IMPLICATIONS OF IMPORTANCE FACTOR ON SEISMIC DESIGN - FROM 2000 SAC-FEMA PERSPECTIVE

Prateek Kumar Dhir B. Tech., M. Tech. (Corresponding author)

Ph. D Scholar, Department of Civil and Environmental Engineering, University of Strathclyde, Glasgow G11XQ, UK, Email: prateek.dhir@strath.ac.uk, Ph.: +44-7424825460

Nikhil P. Zade B. E., M. Tech.

Ph. D. Candidate, Department of Civil Engineering, National Institute of Technology Rourkela, Odisha 769008, India, Email: nikhilzade648@gmail.com, Ph.: +91-9284380929

Akhila Basu B. Tech., M. Tech.

Assistant Executive Engineer, Vigilance and Quality Control Division, Panchayati Raj Engineering Department, Medak, Govt. of Telengana, India, Email: akhilabasu15@gmail.com, Ph.: +91-9493403738

Robin Davis B.E., M.E., Ph.D.

Assistant Professor, Department of Civil Engineering, National Institute of Technology Calicut, NIT Campus P.O 673 601, Kozhikode, India, Email: robin@nitc.ac.in, Ph.: +91-9583066760

Pradip Sarkar B.E., M.E., Ph.D.

Associate Professor, Department of Civil Engineering, National Institute of Technology Rourkela, Odisha 769008, India, Email: sarkarp@nitrkl.ac.in, Ph.: +91-9439429739

IMPLICATIONS OF IMPORTANCE FACTOR ON SEISMIC DESIGN - FROM 2000 SAC-FEMA PERSPECTIVE

Prateek Kumar Dhir[1*], Nikhil P. Zade[2], Akhila Basu[3], Robin Davis[4], Pradip Sarkar[5]

[1*]Ph. D Scholar, University of Strathclyde, Glasgow, UK (Corresponding author)

Email: prateek.dhir@strath.ac.uk, Ph.: +44-7424825460

[2]Ph. D Scholar, National Institute of Technology Rourkela, Odisha 769008, India

Email: nikhilzade648@gmail.com, Ph.: +91-9284380929

[3]Formerly, M. Tech Scholar, National Institute of Technology Rourkela, Odisha 769008, India

Email: akhilabasu15@gmail.com, Ph.: +91-9493403738

[4]Assistant Professor, National Institute of Technology Calicut, NIT Campus P.O 673 601, Kozhikode, India

Email: robin.davisp@gmail.com, Ph.: +91-9583066760

[5]Associate Professor, National Institute of Technology Rourkela, Odisha 769008, India

Email: sarkar.pradip@gmail.com, Ph.: +91-9439429739

ABSTRACT

The seismic design of buildings uses global ductility factor and occupancy importance factor (*IF*) as two major fixed parameters in defining the safety of the structure. The study of performance variation of the structure with global ductility factor is available but, there is hardly any study that provides information regarding the increase in the level of safety achieved by increasing the *IF* values. Being a building categorical dependent parameter, *IF* is used by the international seismic design codes for increasing the design loads of the structure. The change in the level of safety achieved through the variation in the value of the *IFs* for reinforced concrete (RC) framed buildings will perhaps be an important useful representation of the stakeholders for the approximate damage cost estimation. This article performs the structural safety assessment against seismic load using a standard structural reliability method with second-order hazard approximation to evaluate the effect of the *IF* on the level of safety and the cost associated with the building. Results show that, an overall reduction of 50-60% in the

damage index of the selected buildings can be achieved by increasing the *IF* from a value of 1.0 to 2.0 with a consequent increase in the cost of the building.

Keywords: Importance Factor, Fragility curve, Reliability index, Cost index

INTRODUCTION

In seismic design, occupancy importance factor (*IF*) is a multiplier to increase or decrease the design base shear, according to different occupancy category or importance class of a building. In order to obtain an acceptable performance and affordable cost, international seismic design codes have suggested different values of *IFs*. The higher the value of *IF* of the building, the higher is the expected seismic performance at the expense of higher cost. The value of *IF* recommended by major international codes (Table 1) is based on the functional use of the buildings, the nature of the hazardous consequences of a severe event, post-earthquake functional needs, and historical value or economic importance. All the above factors were considered by the building design codes through engineering justification and experience as there are no detailed experimental or analytical studies on the occupancy importance factor available in published literature. The building lots are mainly divided into two or three broad categories by the design codes as shown in Table 1. The higher occupancy category corresponds to critical and lifeline buildings. The moderate occupancy category corresponds to business continuity type buildings. All other buildings are included as a lower occupancy category. Table 1 presents the values of *IF* recommended in the major international design codes. It can be seen from this table that in higher occupancy category buildings, *IF*s increase the design horizontal seismic base shear up to a range of 50-100%.

The selection of the seismic design load level takes the safety and the economic aspects into account which balance the benefit and the cost for the structural lifecycle. The research on the optimum seismic design level for buildings is not new and has been reported by various studies (Rosenblueth 1976, 1987; Kanda and Ellingwood 1991; Rosenblueth and Jara 1991; Ang and De Leon 1997; Rackwitz 2000; Kang and Wen 2000; Ellingwood 2001, Esteva *et al*. 2002, 2012; Ellingwood and Wen 2005; Goda and Hong 2006). Ang and De Leon (1997) proposed a systematic approach for formulating risk-based, cost-effective criteria for the design of structures. Target reliabilities (or acceptable risks) for damage control and life safety performance levels are determined on the basis of minimum expected life-cycle cost, and which develops the risk-consistent criteria for the structural design. Hong *et al*. (2006) suggested an optimum design approach for buried pipelines which can maintain a balance between the cost and benefits. Several studies (Rosenblueth 1976; Ellingwood and Wen 2005; Garcia-Perez *et al*. 2005; Pozoz-Estrada *et al.* 2016) have investigated the economic efficiency of a class of structures on increasing the level of safety and concluded that, the design codes use an *IF* to cope with the extraordinary economic loss or a loss of irreplaceable buildings.

Previous literature (Zahid *et al*. 2013; Ergun *et al.* 2015; Allahvirdizadeh *et al.* 2017; Bojorquez *et al*. 2017; Mosleh *et al.* 2017; Loulelis 2017) have employed deterministic approaches for the performance evaluation of buildings subjected to seismic excitations. But a probabilistic approach for the better estimate of the performance is always profound because the deterministic approach does not take account of the uncertainties associated with earthquake load and structural capacity. The seismic performance of the buildings cannot be estimated accurately by ignoring the associated uncertainties, and sometimes it also cannot be guaranteed that considering such uncertainties will improve the performance of the buildings as a decrease in damage index may not correspond to the decrease in the exceedance probability and the estimated loss. So considering the uncertainties of sensitive random variables, fragility assessment can be a reliable solution to evaluate the efficiency of *IF* in the seismic design. Many studies (Hwang *et al*. 1990; Barron *et al*. 2001; Ellingwood 2001; Cornell *et al.* 2002; Ramamoorthy *et al*. 2006; Rajeev and Tesfamariam 2012; Bakhshi and Ashadi 2013) have employed fragility curves for the evaluation of seismic excitations of buildings using random parameters and, among these random parameters *IF* plays a major role in controlling the damage index of the building.

RESEARCH SIGNIFICANCE

Studies on the variation of the performance of the structure with major design parameter like global ductility factor are available in the literature. However, the information on seismic performance of the structure with the change in the design parameter such as *IF* is very limited. The present study focuses on the change in the level of safety and the associated cost of the building with the change in *IF*. The seismic performance of the buildings with the variations of *IF* values are evaluated in terms of seismic fragility curves and mean annual probability of collapse.

SAFETY ASSESSMENT METHODOLOGY

A joint venture research committee, which was formed by four agencies namely the Structural Engineers Association of California (SEAOC), the Applied Technology Council (ATC), the Consortium of Universities for Research in Earthquake Engineering (CUREE), and the Federal Emergency Management Agency (FEMA), is combinedly abbreviated as the SAC-FEMA and proposed the popularly known SAC-FEMA method (Cornell *et al.* 2002). It is used in the present study for the probabilistic seismic safety assessment that characterizes the randomness and uncertainty both in seismic demand and capacity of the building. The SAC-FEMA method is formulated as a closed-form expression to analytically estimate the value of the risk integral convolving seismic hazard and structural response. This method is widely used by several researchers (Cornell *et al*. 2002; Ramamoorthy *et al*. 2006; Ellingwood *et al*. 2007; Wu *et al*. 2009; Celik and Ellingwood 2009, 2010; Davis *et al*. 2010; Rajeev and Tesfamariam, 2012, Haran *et al*. 2015, 2016; Bhosale *et al*. 2017, 2018; Dhir *et al*. 2018; Sahu *et al*. 2019) for the evaluation of seismic risks. An incremental dynamic analysis (*IDA*) framework introduced by previous researchers (Vamvatsikos and Cornell 2002; Dolsek 2009; Vamvatsikos and Fragiadakis 2010; Ferracuti *et al.* 2009; Azarbakht and Dolšek 2011; Brunesi *et al.* 2015; Kiani and Khanmohammadi 2015) is adopted in this study to develop the probabilistic seismic demand models (PSDMs) and fragility curves. *IDA* involves subjecting a building model to one or more ground motion records, of which each scaled to multiple levels of intensity, and plotting a response parameter as a function of intensity level.

PSDM and Fragility Curves

A fragility function represents the probability of exceedance of the seismic demand (*D*) for a selected performance level (*C*) at a specific intensity measure, characterized here by the level of peak ground acceleration (*PGA*). This can be obtained for each of the damage state and can be expressed in a closed-form equation (Celik and Ellingwood 2010) as follows:

$$f(PGA) = \mathrm{P}\ (\mathrm{C\text{-}D}\ \leq\ 0|\mathrm{PGA})\ = \phi\left(\frac{\ln\left(\hat{D}/\hat{C}\right)}{\sqrt{\beta_{D|PGA}^2 + \beta_c{}^2 + \beta_m{}^2}}\right) \tag{1}$$

Where ϕ is the standardized Gaussian distribution function, $\hat{D}$ is the median drift demand, $\hat{C}$ is the median drift limit state defining the capacity of the structure at selected performance levels, $\beta_{D|PGA}$ is the dispersion in drift demand at a given PGA level, β_c is the dispersion in capacity, and β_m is the dispersion in modelling. A series of nonlinear time history analysis is carried out to obtain the probabilistic representation of demand parameter. An analytical approximation of this representation is considered (Cornell *et al*. 2002) that says, at a given level of the *PGA*, the predicted median drift demand ($\hat{D}$) can be represented approximately by the form:

$$\hat{D} = a(PGA)^b \tag{2}$$

Where *a* and *b* are the constant coefficients. The drift demands (*D*) are assumed to be distributed log-normally about the median (Shome and Cornell 1999) with a standard deviation $\beta_{D|PGA}$ (the dispersion of the drift, *D* considers the natural logarithm at a given *PGA* level) for the considered frame. The three parameters, *a*, *b*, and $\beta_{D|PGA}$ are obtained by performing a regression analysis of nonlinear building response. The power-law relationship presented in Eq. (2) represents the PSDM for the corresponding frame. Eq. (1) can be re-written using this PSDM as follows:

$$f(PGA) = \mathrm{P}\ (\text{C-D} \le 0|\text{PGA}) = 1 - \phi\left(\frac{\ln\left(\hat{C}\right) - \ln\left(a.\left(PGA\right)^b\right)}{\sqrt{\beta_{D|PGA}^2 + \beta_c{}^2 + \beta_m{}^2}}\right) \tag{3}$$

The median inter-storey drift limit states ($\hat{C}$) for RC moment-resisting frame suggested by Haran *et al*. (2015, 2016) for various performance levels as 1%, 2% and 4% for immediate occupancy (IO), life safety (LS), and collapse prevention (CP) performance level respectively are considered in this present study. The value of β_c depends on the building type and construction quality, and it has been assumed as 0.25, according to ATC 58 (ATC 2012) in the present study.

Mean Annual Probability of Exceedance

In order to study the actual effect of the *IF* on the seismic safety of a building, it is important to consider the hazard data on that particular selected site. Seismic hazard function, $H(PGA)$, which, gives the annual probability of occurrence of the earthquake at any given site and the probabilities of the buildings exceeding any performance level is achieved by combining the probabilistic representations of the three elements in two steps. The first step couples the hazard function $H(PGA)$ and drift demand function in terms of PSDM to produce a drift hazard curve $H_D(d)$ and $H_D(d)$ provides the annual probability that the drift demand (D) exceeds any specified drift value (d). The second step combines this curve with the drift capacity (C) to produce P_{PL} which is defined as the annual probability of the performance level not being met.

Using the total probability theorem (Benjamin and Cornell 2014) $H_D(d)$ can be written as:

$$H_D(d) = \int P[D \geq d \,|PGA = x_i]\,|dH(x)| \tag{4}$$

Where $dH(x)$ can be obtained from a standard hazard curve, $H(PGA)$. The hazard curve is assumed to be a second-order polynomial in log-space in the region of interest (Vamvatsikos 2013) as follows:

$$H\left(PGA\right) = k_0 \exp\left[-k_2 \ln^2(PGA) - k_1 \ln(PGA)\right] \tag{5}$$

Where k_0, k_1, and k_2 are the constant coefficients. Using Eq. (2) and the log normality assumption of drift demand, the first factor of Eq. (4) can be written as:

$$P[D \geq d \,|PGA = x_i] = 1 - \Phi\left(\ln\left[d / ax^b\right] \big/ \beta_{D|PGA}\right) \tag{6}$$

Using Eq. (5) and Eq. (6), Eq. (4) for the drift hazard curve can be written in a simplified form as

$$H_D(d) = \sqrt{q}\, k_0^{1-q} \left[H\left(PGA^d\right)\right]^q exp\left[\frac{k_1^{\,2}}{4k_2}(1-q)\right] \tag{7a}$$

$$q = \frac{1}{1 + 2k_2 \beta_{d/IM}^2 \,/\, b^2} \tag{7b}$$

PGA^d is the peak ground acceleration corresponding to the drift demand level, d i.e.

$$PGA^d = \left(\frac{d}{a}\right)^{\frac{1}{b}} \tag{8}$$

Where 'a' and 'b' are the regression coefficients of the corresponding PSDM. The detailed derivation of Eq. (7) is available in Vamvatsikos (2013). Using the total probability theorem, the annual probability of unacceptable performance (P_{PL}) can be defined as:

$$P_{PL} = P[C \le D] = \sum_{all\,d_i} P[C \le D | D = d_i] P[D = d_i] \tag{9}$$

The second factor, in the above equation, represents the likelihood of a given drift demand level $P[D = d]$ which can be determined from the drift hazard curve derived in Eq. (4). Now, Eq. (9) can be represented in a continuous form as follows:

$$P_{PL} = \int P[C \le d] \left| dH_D(d) \right| \tag{10}$$

In order to facilitate an analytical treatment of the rather highly complicated problem of considering uncertainties in the capacity and seismic demand of an RC structure, SAC-FEMA method has introduced various simplifications. The first approximation is the power-law formulation of the PSDM where the engineering damage parameter is approximated to have a linear relation to the seismic intensity measure in log space (Eq. 2). In addition, the engineering demand parameter is assumed to be log-normally distributed about the median. However, the previous literature (Dhir *et al*. 2018) has reported that power-law assumption and log normality assumption of drift demand in RC framed building is in agreement with the results of Monte Carlo simulation. The other important approximation in the original SAC-FEMA method is the first- order power- law fit of the hazard curve, which has later been replaced with second-order hazard approximation (Eq. 5) by Vamvatsikos (2013).

FRAMES CONSIDERED

Typical RC regular bare frames of two, four, six and eight-storied buildings with a uniform storey height of 3.2 m and uniform bay width of 5 m are selected for the present study (Fig. 1). These buildings are assumed to be located in Guwahati (India) lying in the seismic zone V (PGA of 0.36 g) as per IS 1893 (BIS 2016b) and detailed as per IS 13920 (BIS 2016a), considering medium soil conditions (N-value in the range 10-30). The characteristic strength of concrete and steel is taken as 25 MPa and 415 MPa respectively. The buildings are assumed to be symmetric in plan and elevation. The dead load of the slab, including floor finishes, is taken as 3.75 kN/m^2 and live load as 3 kN/m^2. The self-weight of the partition walls (230 mm) is applied separately as the uniformly distributed load on the respective beams. As the focus of the present study is to evaluate the influence of *IF* on the fragility of the structures, a wide range of *IF*s has been studied for four different frame geometries. Selected building frames are designed as per relevant Indian Standards that satisfy the requirement of both ultimate and serviceability limit states. However, the design is found to be governed by the criteria of the ultimate limit state. The design details of the building frames of two storey, four storey, six storey and eight storey are shown in Table 2, 3, 4 and 5 respectively. The notations used *n*S*m*B-*p* to represent a frame with *n* storeys and *m* bays, designed considering an *IF* value of *p*. For example, 2S2B-1 represents a 2-storey, 2-bay frame designed with *IF* value of 1.0. The design base shear (V_b) calculated as per equivalent static method (BIS 2016b) is presented in Table 6.

STRUCTURAL MODELLING AND ANALYSIS

Selected buildings frames are modelled, and nonlinear time history analysis was performed during the seismic risk assessment. The Open System for Earthquake Engineering Simulation (OpenSEES) Laboratory tool developed by McKenna *et al.* (2014) was used for the present analysis. A force-based nonlinear beam-column fiber element (Lee and Mosalam 2004) that considers the spread plasticity approach along the element was adopted for modelling the beams and columns. Five integration points are considered for fiber elements as per Kunnath (2007). Core and cover concrete are modelled as confined and unconfined concrete respectively as per Kent and Park (1971). Giuffre-Menegotto-Pinto steel material model (Filippou *et al.* 1983) has been used for the modelling of steel reinforcing bars.

In the current study, a lumped mass approach is taken into consideration, in which all the permanent weights that move with the structure is lumped at the suitable nodes. It comprises of all the dead loads and a part of the live load (25%) which are expected to be present in the structure during the ground shaking. The in-plane stiffness of the floor is modelled using rigid diaphragm constraint. Mass and stiffness proportional Raleigh damping model is used for dynamic analysis as per Filippou *et al*. (1992). As the number of available earthquake records in the Indian region is limited, a suite of 44 ground motions (22 pairs) from other regions collected from Haselton *et al*. (2012) are used in the present study for representing the uncertainty in the earthquake loading. These ground motions are converted to match with the design spectrum of Indian Standard IS 1893 (BIS 2016b), using a computer program (Mukherjee and Gupta 2002). These modified spectrum consistent ground motion records are used for the nonlinear dynamic analyses (NLDA). Uncertainties associated with the structural capacity are considered through concrete compressive strength and the yield strength of reinforcing steel. The uncertainty, in the global damping ratio, is additionally considered in the analysis. Table 7 presents the details of the uncorrelated random variables which have been explored in the present study. Further details of the computational modelling can be available in Dhir *et al.* (2018).

PROBABILISTIC SEISMIC DEMAND MODEL (PSDM)

The earthquake ground motions are linearly scaled from 0.1 g to 1.0 g and each computational model (44) is analysed for a randomly selected earthquake. The maximum inter-storey drift (*ISD*) is plotted with the corresponding *PGA* on a logarithmic graph for two storey, four storey, six storey and eight storey frames in Fig. 2, 3, 4, and 5 respectively. Using regression analysis, a power law (refer to Eq. 2) relationship is fitted for each frame which represents the PSDM for the corresponding frames. The higher is the value of inter-storey drifts, the higher will be the vulnerability of the building. The regression coefficients, *a* and *b*, of the PSDMs found in each frame are reported in Table 8.

As observed in all the PSDM plots, for a given *PGA*, the maximum *ISD* demand decreases with the increase in *IF*. This can be attributed to the improved structural capacity of the building frames in terms of stiffness due to the increase in *IF*. The percentage decrease in the maximum *ISD* demand of building frames with *IF* values of 1.2, 1.4, 1.5, 1.6, 1.8 and 2.0 with respect to the frame with *IF* = 1 is calculated at a typical PGA of 0.3 g and presented in Fig. 6. This figure shows that, in general, the reduction in *ISD* demand is higher for the higher *IF*. The reduction can be due to the increase in the cross-section of the members and the change in design.

COMPARISON OF FRAGILITY CURVES

Fragility curves were developed for all the selected building frames and the effect of *IF*s on the fragility function was studied at three performance limit states (IO, LS, and CP) as presented in Figs. 7, 8, 9 and 10. The values of the constants *a* and *b*, and the dispersions of *ISD* demand, $\beta_{D/IM}$ are calculated from the corresponding PSDM. The probability of exceedance is found to decrease with the increase in *IF*. It can be seen from the figures that, the building frames designed with *IF* = 2 have the least probability of exceeding the limit state among all the selected frames. The probability of exceedance for the building frames designed with *IF* of 1.2, 1.4, 1.6, 1.8, and 2.0 normalized to that of building frame with *IF* = 1 for a typical *PGA* of

0.3 g is presented in Table 9. It can be seen from the table that the normalised probability of exceedance reduces with the increase of *IF*. This trend is similar to all the four selected categories of buildings.

MEAN ANNUAL PROBABILITY OF EXCEEDANCE

The selected building frames are designed for the limit state of collapse against design basis earthquake (approximately 10% probability of occurrence in 50 years) for a site located in the seismic zone V (Guwahati) as per IS 1893 (BIS 2016b). The hazard curve of Guwahati (Fig. 11) prepared using data collected from the published literature (Iyengar *et al*. 2010) is considered to calculate the seismic performance of the building frames in terms of mean annual probability of exceedance (using Eq. 10). Table 10 presents the values of the annual probability of the collapse (P_{PL}) of all the selected frames at each performance limit state. It can be seen that the annual probability of collapse decreases as the *IF* increases. This implies that the frames designed with a higher *IF* have greater seismic safety.

EFFECT OF *IF* ON DAMAGE AND COST INDICES

It is clear from the previous discussion that, on the one hand, the building performances can be significantly improved by a higher value of *IF* in the design. On the other hand, a higher value of *IF* in the design leads to the increased cost of the building. This section investigates the effect of *IF* on the structural performance and the associated costs of the building. A parameter called Damage Index (*DI*) is introduced and defined as follows:

$$DI = \frac{P_{PL,IF}}{P_{PL,IF=1}} \tag{11}$$

Where, $P_{PL,IF=1}$ is the mean annual probability of exceedance of building designed with *IF* = 1 and $P_{PL,IF}$ is the mean annual probability of exceedance of a building designed with *IF* greater than unity. *DI* is expected to be less than unity for buildings designed with *IF* greater than one. Similarly, another parameter, Cost Index (*CI*) can be expressed as follows:

$$CI = \frac{Cost_{IF}}{Cost_{IF=1}} \tag{12}$$

Where, $Cost_{IF}$ and $Cost_{IF=1}$ are the cost of construction for the building under consideration (with an *IF* greater than unity) and a similar building designed with *IF* = 1, respectively. However, it is to be noted that the cost index as defined in Eq. (12) does not consider the damage-cost and the risk aversion.

Fig. 12 and Fig. 13 present the cost index and damage index of the selected building as a function of *IF* respectively. It can be seen from these figures that *IF* has a linear relationship between the damage and cost index. The higher the value of *IF*, the higher is the cost of the building, but the lower will be the expected damage.

SUMMARY AND CONCLUSIONS

The international codes and standards have recommended different values of occupancy importance factors for the seismic design of buildings considering an acceptable performance and affordable cost. This study investigates the effect of the importance factor in the seismic performance of buildings in a probabilistic framework. The SAC-FEMA method of structural safety assessment with second-order hazard approximation (Vamvatsikos 2013) is used to evaluate the influence of the importance factor on the probabilistic demand model, fragility function and the annual probability of collapse. The performance of the building, in general, is found to be improving with the increase in the importance factor. The importance factor is also found to have a proportional linear relationship with the cost index and an inversely proportional linear relationship with the damage index of the buildings. Salient conclusions of this study are listed as follows.

- When the *IF* increases from 1.0 to 1.5, the damage index decreases by 25-50% and increase in the *IF* from 1.5 to 2.0 the damage index further decreases by 25-35%. An overall decrease in damage index by increasing the *IF* from 1.0 to 2.0 is around 50-60%.
- As the importance factor increases, the failure probability decreases, and cost increases. Design codes may also introduce an indicator of the percentage increase in cost and the percentage decrease in failure probability (increase in reliability) for the respective importance factor values for each class of buildings.

DATA AVAILABILITY STATEMENT

Some data, models, or code generated or used during the study are available from the corresponding author by request (OpenSEES modelling files and generated outputs)

REFERENCES

Allahvirdizadeh, R., Khanmohammadi, M., & Marefat, M. S. (2017). Probabilistic comparative investigation on introduced performance-based seismic design and assessment criteria. *Engineering Structures*, *151*, 206-220.

Ang, A. S., & De Leon, D. (1997). Determination of optimal target reliabilities for design and upgrading of structures. *Structural Safety*, *19*(1), 91-103.

ASCE (*American Society of Civil Engineers),* 2010. Minimum design loads for buildings and other structures. ASCE/SEI 7-10. Reston, *VA.*

ATC (*Applied Technology Council),* 2012. Guidelines for seismic performance assessment of buildings, ATC 58. Redwood City*, CA.*

Azarbakht, A., & Dolšek, M. (2010). Progressive incremental dynamic analysis for first-mode dominated structures. *Journal of Structural Engineering*, *137*(3), 445-455.

Bakhshi, A., & Asadi, P. (2013). Probabilistic evaluation of seismic design parameters of RC frames based on fragility curves. *Scientia Iranica*, *20*(2), 231-241.

Barron, R., Reinhorn, A. and Ayala, A. (2001). Spectral evaluation of seismic fragility of structures. *Technical Report MCEER, Multidisciplinary Center for Earthquake Engineering Research, Buffalo,* NY

Benjamin, J. R., & Cornell, C. A. (2014). *Probability, statistics, and decision for civil engineers*. Courier Corporation.

Bhosale, A. S., Davis, R., & Sarkar, P. (2017). Vertical irregularity of buildings: Regularity index versus seismic risk. *ASCE-ASME Journal of Risk and Uncertainty in Engineering Systems, Part A: Civil Engineering*, *3*(3), 04017001.

Bhosale, A. S., Davis, R., & Sarkar, P. (2018). Seismic Safety of Vertically Irregular Buildings: Performance of Existing Indicators. *Journal of Architectural Engineering*, *24*(3), 04018013.

Billah, A. M., & Alam, M. S. (2014). Seismic performance evaluation of multi-column bridge bents retrofitted with different alternatives using incremental dynamic analysis. *Engineering Structures*, *62,* 105-117.

BIS (Bureau of Indian Standards), 2016a. *Ductile Detailing of Reinforced Concrete Structures Subjected to Seismic Forces – Code of Practice*, *IS 13920-16*, New Delhi, India.

BIS (Bureau of Indian Standards), 2016b. *Indian Standard Criteria for Earthquake Resistant Design of Structures*. *IS 1893-16*, New Delhi, India.

Bojórquez, J., Ruiz, S. E., Ellingwood, B., Reyes-Salazar, A., & Bojórquez, E. (2017). Reliability-based optimal load factors for seismic design of buildings. *Engineering Structures*, *151*, 527-539.

Brunesi, E., Nascimbene, R., Parisi, F., & Augenti, N. (2015). Progressive collapse fragility of reinforced concrete framed structures through incremental dynamic analysis. *Engineering Structures*, *104*, 65-79.

Celik, O. C., & Ellingwood, B. R. (2009). Seismic risk assessment of gravity load designed reinforced concrete frames subjected to Mid-America ground motions. *Journal of structural engineering*, *135*(4), 414-424.

Celik, O. C., & Ellingwood, B. R. (2010). Seismic fragilities for non-ductile reinforced concrete frames–Role of aleatoric and epistemic uncertainties. *Structural Safety*, *32*(1), 1-12.

Code, G. S. (2000). EAK 2000. *Technical Chamber of Greece, Greek Government Journal—FEK*, *2184*.

Code, I. S. (2005). Iranian code of practice for seismic resistant design of buildings. *Standard*.

Cornell, C. A., Jalayer, F., Hamburger, R. O., & Foutch, D. A. (2001). The probabilistic basis for the 2000 SAC/FEMA steel moment frame guidelines. Submitted to. *J. Struct. Engrg*.

Davis, P. R., Padhy, K. T., Menon, D., & Prasad, A. M. (2010). Seismic fragility of open ground storey buildings in India. In *9th US National and 10th Canadian Conference on Earthquake Engineering*.

Dhir, P. K., Davis, R., & Sarkar, P. (2018). Safety Assessment of Gravity Load–Designed Reinforced Concrete–Framed Buildings. *ASCE-ASME Journal of Risk and Uncertainty in Engineering Systems, Part A: Civil Engineering*, *4*(2), 04018004.

Dolsek, M. (2009). Incremental dynamic analysis with consideration of modeling uncertainties. *Earthquake Engineering & Structural Dynamics*, *38*(6), 805-825.

dos Santos, K. R., Kougioumtzoglou, I. A., & Beck, A. T. (2016). Incremental Dynamic Analysis: A Nonlinear Stochastic Dynamics Perspective. *Journal of Engineering Mechanics, 142*(10), 06016007.

Ellingwood BR, Celik OC, & Kinali K. (2007). Fragility assessment of building structural systems in Mid- America. *Earthquake Engineering & Structural Dynamics*, *36*(13), 1935-1952.

Ellingwood, B. R. (2001). Earthquake risk assessment of building structures. *Reliability Engineering & System Safety*, *74*(3), 251-262.

Ellingwood, B. R., & Wen, Y. K. (2005). Risk- benefit- based design decisions for low-probability/high consequence earthquake events in Mid- America. *Progress in Structural Engineering and Materials*, *7*(2), 56-70.

Ergün, A., Kıraç, N., & Başaran, V. (2015). The evaluation of structural properties of reinforced concrete building designed according to pre-modern code considering seismic performance. *Engineering Failure Analysis*, *58*, 184-191.

Esteva, L., Campos, D., & Diaz-Lopez, O. (2011). Life-cycle optimisation in earthquake engineering. *Structure and Infrastructure Engineering*, *7*(1-2), 33-49.

Esteva, L., D1az-Lopez, O., Garcıa-Perez, J., Sierra, G., & Ismael, E. (2002). Life-cycle optimization in the establishment of performance-acceptance parameters for seismic design. *Structural Safety*, *24*(2-4), 187-204.

Eurocode, C. E. N. (2004). 8: Design of structures for earthquake resistance. *Part*, *1*, 1998-1.

Ferracuti, B., Pinho, R., Savoia, M., & Francia, R. (2009). Verification of displacement-based adaptive pushover through multi-ground motion incremental dynamic analyses. *Engineering Structures, 31*(8), 1789-1799.

Filippou, F. C., Bertero, V. V., & Popov, E. P. (1983). Effects of bond deterioration on hysteretic behavior of reinforced concrete joints.

Filippou, F. C., d'Ambrisi, A., & Issa, A. (1992). Nonlinear static and dynamic analysis of RC subassemblages. *Rep. No. UCB/EERC-92*, *8*.

García-Pérez, J., Castellanos, F., & Díaz, O. (2005). Occupancy importance factor in earthquake engineering. *Engineering structures*, *27*(11), 1625-1632.

Goda, K., & Hong, H. P. (2006). Optimal seismic design considering risk attitude, societal tolerable risk level, and life quality criterion. *Journal of Structural Engineering*, *132*(12), 2027-2035.

Haran, P. D., Davis, R., & Sarkar, P. (2015). Reliability evaluation of RC frame by two major fragility analysis methods, 47-66.

Haselton, C. B., Whittaker, A. S., Hortacsu, A., Baker, J. W., Bray, J., & Grant, D. N. (2012). Selecting and scaling earthquake ground motions for performing response-history analyses. In *Proceedings of the*

15th World Conference on Earthquake Engineering (pp. 4207-4217). Earthquake Engineering Research Institute.

Hong, H. P., Allouche, E. N., & Trivedi, M. (2006). Optimal scheduling of replacement and rehabilitation of water distribution systems. *Journal of Infrastructure Systems*, *12*(3), 184-191.

Hwang, H. H., & Huo, J. R. (1994). Generation of hazard-consistent fragility curves. *Soil Dynamics and Earthquake Engineering*, *13*(5), 345-354.

Iranian code of practice for seismic resistant design of buildings, 2007. Standard no. 2800. 3rd edition. Building and Housing Research Center.

Kanda, J., & Ellingwood, B. (1991). Formulation of load factors based on optimum reliability. *Structural safety*, *9*(3), 197-210.

Kang, Y. J., & Wen, Y. K. (2000). *Minimum life-cycle cost structural design against natural hazards*. University of Illinois Engineering Experiment Station. College of Engineering. University of Illinois at Urbana-Champaign.

Kent, D. C., & Park, R. (1971). Flexural members with confined concrete. *Journal of the Structural Division*, *97*(7), 1969-1990.

Kiani, J., & Khanmohammadi, M. (2015). New approach for selection of real input ground motion records for incremental dynamic analysis (IDA). *Journal of Earthquake Engineering*, *19*(4), 592-623.

Kunnath, S. K. (2006). *Application of the PEER PBEE Methodology to the I-880 Viaduct: I-880 Testbed Committee*. Pacific Earthquake Engineering Research (PEER) Center, College of Engineering, University of California.

Lee, T. H., & Mosalam, K. M. (2004). Probabilistic fiber element modeling of reinforced concrete structures. *Computers & structures*, *82*(27), 2285-2299.

Loulelis, D. G., Papagiannopoulos, G. A., & Beskos, D. E. (2018). Modal strength reduction factors for seismic design of steel moment resisting frames. *Engineering Structures*, *154*, 23-37.

McKenna, F., Fenves, G. L., & Filippou, F. C. (2010). OpenSees. *University of California, Berkeley: nd*.

Mosleh, A., Rodrigues, H., Varum, H., Costa, A., & Arêde, A. (2016). Seismic behavior of RC building structures designed according to current codes. In *Structures*, *7*,1-13.

Mukherjee, S., & Gupta, V. K. (2002). Wavelet-based generation of spectrum-compatible time-histories. *Soil Dynamics and Earthquake Engineering*, *22*(9-12), 799-804.

NBC (Nepal National Building Code) -105, 1994). Seismic Design of Buildings in Nepal.

NZS (New Zealand Standard) 1170.5. (2004). Structural Design Actions, Part 5: Earthquake Actions—New Zealand.

Pozos-Estrada, A., Liu, T. J., Gomez, R., & Hong, H. P. (2016). Seismic design and importance factor: Benefit/cost for overall service time versus per unit service time. *Structural Safety*, *58*, 40-51.

Pragalath, D. H., Avadhoot, B., Robin, D. P., & Pradip, S. (2016). Multiplication factor for open ground storey buildings–a reliability based evaluation. *Earthquake Engineering and Engineering Vibration*, *15*(2), 283-295.

Rackwitz, R. (2000). Optimization—the basis of code-making and reliability verification. *Structural safety*, *22*(1), 27-60.

Rajeev, P., & Tesfamariam, S. (2012). Seismic fragilities for reinforced concrete buildings with consideration of irregularities. *Structural Safety*, *39*, 1-13.

Ramamoorthy, S. K., Gardoni, P., & Bracci, J. M. (2006). Probabilistic demand models and fragility curves for reinforced concrete frames. *Journal of Structural Engineering*, *132*(10), 1563-1572.

Ranganathan, R. (1999). *Structural reliability analysis and design*. Jaico Publishing House.

Rosenblueth, E. (1976). Optimum design for infrequent disturbances. *Journal of the Structural Division*, *102*(9), 1807-1825.

Rosenblueth, E. (1987). What should we do with structural reliabilities. In *Proc.*, *ICASP*, *5*(1), 24-34.

Rosenblueth, E., & Jara, J. M. (1991). design coefficients. Constant versus time dependent seismic In *Reliability and Optimization of Structural Systems' 90* (pp. 315-327). Springer, Berlin, Heidelberg.

Sahu, D., Nishanth, M., Dhir, P. K., Sarkar, P., Davis, R., & Mangalathu, S. (2019). Stochastic response of reinforced concrete buildings using high dimensional model representation. *Engineering Structures*, *179*, 412-422.

Shome, N. (1999). *Probabilistic seismic demand analysis of nonlinear structures*. Stanford University.

Vamvatsikos, D. (2013). Derivation of new SAC/FEMA performance evaluation solutions with second-order hazard approximation. *Earthquake Engineering & Structural Dynamics*, *42*(8), 1171-1188.

Vamvatsikos, D., & Cornell, C. A. (2002). Incremental dynamic analysis. *Earthquake Engineering & Structural Dynamics*, *31*(3), 491-514.

Vamvatsikos, D., & Fragiadakis, M. (2010). Incremental dynamic analysis for estimating seismic performance sensitivity and uncertainty. *Earthquake engineering & structural dynamics*, *39*(2), 141-163.

Wu, D., Tesfamariam, S., Stiemer, S. F., & Cui, J. (2013). Comparison of seismic performance of a RC frame building before and after the Wenchuan earthquake in Sichuan province. *Journal of performance of constructed facilities*, *29*(1), 04014038.

Zahid, M. Z. A. M., Robert, D., & Shahrin, F. (2013). An evaluation of overstrength factor of seismic designed low rise RC buildings. *Procedia Engineering*, *53*, 48-51.

Table 1: *IF* suggested by various international codes

Seismic Code	Occupancy Category		
	Higher	Moderate	Lower
IS 1893- Part1 (2016)	1.5	1.2	1.0
Canadian Building code Act, 1992	1.5	-	1.0
BS EN 1998-1:2004	1.5	-	0.8
SEI/ASCE 7-16	1.5	1.25	1.0
NZS 1170 Part 5: 2004	1.3	-	0.6
NBC 105 : 1994	2.0	1.5	1.0
EAK 2000	1.3	-	0.85
Iranian Standard 2800 (2007)	1.2/1.4	1.0	0.8

Table 2: Design details of two storey frames

Frame ID	Member	Floor/ Storey No.	Width (mm)	Depth (mm)	Longitudinal Reinforcement		Transverse reinforcement
					Top	Bottom	
2S2B-1	Beam	1	300	300	6-16φ	4-16φ	8φ @100 c/c
	Beam	2	300	300	5-16φ	3-16φ	8φ @100 c/c
	Column	1-2	380	380	8-20φ		8φ @220 c/c
2S2B-1.2	Beam	1	300	300	5-16φ	4-16φ	8φ @100 c/c
	Beam	2	300	300	6-16φ	4-16φ	8φ @100 c/c
	Column	1-2	400	400	8-20φ		8φ @220 c/c
2S2B-1.4	Beam	1	300	300	5-16φ	4-16φ	8φ @100 c/c
	Beam	2	300	300	4-16φ	3-16φ	8φ @100 c/c
	Column	1-2	450	450	10-20φ		8φ @200 c/c
2S2B-1.5	Beam	1	300	300	7-16φ	4-16φ	8φ @100 c/c
	Beam	2	300	300	6-16φ	3-16φ	8φ @100 c/c
	Column	1-2	450	450	10-20φ		8φ @200 c/c
2S2B-1.6	Beam	1	330	330	8-16φ	4-16φ	8φ @100 c/c
	Beam	2	330	330	7-16φ	3-16φ	8φ @100 c/c
	Column	1-2	450	450	10-20φ		8φ @200 c/c
2S2B-1.8	Beam	1	330	330	9-16φ	4-16φ	8φ @100 c/c
	Beam	2	330	330	8-16φ	3-16φ	8φ @100 c/c
	Column	1-2	500	500	14-20φ		8φ @185 c/c
2S2B-2	Beam	1	350	350	9-16φ	4-16φ	8φ @100 c/c
	Beam	2	350	350	9-16φ	3-16φ	8φ @100 c/c
	Column	1-2	550	550	16-20φ		8φ @160 c/c

Table 3: Design details of four storey frames

Frame	Member	Floor/ Storey No.	Width (mm)	Depth (mm)	Longitudinal Reinforcement		Transverse reinforcement
					Top	Bottom	
4S2B-1	Beam	1	300	300	10-16φ	4-16φ	8φ @100 c/c
	Beam	2-4	300	300	7-16φ	3-16φ	8φ @100 c/c
	Column	1-4	400	400	10-20φ		8φ @180 c/c
4S2B-1.2	Beam	1	400	400	8-20φ	5-16φ	8φ @100 c/c
	Beam	2-4	400	400	5-20φ	3-16φ	8φ @100 c/c
	Column	1-4	500	500	14-20φ		8φ @150 c/c
4S2B-1.4	Beam	1	400	400	9-20φ	5-16φ	8φ @120 c/c
	Beam	2-4	400	400	6-20φ	3-16φ	8φ @120 c/c
	Column	1-4	500	500	16-20φ		8φ @110 c/c
4S2B-1.5	Beam	1	450	450	10-20φ	6-16φ	8φ @100 c/c
	Beam	2-4	450	450	5-20φ	3-16φ	8φ @100 c/c
	Column	1-4	500	500	18-20φ		8φ @200 c/c
4S2B-1.6	Beam	1	450	450	12-20φ	7-16φ	8φ @100 c/c
	Beam	2-4	450	450	7-20φ	4-16φ	8φ @100 c/c
	Column	1-4	550	550	18-20φ		8φ @200 c/c
4S2B-1.8	Beam	1	450	450	13-20φ	8-16φ	8φ @100 c/c
	Beam	2-4	450	450	8-20φ	5-16φ	8φ @100 c/c
	Column	1-4	600	600	24-16φ		8φ @180 c/c
4S2B-2	Beam	1	500	500	5-20φ	9-16φ	8φ @100 c/c
	Beam	2-4	500	500	9-20φ	5-16φ	8φ @100 c/c
	Column	1-4	600	600	26-20φ		8φ @160 c/c

Table 4: Design details of six storey frames

Frame	Member	Floor/ Storey No.	Width (mm)	Depth (mm)	Longitudinal Reinforcement		Transverse reinforcement
					Top	Bottom	
6S2B-1	Beam	1	400	400	11-20φ	7-16φ	8φ @100 c/c
	Beam	2-6	400	400	5-20φ	3-16φ	8φ @100 c/c
	Column	1-6	500	500	18-20φ		8φ @160 c/c
6S2B-1.2	Beam	1	400	400	12-20φ	8-16φ	10φ @100 c/c
	Beam	2-6	400	400	6-20φ	3-16φ	10φ @100 c/c
	Column	1-6	550	550	20-20φ		8φ @110 c/c
6S2B-1.4	Beam	1	400	400	15-20φ	10-16φ	10φ @100 c/c
	Beam	2-6	400	400	7-20φ	4-16φ	10φ @100 c/c
	Column	1-6	550	550	20-20φ		10φ @130 c/c
6S2B-1.5	Beam	1	450	450	11-25φ	7-20φ	10φ @100 c/c
	Beam	2-6	450	450	7-20φ	4-16φ	10φ @100 c/c
	Column	1-6	550	550	22-20φ		10φ @150 c/c
6S2B-1.6	Beam	1	450	450	12-25φ	6-20φ	10φ @100 c/c
	Beam	2-4	450	450	9-20φ	4-16φ	10φ @100 c/c
	Column	1-4	550	550	16-25φ		10φ @130 c/c
6S2B-1.8	Beam	1	450	450	14-25φ	9-20φ	10φ @100 c/c
	Beam	2-6	450	450	5-25φ	3-16φ	10φ @100 c/c
	Column	1-6	600	600	24-25φ		10φ @150 c/c
6S2B-2	Beam	1	500	500	15-25φ	10-20φ	12φ @110 c/c
	Beam	2-4	500	500	6-25φ	4-16φ	12φ @110 c/c
	Column	1-4	600	600	24-25φ		12φ @160 c/c

Table 5: Design details of eight storey frames

Frame	Member	Floor/ Storey No.	Width (mm)	Depth (mm)	Longitudinal Reinforcement		Transverse reinforcement
					Top	Bottom	
8S4B-1	Beam	1	400	400	11-12φ	3-16φ	8φ @100 c/c
	Beam	2-8	400	400	4-16φ	3-16φ	8φ @100 c/c
	Column	1-8	600	600	24-32φ		8φ @160 c/c
8S4B-1.2	Beam	1	450	450	7-20φ	4-16φ	10φ @100 c/c
	Beam	2-8	450	450	4-16φ	3-16φ	10φ @100 c/c
	Column	1-8	600	600	28-32φ		8φ @110 c/c
8S4B-1.4	Beam	1	500	500	11-16φ	4-16φ	10φ @100 c/c
	Beam	2-8	500	500	5-16φ	3-16φ	10φ @100 c/c
	Column	1-8	800	800	32-32φ		10φ @130 c/c
8S4B-1.5	Beam	1	500	500	8-20φ	5-16φ	12φ @110 c/c
	Beam	2-8	500	500	6-20φ	2-20φ	12φ @110 c/c
	Column	1-8	850	850	40-32φ		12φ @100 c/c
8S4B-1.6	Beam	1	500	500	11-20φ	6-16φ	10φ @100 c/c
	Beam	2-8	500	500	4-20φ	4-16φ	10φ @100 c/c
	Column	1-8	900	900	42-32φ		10φ @150 c/c
8S4B-1.8	Beam	1	550	550	11-20φ	6-16φ	10φ @150 c/c
	Beam	2-8	550	550	4-20φ	4-16φ	10φ @150 c/c
	Column	1-8	900	900	24-25φ		10φ @100 c/c
8S4B-2	Beam	1	550	550	12-20φ	4-16φ	10φ @100 c/c
	Beam	2-8	550	550	5-20φ	4-16φ	10φ @100 c/c
	Column	1-8	900	900	48-32φ		10φ @180 c/c

Table 6: Design base shear (kN) for different *IF*

Building Frame (fundamental period)	Importance Factor						
	1.0	1.2	1.4	1.5	1.6	1.8	2.0
2S2B (0.301s)	76	92	109	127	143	180	229
4S2B (0.507s)	187	236	316	367	429	529	631
6S2B (0.688s)	295	416	503	594	651	682	1017
8S4B (0.854s)	301	411	532	595	659	810	1056

Table 7: Details of random variables used

Random variables	Mean	COV (%)	Probability Distribution	Source
Concrete compressive strength	33.66 MPa	21.0	Normal	Ranganathan (1999)
Steel yield strength	483.47 MPa	10.0	Normal	Ranganathan (1999)
Global damping ratio	5 %	76.0	Lognormal	Celik and Ellingwood (2009)

Table 8: Regression output from NLDA analysis for considered frame

Frame	PSDM	R^2	$\beta_{D\|PGA}$
2S2B-1.0	3.215 $(PGA)^{0.9206}$	0.8454	0.226
2S2B-1.2	3.259 $(PGA)^{0.9397}$	0.8839	0.206
2S2B-1.4	2.795 $(PGA)^{0.9575}$	0.8899	0.211
2S2B-1.5	2.616 $(PGA)^{0.9282}$	0.8503	0.244
2S2B-1.6	2.184 $(PGA)^{0.9295}$	0.8412	0.253
2S2B-1.8	1.850 $(PGA)^{0.9644}$	0.8460	0.258
2S2B-2.0	1.843 $(PGA)^{0.9583}$	0.8364	0.260
4S2B-1.0	3.306 $(PGA)^{0.7567}$	0.7766	0.254
4S2B-1.2	2.352 $(PGA)^{0.7421}$	0.7150	0.230
4S2B-1.4	2.338 $(PGA)^{0.7457}$	0.7962	0.236
4S2B-1.5	2.236 $(PGA)^{0.7396}$	0.8113	0.223
4S2B-1.6	1.939 $(PGA)^{0.8128}$	0.8533	0.211
4S2B-1.8	1.700 $(PGA)^{0.7627}$	0.8798	0.176
4S2B-2.0	1.512 $(PGA)^{0.8128}$	0.7608	0.241
6S2B-1.0	2.567 $(PGA)^{0.8109}$	0.8267	0.232
6S2B-1.2	2.284 $(PGA)^{0.8129}$	0.8442	0.219
6S2B-1.4	2.221 $(PGA)^{0.7999}$	0.8196	0.235
6S2B-1.5	2.217 $(PGA)^{0.7877}$	0.8304	0.223
6S2B-1.6	2.381 $(PGA)^{0.7786}$	0.8242	0.225
6S2B-1.8	2.327 $(PGA)^{0.7778}$	0.8386	0.214
6S2B-2.0	1.442 $(PGA)^{0.7565}$	0.8460	0.205
8S4B-1.0	3.869 $(PGA)^{0.8113}$	0.8003	0.254
8S4B-1.2	3.169 $(PGA)^{0.8240}$	0.8686	0.201
8S4B-1.4	3.022 $(PGA)^{0.8803}$	0.8855	0.198
8S4B-1.5	2.864 $(PGA)^{0.8614}$	0.8450	0.231
8S4B-1.6	2.351 $(PGA)^{0.8267}$	0.8811	0.190
8S4B-1.8	2.348 $(PGA)^{0.8720}$	0.8764	0.205
8S4B-2.0	2.240 $(PGA)^{0.8447}$	0.8820	0.193

Table 9: Normalized probability of exceedance of selected frames at typical *PGA* of 0.3g

IF	2S2B	4S2B	6S2B	8S4B
1	1	1	1	1
1.2	0.80	0.66	0.58	0.80
1.4	0.71	0.65	0.49	0.35
1.5	0.46	0.62	0.35	0.34
1.6	0.20	0.36	0.11	0.33
1.8	0.08	0.29	0.09	0.17
2	0.02	0.15	0.07	0.09

Table 10: Annual probability of collapse $\left(P_{PL}\right)$ for selected buildings

Frame	Annual probability of collapse (× 10^{-2})		
	IO	LS	CP
2S2B-1.0	0.924	0.224	0.041
2S2B-1.2	0.918	0.17	0.044
2S2B-1.4	0.666	0.16	0.035
2S2B-1.5	0.609	0.138	0.024
2S2B-1.6	0.422	0.089	0.015
2S2B-1.8	0.286	0.06	0.01
2S2B-2.0	0.285	0.059	0.01
4S2B-1.0	1.433	0.282	0.037
4S2B-1.2	0.711	0.112	0.012
4S2B-1.4	0.692	0.109	0.011
4S2B-1.5	0.636	0.096	0.01
4S2B-1.6	0.396	0.065	0.008
4S2B-1.8	0.315	0.043	0.004
4S2B-2.0	0.212	0.031	0.003
6S2B-1.0	0.743	0.139	0.019
6S2B-1.2	0.576	0.102	0.013
6S2B-1.4	0.549	0.094	0.011
6S2B-1.5	0.464	0.087	0.009
6S2B-1.6	0.276	0.075	0.007
6S2B-1.8	0.247	0.061	0.004
6S2B-2.0	0.201	0.025	0.002
8S4B-1.0	1.677	0.386	0.063
8S4B-1.2	1.135	0.241	0.037
8S4B-1.4	0.907	0.203	0.034
8S4B-1.5	0.837	0.179	0.028
8S4B-1.6	0.606	0.112	0.015
8S4B-1.8	0.551	0.11	0.016
8S4B-2.0	0.526	0.098	0.013

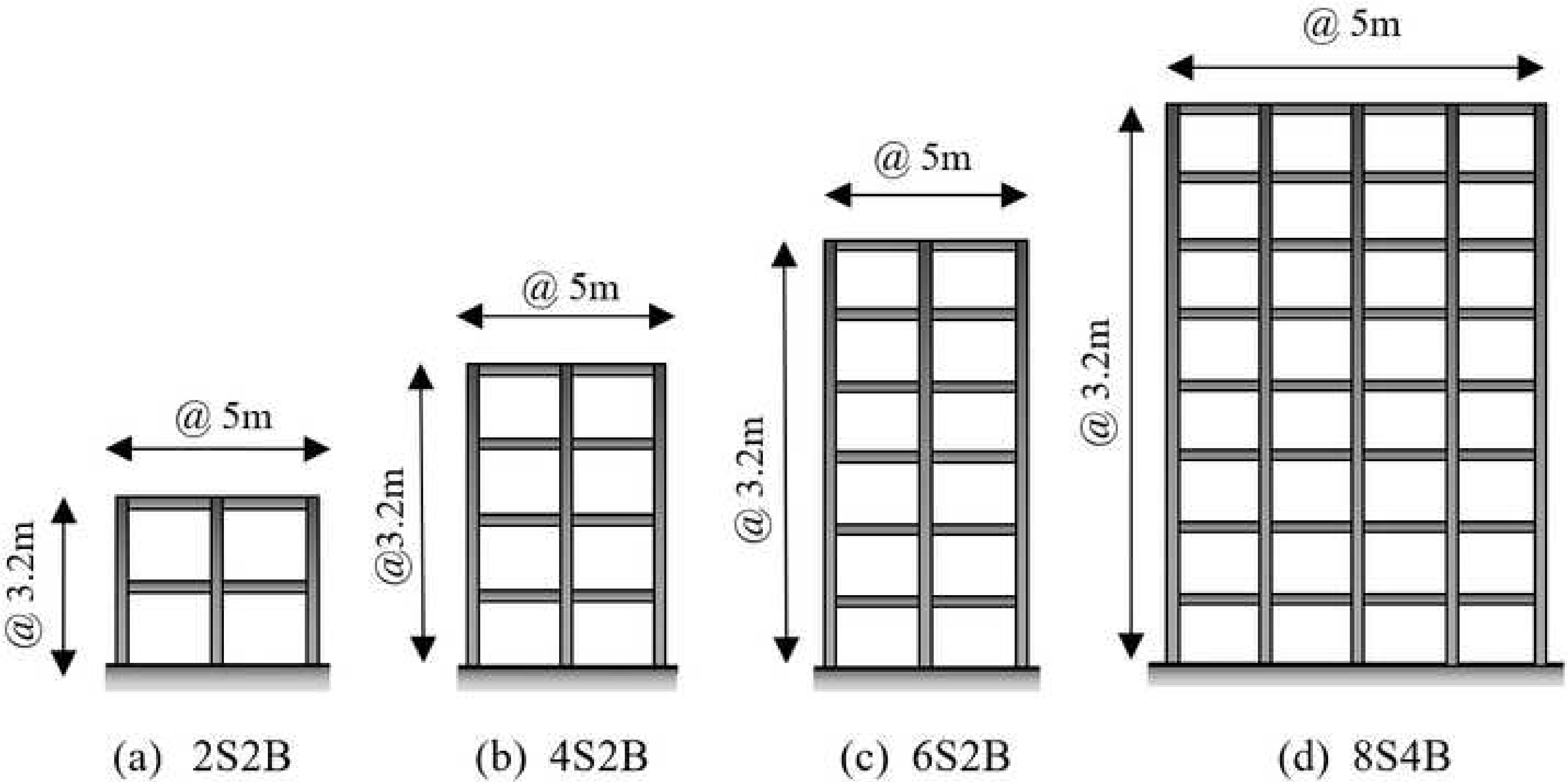

Fig. 1: RC frames considered in the study

Figure 2

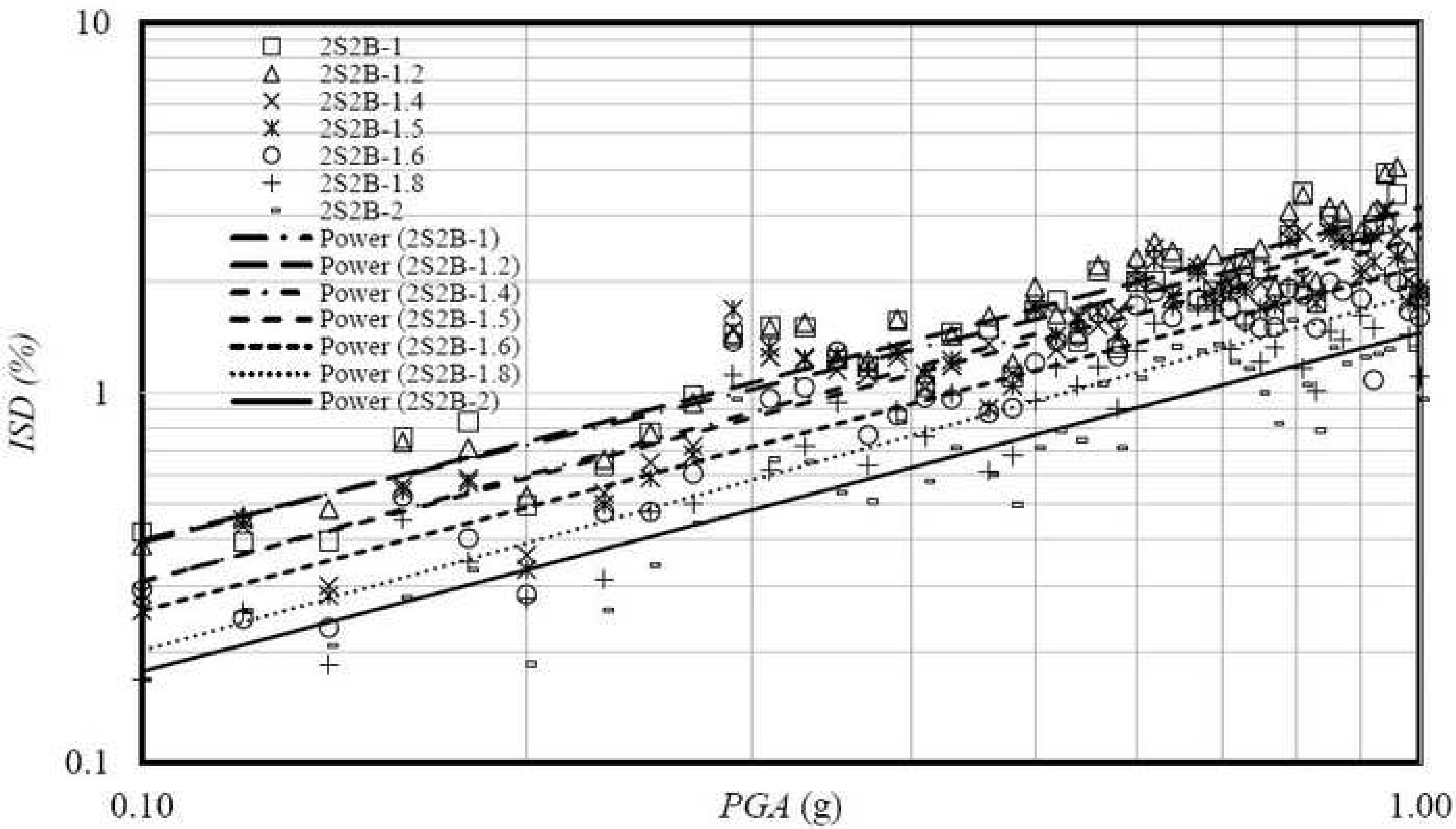

Fig. 2: PSDM for 2S2B frame

Figure 3

Click here to access/download;Figure;Fig 3.tif

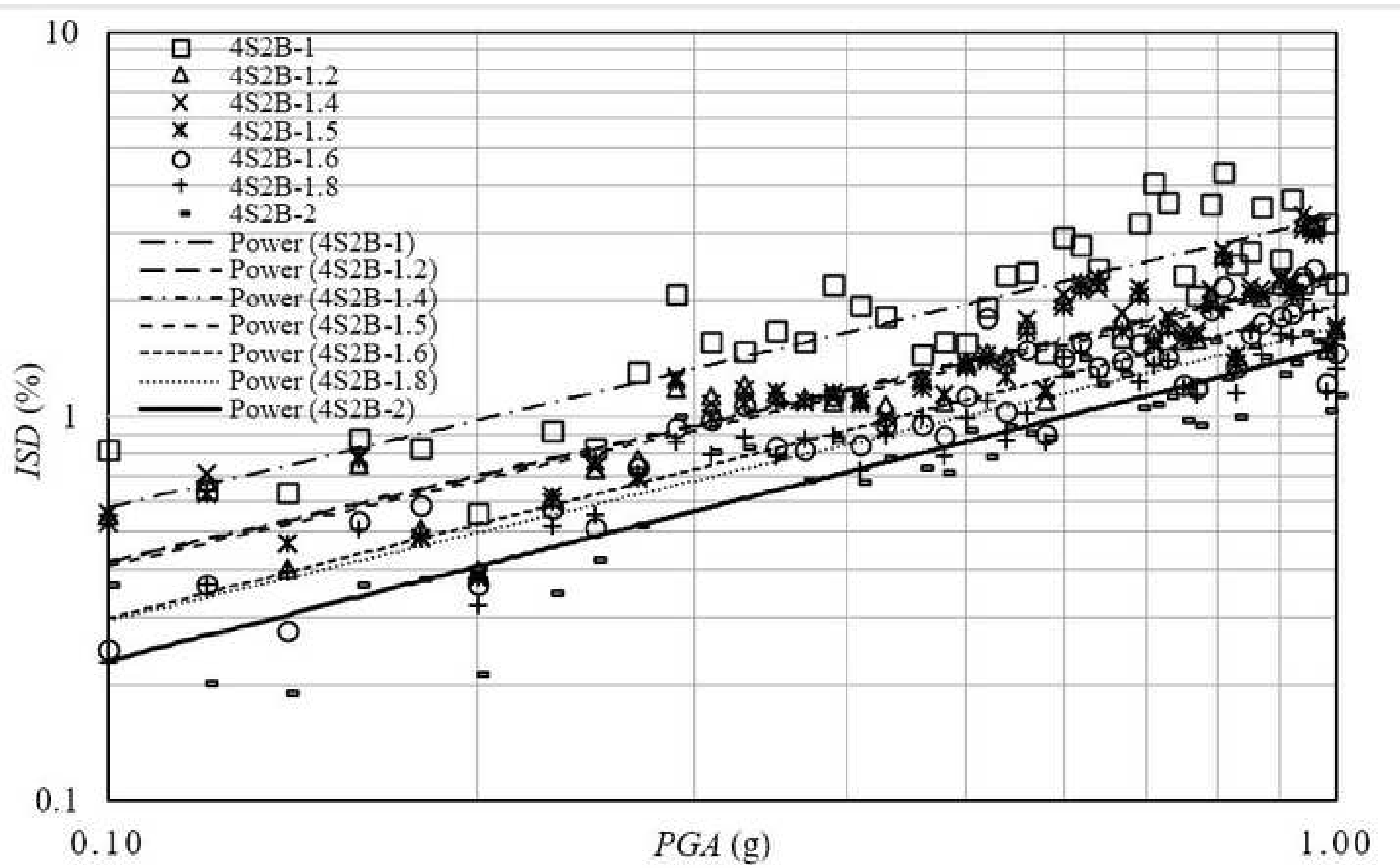

Fig. 3: PSDM for 4S2B frame

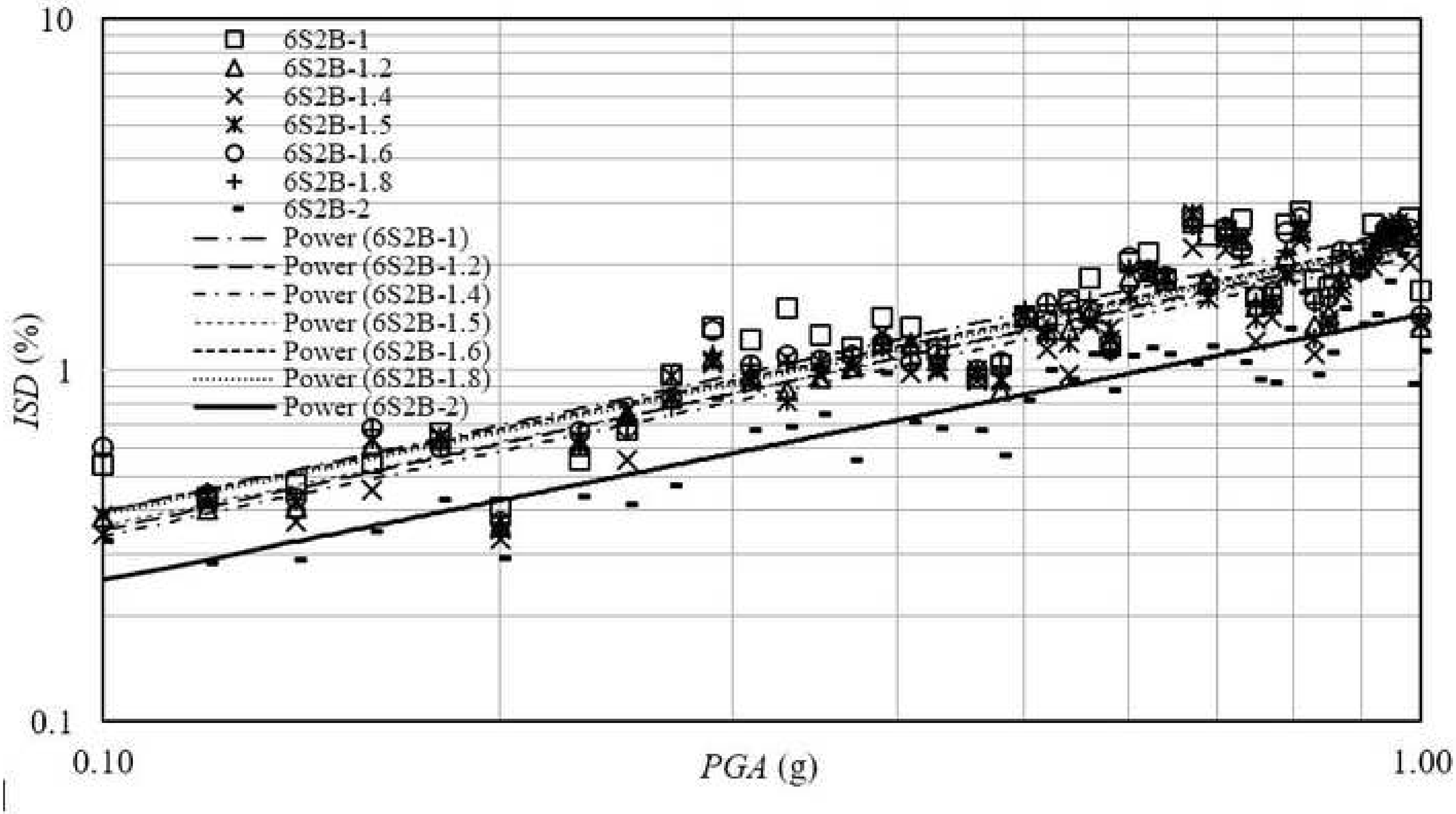

Fig. 4: PSDM for 6S2B frame

Figure 5

Click here to access/download;Figure;Fig 5.tif

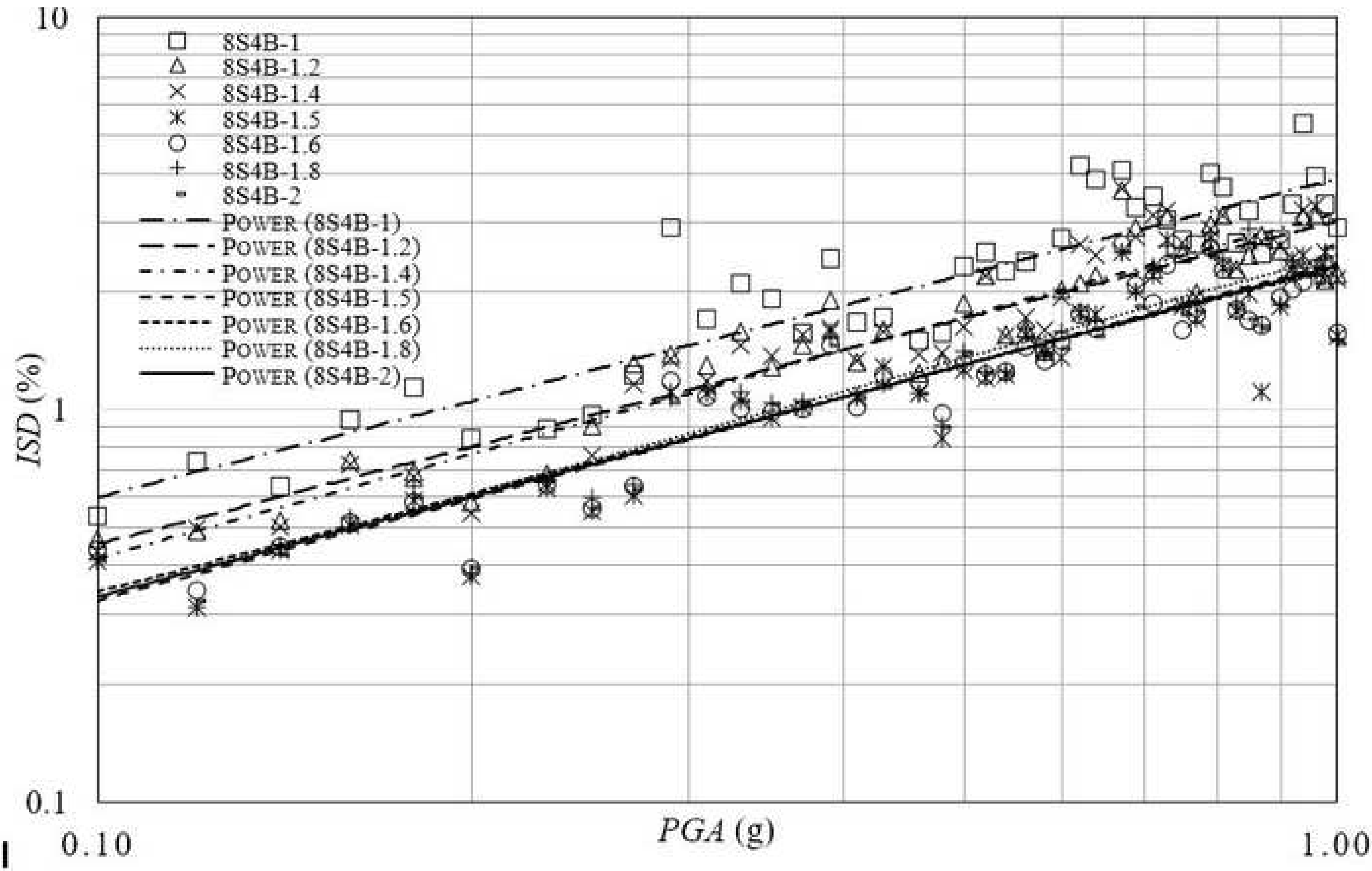

Fig. 5: PSDM for 8S4B frame

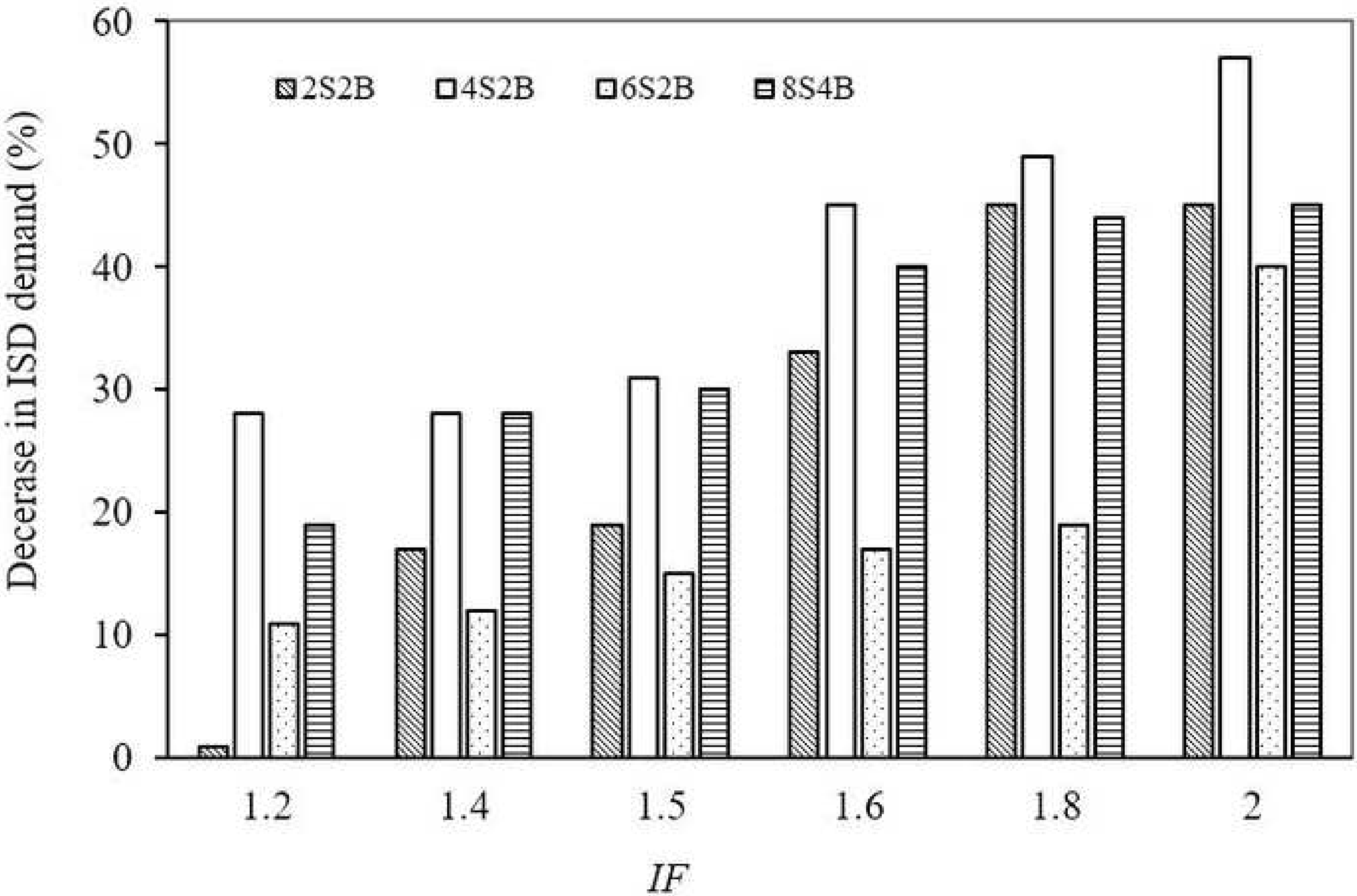

Fig. 6: Decrease in *ISD* as a function of *IF* at *PGA* = 0.3g

Figure 7

Click here to access/download;Figure;Fig 7.tif

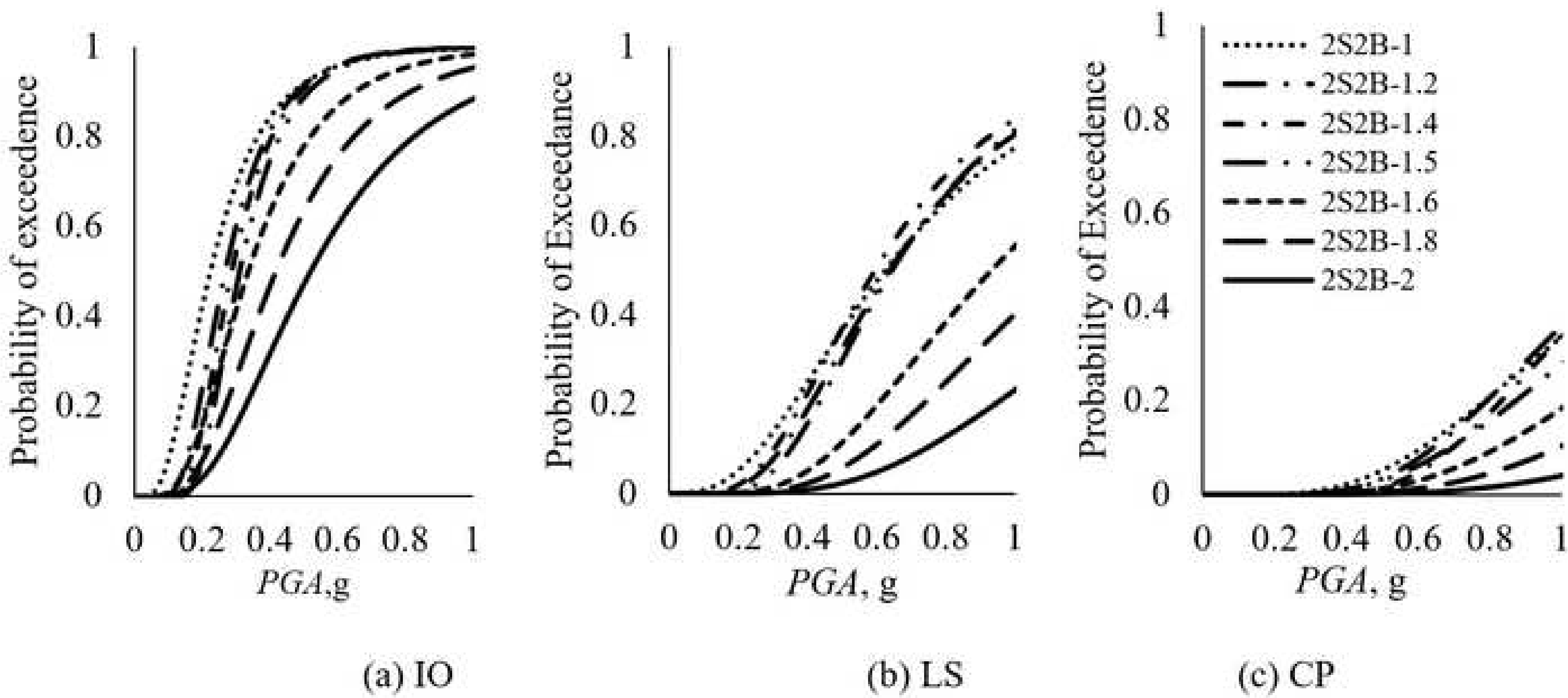

Fig.7: Fragility curve for 2S2B frame

Figure 8

Click here to access/download;Figure;Fig 8.tif

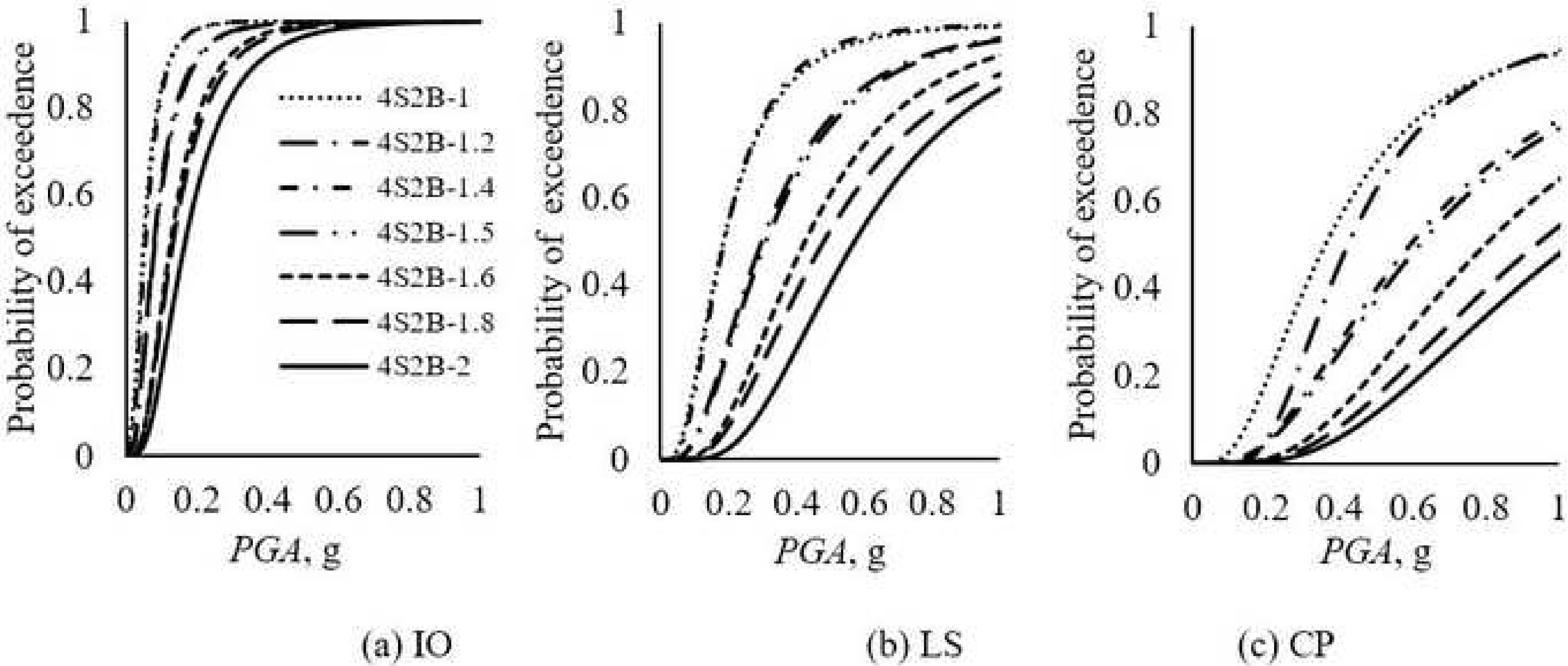

Fig.8: Fragility curve for 4S2B frame

Figure 9

Click here to access/download;Figure;Fig 9.tif

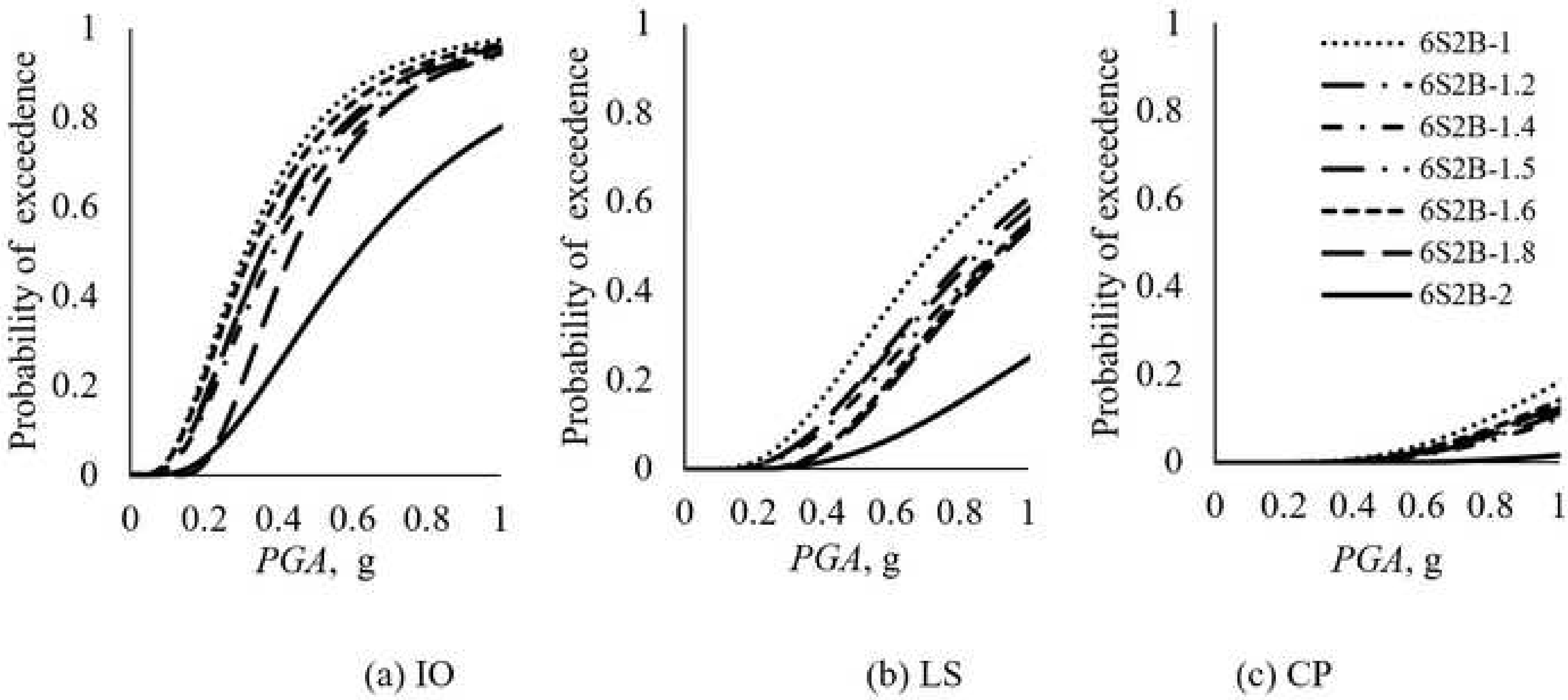

Fig.9: Fragility curve for 6S2B frame

Figure 10

Click here to access/download;Figure;Fig 10.tif

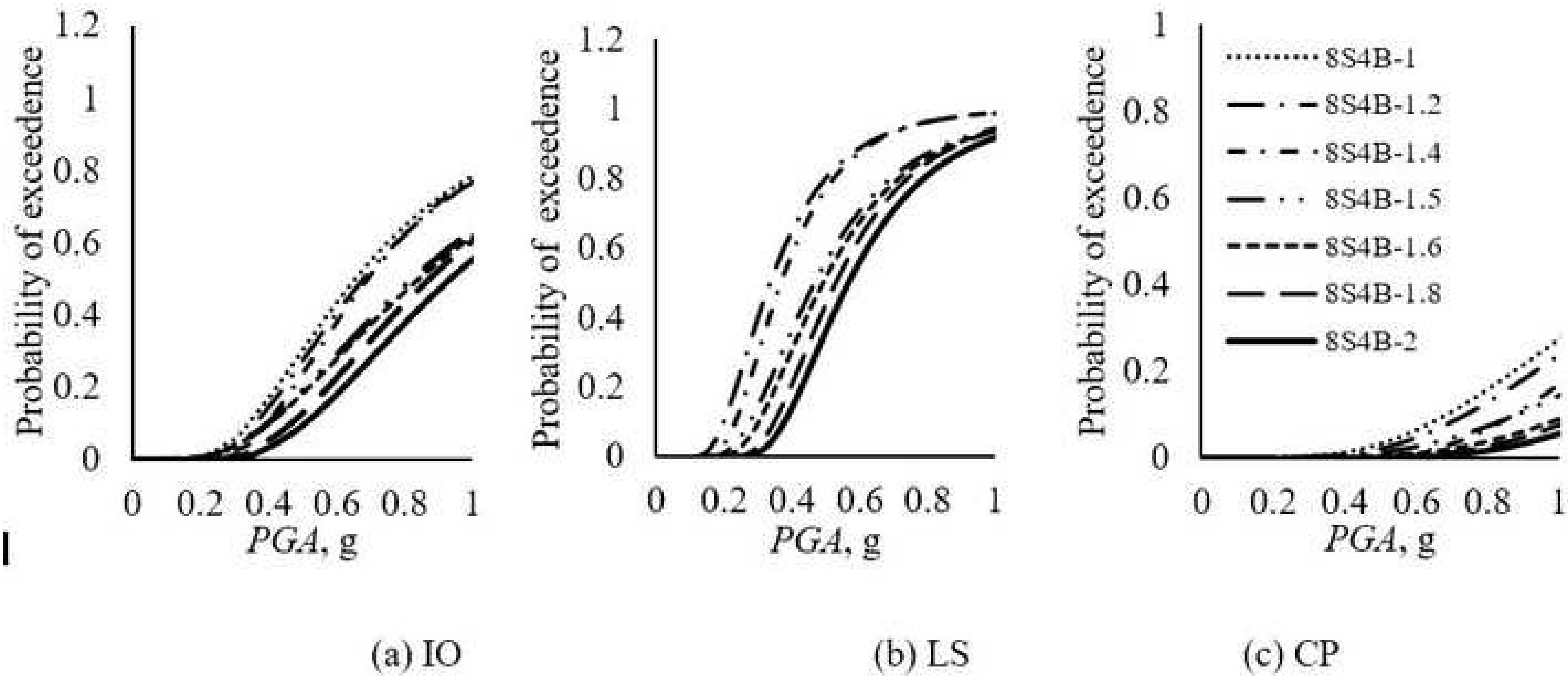

Fig.10: Fragility curve for 8S4B frame

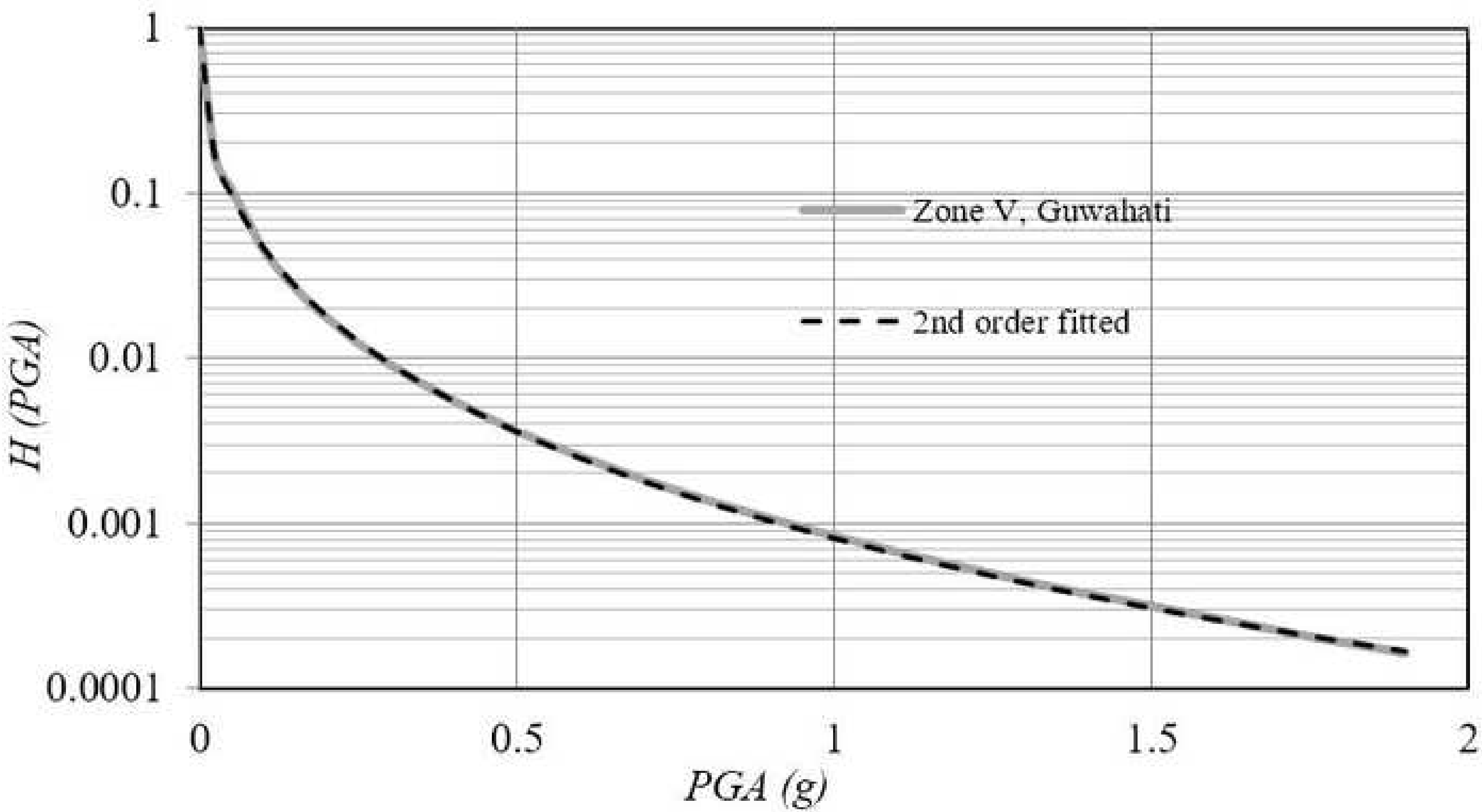

Fig. 11. Selected seismic hazard curve for Guwahati

Figure 12

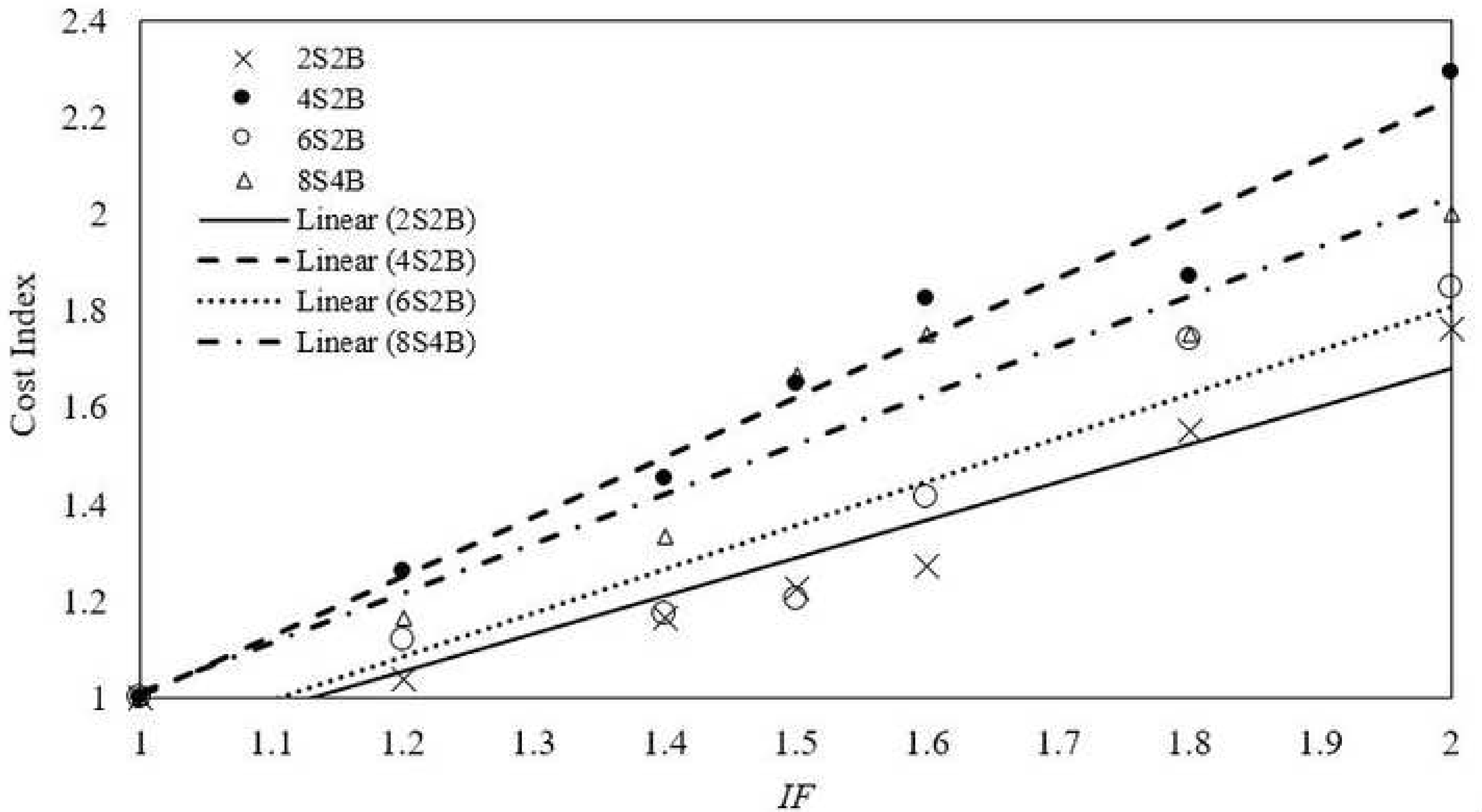

Fig. 12: Effect of *IF* on the cost index of the building

Figure 13

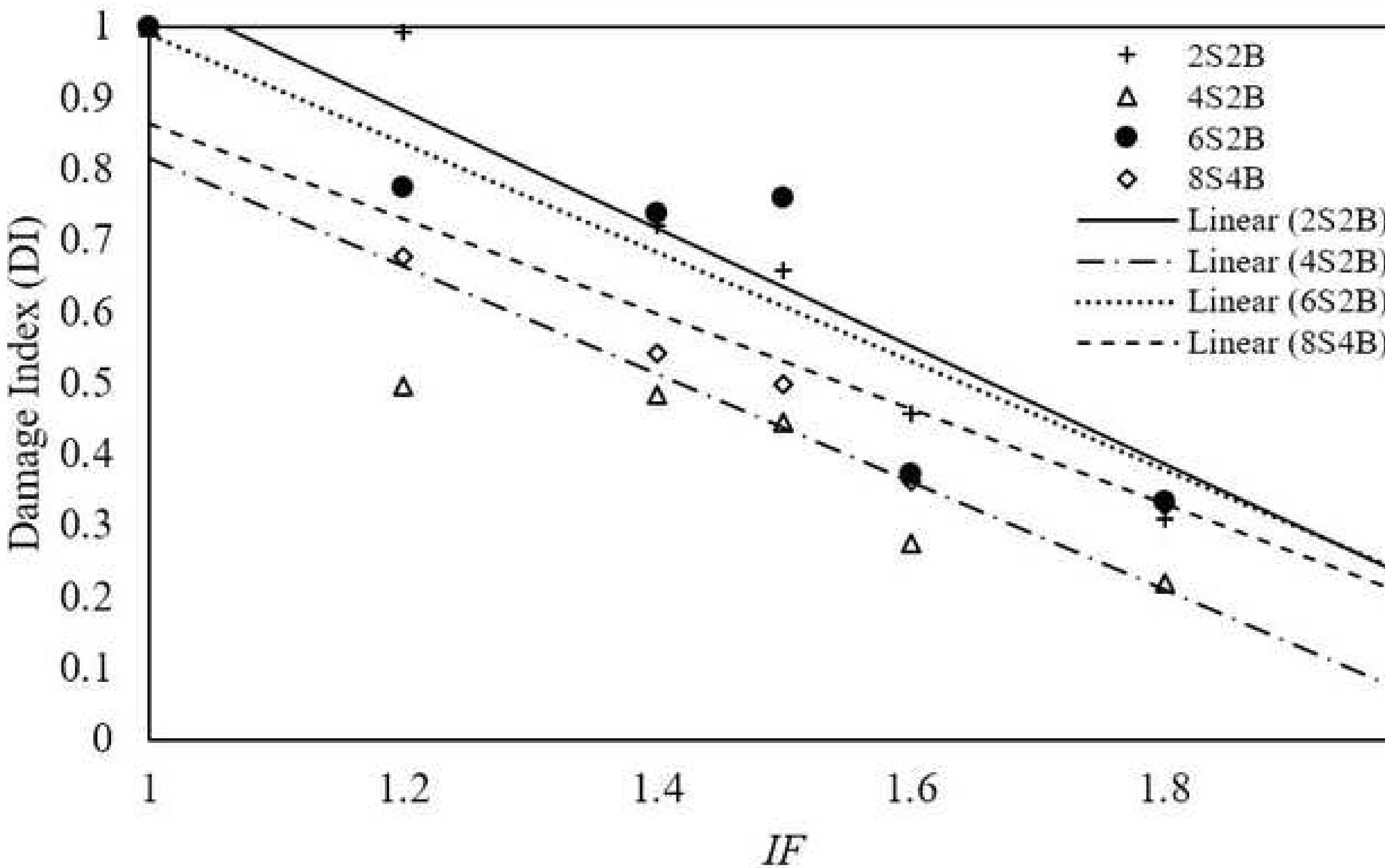

Fig. 13: Effect of *IF* on the damage index of the building for a typical performance objective

Figure Caption List

Change in Author Byline Form

Please use this form to submit changes to your author byline. Note, changes cannot be made after your manuscript has been published. A digital signature is acceptable as long as the name is not merely typed.

Publication Title: ASCE-ASME Journal of Risk and Uncertainty in Engineering Systems, Part A: Civil Engineering

Manuscript Title: IMPLICATIONS OF IMPORTANCE FACTOR ON SEISMIC DESIGN - FROM 2000 SAC-FEMA PERSPECTIVE

Explain why the author order differs from the author order on the manuscript when it was originally submitted:

The original manuscript has been revised significantly to comply with the reviewers' comments. The final version of the manuscript submitted includes new analysis results considering a major change in the analysis methodology. Recognizing the contribution of Nikhil P. Zade in this revision of this manuscript all authors have agreed to include Nikhil P. Zade as the 2nd author.

Author signature(s) – ALL AUTHORS (required)

Print name	Signature	Date
PRATEEK KUMAR DHIR	Prateek Kumar Dhir	17/05/2019
NIKHIL P. ZADE		17/05/2019
AKHILA BASU		17/05/2019
ROBIN DAVIS		17/05/2019
PRADIP SARKAR		17/05/2019

E-mail completed form to:

ASCE Authorship, Originality, and Copyright Transfer Agreement

Publication Title: ASCE-ASME Journal of Risk and Uncertainty in Engineering Systems, Part A: Civil Engineering

Manuscript Title: IMPLICATIONS OF IMPORTANCE FACTOR ON SEISMIC DESIGN - FROM A 2000 SAC-FEMA PERSPECTIVE

Author(s) – **Names, postal addresses, and e-mail addresses of all authors**

Prateek Kumar Dhir, University of Strathclyde, Glasgow, UK, G1 1XJ, prateek.dhir@strath.ac.uk

Nikhil P. Zade, National Institute of Technology Rourkela, India, 769008, nikhilzade648@gmail.com

Akhila Basu, National Institute of Technology Rourkela, India, 769008, akhilabasu15@gmail.com

Robin Davis, National Institute of Technology Calicut, India, 673601, robin.davisp@gmail.com

Pradip Sarkar, National Institute of Technology Rourkela, India, 769008, sarkarp@nitrkl.ac.in

I. Authorship Responsibility

To protect the integrity of authorship, only people who have significantly contributed to the research or project and manuscript preparation shall be listed as coauthors. The corresponding author attests to the fact that anyone named as a coauthor has seen the final version of the manuscript and has agreed to its submission for publication. Deceased persons who meet the criteria for coauthorship shall be included, with a footnote reporting date of death. No fictitious name shall be given as an author or coauthor. An author who submits a manuscript for publication accepts responsibility for having properly included all, and only, qualified coauthors.

I, the corresponding author, confirm that the authors listed on the manuscript are aware of their authorship status and qualify to be authors on the manuscript according to the guidelines above.

Prateek Kumar Dhir	Prateek Kumar Dhir	14th May 2019
Print Name	Signature	Date

II. Originality of Content

ASCE respects the copyright ownership of other publishers. ASCE requires authors to obtain permission from the copyright holder to reproduce any material that (1) they did not create themselves and/or (2) has been previously published, to include the authors' own work for which copyright was transferred to an entity other than ASCE. Each author has a responsibility to identify materials that require permission by including a citation in the figure or table caption or in extracted text. Materials re-used from an open access repository or in the public domain must still include a citation and URL, if applicable. At the time of submission, authors must provide verification that the copyright owner will permit re-use by a commercial publisher in print and electronic forms with worldwide distribution. For Conference Proceeding manuscripts submitted through the ASCE online submission system, authors are asked to verify that they have permission to re-use content where applicable. Written permissions are not required at submission but must be provided to ASCE if requested. Regardless of acceptance, no manuscript or part of a manuscript will be published by ASCE without proper verification of all necessary permissions to re-use. ASCE accepts no responsibility for verifying permissions provided by the author. Any breach of copyright will result in retraction of the published manuscript.

I, the corresponding author, confirm that all of the content, figures (drawings, charts, photographs, etc.), and tables in the submitted work are either original work created by the authors listed on the manuscript or work for which permission to re-use has been obtained from the creator. For any figures, tables, or text blocks exceeding 100 words from a journal article or 500 words from a book, written permission from the copyright holder has been obtained and supplied with the submission.

Prateek Kumar Dhir	Prateek Kumar Dhir	14th May 2019
Print name	Signature	Date

III. Copyright Transfer

ASCE requires that authors or their agents assign copyright to ASCE for all original content published by ASCE. The author(s) warrant(s) that the above-cited manuscript is the original work of the author(s) and has never been published in its present form.

Manuscript MS RUENG-418R3

Title: Implications of importance factor on seismic design - from 2000 SAC-FEMA perspective

Authors Response to the Comments from the Editors

1. Please provide current affiliations for all authors on the first page of your manuscript. Former affiliations are permissible only if an author's affiliation has changed after a manuscript has been submitted for publication.

   Response: A new page is added in the beginning of the manuscript to include current affiliations, current positions, professional designations and postal addresses for all the authors

2. Please include both current positions (for example, Professor, PhD. Candidate, Lecturer, etc.) AND professional designations (for example, PhD, MSc., etc.) for all authors in the affiliation statements on the first page of your manuscript.

   Response: Refer to our response to the 1st comment

3. Please include complete postal addresses in the affiliation statements for each author on the first page of your manuscript.

   Response: Refer to our response to the 1st comment

4. Please distinguish the two BIS (Bureau of Indian Standards), 2016 references by labeling them 2016a and 2016b respectively in the reference list and in-text citations.

   Response: The above two references are distinguished in the revised manuscript by labeling them 2016a and 2016b respectively in the reference list and in-text citations

5. The two references from the Bureau of Indian Standards do not have in-text citations. Every reference must have a citation in the text and every citation must have a reference. All unmatched references will be deleted by the copyeditor. Please provide in-text citations for these references and check your other references and in-text citations.

   Response: In-text citations of the above two references are now provided in the revised manuscript. The same has been checked for all other references given in the paper.

6. Please update the figure captions list. For Figures with multiple parts (a, b, c), please make sure that the labels for each part of the figure are included in the figure caption.

   Response: Figure captions list is now updated as per the above suggestion.

7. Please make sure that Tables 3-6 are mentioned individually in the manuscript file. Each figure and table should be mentioned individually in your manuscript text. While it is okay to refer to Tables 3-6, Tables 3, 4, 5, and 6 need to be mentioned on their own as well.

   Response: This is now updated in the revised manuscript (Line no: 202).