-3.6	0.744	1.764	2.371	280.5778
Sa(2.5Ts,5%)	-3.6	0.741	1.766	2.383	282.504
ASA (0-4)	-3.63	0.884	1.145	1.296	108.0257
ASA (0.5Ts-2Ts)	-4.192	0.939	0.720	0.767	23.11396
ASA (0.9Ts-1.4Ts)	**-4.244**	**0.955**	**0.595**	**0.623**	**0**

Table 5 45 degree skewed bridge: dispersion index vs IMs

IM	*ln* (*a*)	*b*	$\beta_{EDP\|IM}$	$\zeta = \frac{\beta_{EDP\|IM}}{b}$	Differences (%)
PGA(g)	**-4.29**	**0.953**	**0.884**	**0.928**	**79.71**
Sa(0.1Ts,5%)	-4.47	0.934	1.015	1.087	110.48
Sa(0.2Ts,5%)	-4.66	0.875	1.386	1.584	206.81
Sa(0.3Ts,5%)	-5.01	0.881	1.380	1.566	203.29
Sa(0.4Ts,5%)	-5.12	0.825	1.580	1.915	270.80
Sa(0.5Ts,5%)	-5.15	0.794	1.698	2.138	314.05
Sa(0.6Ts,5%)	-5.12	0.847	1.426	1.683	225.99
Sa(0.7Ts,5%)	-5.05	0.882	1.251	1.419	174.73
Sa(0.8Ts,5%)	-5.03	0.887	1.222	1.378	166.86
Sa(0.9Ts,5%)	-4.99	0.931	0.842	0.904	75.05
Sa(Ts,5%)	**-4.97**	**0.965**	**0.965**	**0.546**	**9.72**

Table 5 Continued

Sa(1.1Ts,5%)	-4.9	0.958	0.635	0.662	28.19
Sa(1.2 Ts,5%)	-4.8	0.924	0.958	1.037	100.82
Sa(1.3Ts,5%)	-4.7	0.914	1.043	1.141	120.92
Sa(1.4Ts,5%)	-4.7	0.896	1.113	1.242	140.51
Sa(1.5Ts,5%)	-4.59	0.903	1.088	1.205	133.37
Sa(1.6Ts,5%)	-4.57	0.915	1.061	1.159	124.45
Sa(1.7Ts,5%)	-4.57	0.895	1.212	1.355	162.31
Sa(1.8Ts,5%)	-4.54	0.868	1.365	1.572	204.45
Sa(1.9Ts,5%)	-4.41	0.865	1.352	1.563	202.74
Sa(2Ts,5%)	-4.28	0.853	1.455	1.705	230.20
Sa(2.1Ts,5%)	-4.27	0.829	1.578	1.904	268.66
Sa(2.2Ts,5%)	-4.27	0.789	1.731	2.194	324.81
Sa(2.3Ts,5%)	-4.28	0.775	1.772	2.287	342.86
Sa(2.4Ts,5%)	-4.3	0.73	1.891	2.590	401.57
Sa(2.5Ts,5%)	-4.3	0.7	2.018	2.883	458.28
ASA (0-4)	-4.01	0.925	1.043	1.127	118.25
ASA (0.5Ts-2Ts)	-4.88	0.96	0.715	0.745	44.40
ASA (0.9Ts-1.4Ts)	**-4.94**	**0.971**	**0.501**	**0.516**	**0**

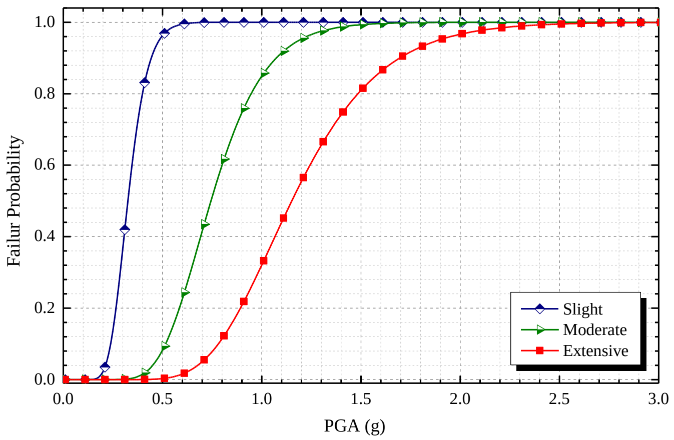

Fig. 9(a) Fragility curves of the bridge pier respect to PGA for 0^{o} skewed bridge

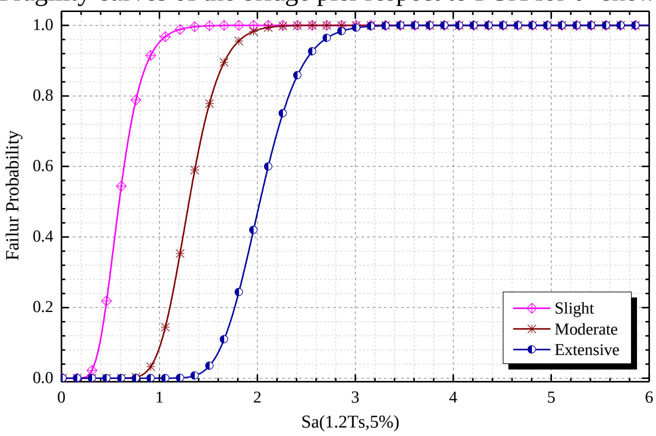

Fig. 9(b) Fragility curves of the bridge pier respect to *Sa* (1.2*Ts*, 5%) for 0^{o} skewed bridge

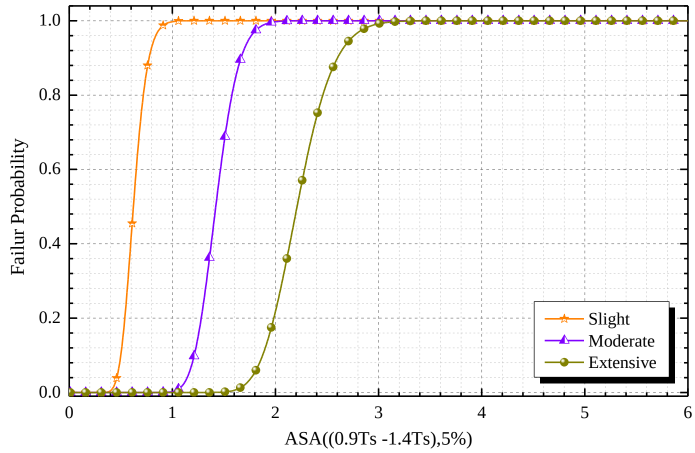

Fig. 9(c) Fragility curves of the bridge pier respect to ASA optimized for 0^{o} skewed bridge

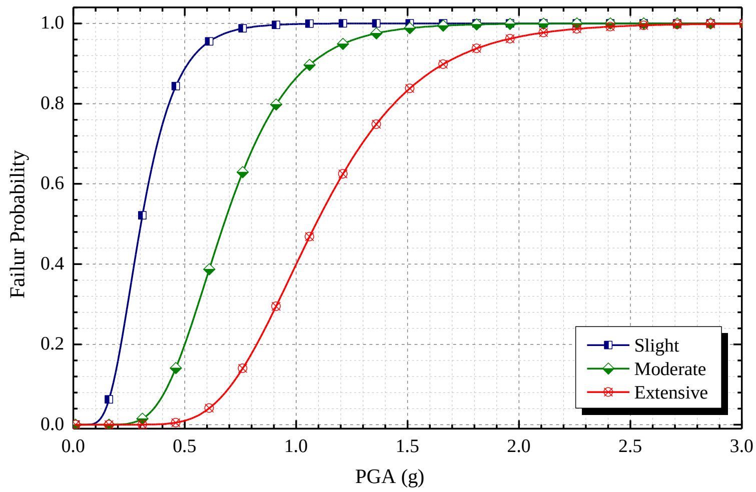

Fig. 10(a) Fragility curves of the bridge pier respect to PGA for 30^{o} skewed bridge

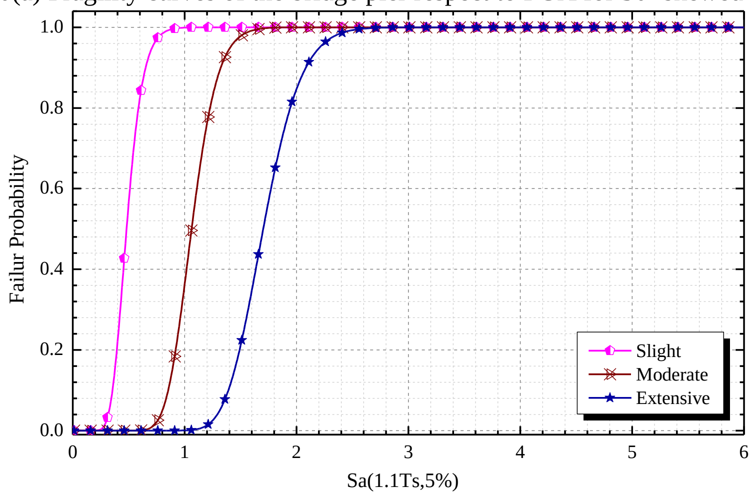

Fig. 10(b) Fragility curves of the bridge pier respect to $Sa(1.1Ts,5\%)$ for 30^{o} skewed bridge

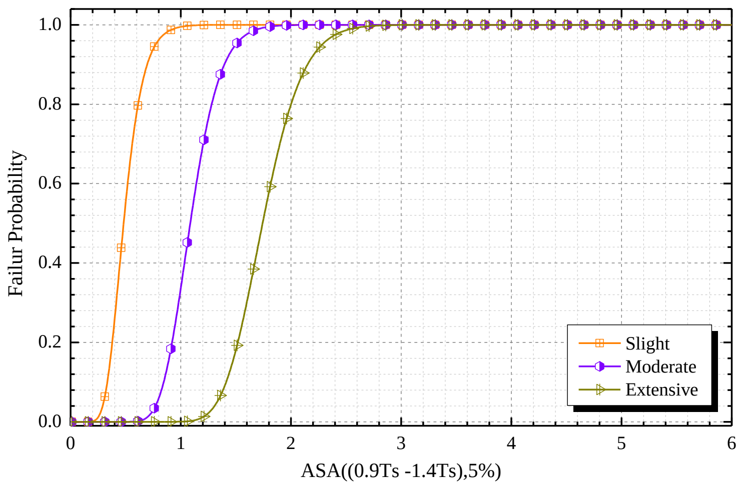

Fig. 10(c) Fragility curves of the bridge pier respect to ASA optimized for 30^o skewed bridge

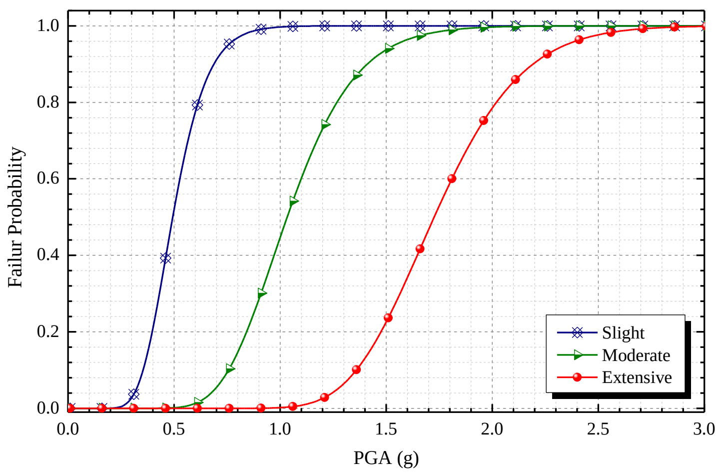

Fig. 11(a) Fragility curves of the bridge pier respect to PGA for 45^o skewed bridge

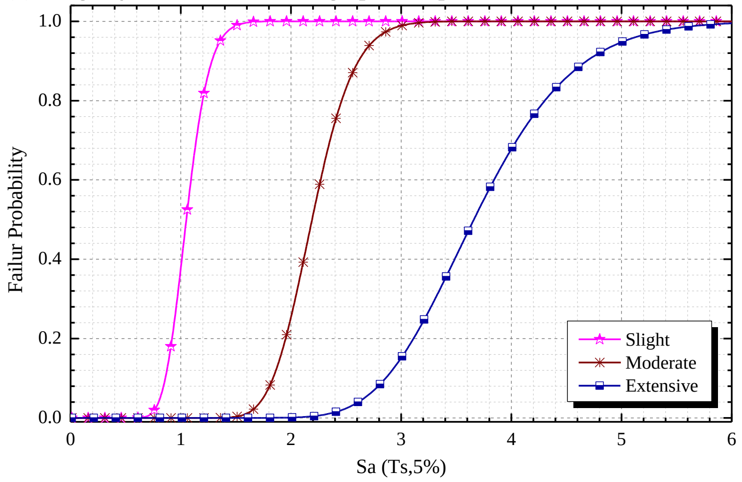

Fig. 11(b) Fragility curves of the bridge pier respect to $Sa(T_s,5\%)$ for 45^o skewed bridge

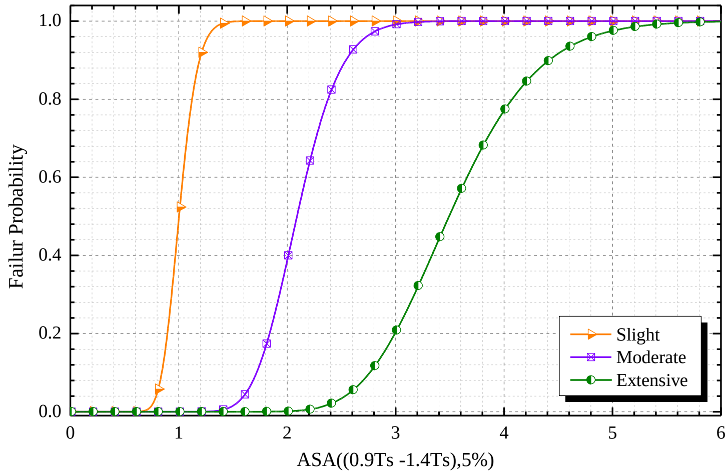

Fig. 11(c) Fragility curves of the bridge pier respect to ASA optimized for 45^{o} skewed bridge

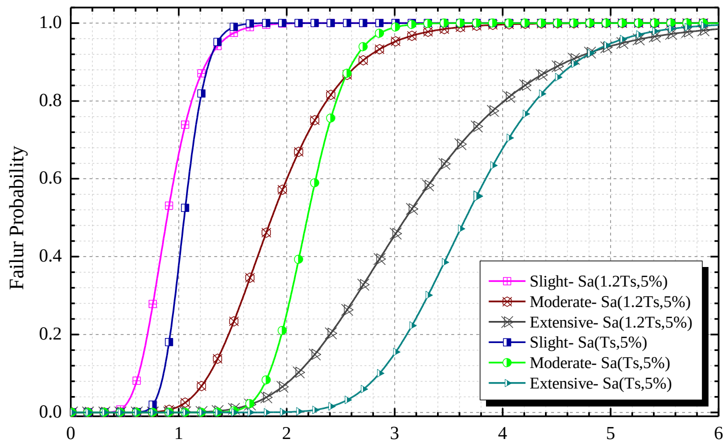

Fig. 12 Comparison *Sa*(1.2*Ts*, 5%) and *Sa*(*Ts*, 5%) fragility curves

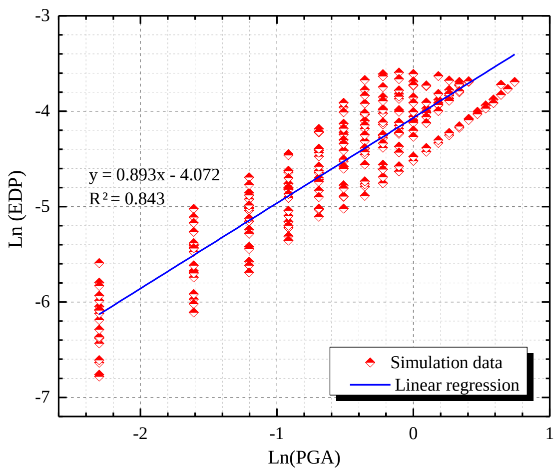

Fig. 13(a) Non-skewed bridge: regression analyses. Maximum column drift ratio (EDP) vs PGA (IM)

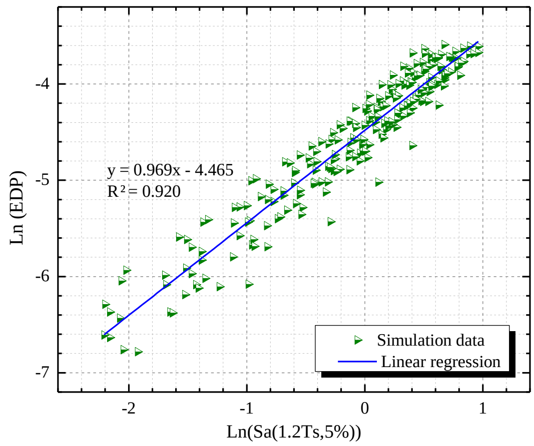

Fig. 13(b) Non-skewed bridge: regression analyses. Maximum column drift ratio (EDP) vs optimized ASA (IM)

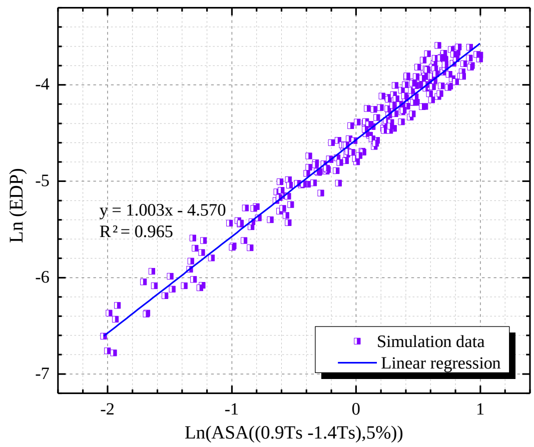

Fig. 13(c) Non-skewed bridge: regression analyses. Maximum column drift ratio (EDP) vs optimized ASA (IM)

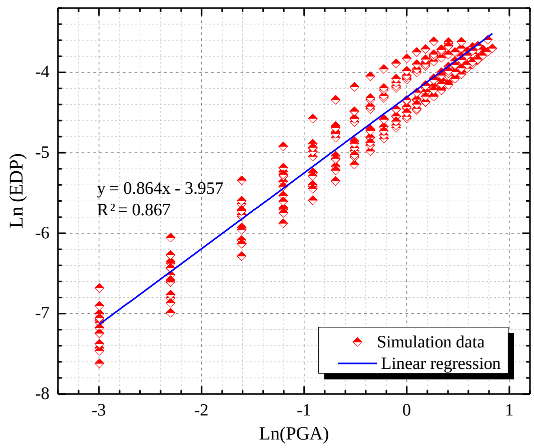

Fig. 14(a) 30 degree-skewed bridge: regression analyses. Maximum column drift ratio (EDP) vs PGA (IM)

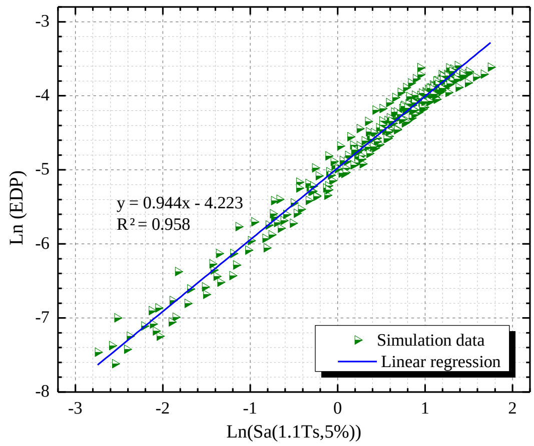

Fig. 14(b) 30 degree-skewed bridge: regression analyses. Maximum column drift ratio (EDP) vs optimized S_a (IM)

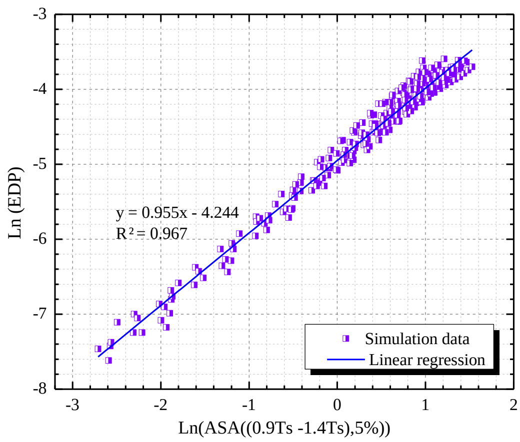

Fig. 14(c) 30 degree-skewed bridge: regression analyses. Maximum column drift ratio (EDP) vs optimized ASA (IM)

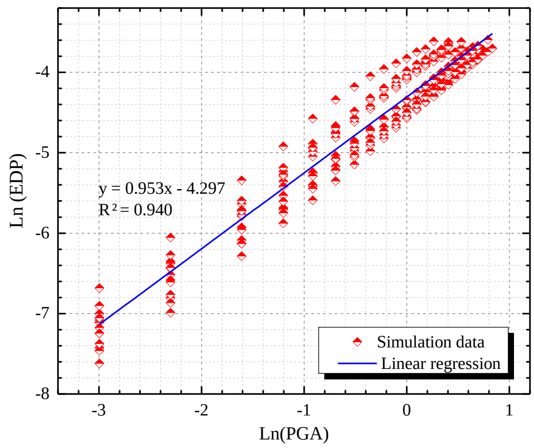

Fig. 15(a) 45 degree-skewed bridge: regression analyses. Maximum column drift ratio (EDP) vs PGA (IM)

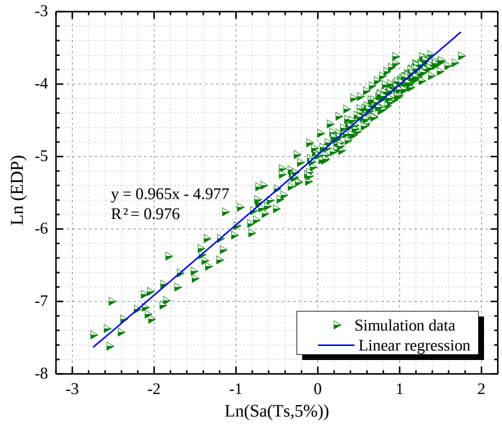

Fig. 15(b) 45 degree-skewed bridge: regression analyses. Maximum column drift ratio (EDP) vs optimized S_a (IM)

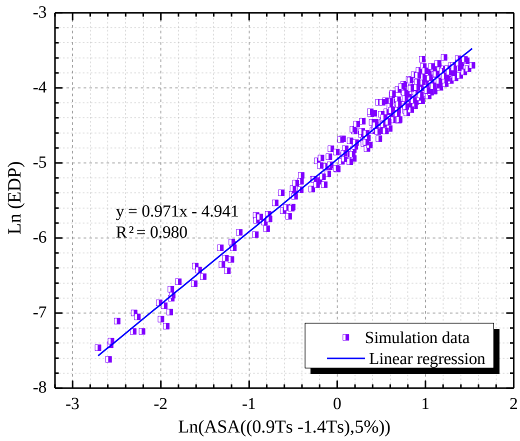

Fig. 15(c) 45 degree-skewed bridge: regression analyses. Maximum column drift ratio (EDP) vs optimized ASA (IM)

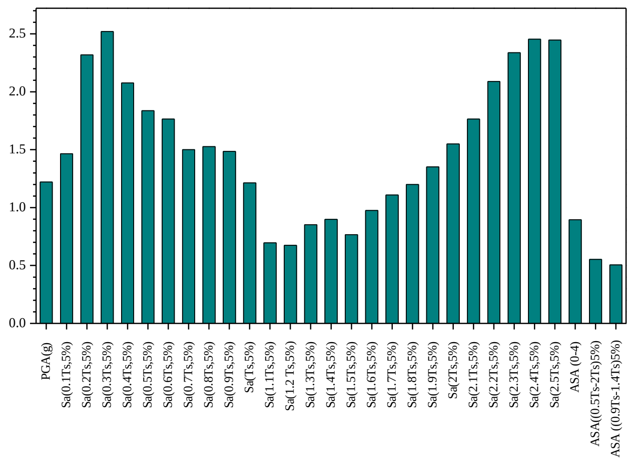

Fig. 16(a) Non-skewed bridge: IM proficiency values

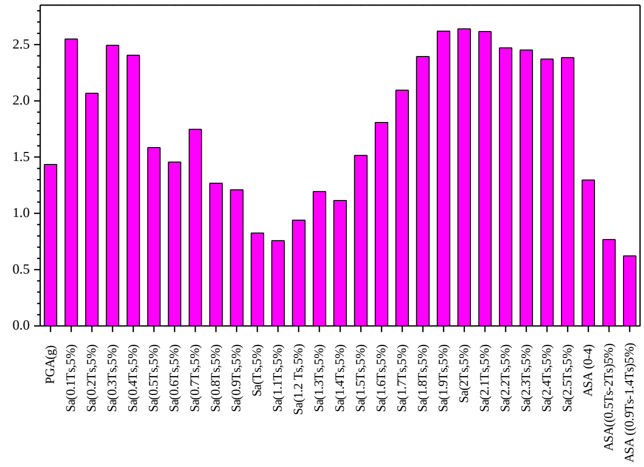

Fig. 16(b) 30 degree skewed bridge: IM proficiency values

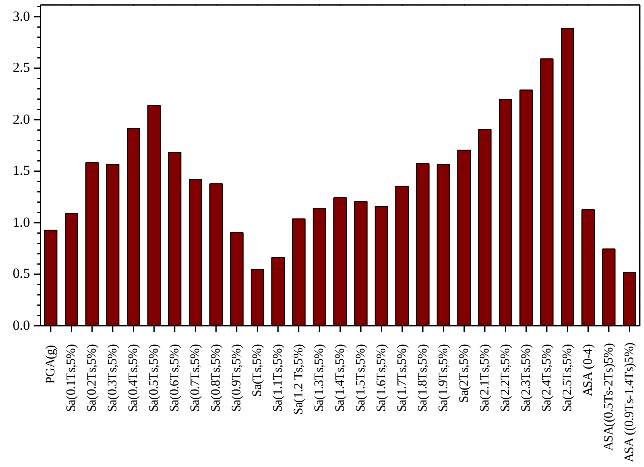

Fig. 16(c) 45 degree skewed bridge: IM proficiency values

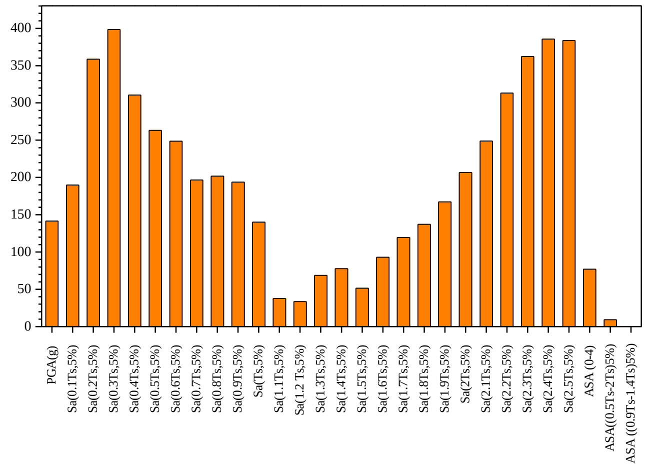

Fig. 17(a) Non-skewed bridge: IM error percentage

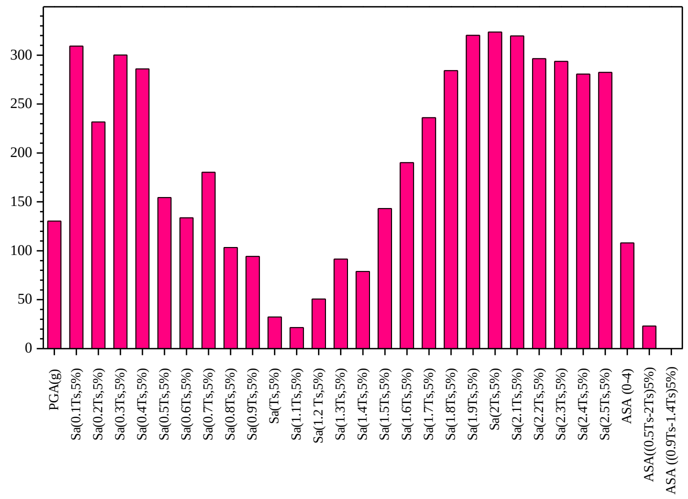

Fig. 17(b) 30 degree skewed bridge: IM error percentage

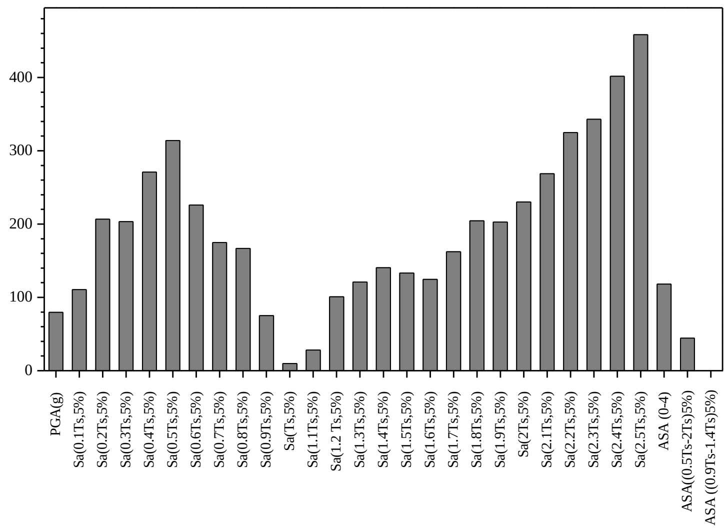

Fig. 17(c) 45 degree skewed bridge: IM error percentage

6. Conclusions

In this paper, probabilistic seismic behavior of skewed highway bridges using fragility function methodology has been studied. Incremental dynamic analysis (IDA) was applied successfully to the bridge models in order to study the behavior of the structure from linear range to extreme nonlinear behavior. A new proficient and sufficient intensity measure named optimized Averaged Spectral Acceleration (ASA) was introduced to decrease (up to 10%) the dispersion in the results. ASA were optimized to achieve to the highest efficiency in the results. We have observed that the range of structural period between T_1=0.9T_s, T_2=1.4T_s is the critical range in acceleration spectrum. By considering the shape effect of this range we have achieved to the highest accuracy in presenting a proficient intensity measure. A sensitive analysis was done on the different spectral intensity measures. The effect of the referring period on the intensity measures and the fragility curves have been studied, inferring the necessity to take in account the spectral shape by means the ASA index that resulted suitable intensity measure if compared with PGA and other spectral intensity measures, as far as skewed highway bridges are concerned.

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