CODIFICATION OF PERFORMANCE-BASED SEISMIC DESIGN CONSIDERING MULTIPLE PERFORMANCE OBJECTIVES

P. Badal[1], R. Sinha[2]

[1] *Doctoral Candidate, Indian Institute of Technology Bombay, Mumbai, India-400076. aprakashn@gmail.com*
[2] *Professor, Indian Institute of Technology Bombay, Mumbai, India-400076. rsinha@civil.iitb.ac.in*

Abstract

The three-level seismic design objective of negligible damage in case of frequent earthquakes, limited damage in moderate events, and safety of occupants in case of severe events has long-served as an unquantified quest during the seismic design of buildings. Capacity-based design, strong-column-weak-beam design and special confinement reinforcement near beam-column joints are some concepts that have been in use for long. Such provisions in seismic design codes have evolved from observed building behavior during various earthquakes. These principles safeguard buildings against unexpected or sudden failure modes by ensuring a preferred order of failure. But, they do not reveal any information about the probabilities of exceeding any structural limit state due to their prescriptive nature. It is also observed that despite the widespread use of modern seismic codes and better understanding of seismic behavior of the buildings, there has been a steady increase in the economic losses to the infrastructure from earthquakes. The excessive seismic losses have raised questions about the value and nature of acceptable or target seismic risk. Due to considerable variation in the perception of seismic losses from region-to-region, consensus on the values of acceptable risk has not been reached; however, it has been underlined in published literature that targeting an acceptable collapse risk as the only objective cannot limit the economic losses. It is also worth noting that though collapse risk has its highest contributor from severe low-probability earthquakes, economic losses are often governed by more frequently-occurring moderate earthquakes.

The paper investigates the additional considerations required in the prescriptive design methodology in order to meet multiple seismic performance objectives obtained from performance-based design considerations. Two regular reinforced concrete moment frame buildings—four-story and twelve-story tall representing short-period and long-period buildings—designed as per Indian design standard have been considered. Nonlinear time history analyses of the buildings incorporating the strength and stiffness deterioration have been used to quantify their behavior. Multiple seismic performance objectives have been investigated to constrain both collapse risk and economic loss. To evaluate dynamic characteristics of buildings incorporating the uncertainties in seismic demand and capacity, incremental dynamic analysis for each building has been carried out for a suite of twenty-two pairs of far-field ground motion records specified in FEMA P695. Using deaggregation of seismic risk, it has been demonstrated that Design Basis Earthquake (DBE)-level hazard governs the seismic performance of short-period buildings. However, rarer ground motions than DBE-level are more critical for long-period buildings. In order to provide information for risk mitigation planning, mean Repair Cost Ratio of the buildings are estimated as a function of intensity measure. It is found that for seismic events corresponding to both 475-year and 2475-year of return periods, lower damage states have higher contribution to the mean Repair Cost Ratio than the severe damage state corresponding to Collapse Prevention. This illustrates that a simple targeting of collapse risk is unable to control the decision variable of Repair Cost Ratio along with that of collapse risk. Results from the present study highlight the need to consider multiple performance objectives, especially the ones with higher performance levels at more frequent earthquake events, for incorporation of performance-based seismic design in the design standards.

Keywords: performance-based seismic design; multiple performance objectives; seismic hazard specifications;

1. Introduction

Conventionally, seismic design of a building follows a prescriptive force-based methodology for analysis and design of the structural members of a building. The design base shear for a building of given configuration is computed as a function of the building's location, constituent lateral load-resisting system, importance, and natural period [1–3]. Recent advances in the seismic design of buildings is driven by a shift in focus from controlling the design force to controlling the expected performance of the buildings [4–6]. This new approach also allows for considering several uncertainties in the assessment of seismic demand and capacity. Due to an emphasis of safe building designs in regions with high seismic hazard, the primary objective of typical earthquake-resistant design frameworks that incorporate the next-generation performance-based design has been centered around limiting the risk of collapse [7,8]. Several studies have shown that the approach of targeting collapse risk may be misleading for moderate seismic regions where despite very low collapse risk, the economic losses arising out of more frequently-occurring moderate-level seismic events are critical [9,10].

The objective of the present study is to investigate whether there is a need to target multiple performance objectives in designs of buildings. Two special moment-resisting RC buildings, one with 4-storys representing short-period and another with 12-storys representing long-period, have been selected. A detailed nonlinear time-history analysis has been carried out to ascertain seismic risk corresponding to different damage states. Risk deaggregation demonstrates that DBE-level hazard governs the design for 4-story building but rarer hazard levels are important for 12-story building. Further, a decision variable namely Repair Cost Ratio of the buildings has been assessed. Contributions to the decision variable at different hazard levels indicate that even for rare events such as those corresponding to maximum considered earthquake, damage states lower than collapse can be crucial. This indicates that increased attention to reduce the losses from lower damage states can be highly beneficial in reducing the annualized economic losses associated with RC buildings.

2. PEER Framework

Starting with late 1990's, performance-based seismic design has evolved as a more realistic and controlled methodology. First-generation performance-based design guidelines described a matrix of expected performance objectives [11–13]. Each entry of this matrix is defined in terms of an expected performance level conditional on a seismic hazard. A part of this matrix has been presented in Table 1. Performance objectives corresponding to the cells *f*, *k*, and *p* are central to the present study. Different threshold values of *maximum interstory drift ratio* has been considered from the literature to define performance limits [12,14].

The extensively-used expression for evaluating annual frequency of exceedance of a decision variable developed by Pacific Earthquake Engineering Research (PEER) Center is given as follows [4,5,15]:

$$\lambda_{dv} = \int_{dm}\int_{edp}\int_{im} P(dv\,|\,dm)\,|\,p(dm\,|\,edp)\,\|\,p(edp\,|\,im)\,\|\,\mathrm{d}(\lambda_{im})\,| \tag{1}$$

where *dv* is a realization of the decision variable DV (say, dollar loss, repair cost, injuries, etc.), *dm* denotes damage measure, *edp* denotes engineering demand parameter, and λ_{im} denotes annual frequency of exceedance of hazard for intensity measure IM. Finally, $P(x|y)$ and $p(x|y)$ denote the conditional cumulative distribution function and conditional probability density function of random variable X conditional on another random variable Y = *y*, respectively.

Above expression facilitates the seismic performance assessment by breaking it down into four investigations, *viz.*, hazard assessment, structural fragility analysis, damage analysis, and loss analysis. Though questionable, the assumption of independence of these investigations from one another has been generally ignored by the researchers.

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3. Probabilistic Seismic Performance Assessment

3.1. Building Selection, Design, and Detailing

Two regular bare reinforced concrete special moment-resisting frame buildings—four-story and twelve-story tall—located in seismic zone-V as per IS 1893 (part 1) [1], have been selected for the present study. Design and detailing of the buildings conform to Indian standards [1,16,17]. Peak ground acceleration corresponding to maximum considered earthquake for the location is 0.36g. Both buildings have identical plan of 9 × 3 bays with the bay-width of 8.2 m along both direction. Longer ground floor columns (4.5 m) compared to other floors (3.9 m) are considered to incorporate the effect of depth of foundation. Plan and elevation along with required longitudinal reinforcement of these buildings are shown in Fig. 1. Prevalent structural engineering practices in the Indian industry have been used for design and detailing of the buildings. Table 2 provides general design details of these buildings. The buildings are taken from a larger set of *index archetypical buildings* representing the special RC frames. Further details of the building and detailing corresponding to the building ID 2221 and 2225 can be found in Badal & Sinha [18].

Seismic hazard curves are not explicitly provided in the Indian standards. Therefore, a two-parameter power law model, $\lambda_{im} \sim k_0 (im)^{-k}$, based on the MCE and DBE-level spectral coordinates, has been considered to approximate the hazard curve [5,19,20]. Table 3 shows the hazard values corresponding to different events and intensity measures.

3.2. Nonlinear Analytical Modeling and Ground Motion Selection

Two-dimensional nonlinear analytical model considering the effects of cyclic and in-cycle strength and stiffness reduction is employed for the seismic assessment. Hysteretic rules defined by Ibarra-Medina-Krawinkler are used to model the member's nonlinearity [21]. Concentrated flexural hinges have been modeled at the member ends. Recognizing that in capacity-based ductile detailing for shear force in the beams and columns, shear failure is not critical. Consequently, shear hinges are not modeled so as to reduce the cases of numerical non-convergence and computational efforts [22]. Flexural backbone curve for the members have been obtained using the semi-empirical expressions from the literature [23,24]. The finite dimension of the beam-column joint has been modeled using diagonal strut mechanism [25]. Nominal material strength for concrete and reinforcement steel are considered. Rayleigh damping of 5% in first and third mode has been considered for the members before yield. The adopted analytical model has been validated with experimental observations. More details of the analytical model and its validation can be found in Badal & Sinha [22]. Maximum base shear capacity and the Overstrength factor obtained from the nonlinear static analysis for the buildings are shown in last two columns of Table 2. For the time-history analysis, twenty-two pairs of far-field ground motion records have been selected [26].

3.3. Fragility Analysis and Uncertainty Propagation

Incremental dynamic analysis (IDA) is carried out by subjecting the structure to increasingly scaled ground motion record until the building collapse [27]. Results of IDA for all the considered suite of ground motion records is used to evaluate the fragility parameters of the buildings corresponding to different damage states. Spectral acceleration at approximate natural period has been considered as the intensity measure for the present study. Concept of controlling horizontal ground motion component has been utilized to consider the three-dimensional effect of ground motion. Record-to-record variability β_{RTR} is thus obtained from IDA. Other uncertainty component β_{DR}, β_{TD}, and β_{MDL} arising from uncertainties in Design Requirements, Test Data, and Modeling, respectively, are combined with it to compute total uncertainty as follows:

$$\beta_{TOT} = \sqrt{(\beta_{RTR}^2 + \beta_{DR}^2 + \beta_{TD}^2 + \beta_{MDL}^2)} \tag{2}$$

Recommendations of FEMA P695 are used to select values for these components. Assuming a lognormal distribution, Table 4 lists the fragility parameters for both buildings corresponding to three

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damage states, Immediate Occupancy (IO), Life Safety (LS), and Collapse Prevention (CP). Fig. 2a-b graphically represents fragility functions for both buildings.

4. Analysis Results

4.1. Observed Seismic Performance, Associated Risk, and Deaggregation

Annual frequency of exceedance of a damage state *ds*, also known as seismic risk associate with that damage state is defined as the convolution of hazard and fragility corresponding to *ds*:

$$\lambda_{ds} = \int_0^\infty \lambda_{im}(a) f_{ds}(a) da \tag{3}$$

where λ_{im} is the annual frequency of exceedance of intensity measure, IM, and f_{ds} is the probability mass function for the structural fragility. Table 5 presents associated seismic risk of both buildings with damage states of Immediate Occupancy (IO), Life Safety (LS), and Collapse Prevention (CP). The table also reports point estimate of seismic performance objectives corresponding to cells *f*, *k*, and *p* from Table 1. Considering a commonly accepted *low probability of exceedance* as 10% [7], it is found that the 4-story building fails to meet the performance objective of "CP @ MCE" whereas the 12-story building meets this objective. This indicates the deficiency in the design of low-rise buildings or short-period buildings. This behavior has been also observed elsewhere [22]. It is also noticed from the table that exceedance probability for performance objective "IO at 20%/50y" is disproportionately higher than that for other two objectives. This observation indicates a possible disparity between the hazard return period and performance level for "IO at 20%/50y". In other words, performance objective corresponding to cell *f* is disproportionately more stringent than the other two cells, *k* and *p*, constituting basic safety objective.

Risk deaggregation is carried out to assess the relative importance of intensity measure realization that has highest contribution to seismic risk [28]. Modal intensity measure corresponding to a damage state *ds* is defined as:

$$\mu_{0,ds} = \arg\max_{a}(\lambda_{im}(a) f_{ds}(a)) \tag{4}$$

Table 6 shows values of $\mu_{0,ds}$ for two damage states, LS and CP. The table also presents the return periods corresponding to modal intensity measure values. It is observed that DBE-level hazard governs the design for 4-story building (with return period corresponding to $\mu_{0,ds}$ as 294 year and 405 year) but rarer hazard levels than DBE-level are more critical for 12-story building (with return period corresponding to $\mu_{0,ds}$ as 511 year and 1184 year). This finding indicates that an accurate specification of a single hazard level is not sufficient to address the multiple performance objectives for both short-period and long-period buildings. Therefore, the traditional approach of specifying a single hazard value and computation of other hazard values based on it as incorporated in ASCE 7 and IS 1893 (part 1) [1,3], needs reconsideration.

4.2. Consequence Model and Vulnerability Functions

For the purpose of risk mitigation planning, decision variables like repair cost ratio as a fraction of building replacement cost conditional on intensity measure is one of the key outcomes of probabilistic seismic performance assessment. Such relationship, termed as vulnerability function, can be derived by weighted sum of (1) the marginal fragility functional values to quantify the probability of a building being in a particular damage state (rather than exceeding it) with (2) consequence model, which approximates the cost of repair of a building in each damage state [29].

$$\lambda_{RCR}(im) = \sum_k \Delta F_{ds,k}(im) D_k \tag{5}$$

where $\Delta F_{ds,k}$ is the marginal fragility function for a building to be in the k^{th} damage state and D_k is the repair cost ratio of k^{th} damage state, taken from the consequence model for the occupancy class of interest.

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In the absence of rigorous reconnaissance reports and statistical data from past earthquakes in India, we have adopted consequence model for commercial buildings for professional/business services from HAZUS [30]. Table 7 gives the consequence model for this occupancy class and tabulates the repair cost ratio for different damage states as a percentage of the building replacement cost.

Fig. 2c-d shows vulnerability functions for both buildings under the study. It should be noted that due to different natural period, intensity measure for both buildings are not same. The figure also displays the contributions to respective vulnerability from each of the three damage states. It is observed that for smaller intensity measure values, IO damage state has a higher contribution to vulnerability. This diminishes with increase in the intensity measure values. For very high intensity measure values, CP damage state controls the vulnerability.

Table 8 shows presents contribution from different damage states to the vulnerability of buildings at five hazard levels, namely, 20%, 10%, 2%, 1%, and 0.5% in 50 year. The highest contributor to the vulnerability for each building at a certain hazard level is marked in **bold**. It is observed from the table that even for a DBE-level (475-year return period) hazard, IO damage state has single largest contribution to the mean Repair Cost Ratio of both buildings. Further, for MCE-level hazard, IO and LS damage states combined together contribute to 50% or more of the repair cost ratio.

5. Conclusions

Performance-based seismic design of buildings is being gradually incorporated in the design standards across the globe. Traditionally, a single performance objective targeting collapse risk has been used as the basis for structural design. However, a more desired objective related to the economic loss is not directly targeted. The present paper investigates the importance of multiple performance objectives by considering a 4-story (short-period) and a 12-story (long-period) archetypical building. With due consideration to several sources of uncertainties in the seismic demand and capacity, a probabilistic framework has been adopted to predict building behavior. Based on the risk-deaggregation it is found that DBE-level hazard is critical for the short-story building whereas rarer earthquakes govern the performance for the long-period building.

Further, vulnerability functions have been developed to estimate the mean repair cost ratio of buildings as a fraction of building replacement cost conditional on the intensity measure. It is observed that for hazard levels as high as 475-year and 2475-year return period, lower damage states like Immediate Occupancy and Life Safety contribute more to the annualized building repair cost than severe damage state of Collapse Prevention. This implies that a simple targeting of collapse risk is unable to control the decision variable of Repair Cost Ratio. The present study therefore highlights the need for considering multiple performance objectives, especially the ones with higher performance levels at more frequent earthquake events, for more comprehensive incorporation of performance-based seismic design in the design standards.

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Table 1 – A subset of performance objective matrix used in the current study [12,14].

Performance limit → / Hazard level ↓	IO	LS	CP
20%/50y	*f*		
10%/50y (DBE)	*j*	*k*	
2%/50y (MCE)	*n*	*o*	*p*

(*k* + *p*) constitutes basic safety objectives
(*k* + *p* + any of *f*, *j*, or *n*) constitutes enhanced objectives

Table 2 – General design details of the buildings under consideration and Overstrength factor Ω_0 based on nonlinear static analysis.

Building ID	N_{st}	Height (m)	T_a (sec)	T_1 (sec)	C_s	$C_{s,max}$	Ω_0
2221	4	16.2	0.61	1.50	5.9%	17.5%	2.94
2225	12	47.4	1.35	3.65	2.7%	6.5%	2.45

N_{st}: number of stories
T_a: approximate natural period as per IS 1893 (part 1) [1]
T_1: analytical vibration period corresponding to first mode
C_s: design-level seismic response coefficient
$C_{s,max}$: maximum base shear coefficient

Table 3 – Seismic hazard levels for a site located in seismic zone-V as per IS 1893 (part 1) [1].

Event	Return Period (year)	POE in 50 year	PGA (g)	*Sa*(0.61) (g)	*Sa*(1.35) (g)
Occasional	225	20%	0.13	0.22	0.10
Rare	475	10%	0.18	0.30	0.13
Very Rare	2475	2%	0.36	0.59	0.27

Table 4 – Fragility parameters of 4-story (ID-2221) and 12-story (ID-2225) buildings corresponding to different damage states.

Building ID	IO		LS		CP	
	μ_{ds}	β_{TOT}	μ_{ds}	β_{TOT}	μ_{ds}	β_{TOT}
2221	0.32g	0.55	0.63g	0.53	1.12g	0.64
2225	0.18g	0.50	0.33g	0.51	0.76g	0.63

μ_{ds}: median fragility parameter for damage state, *ds*
β_{TOT}: total logarithmic standard deviation in the fragility

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Table 5 – Seismic performance for different objectives and associated risk for different damage state of 4-story (ID-2221) and 12-story (ID-2225).

Building ID	Associated Seismic Risk			Performance Objectives		
	λ_{IO} (10^{-4})	λ_{LS} (10^{-4})	λ_{CP} (10^{-4})	IO at 20%/50y	LS at DBE	CP at MCE
2221	41.4	7.8	2.9	23.7%	7.9%	16.2%
2225	21.6	5.0	1.0	11.4%	3.6%	4.9%

Table 6 – Deaggregation of seismic risk corresponding to Life Safety and Collapse Prevention damage states

Building ID	λ_{LS} (10^{-4})	$\mu_{0,LS}$ (g)	$RP_{\mu 0,LS}$ (year)	λ_{CP} (10^{-4})	$\mu_{0,CP}$ (g)	$RP_{\mu 0,CP}$ (year)
2221	7.8	0.24	294	2.9	0.28	405
2225	5.0	0.14	511	1.0	0.20	1184

λ_{ds}: seismic risk associated with damage state *ds*
$\mu_{0,ds}$: highest contributing intensity measure to the risk corresponding to damage state *ds*
$RP_{\mu 0,ds}$: return period corresponding to $\mu_{0,ds}$

Table 7 – Consequence model for a professional commercial building (COM4) as per HAZUS [30]

Damage State	Repair cost ratio (% of building replacement cost)			
	Structural repair cost ratio	Acceleration sensitive non-structural component	Drift sensitive non-structural component	Total
Moderate	1.9	4.8	3.3	10.0
Extensive	9.6	14.4	16.4	40.4
Complete	19.2	47.9	32.9	100.0

Table 8 – Contribution to decision variable, Repair Cost Ratio, at different hazard levels.

Building ID	DS	224-year RP	475-year RP	2475-year RP	4975-year RP	9975-year RP
2221	IO	**64%**	**45%**	13%	6%	2%
	LS	20%	30%	37%	32%	24%
	CP	16%	24%	**50%**	**62%**	**74%**
2225	IO	**75%**	**60%**	22%	11%	5%
	LS	21%	33%	**54%**	**53%**	45%
	CP	4%	7%	24%	36%	**50%**

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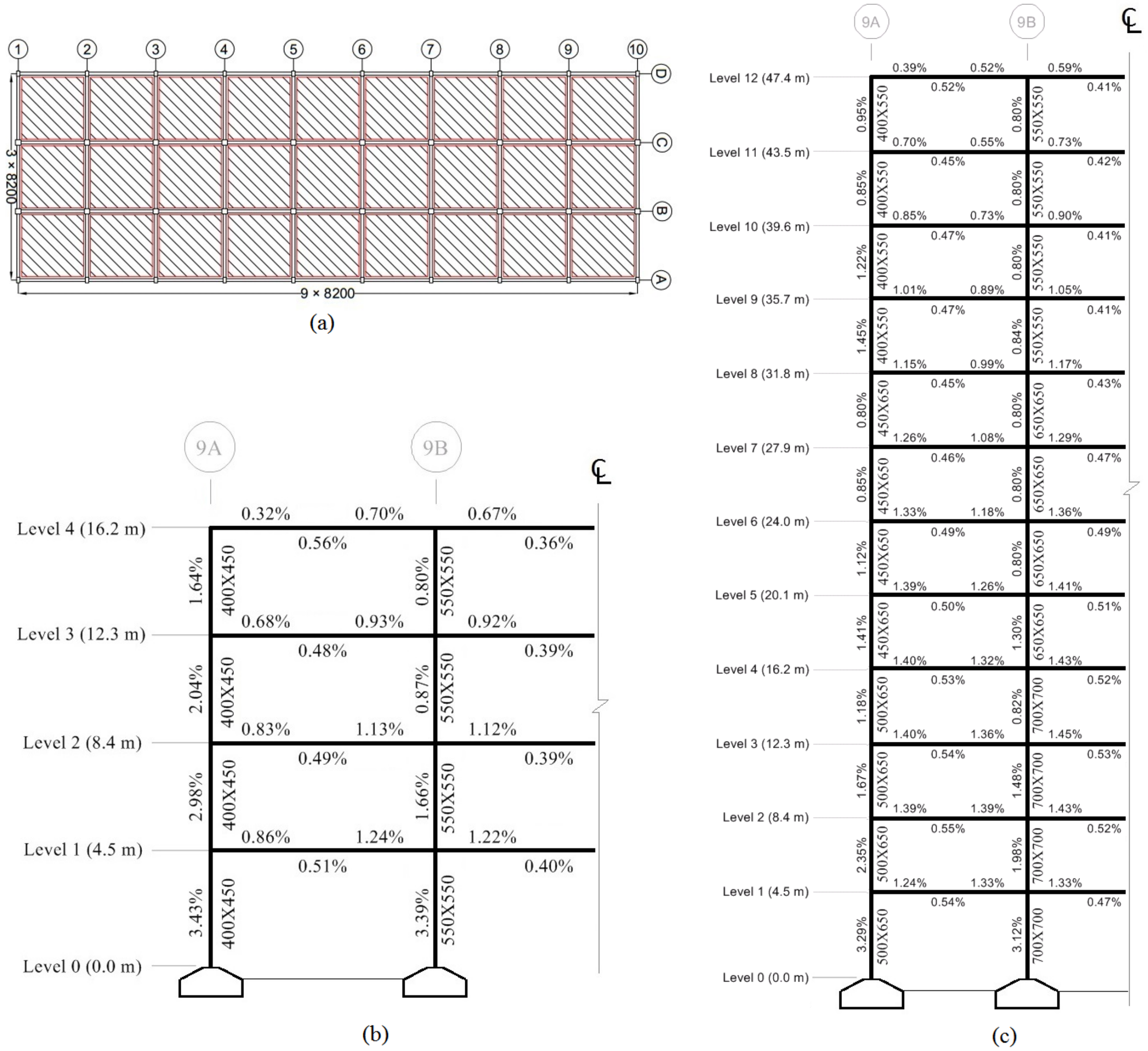

Fig. 1 – (a) Plan and section details of (b) 4-story building (ID-2221) and (c) 12-story building (ID-2225). All beams of both buildings are 400×750 mm in size.

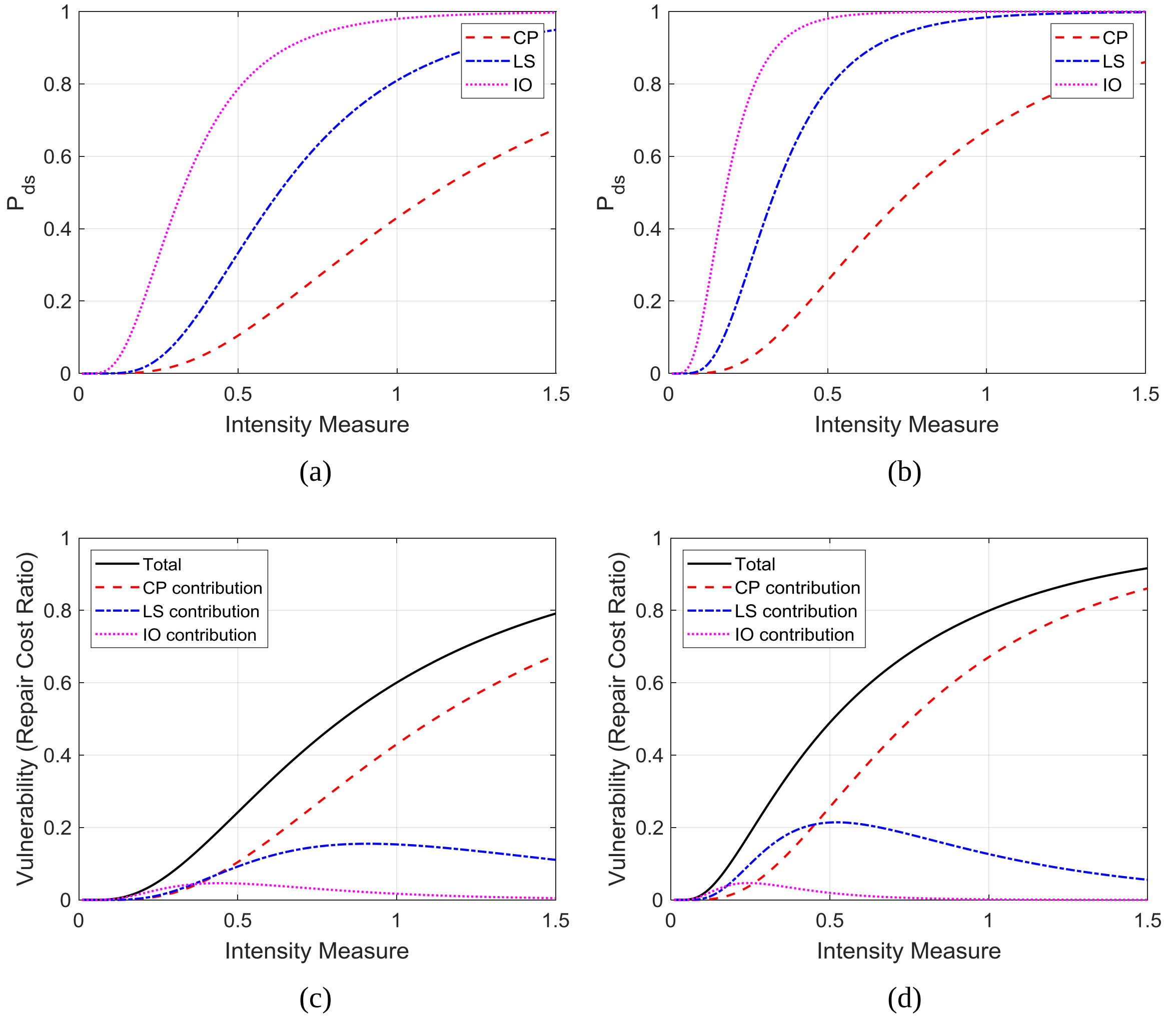

Fig. 2 – Fragility functions of (a) 4-story building (ID-2221) and (b) 12-story building (ID-2225). Vulnerability function of (c) building ID-2221 and (d) 2225, along with contributions to vulnerability from exclusive particular damage state. Intensity measure for ID-2221 is *Sa*(0.61) and for ID-2225 is *Sa*(1.35).

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