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Multilevel Seismic Demand Prediction for Acceleration-Sensitive Non-Structural Components

Vahid Mohsenian[1], Nima Gharaei-Moghaddam[2], Iman Hajirasouliha[3*]

[1] Postgraduate Researcher, Department of Civil Engineering, University of Science and Culture, Tehran, Iran

[2] PhD Candidate, Department of Civil Engineering, School of Engineering, Ferdowsi University of Mashhad, Mashhad, Iran

[3] Associate Professor, Department of Civil and Structural Engineering, The University of Sheffield, Sheffield, UK

* Corresponding Author; E-mail: i.hajirasouliha@sheffield.ac.uk

Abstract: Existing methods to predict the seismic demand of non-structural components in current seismic design guidelines do not generally consider the intensity of the design earthquake and the expected performance level of the lateral load bearing system. This limitation is especially important in performance-based design of buildings and industrial facilities in seismic regions. In this study, a novel multilevel approach is proposed to predict the seismic demand of acceleration-sensitive non-structural components using two new parameters obtained based on site seismicity and seismic capacity of the lateral load carrying system. The main advantage of the new method is to take into account the seismic hazard level and the expected performance level of structure in the calculation of the seismic demand of non-structural components. Based on the results of a comprehensive reliability study on 5 and 10-storey steel frame structures, the efficiency of the proposed approach is demonstrated compared to the conventional seismic design methods. The results, in general, indicate that the current standards may provide inaccurate predictions and lead to unsafe design solutions for acceleration-sensitive non-structural components, especially in the case of higher seismic intensity or medium performance levels. It is shown that the estimated accelerations by NIST and ASCE suggested equations are up to 50% and 80% lower than the minimum demand accelerations calculated for the studied structures, respectively, under the selected design condition. Based on the results of this study, a simple but efficient design equation is proposed to estimate the maximum acceleration applied to non-structural components for different earthquake intensity levels and performance targets.

Keywords: Acceleration-sensitive equipment; Non-structural components; Absolute demand acceleration; Absolute capacity acceleration; Seismic reliability; Fragility curves

1. Introduction

Non-structural components are the members which are attached to a structure but do not participate considerably in resisting loads, whether gravitational or lateral [1]. Architectural components such as walls, facades, partitions, shelves and suspended ceilings as well as mechanical and electrical components with their holders and connections are instances of these members. Different classifications have been proposed for non-structural components, for example based on their functionality or sensitivity to different properties of structure or excitations [1-3]. Extensive seismic damages to architectural non-structural components such as partition walls, exterior facade walls or suspended ceilings in previous earthquakes, have demonstrated the important role of these components in the seismic design and assessment of structures. The collapse of these non-structural components had drastic consequences in some cases, while it resulted in temporary loss of function in many residential and industrial structures [4]. There are several challenges to propose a unified suitable strategy for analysis and design of all types of non-structural components. The diversity of these elements, their various degrees of importance, inherent uncertainties associated with the future earthquakes, and the ambiguities regarding the distribution and amplification of applied acceleration at the height of the structure are among these challenges. At the initial design stage of typical structures, generally the participation of non-structural components in carrying the lateral design loads is neglected. Therefore, in most cases sufficient attention is not paid to seismic design and adequate detailing of these elements. The poor performance and widespread damages of such elements during recent major earthquakes provide a strong evidence for the shortcomings in this regard [5-7]. It should be noted that even if the lateral load bearing system can satisfy the target damage level under a severe earthquake, extensive damage to non-structural components can potentially violate the performance objective (e.g. Life Safety) and make the building inoperative (see Fig. 1). Initial estimates indicated that a large part of the capital required for building a structure is spent on non-structural components [4]. Thus, the importance of using appropriate seismic design and implementation approaches for non-structural components is evident from different aspects such as safety requirements or financial aspects.

Figure 1. Extensive damages to exterior and facade walls of buildings despite the acceptable performance of the lateral load-bearing systems in the Kermanshah earthquake, 2017

Over the past few decades, several studies have been performed on different aspects of non-structural components, in which most of the investigations were focused on seismic performance of these members. This subject attracted more attentions after the San Fernando earthquake in 1971 that caused extensive economic loses as well as high casualties due to collapse of non-structural components [8]. Fiorino et al. [9] performed shake table tests on several prototypes of architectural non-structural components. They utilized fragility curves as a means to assess the seismic performance of these components and also to evaluate the effect of connections on the extent of damage to the non-structural members. In addition, they performed cost estimation analysis and found that the repair cost for interior partitions is higher than that of exterior facade walls. In another relevant study, Sousa and Monteiro [10] conducted cost-benefit analysis on retrofitting of non-structural partition walls and concluded that for the buildings located in high seismic zones, retrofitting of only non-structural components can results in considerable reduction of seismic losses. Hou et al. [11] performed an experimental study to evaluate lateral seismic demand of non-structural components in low-rise steel frame building with tension-only braces. They designed and tested two full-scale three-storey models and based on the attained results, they found that the existing relationships for estimating seismic demand of non-structural components are inadequate for the studied structural system. Moreover, they concluded that participation of exterior walls in resisting lateral forces increases the seismic demand of internal non-structural components. Similarly, Lim et al. [12] performed large-scale experiments on multiply-supported non-structural components under multi-direction excitation. They investigated the interaction of primary structure and non-structural components and showed that the current specifications are generally inadequate in estimating the seismic demand of non-structural components, since they are developed based on one directional seismic excitation.

Lima and Martinelli [13] investigated the main mechanical parameters, which control the behaviour of acceleration-sensitive non-structural components. They used a two-degrees of freedom system to model both structural and non-structural components. Their study showed that the current code provisions for estimating the seismic demand of non-structural components, neglect some of the effective parameters. It is noteworthy that a similar conclusion was made previously by Martinelli and Faella [14]. According to their findings, Lima and Martinelli [13] suggested that further investigations are required to propose more accurate and reliable code provisions for determining the seismic demand of non-structural components. In an attempt to reduce the seismic design forces of non-structural components, Miranda et al. [15] proposed a new approach, in which bracings of non-structural elements are designed and detailed in a manner to act as a seismic fuse and limit the applied seismic forces to the components and their attachments.

Petrone et al. [16] performed a parametric study on ten reinforced concrete buildings with different numbers of storeys varying from 1 to 10. They used a set of frequent earthquakes with 63% probability of exceedance in 50 years, and performed dynamic nonlinear analysis. They found that the relations proposed by Eurocode 8 [17] underestimate the seismic demand of light non-structural components in a wide range of excitation frequencies. According to their results, only for the natural vibration periods which are sufficiently higher than the natural frequency of the structure, the Eurocode proposed equations provide acceptable estimations. Accordingly, they proposed a new formulation to improve the accuracy of the predictions. In another similar study, Magliulo et al. [18] evaluated the adequacy of Eurocode 8 [17] provisions for calculating the seismic demand of light non-structural components using a series of multi-storey structures. Similarly, Fathali and Lizundia [19] investigated the accuracy of ASCE 7-05 [20] provisions to estimate the seismic demands of non-structural components in tall buildings. They found that the code provisions may not provide satisfactory estimations, and accordingly modified the design relations in two aspects. First, they suggested a nonlinear relationship instead the code linear relationship between the peak floor acceleration (PFA) and the relative height of the component. Then, they suggested to use a three-segment spectrum for component amplification factor. They verified the performance of their modified relationships with the recorded results for tall buildings with over 15 storeys. In a more recent study, Anajafi and Medina [21] assessed the efficiency of ASCE 7-16 [22] suggested equation for estimating the seismic demand of non-structural components using data from instrumented buildings as well as numerical models. The results of their study indicated that this relationship does not provide accurate estimations.

Chauhuri and Villaverde [23] performed an extensive parametric study on steel moment resisting frames to investigate the effect of nonlinear behaviour of structures on amplification of seismic response of non-structural components. They found that the seismic response of the non-structural components is reduced in nonlinear structures compared to their linear counterparts. To address the shortcomings of existing code design equations, Singh et al. [24, 25] suggested new design relationships to estimate the seismic demand of rigid and flexible non-structural components, which could provide more conservative estimations. In their review paper, Filiatrault and Sullivan [26] demonstrated the major research needs to fill the research gaps in the seismic design and analysis of non-structural components, and discussed the possibility of using the performance-based design approach for seismic design of non-structural components. In follow-up study, Filiatraul et al. [27] suggested a modification of direct displacement-based approach for performance-based seismic design of non-structural components. Calvi and Sullivan [28] proposed a simplified modal combination approach to estimate floor spectra in multiple degree of freedom elastic systems. In a more recent study, Vukobratovic and Fajfar [29] developed a method for more accurate estimation of floor spectra obtained from input ground motion spectra, by taking into account the dynamic properties of the structure. Their proposed method can be used for both elastic and inelastic systems.

Based on the results of the above mentioned studies, it is clear that the standard seismic code suggested equations do not generally lead to accurate predictions of seismic demands in non-structural components. This is also evident from the extensive damage to non-structural elements observed even under moderate earthquakes in the recent years. On the other hand, the adequacy of the current specifications is ambiguous under different earthquake intensity levels and various performance targets. To address these issues, this study aims to develop a novel multilevel approach to predict the seismic demand of acceleration-sensitive non-structural components. For this purpose, two new parameters namely absolute demand acceleration (A_{Demand}) and absolute capacity acceleration (A_{Supply}) are introduced, which can be obtained based on the seismicity of the site and capacity of the lateral load system, respectively. The main advantage of the proposed method is to take into account the seismic hazard level and the expected performance level of structure in the calculation of the seismic demand of non-structural components. This feature is highly desirable in performance-based design of these systems. The efficiency of the proposed approach is then demonstrated compared to the conventional seismic design code relationships through a comprehensive reliability study on 5 and 10-storey steel frame structures. Finally, the results are used to suggest a simple equation to estimate the maximum acceleration applied to non-structural components under different scenarios.

2. Code-based equations for non-structural seismic demand prediction

In this section, the widely used relationships for estimation of the seismic demand of non-structural components are briefly explained. In seismic design standards such as ASCE 7-16 [22] and standard No. 2800 [30], the effective design lateral force applied to the non-structural components (F_p) is calculated using the following general equation:

$$\frac{F_p}{W_p} = \frac{0.4\, a_p A B_s I_p}{R_p}(1 + 2\frac{Z}{H}) \tag{1}$$

In this equation, A is the design acceleration and B_S is the reflection coefficient for low vibration periods at 5% damping (for conservative evaluation, this parameter can be selected based on the maximum values of the design spectrum). a_p is the amplification factor, which includes the effects of proximity of the natural frequencies of the structure and non-structural components. This factor varies between 1 and 2.5 and is higher if the vibration period of the structure and non-structural components are closer. I_P is the importance factor that generally varies from 1.0 to 1.5. For non-structural components and Life Safety performance level this factor is considered equal to 1. W_p is weight of non-structural components during service. R_p is the response modification factor, which is a criterion for evaluating ductility or fragility of the components and their accessories. This factor increases by increasing the ductility and energy absorption capacity of non-structural components. H and Z are the roof height and elevation of the component mass centre from the base elevation, respectively (See Fig. 2).

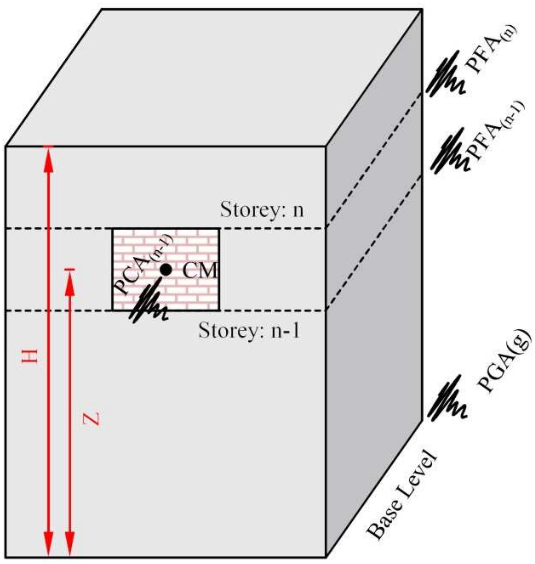


Figure 2. Effective parameters used in current non-structural seismic demand prediction equations

In seismic design guidelines such as ASCE 7-16 [22] and standard No. 2800 [30], it is required to apply lateral earthquake loads to non-structural components at least in two perpendicular directions. These lateral forces must be applied along with the expected dead and service loads to the non-structural components in order to produce maximum stresses in their supports and connections. The following higher and lower limits are generally defined:

$$F_{p_{min}} = 0.3AB_sW_pI_p \tag{2}$$

$$F_{p_{max}} = 1.6AB_sW_pI_p \tag{3}$$

NIST [31] suggests the following equation to estimate the applied load to non-structural components:

$$\frac{F_p}{W_p} = PGA\left[\frac{\left(\frac{PFA}{PGA}\right)\gamma_{tor}\gamma_{diaph}}{R_{\mu bldg}}\right]\left[\frac{\left(\frac{PCA}{PGA}\right)B_{\beta comp}}{R_{\mu comp}}\right]I_p \tag{4}$$

where PGA is the maximum ground acceleration in the base level of structure. This parameter can be calculated using $0.4S_{DS}$ and $0.4A(S+1)$ according to ASCE 7-16 [22] and standard No. 2800 [30], respectively, where S_{DS} is the short period spectral acceleration for the building site and S is a constant parameter (ranging between 1.5 to 2.25) determined based on site soil classification. PFA and PCA are maximum storey acceleration and maximum acceleration of non-structural components, respectively. I_p is importance factor of non-structural component. γ_{tor} represents intensification factor for storey acceleration in surrounding areas with respect to the centre of rigidity for structures with torsional irregularity. γ_{diaph} is intensification factor for storey acceleration due to diaphragm deformability. $B_{\beta comp}$ denotes spectrum factor for non-structural component with respect to the 5% damping spectrum developed based on PGA (for 5% damping this factor is equal to 1). $R_{\mu comp}$ is coefficient of nonlinear performance for connections of non-structural components or the components themselves. $R_{\mu bldg}$ is the reduction factor for overall ductility of structure and can be calculated using Equation (5), in which R and Ω_0 are response modification and over-strength factors, respectively.

$$R_{\mu bldg} = \left(\frac{1.1R}{\Omega_0}\right)^{\frac{1}{2}} \geq 1.0 \tag{5}$$

Equation (4) can be simplified in the following form by taking into account the relationship between dynamic magnification due to ductility of the members and damping of components:

$$\frac{F_p}{W_p} = PGA \times \left[\frac{\left(\frac{PFA}{PGA}\right)}{R_{\mu bldg}}\right] \times \left[\frac{\left(\frac{PCA}{PFA}\right)}{R_{pocomp}}\right] \times I_p \tag{6}$$

In this relation, R_{pocomp} is the marginal coefficient of residual storey strength which is used to include the inherent over-strength in common design of non-structural components (according to the current design specifications, a value of 1.3 is considered for this variable). $\left(\frac{PFA}{PGA}\right)$ used in Equation (6) is defined as follows:

$$\left(\frac{PFA}{PGA}\right) = 1 + a_1\left(\frac{Z}{H}\right) + a_2\left(\frac{Z}{H}\right)^{10} \tag{7}$$

where a_1 and a_2 are constant parameters calculated based on the following equations:

$$a_1 = \frac{1}{T_{abldg}} \leq 2.5 \tag{8}$$

$$a_2 = \left[1 - \left(\frac{0.4}{T_{abldg}}\right)^2\right] > 0 \tag{9}$$

In the above equations, T_{abldg} represents the empirical vibration period of the structure. It should be noted that $\frac{PCA}{PFA}$ indirectly considers the effects of the magnification, inherent damping and ductility of non-structural components. NIST [31] presented a table of values for this variable. The minimum lateral force ($F_{p_{min}}$) in this standard, is also determined by Equation (2). To calculate the maximum lateral force ($F_{p_{max}}$), however, the following equation is proposed:

$$F_{p_{max}} = 2AB_sW_pI_p \tag{10}$$

In the present study, to verify the adequacy of the described relationships, periphery facade walls are considered as non-structural components. Subsequently, by replacing the parameters related to this group of components, the applied lateral forces are calculated and compared with the results of non-linear dynamic analyses. To provide more information, all input parameters used in the above ASCE 7-16 [22] and NIST [31] proposed formula are presented in Appendix A.

3. Specifications of the studied models

For this study, three-dimensional steel structures with the plan depicted in Fig. 3 are used. The structures are designed to satisfy Intermediate Moment Frames (IMFs) requirements in accordance to AISC 360-10 [32] (i.e. medium ductility level). Span length and storey height for the modelled structures are 5 and 3.2 m, respectively. The connections of the steel frames were assumed to be rigid. To study effect of the height on the seismic performance of structures, 5 and 10-storey structures are considered. The structures are assumed to have residential use and to be located in an area of high seismic hazard in Iran (PGA=0.35g).

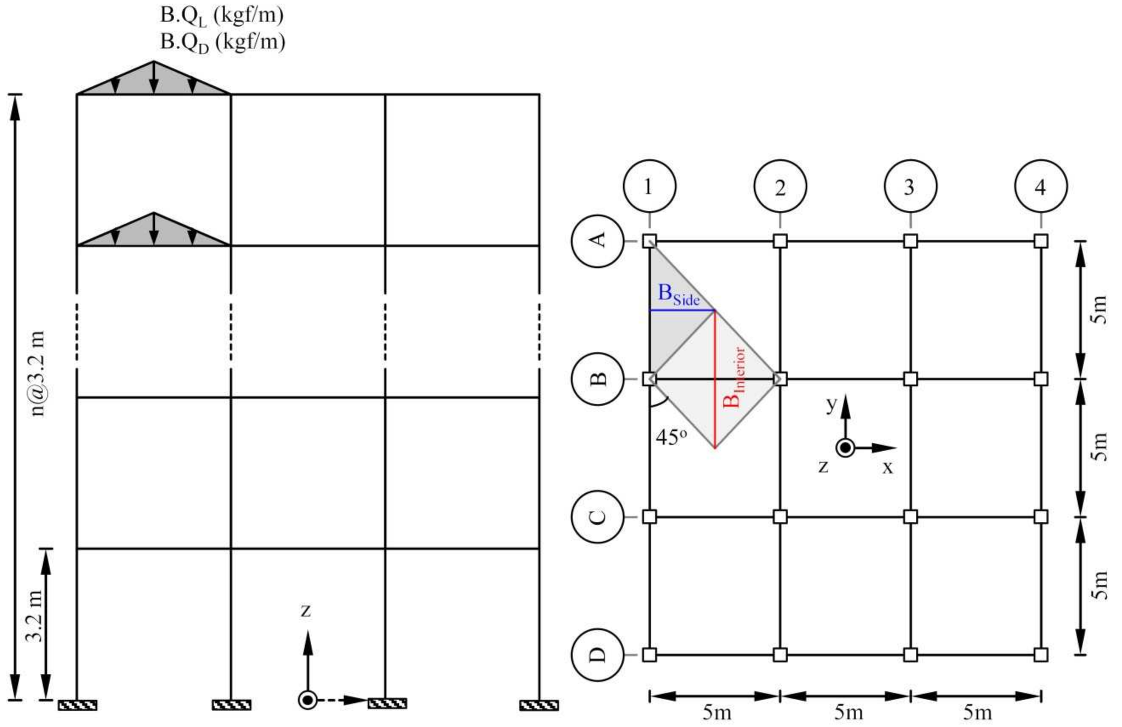


Figure 3. Plan of the studied structures and the distribution of vertical loads on beam elements

Using two-way slab floor system, the dead load (Q_D) was calculated to be 630 kgf/m^2 by taking into account the weight of the partitions. The live (Q_L) load was considered to be 200 kgf/m^2 and 150 kgf/m^2 for interior storeys and roof, respectively. Fig. 3 shows the distribution pattern of the design vertical loads on beam elements. The site soil was assumed of type "II" (with the shear wave velocity between 375 to 750 m/s) according to ASCE 7-16 [22]. The sectional characteristics for beam and column elements are listed in Table (1) based on the identification codes (IDs) shown in Fig. 4. Since the studied structures are all symmetric with respect to the z-axis (see Fig. 3), only half of the middle frames are shown in this figure.

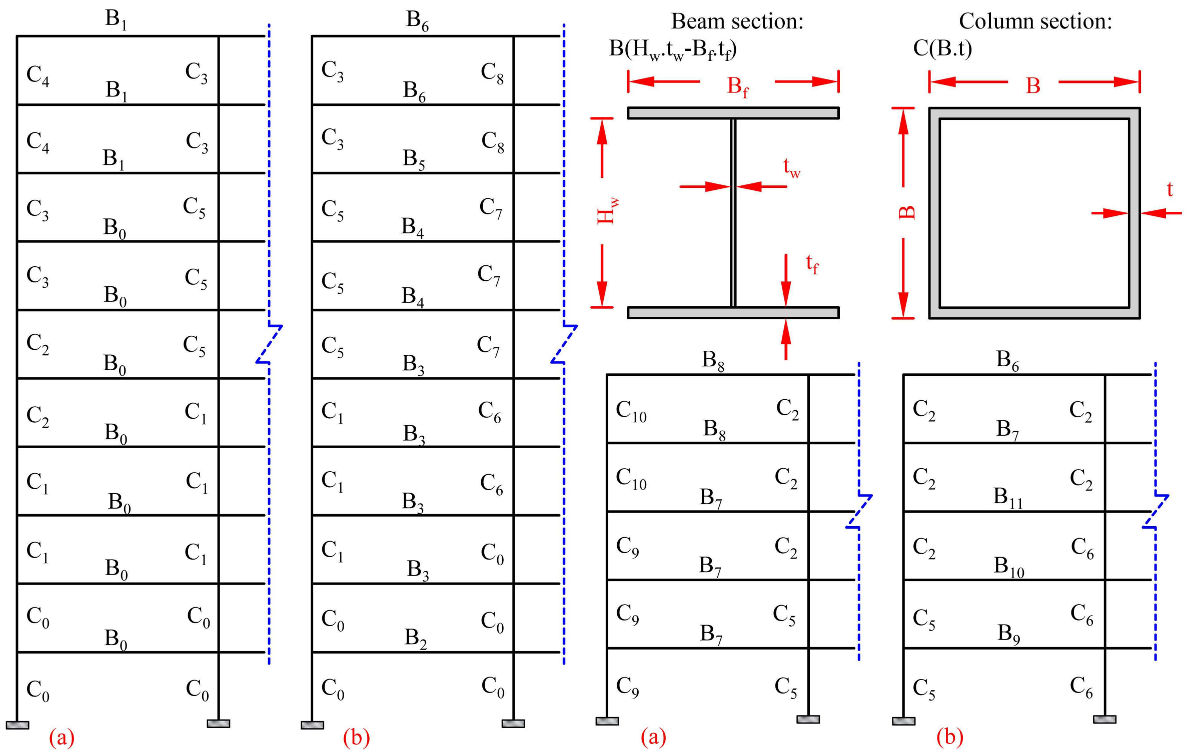


Figure 4. Geometric features of the studied structures (a) Side frames (axes 1, 4, A and D) (b) Interior frames (axes 2, 3, B and C)

The mild steel material is used with yield stress of 240 MPa, modulus of elasticity of 200 GPa and Poisson's ratio of 0.3 [33]. The effect of the rigid diaphragm and P-Delta effects are included in the analyses and design of the models. The studied structures are designed using ETABS software [34] based on AISC 360-10 steel design requirements [32]. The vibration modes of the two designed case-study buildings are listed in Table (2).

Table 1. Specifications of utilized sections for beams and column members (Units: mm)

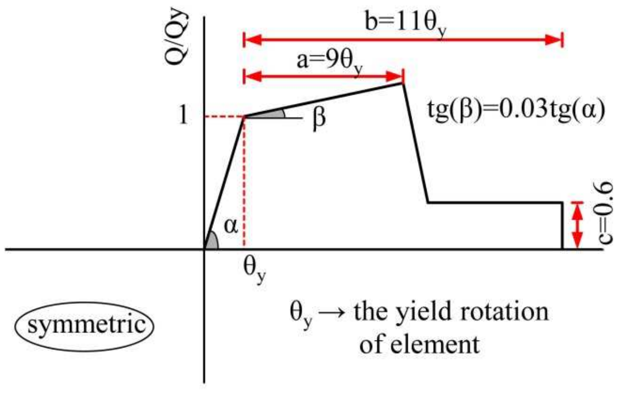
Columns		Beams	
ID	Section	ID	Section
C_0	500.20	B_0	300.10-150.20
C_1	400.15	B_1	300.10-150.15
C_2	300.15	B_2	300.15-180.15
C_3	250.15	B_3	350.10-250.25
C_4	200.15	B_4	350.10-200.25
C_5	350.15	B_5	270.10-180.20
C_6	400.20	B_6	240.10-150.20
C_7	300.20	B_7	270.10-150.20
C_8	200.20	B_8	200.10-150.15
C_9	270.15	B_9	350.10-180.20
C_{10}	240.15	B_{10}	350.10-200.20
-	-	B_{11}	300.10-200.20

Table 2. Vibration period and effective translational mass coefficient of the first five vibration modes of the case study buildings in X and Y directions

Mode No.	5-Storey building			10-Storey building		
	T(s)	M_x(%)	M_y(%)	T(s)	M_x(%)	M_y(%)
1	1.07	0	77.22	1.76	0	73.54
2	1.07	77.22	0	1.76	73.54	0
3	1.06	0	0	1.47	0	0
4	0.38	0.03	13.92	0.68	11.72	0.73
5	0.38	13.92	0.03	0.68	0.73	11.72

4. Modelling nonlinear behaviour

The PERFORM-3D [35] software is used for modelling and analysis of the structures in nonlinear range of behaviour as 3D systems. Considering rigid beam to column connections of the studied structures, only the two ends of the beams and columns will be prone to the development of plastic joints. Thus, concentrated plastic hinges at both ends of the beam and column members are modelled utilising M and P-M-M interaction diagrams, respectively. For the modelling of beams and columns in the software, the generalized load-deformation depicted in Fig. 5 is adopted. The parameters a, b and c in this figure are attained from the acceptance criteria of steel members for nonlinear methods as suggested by ASCE 41-17 [3]. The contribution of the non-structural components on the overall structural behaviour of the frames are neglected in this study. For non-linear dynamic analyses, Rayleigh damping model is used with a constant damping ratio of 0.05 assigned to the first mode and to the mode at which the cumulative mass participation exceeds 95%.



Performance Levels →	IO	LS	CP
	θ=1.25θy	θ=10θy	θ=12θy

Figure 5. Force-deformation curve for steel members

5. Seismic reliability analysis of the lateral load bearing system

For nonlinear analyses, the following equation is used to calculate the upper limit of the gravity loads Q_G in accordance with ASCE 41-17 [3]:

$$Q_G = 1.1[Q_D + 0.2Q_L] \tag{11}$$

where Q_D and Q_L represent dead and live loads, respectively.

In this study, fragility curves are used to evaluate seismic reliability of the studied structures at different levels of seismic intensity. The fragility curves have been obtained by using dynamic incremental analysis for constant performance levels (Immediate Occupancy, Life Safety and Collapse Prevention) of the structures (IM-Based) [36], where structures are initially subjected to incremental dynamic analysis using the possible ground motions.

To perform incremental dynamic analysis, according to the soil conditions of the site (375(m/s) ≤ Vs ≤ 750(m/s)), 12 pairs of accelerograms are taken from the PEER database [37]. The selected records fall in the class of far-fault ground motions. Among the horizontal components of each earthquake, the one which has higher spectral acceleration in the range of vibration frequency of the structures has been selected as the main component of the earthquake for incremental dynamic analysis. Specifications and features of the selected components are described in Table (3). Spectral accelerations of the opted components are also compared in Fig. 6, after scaling based on the maximum peak ground motion.

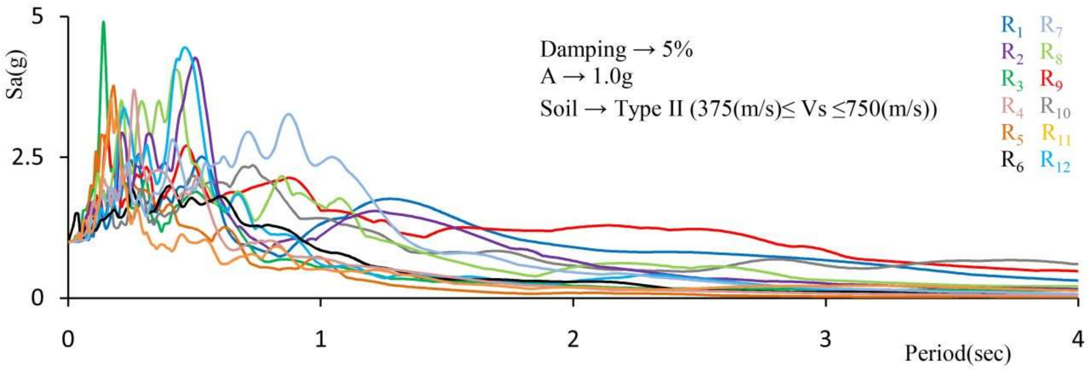


Figure 6. Spectral acceleration of selected records for incremental dynamic analysis

In the present study, the peak ground motion (PGA(g)), is opted as the intensity measure (IM) and the maximum relative lateral displacement of the storeys (inter-storey drift) is selected for the damage measure (DM) of incremental dynamic analysis [38]. Figs. 7 and 8, demonstrate the curves resulting from the IDA analysis, along with a quantitative values corresponding to different performance levels for the studied structures [3].

Table 3. The selected ground motion records that are consistent with the site condition to produce modified accelerograms for IDA analysis

Record No.	Earthquake& Year	Station	R[a] (km)	Component	Mw	PGA(g)
R_1	Imperial Valley, 1979	El Centro Array#11	29.4	230	6.5	0.38
R_2	Chi Chi(Taiwan), 1999	TAP095	109.01	90	7.6	0.15
R_3	Loma Prieta, 1989	CDMG58224	72.2	290	6.9	0.24
R_4	Loma Prieta, 1989	CDMG58472	74.26	270	6.9	0.26
R_5	Kobe (Japan), 1995	HIK	95.72	0	6.9	0.14
R_6	Loma Prieta, 1989	CDMG58223	58.65	0	6.9	0.23
R_7	Manjil (Iran), 1990	Qazvin	49.97	336	7.4	0.13
R_8	Loma Prieta, 1989	Capitola	27.0	0	7.1	0.53
R_9	Landers, 1992	Yermo Fire Station	86.0	270	7.3	0.24
R_{10}	Duzce (Turkey), 1999	Bolu	41.3	90	7.1	0.82
R_{11}	Imperial Valley, 1979	Delta	33.7	352	6.5	0.35
R_{12}	Northridge, 1994	Canyon Country-WLC	26.5	270	6.7	0.48

[a] Closest Distance to Fault Rupture

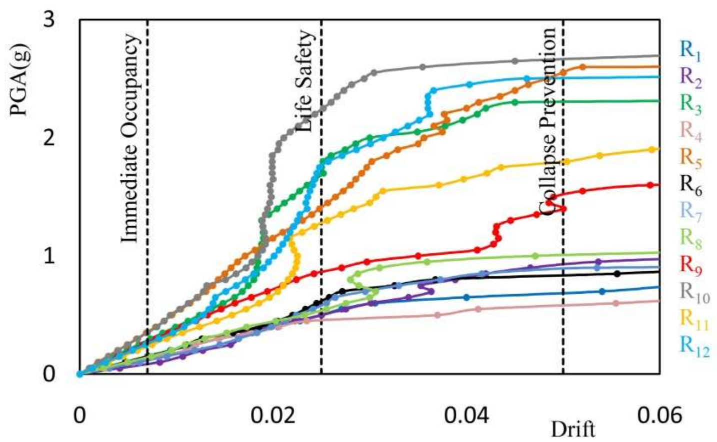


Figure 7. Curves derived from incremental dynamic analysis for 5-storey structure

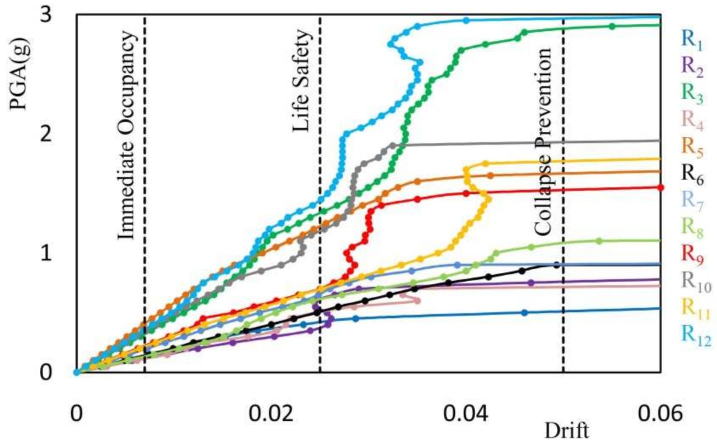


Figure 8. Curves derived from incremental dynamic analysis for 10-storey structure

To calculate the fragility curves based on IM-Based method, the maximum acceleration values corresponding to a given performance level are taken from the curves derived from incremental dynamic analysis. In the next step, by assuming a lognormal distribution for the recorded values and after calculating the mean values and standard deviations of the results at this damage level, a probability density function is extracted. According to Fig. 9, by substituting a value for X_0 as a certain intensity level, the area under the curve of the probability density function from -∞ to X_0 indicates the failure probability or fragility of the structure. This means that at this level of intensity, the structure will experience the considered performance level with a probability of P [39]. Consequently, $1 - P$ indicates the reliability of the system. For the studied structures, fragility curves are extracted using the above-mentioned approach based on the process described in Fig. (10).

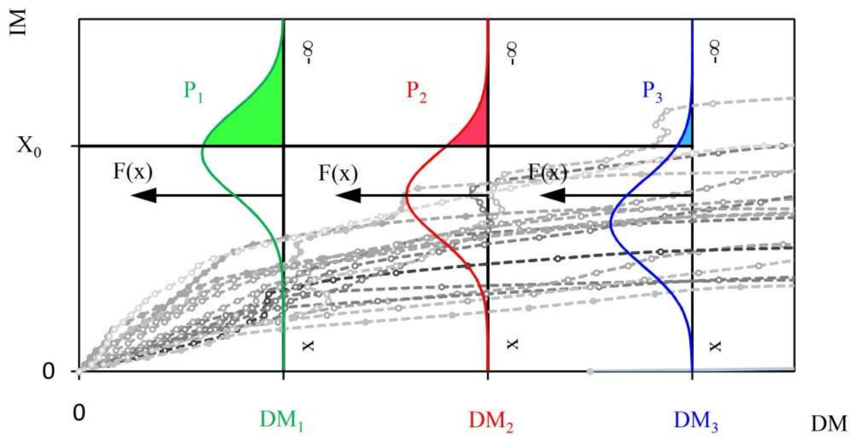


Figure 9. Probability of exceedance of responses from specific performance levels at a specified intensity "X_0"

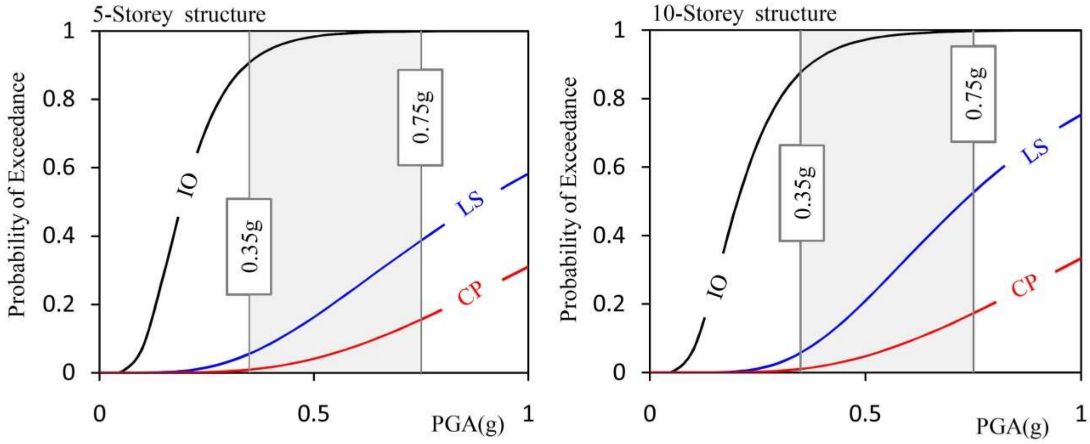


Figure 10. Fragility curves for the 5 and 10-storey studied models

The reliability of the lateral load system is measured at several intensity levels according to Table (4). It should be noted that the selected intensity levels are optional and can be replaced by other values.

Table 4. Reliability of the lateral load bearing system of the structures for different intensity and performance levels (%)

Selected intensity levels	5-Storey structure			10-Storey structure		
	Performance levels			Performance levels		
	IO	LS	CP	IO	LS	CP
PGA_A=0.35g	9.22	94.50	99.14	12.40	94.20	98.93
PGA_B=0.55g	0.97	79.42	94.20	1.72	72.64	93.26
PGA_C=0.75g	0.12	61.34	84.46	0.28	47.43	82.70
PGA_D=1.0g	0	41.83	69.00	0.04	24.75	66.62

Under the (PGA_A), the reliability of the lateral load bearing system in the studied models for the selected design performance level (i.e. Life Safety) is approximately 94%. However, it should be noted that the overall seismic performance of a building is affected by the performance of both structural and non-structural elements especially in Immediate Occupancy and Life Safety performance levels. As discussed before, extensive damage to non-structural components can potentially violate the target performance objectives as widely observed in the Kermanshah earthquake 2017 earthquake in Iran. This indicates that in addition to the level of reliability of the lateral load bearing system, non-structural components also require a high degree of reliability to ensure the satisfactory performance of the buildings. It should be noted that for higher levels of intensity such as (PGA_D), although the reliability of the lateral load bearing system is reduced at the Life Safety performance level and damage to non-structural elements is inevitable, collapse of non-structural elements should be prevented.

5. Estimation of the minimum design acceleration for non-structural components

In this study, two different scenarios are adopted to estimate the seismic demand of non-structural components and to assess the relationships described in Section (1-1). In the first scenario, the approximate acceleration applied to non-structural components under the intensity corresponding to a certain hazard level (in this study, the design hazard level with a return period of 475 years) is extracted. To achieve complete consistency of the utilized accelerograms with seismicity conditions of the site, all of the records introduced in Table (3) are modified according to the site design spectrum (return period of 475 years) using wavelet transformation method [40]. In this method, the records are transferred to the wavelet space, a baseline correction is then performed to match their response to the predefined target spectrum, and finally they are reverted to the time space. This operation is iterated until it reaches the acceptable level of accuracy. While this method generally preserves the nonstationary character of the reference records, the frequency content of the accelerograms are changed to provide the best match. Fig. 11 confirms the perfect good agreement between the response spectrum of the modified records and the site demand spectrum. For each accelerogram, the maximum absolute acceleration responses along the height of the structure are recorded. The recorded accelerations in this scenario are based on site seismicity and therefore are named demand accelerations (A_{Demand}).

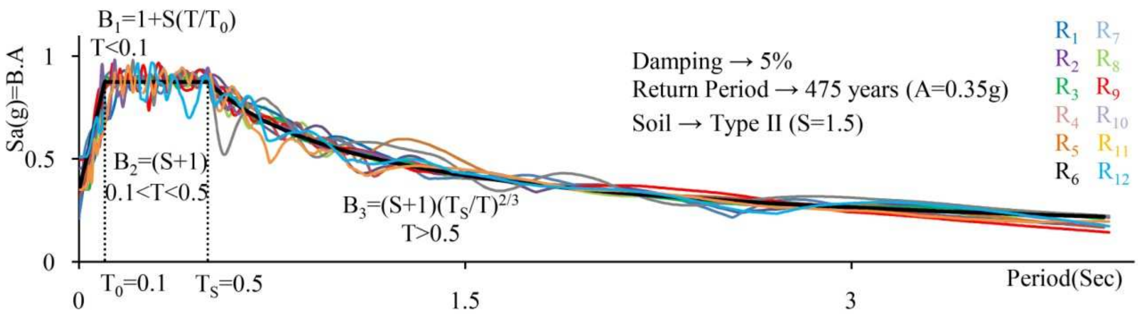


Figure 11. Comparison of the modified records produced using site demand spectrum

In the second scenario, applied accelerations to the non-structural components at a specific performance level of lateral load system (in this study, Life Safety level) are determined. In this approach, the accelerograms are scaled based on the earthquake intensity corresponding to the Life Safety performance level extracted from the incremental dynamic analysis (IDA) results (see Figs. 7 and 8). Subsequently, the scaled accelerograms are applied to the structure and the maximum absolute acceleration responses along the height of the structure are recorded. The recorded accelerations in this scenario are based on the capacity of the lateral loading bearing system of the structure, and accordingly are termed the capacity acceleration (A_{Supply}). It should be mentioned that the contribution of the non-structural components on the overall structural performance of the systems were considered to be negligible in the calculation of the IDA curves.

The absolute storey accelerations of 5 and 10-storey structures using the above mentioned scenarios are depicted in Fig. 12. In this figure, A_{demand} represents the minimum acceleration that is obtained from the code based design spectrum (or scaled accelerograms) with the predefined probability of exceedance, while A_{supply} is the maximum acceleration which comes from maximum performance point obtained from IDA curves using an accepted probability of exceedance.

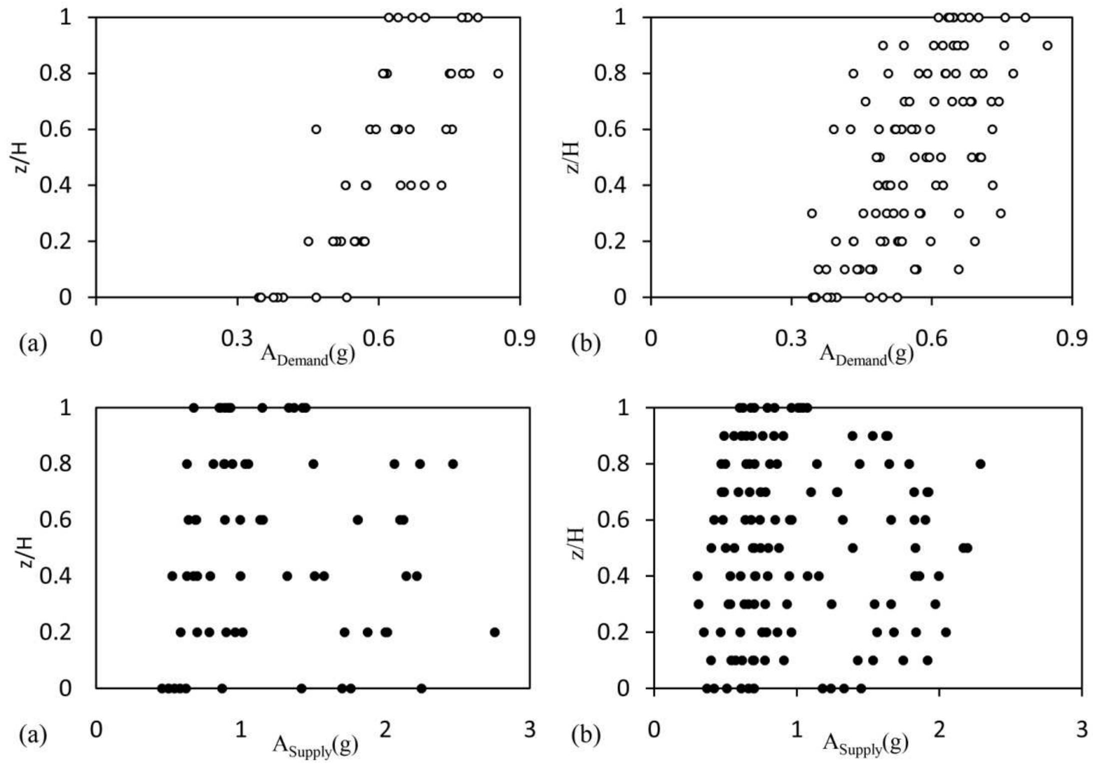


Figure 12. Absolute storey accelerations (a) 5-storey structure (b) 10-storey structure

For better comparison, Fig. 13 shows the values corresponding to 16th, 50th and 84th percentiles at each storey level, specified by using Rosenblueth method and assuming normal distribution for the recorded accelerations in the previous step [41]. Finally, for each scenario, the distribution of the absolute acceleration at the height of the studied structures is approximated by the following equation, which is derived by linear fitting of the resulting values (see Fig. 14).

$$A(g) = A_0\left[1 + k\left(\frac{Z}{H}\right)\right] \tag{12}$$

In above equation, parameters A_0 and k are the maximum accelerations at ground level (base level) and the coefficient which determines the slope of the best fitting line, respectively. The parameter $(\frac{Z}{H})$ denotes the relative height, while z and H are common with Equation (1). In the present study, the parameters A_0 and k are derived for various states and reported in Table (5).

Table 5. Quantitative values of A_0 and k for the best fitted lines

Percentiles	5-Storey				10-Storey			
	Demand		Supply		Demand		Supply	
	A_0(g)	k	A_0(g)	k	A_0(g)	k	A_0(g)	k
16%	0.35	1	0.45	0.88	0.35	0.76	0.38	1.17
50%	0.43	0.83	1.06	0.36	0.43	0.60	0.83	0.49
84%	0.52	0.76	1.65	0.24	0.51	0.54	1.25	0.35

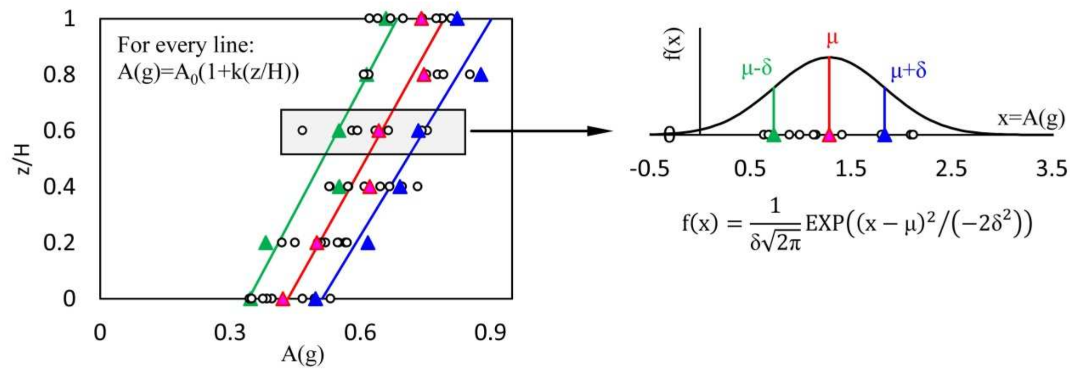


Figure 13. The process of determining percentiles for absolute acceleration values at each storey level and the linear regression of each percentile in height

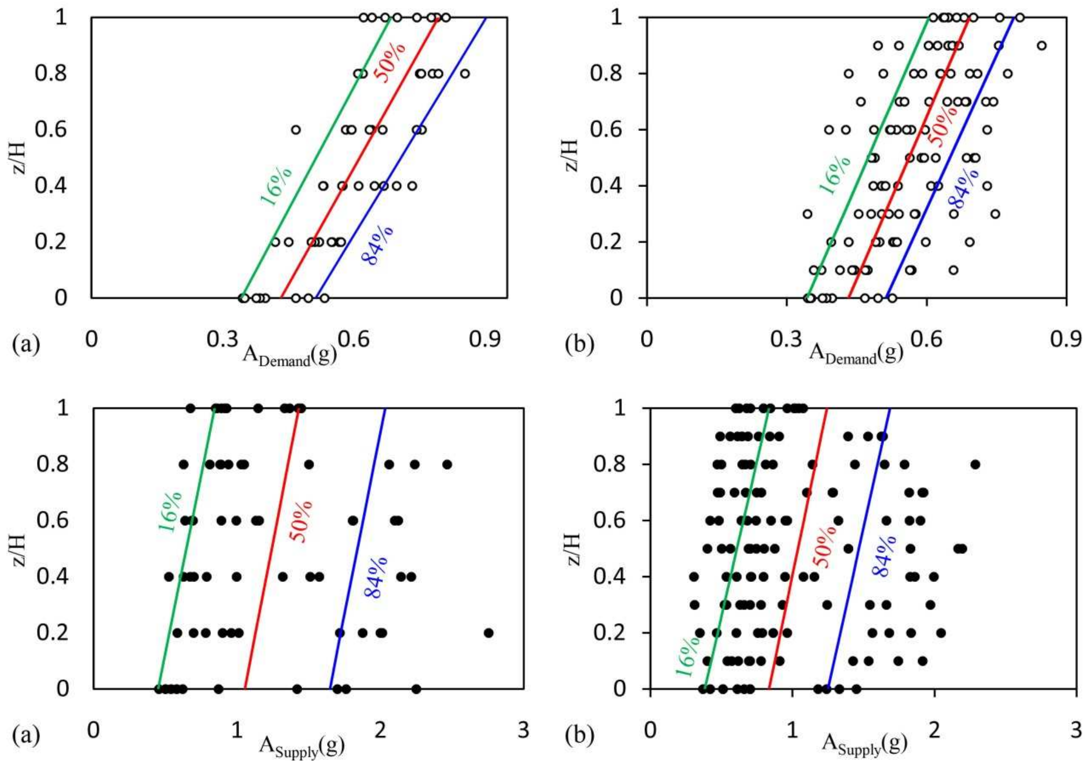


Figure 14. Fitted lines at 16^{th}, 50^{th} and 84^{th} percentiles at the height of the structure: (a) 5-storey structure, (b)10-storey structure

In the proposed approach, it is suggested to use the 16^{th} percentile of the linear distribution of absolute demand accelerations as the minimum acceleration for design of non-structural components. It is expected that such selection leads to a safe design solution for the non-structural components under the corresponding intensity level (here PGA=0.35g). Using design accelerations below this value will be outside the safe limits and will underestimate the lateral design loads for non-structural components.

For the maximum design acceleration, the 84th percentile of the linear distribution of the absolute capacity accelerations applied to the lateral load system at the height of the structure is proposed. In this condition, it is expected that under the intensity corresponding to the selected performance level of the lateral load bearing system, the non-structural components remain functional. Using higher acceleration values for the design of non-structural components will not be cost-effective unless higher levels of intensity or lower damage levels are considered for the lateral load carrying system.

It is shown in Fig. 15 that using design values between the 16th percentile of the demand accelerations and the 84th percentile of the capacity accelerations (the grey shaded area) leads to safe design solutions for non-structural components. The designer can choose the best component acceleration in this range according to the fragility curve for the structure and the required reliability index for the system at a certain level of intensity (see Fig. 10 and Table (4)).

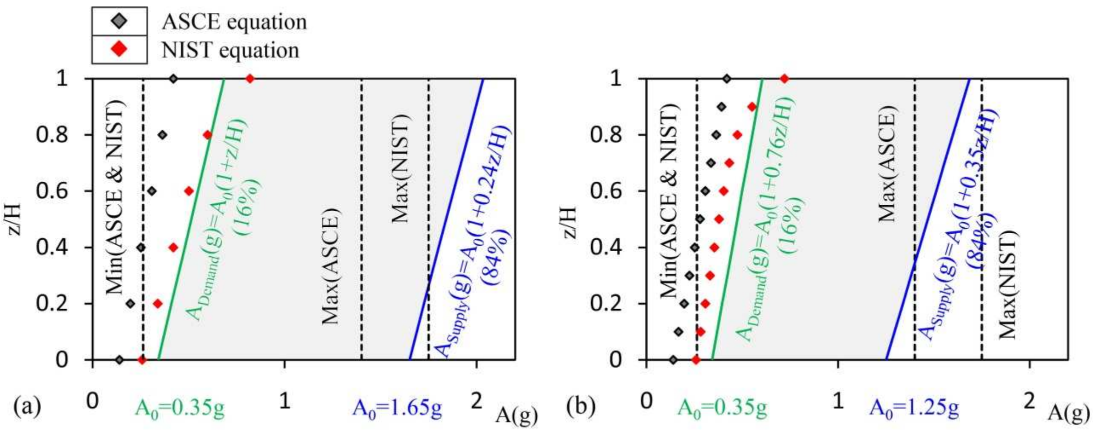


Figure 15. Safe range for design acceleration and values derived from the relationships proposed by ASCE and NIST, (a) 5-storey structure, (b) 10-storey structure

As it is evident in Fig. 15, although the reliability of the relationship provided in NIST [31] is higher than the equation suggested by ASCE 7-16 [22], the estimated values by both relations are considerably less than the demand values calculated for the structures studied here. Using the extracted acceleration from the linear fit of the 16th percentile of the demand values, it can be seen that the absolute acceleration in the storey levels of the 5th and 10th structures are 1.8 and 1.7 times higher than ASCE 7-16 [22] estimated values, respectively. Similarly, these accelerations are 1.7 and 1.5 times higher than estimations of the equation proposed by NIST [31]. It means that the estimated accelerations by ASCE 7-16 [22] and NIST [31] proposed equations are up to 80% and 50% lower than the minimum demand accelerations calculated for the selected structures under the design hazard level (return period of 475 years). This underestimation considerably increases (up to 2.5 times lower than the actual values) by using the extracted accelerations from the linear fit of the 50th and 84th percentiles. It is also shown in Fig. 15 that the extracted accelerations from the linear fit of the 16th percentile of the capacity values are up to 120% and 60% times higher than the ASCE 7-16 [22] and NIST [31] estimated values, respectively. This means that under the minimum acceleration that reaches the structural system to Life Safety performance level, the acceleration applied to non-structural components can be up to 2.2 times higher than the values estimated by these standards. Similar to the previous case, by using the extracted accelerations from the linear fit of the 50th and 84th percentiles, these errors considerably increase and the predictions can be up to 7 times lower than the actual values.

The above findings demonstrate the inadequacy of these standard code relationships to predict the minimum acceleration and accordingly applied forces to non-structural components, especially in the cases when high intensity levels or medium performance levels for lateral load bearing system are considered.

The results in Fig. 15 indicate that the maximum acceleration values proposed by ASCE 7-16 [22] are sufficiently safe for the demand earthquake. For the capacity earthquake and medium performance levels of lateral load carrying system, using the maximum values suggested by NIST [31] are also acceptable. Please see Appendix A for all the input parameters.

It is possible to predict the safe region for estimating the acceleration applied to non-structural components in the height of the structure by combining the curves presented in Fig. 15. In order to increase the reliability margin, the obvious choice is to select equations which result in higher demand and lower capacity accelerations. For the studied structures, the coloured area specified in Fig. 16 represents the appropriate range. In this case, the design equation will be in the general form of Equation (11), in which the coefficient K is equal to 1 and A_0 is selected between (0.35g) to (0.75g) according to the fragility curves and seismic reliability analysis of the lateral load system (see Fig. 10 and Table (4)). The lower bound is the acceleration corresponding to the typical design hazard level suggested by most standards (return period of 475 years). The upper bound is the acceleration corresponding to the reliability of nearly 50% for the Life Safety performance level, where the lateral load bearing system has passed the Immediate Occupancy level with a high probability but it is more than 80% reliable for the collapse prevention performance level. In fact, the upper bound of A_0 range is selected based on the minimum intensity corresponding to the expected performance level of the system.

The coefficient K can be varied within its permissible range based on the designer decision. For example, considering the various economic and safety aspects, choice of K between 0.4 and 0.7 leads to approximations close to the 84th percentile of demand and 50th percentile of capacity.

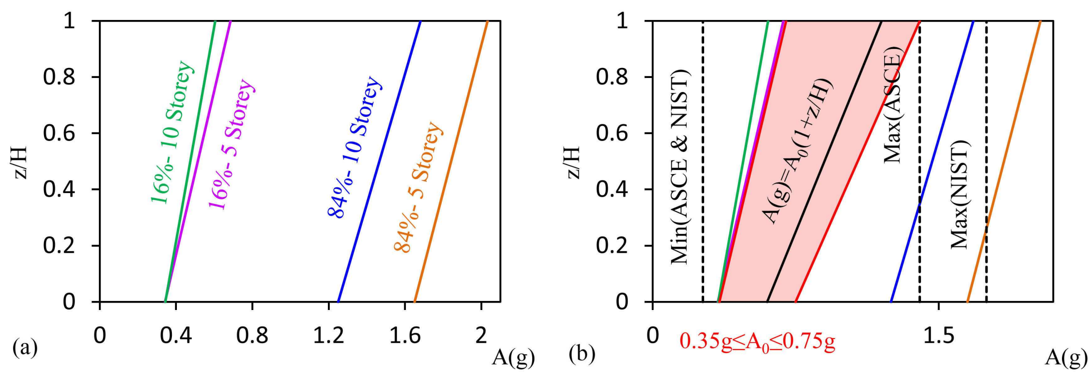


Figure 16. (a) Comparison of demand and capacity equations for the studied structures and the proposed range; (b) Safe regions compared to the code suggestions

It should be noted that the above results are expected to change with the change of the properties of structures such as lateral load system, plan and height. However, the proposed design equation is general and optimum values of A_0 and k can be easily obtained for each case. Since the acceleration distribution is linear, the absolute accelerations are approximated in the floor level, and the behaviour of non-structural components is assumed to be rigid, the product of the acceleration of the mass of the components can be considered as the lateral force of the non-structural component provided that the magnification of accelerations transferred from storey to non-structural components is neglected. In cases where non-structural elements are not rigid, it is suggested to calculate the acceleration at the elevation of their mass centres using the proposed equation.

The proposed multilevel approach can be considered as an important step toward the inclusion of reliability principles and performance-based design concepts in the new generation of seismic design regulations for non-structural components. While the general methodology proposed in this study to predict the acceleration demand of non-structural components can be easily applied to different structural systems, some of the presented results may be changed by the variation of the lateral load carrying system and geometry of the structures. To simplify the proposed method for adoption in current design standards, the incremental dynamic analyses (IDA) required to calculate fragility curves can be replaced by nonlinear pushover analyses. Alternatively, the reliability analyses can be performed based on less computationally expensive methods such as EDP-based method [36]. However, the accuracy of such approaches should be carefully investigated for different structural systems and case study examples.

6. Summary and Conclusion

In this study, a novel multilevel approach was developed to predict the seismic demand of acceleration-sensitive non-structural components. Unlike most conventional methods, the proposed approach can take into account the intensity of the design earthquake and the expected performance level of the lateral load bearing system. Through a comprehensive reliability study on 5 and 10-storey steel frame structures, the efficiency of the proposed method was demonstrated compared to the conventional seismic design code relationships. Based on the presented results, the following conclusions can be drawn:

- The estimated accelerations by ASCE 7-16 [22] and NIST [31] proposed equations were up to 80% and 50% lower, respectively, than the demand values calculated for the selected structures under the design hazard level. Besides, under the minimum acceleration at which the main structural system reaches Life Safety performance level, it was shown that the acceleration applied to non-structural components could be up to 2.2 times higher than the values estimated by these standards. Therefore, the code suggested methods can lead to unsafe design solutions for acceleration-sensitive non-structural components. This indicates that the influence of the main structural system on the maximum acceleration transferred to the non-structural component (e.g. due to non-linear behaviour and higher mode effects) cannot be captured accurately in these methods.

- For the studied structures, the initial capacity acceleration was greater than the initial demand acceleration for all percentiles. This indicates the safety of the lateral load bearing system under intensities even greater than the intensity of the design earthquake. However, the safety of non-structural components was only achieved for acceleration values between the maximums of the 16^{th} percentile of demand and the minimums of the 84^{th} percentile of capacity. This can be due to the non-ductile behavior of non-structural components, which is not adequately considered in the current design equations.

- It was shown that the minimum and maximum accelerations proposed for the relationships presented in ASCE 7-16 [22] and NIST [31] standards are generally ineffective. To address this issue, two new parameters were defined namely, absolute demand acceleration (A_{Demand}) and absolute capacity acceleration (A_{Supply}), which could be based on the intensity of the design earthquake and expected performance level of the lateral load system, respectively.

- By conducting a comprehensive reliability study, a new design equation was proposed to estimate the maximum accelerations (or forces) applied to non-structural components. Despite its simplicity, the proposed equation can take into account the effects of different earthquake intensity levels and performance targets, and therefore, it is suitable for performance-based design purposes. While the accuracy of the proposed equation may be limited to the developed models and design assumptions, the general equation can be easily adopted for other structural systems using the fragility curves corresponding to different performance levels.

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Appendix A:

Table A-1- Value of the parameters used in ASCE formulation

Parameter	Value	Description
a_p	1	Amplification factor for the exterior walls and connections
R_p	2.5	Response modification factor for the exterior walls and connections
A	0.35g	Base ground acceleration for the site with high seismicity
I_p	1	Importance factor for residential building, in which it is not expected that the non-structural component remain in Immediate Occupancy (IO) performance level after earthquake
B_s	2.5	Corresponding to site soil type with shear wave velocity varying between 375 to 750 m/s

Table A-2- Value of the parameters used in NIST formulation

Parameter	Value	Description
A	0.35g	Base ground acceleration for the site with high seismicity
R	5	Response modification factor for intermediate steel frame
Ω_0	3	Overstrength factor for intermediate steel moment frame
T_{abldg}	$0.08(H)^{0.75}$	The actual first mode period for the building as a function of building height, H, in meter
R_{pocomp}	1.3	The inherent component reserve strength margin factor
I_p	1	Importance factor for residential building, in which it is not expected that the non-structural component remain in Immediate Occupancy (IO) performance level after earthquake