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Simplified methods for seismic assessment of existing buildings

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Abstract. Besides the complex instructions of guidance documents for seismic rehabilitation of existing buildings, some institutions have provided simple criteria in terms of simplified rehabilitations.

ASCE 41-06 is one of documents that introduced a simple method for assessment of certain buildings that do not require advanced analytical procedures. Furthermore the New Zealand guideline has presented a simple lateral mechanism analysis that is a hand static analysis for determining the probable collapse mechanism, lateral strength and displacement capacity of the structure. The present study is focused on verifying the results of the simplified methods which is used by NZSEE and ASCE 41-06 in assessment of existing buildings. For this, three different special steel moment and braced frames are assessed under these two guidelines and the accuracy of the results is checked with the results of nonlinear static and dynamic analysis. After comparison of obtained results, suggestions are presented to improve seismic retrofit criteria.

Keywords: pushover analysis; nonlinear dynamic analysis; moment frames; braced frames; peak ground acceleration failure (PGAf)

1. Introduction

The nonlinear time history analysis of structures is one of the most prominent tools in determining of damage curves for a group of special earthquakes that are extremely time-consuming. So many researchers have been providing simple methods to solve these problems due to the complexity and time-consuming.

It should be noted that the use of complicated methods is not only way to confirm the accuracy of the assessment results and it is possible to achieve the same results with the use of simpler procedures such as simplified analysis methods with an acceptable margin of error.

Simplified criteria for seismic rehabilitation of existing conventional buildings based on various guidelines such as New Zealand guideline and ASCE 41-06 have presented. With regard to assumptions that are considered for Simplification in each of the simplified procedures, the

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comparison of these methods with precise approaches for ensuring the integrity of their responses is essential.

One of the purposes of this article is to present a simple method for seismic evaluation of structures that this approach has been proposed in NZSEE .Selected models for assessment by this method contain the steel moment and concentric braced frame structural system. These models also are assessed by the simplified method of ASCE 41-06.

Nonlinear static and dynamic analysis procedures are applied to assess the accuracy of the seismic performance of structures according to these guidelines. At the end the results are used to improve these guidelines and recommendations are suggested to eliminate uncertainties in those methods.

2. Brief review of simple analytical methods and researches for simplified seismic assessment

The simplified rehabilitation method is less complicated than the complete analytical rehabilitation design procedures found. In many cases, this method represents a cost-effective improvement in seismic performance, and often requires less detailed evaluation or partial analysis to qualify for a specific performance level. FEMA 178, the NEHRP handbook for the seismic evaluation of existing buildings, was the basis for the simplified rehabilitation method that different versions of it have been completed and new analysis techniques have been provided in ASCE 41-06. Simplified rehabilitation method that proposed by ASCE41-06 is intended primarily for use on a selected group of simple buildings being rehabilitated to the Life Safety Performance Level. The term “Simplified Rehabilitation” is intended to reflect a level of analysis and assessment that is appropriate for small, regular buildings and buildings that do not require advance analytical procedures.

Another guidance document for seismic assessment of existing buildings is NZSEE2006 recommendations in New Zealand. Three possible approaches for performing the assessment have been indicated in this document; time history analysis, force analysis and displacement analysis. The first one is the most accurate but the most complex as well, so the others are considered as the main approaches for assessments. In both cases, with simplified consideration of capacity issues, the probable collapse mechanism, lateral strength and displacement capacity can be determined by Simple Lateral Mechanism Analysis (SLaMA). In displacement-based methods, expected displacement demand is based on the structure characteristics (effective stiffness and equivalent viscous damping) at maximum displacement capacity rather than initial elastic characteristics in force based methods and a displacement spectra-set for different levels of elastic damping is used rather than the acceleration spectra set of force-based design. In the document it is stated that “the displacement based approach is generally considered to produce more rational and less conservative assessment outcome but the force based one is more familiar to designers”.

In recent years the aim of many researchers was to evaluate the accuracy of the proposed simplified procedures for the seismic assessment of structures.

Moshref *et al.* (2011, 2011a) carried out a comparison on the results of the evaluation methods proposed by ASCE 41-06 and NZSEE 2002. The benchmark comparison was the results of the incremental dynamic analysis. At the end, it was concluded that the New Zealand force approach has the most compatibility with the nonlinear dynamic analysis.

Tehranizadeh and Yakhchalian (2011) assessed the results of the displacement based and consolidated force displacement based methods. Consolidated force-displacement based method is a combination of force based and displacement based methods. It concluded that for structures with lower height, displacement based method gives more rational results, but as the height of structure increases, the results of consolidated force-displacement based method are more acceptable.

The displacement based assessment procedure assessed by Kam *et al.* (2013) as a practical and effective seismic assessment tool. It is noted that displacement-based assessment may be achieved using direct hand calculation methods (Priestley 1996, Priestley *et al.* 2007, NZSEE 2006) or sophisticated non-linear computer analysis. The focus of the paper is on the practical implementation of the hand-calculation method for realistic buildings of complex configurations.

Borzi *et al*. (2008) defined the nonlinear behavior of RC buildings through a simplified pushover and displacement-based procedure that combines the definition of a pushover curve using a simplified mechanics based procedure (similar to that proposed by Cosenza *et al*. 2005) to define the base shear capacity of the building stock, with a displacement-based framework similar to that in NZSEE. For estimating of seismic demands Borzi *et al*. (2013) in continuation of their previous studies implemented a simplified methodology in simplified pushover-based earthquake loss assessment approach and validated the accuracy of obtained results against the results of more sophisticated nonlinear dynamic analyses for RC buildings.

Grande and Rasulo (2013) proposed a simplified approach for the seismic assessment of concentric steel braced frames (CBFs) according to the displacement based method. In the first step of the approach the possibility of activation of a plastic mechanism controls by the yielding of diagonals using the simple considerations. After that the approximate capacity curve of the CBFs is determine. Finally, the third step of the approach consists of to assess the braced frame according to the DB procedure. In 2015 Grande and Rasulo enhanced their assessment method in order to include also the retrofit in the procedure.

Piazza and Sullivan (2014) proposed a simplified displacement-based procedure for the seismic design and assessment of RC frame structures based on Priestley *et al*. (2007) and Pinho *et al*. (2007). In this way, no estimate is required of the building strength, stiffness or period of vibration, thereby greatly simplifying the task of seismic assessment. Proposals for simplified DBD have already been made by Sullivan (2013, 2013a) but this paper aims for an even more simplified approach.

Fox (2015) formulated a simplified displacement-based seismic assessment procedure to permit the rapid seismic assessment of reinforced concrete (RC) wall buildings. The displacement capacity, shear capacity and shear demand are also estimated simply, using newly developed equations that are a function of wall geometry and material properties.

Lignos *et al*. (2015) evaluated the effectiveness of single and multi-mode nonlinear static procedures as well as the FEMA P58 simplified approach versus rigorous nonlinear response history analyses for estimating seismic demands of steel special moment frames. This work indicated that the simplified analysis procedures in combination with commonly used nonlinear component models can reliably predict story level engineering demand parameters such as, story drift ratios, story shear forces, overturning moments, residual deformations and peak floor absolute accelerations.

The simple method of NZSEE that has been used in this study, is based on the displacement based design approach that proposed by Priestley *et al*. (2007). This method has been presented for concrete moment structures but it is applicable for steel moment frames. Also in this guideline for assessments of braced frames have not been specified recommendations in details.

In this study, we tried to present simplified methods for seismic assessment of steel moment and concentric braced frames by using of NZSEE and ASCE 41-06 recommendation.

At first, samples are assessed with the simplified methods proposed by the NZSEE and ASCE 41-06. After that, the nonlinear static and dynamic analysis was applied to assess the accuracy of seismic performance according to simple methods. For this, frames have been analyzed under the action of 56 Near-Field earthquakes with the use of incremental dynamic analysis to determine the LS Performance level, capacity curves and PGA values that cause their collapse. At the end these results have been compared by their similar values that were determined from the simple methods.

2. Simplified methods

2.1 Simple lateral mechanism analysis (SlaMA) of NZSEE

This method is a hand static analysis that is carried out to determine the probable collapse mechanism, lateral strength and displacement capacity with simplified consideration of capacity issues. So we can determine the capacity curve in terms of base shear versus roof displacement of structures. All steps are explained below;

2.1.1 Moment frames

The procedure starts with the evaluation of members capacities. The probable flexural strengths are calculated according to standard theories. The flexural strengths in beam and column can be calculated by the following equation respectively

$$M_b = ZF_{ye} \tag{1}$$

$$M_c = 1.18ZF_{ye}\left(1-\frac{P}{P_{ye}}\right) \tag{2}$$

Demand shears, $V_{bdr,l}$, in both sides of beams at the moment capacities are determined as

$$\begin{aligned} V_{bdl} &= V_{bgl} + \left(M_{bl} + M_{br}\right)/L_b \\ V_{bdr} &= V_{bgr} + \left(M_{bl} + M_{br}\right)/L_b \end{aligned} \tag{3}$$

Where V_{bgl} and V_{bgr} are shear force due to gravity loads in the left and right ends respectively, M_{bl} and M_{br} are plastic moment capacities of beam in the left and right ends respectively, and L_b is the length of beam.

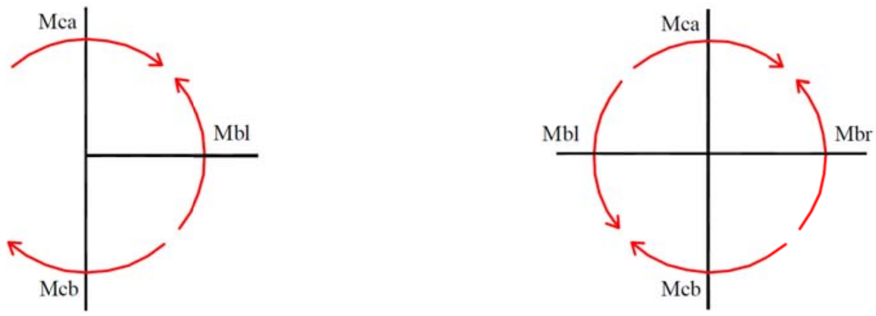
The probable shear capacity is defined by the following relationship

$$V_{br} = 0.55F_{ye}d_c t_p \tag{4}$$

Where d_c and t_p are the outside height and the web thickness of beam respectively.

The initial capacity of shear should be controlled by demand shear. If $V_{br}>V_{bd}$, the flexural capacity of beam in the left and right ends, M^*_{bl} and M^*_{br}, is modified as below respectively

$$\begin{aligned} M^*_{bl} &= \left(V_{brl} - V_{bgl}\right)/L_b - M_{br} \\ M^*_{br} &= \left(V_{brr} - V_{bgr}\right)/L_b - M_{bl} \end{aligned} \tag{5}$$



(a) External joints (b) Internal joints

Fig. 1 Equilibrium at the joint in the case of weak beams

The post-elastic critical mechanism is investigated next. To investigate whether plastic hinges occur in beams or columns, a sway potential index, *Si*, can be defined for the beam-column joints at a horizontal level by comparing the sum of the expected flexural strengths of the beams and the columns at the joints centroids

$$S_i = \frac{\sum (M_{bl} + M_{br})}{\sum (M_{ca} + M_{cb})} \tag{6}$$

Where M_{bl} and M_{br} are beam expected maximum flexural strengths at the left and right of the joint, respectively, at the joint centroid, and M_{ca} and M_{cb} are minimum expected column flexural strengths above and below the joint, respectively, at the centroid of the joint. These are summed for all the joints at that horizontal level. If S_i<0.85, the NZ document suggest that plastic hinges would develop in the beams and at the top and bottom of the column bases, (beam-sway collapse), otherwise they would develop in the columns (column-sway collapse).

Since bay width will normally exceed story height and column curvature is typically less than beam curvatures, beam flexibility is likely to be the major contributor to the deformation. On this basis the yield drift θ_y for a structural steel frame proposed by Priestley *et al*. (1995) according to Eq. (7)

$$\theta_y = 0.65\varepsilon_y \frac{L_b}{h_b} \tag{7}$$

So the roof yield displacement is calculated as below

$$\Delta_y = h\theta_y \tag{8}$$

Where ε_y is the steel yielding strain, L_b is the beam span, h is the total height of building and h_b is the beam section height.

For the evaluation of the structural ductility, μ_{sc}, there is a simple equation based on plastic and yield rotation for the cases of beam sway and column sway mechanisms.

For beam sway

$$\mu_{sc} = 1 + \frac{\theta_p}{\theta_y} \qquad For \quad n \le 4 \tag{9}$$

$$\mu_{sc} = 1 + \frac{(0.64 - 0.0125(n-4))\theta_p}{0.64\theta_y} \qquad For \qquad n > 4 \tag{10}$$

For column sway

$$\mu_{sc} = \frac{\left(0.72 + \frac{2\theta_p}{n\theta_y}\right) + \sqrt{\left(0.72 + \frac{2\theta_p}{n\theta_y}\right)^2 + 1.12}}{2} \tag{11}$$

Where θ_p is the plastic rotation at the top and bottom of columns in the soft story that could be calculated by FEMA356 and n is the numbers of stories. So the displacement capacity is calculated from the following equation

$$\Delta_d = \mu_{sc}\Delta_y \tag{12}$$

The lateral seismic forces cause base moments and axial forces in columns. For a regular building, seismic axial forces will be induced in the exterior columns by the seismic beam shears (V_{bdi}). If the beam negative moment capacities at all critical sections at a given level be equal, and similarly, all beam positive moment capacity at that level be equal (but not necessarily equal to negative moment capacities), for the interior columns, the axial force component from the beam shears at opposite sides of columns will be cancel, and no seismic axial force will be induced. The overturning moment induced by external forces must be equilibrated by the internal forces, thus

$$OTM = \sum_{j=1}^{m} M_{cj} + TL_{base} \tag{13}$$

$$T = \sum_{i=1}^{n} V_{bdi} \tag{14}$$

Where M_{cj} are the column base moments, T=C are the seismic axial forces in the exterior columns, L_{base} is the distance between T and C, and m is the number of base columns.

The base shear capacity could be calculated as below

$$V_{base} = \frac{OTM}{h_{eff}} \tag{15}$$

For frames with beam sway collapse mechanism, the effective height, h_{eff}, of the SDOF structure is given by the following relationship

$$h_{eff} = 0.67h \qquad For \qquad n \le 4 \tag{16}$$

$$h_{eff} = 0.64h \qquad For \qquad n > 4 \tag{17}$$

In the column sway, the effective height is affected by the general structural ductility and is calculated by the following equation

$$h_{eff} = \left(0.64 - 0.14\frac{\mu_{sc} - 1}{\mu_{sc}}\right) \tag{18}$$

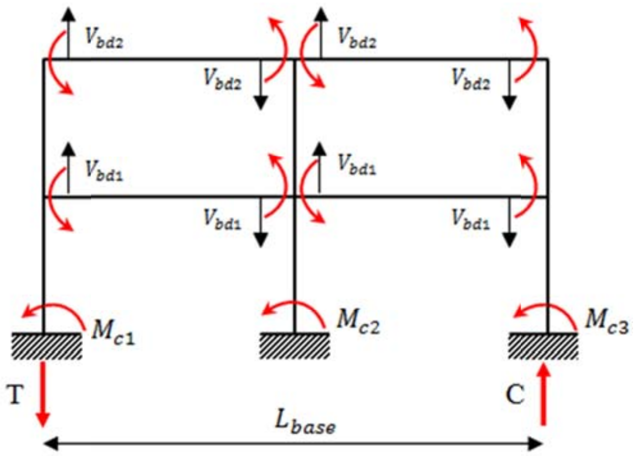


Fig. 2 Seismic overturning moment

2.1.2 Concentric braced frames

In this approach, a demand-capacity ratio analysis is carried out first to search for the location for the formation of the first plastic hinge is formed.

The demand-capacity ratio shall be calculated at any level

$$DCR_i = \frac{F_{Ci}}{F_{Di}} \tag{19}$$

Where F_{Di} is the base shear force at floor level i and F_{Ci} is the capacity of the bracing elements at floor level i which calculated such as Eq. (24) .In its simplest form, a ratio less than 1, implies failure at that level. But this ratio should be lower than the value for upper and bottom stories. If in calculations more than one floor have this property, the story that has the greatest difference with upper and bottom stories choose as the critical floor.

The demand-capacity ratio is calculated for the vertical distribution of the seismic forces which proposed by the ASCE 41-06 accordance with Eq. (20) at each floor

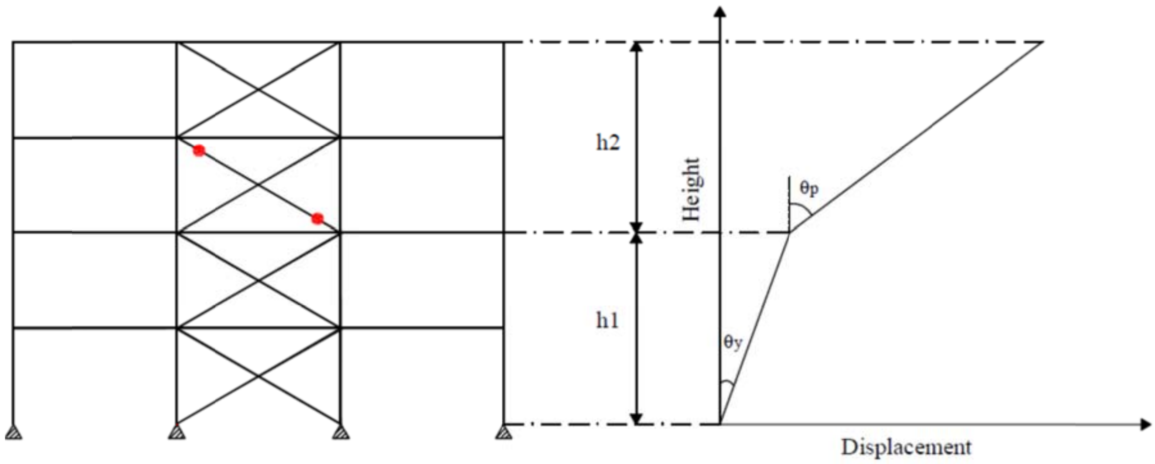
$$F_i = \frac{m_i h_i^k}{\sum_{j=1}^{n} m_j h_j^k} F_b \qquad \begin{array}{lll} k = 1 & for & T < 0.5 \\ k = 0.75 + T/2 & for & 0.5 \le T < 2.5 \\ k = 2 & for & T \ge 2.5 \end{array} \tag{20}$$

Where F_i is seismic forces at floor level i, F_b is the total base shear force, m_i is mass of floor level i, h_i is height of floor level i and T is the fundamental period of structure.

The yield displacement of a concentric braced frame is governed by the conditions to cause yielding of the bracing elements.

From geometry and assuming that strains in the beams and columns are negligible with respect to strains in the brace, the yield drift ratio is calculated by follow equation (see Fig. 4)

$$\theta_y = \frac{\Delta L_y \cos\theta}{h} = \frac{\frac{F_{ye} L_{brace}}{E}\cos\theta}{h} = \frac{\varepsilon_y L_{bay}}{h} \tag{21}$$



(a) Plastic mechanism (b) Displacement profiles

Fig. 3 Determining limit state displacement profile from mechanism

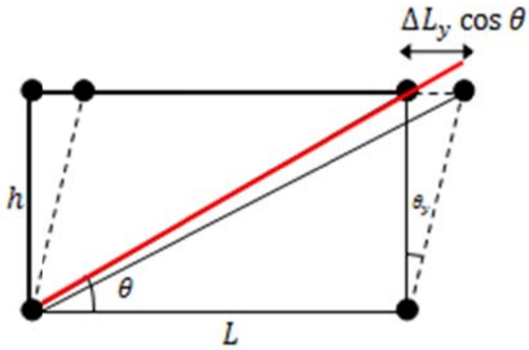


Fig. 4 Deformed shape of a story

Where L_{bay} is the length of the frame bay, and h is the story height. So the yield displacement is calculated by Eq. (8).

The plastic deformation of bracing elements is calculated with the use of ASCE 41-06 guideline that has offered relationships based on the axial deformation at expected tensile yielding load (Table 3). Tensile plastic deformation of braces for CBFs in different building performance levels is according to Table 5. The plastic rotation angle can be calculated by the following equation

$$\theta_p = \frac{\Delta L_P \cos\theta}{h} \tag{22}$$

With the use of Eqs. (21)-(22) and according to Figs. 3(a)-(b) the ultimate roof displacement is obtained as Eq. (23)

$$\Delta_d = h_1\theta_y + h_2\theta_p \tag{23}$$

To calculate the base shear, the axial tensile force and compressive force of the first floor in the horizontal orientation are obtained. The base shear is governed by

$$V_{base} = (P_T + P_C)\cos\theta \tag{24}$$

That P_T and P_C are the plastic tensile and compressive capacity of braces respectively. Where θ is the angle between the brace and horizontal line.

Chen (2011) and Uriz and Mahin (2008) in detail argued on the behaviour of the concentrically braced frame systems, mechanisms and lateral load distributions.

2.2 Simplified rehabilitation method of ASCE41-06

Simplified rehabilitation method that proposed by ASCE41-06, reflects a level of analysis and design that is appropriate for small, regular buildings and buildings that do not require advanced analytical procedures and achieves the Life Safety Performance Level. This method only applies to a select group of simple buildings that conform to the limitations of Table 1.

For assessment of buildings, a linear static analysis should be used. All steps are explained below.

1- The lateral seismic force, V, is calculated in accordance to Eq. (25)

$$V = S_a C W \tag{25}$$

Where W is effective seismic weight of the building including the total dead load and portion of live load, S_a is response spectrum acceleration, at the effective fundamental period of structure and C is a modification factor to relate expected maximum inelastic displacements to displacements calculated for linear elastic response obtained from Table 2.

Table 1 Limitation on use of the simplified rehabilitation method

Model Building Type	Maximum building height in stories by seismic zone* for use of the simplified rehabilitation method		
Model Building	Low	Moderate	High
Steel moment frame			
Stiff diaphragm	6	4	3
Flexible diaphragm	4	4	3
Steel braced frame			
Stiff diaphragm	6	4	3
Flexible diaphragm	3	3	3

*The zone of seismicity shall be defined as High, Moderate, or Low as specified in Sections 1.6.3 of ASCE41-06

Table 2 C-coefficient factor

Model Building Type	Number of stories			
	1	2	3	≥4
Steel Moment Frame	1.3	1.1	1	1
Steel Braced Frame	1.4	1.2	1.1	1

2- Distribution of the lateral seismic force at any floor level shall be determined in accordance with Eq. (26)

$$F_i = \frac{w_i h_i^k}{\sum_{j=1}^{n} w_j h_j^k} \qquad \begin{array}{l} k=2 \text{ for } T \geq 2.5 \text{ sec} \\ k=1 \text{ for } T \leq 0.5 \text{ sec} \\ \text{Linear interpolation for} \\ \text{intermediate values of } T \end{array} \tag{26}$$

Where w_i is portion of the total building weight W assigned to floor level i, w_j is portion of the total building weight W assigned to floor level j, h_i is height from the base to floor level i, h_j is height from the base to floor level j and T is the fundamental period of structure.

3- The design actions, Q_U, in members due to gravity loads, Q_G,and earthquake loads, Q_E, is calculated in accordance with Eq. (28)

$$Q_U = Q_G \pm Q_E \tag{26}$$

$$Q_G = 1.1[Q_D + Q_L] \tag{27}$$

Where Q_D is dead load and Q_L is Effective live load equal to 25% of the unreduced design live load.

4- Expected strength of members, Q_C, is calculated as design codes.

Table 3 Acceptance criteria for linear procedures-structural steel components

Component/Action	*m*-factors for Linear Procedures	
	LS	CP
	Columns - flexure	
	$P / P_{CL} < 0.2$ for	
a. $b_f \leq 0.3\sqrt{\frac{F}{F_{ye}}}$	6	8
b. $b_f > 0.55\sqrt{\frac{F}{F_{ye}}}$	1.25	2
Other	Linear interpolation between the values on lines a and b shall be performed	
	$0.2 < P / P_{CL} < 0.5$ for	
a. $\frac{b_f}{2t_f} < 0.3\sqrt{\frac{F}{F_{ye}}}$	$20(1-1.7P / P_{CL})$	$12(1-1.7P / P_{CL})$
b. $\frac{b_f}{2t_f} > 0.55\sqrt{\frac{F}{F_{ye}}}$	1.25	1.5
Other	Linear interpolation between the values on lines a and b shall be performed	
	Braces in Compression (except EBF braces)	
W or I shape	6	8
Braces in Tension (except EBFbraces)	6	8

5- Design actions in elements shall satisfy Eq. (29)

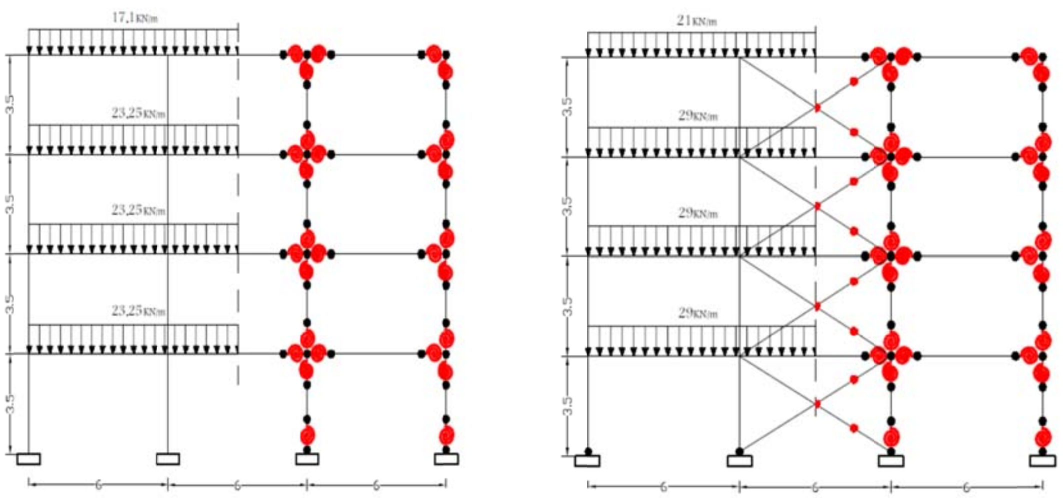
$$Q_C \geq \frac{Q_U}{m} \tag{29}$$

Where m is a component or element demand modifier based on nonlinear behavior of elements. m-factors are specified in Table 3.

3. The studied frames

Three special moment and braced steel frames with 4, 8 and 12 stories were considered in this study. As shown in Fig. 5(a)-(b), the frames have three bays with the width of 6 m and the height of 3.5 m. The gravity load containing both dead and live load was assumed to equal 23.25 KN/m in moment frames and 29 KN/m in braced frames for all the levels except the roof level, which its load is assumed to equal 17 KN/m in moment frames and 21.14 KN/m in braced frames. Tables 4, 5 give cross sections for all members. Analytical models of buildings were developed using nonlinear finite element program OpenSees which is capable of performing nonlinear static and dynamic analyses. In moment frames, beams and columns were modeled as elastic beam column elements and the rotational spring at both ends of beams and columns capture the nonlinear behavior of the frame. In braced frames the nonlinear behavior of beam and columns was the same as moment frames but the braces were modeled as nonlinear beam column elements. The initial camber at the center was assumed to be 0.01% of the total length of the brace. The number of integration points is kept constant at five integration points per braced elements. The interaction of the axial force and bending moment was considered in brace elements.

The force-deformation curve of each member was modeled in accordance with ASCE 41-06 which is shown in Fig. 6(a)-(b). Modeling parameters and acceptance criteria for beams, columns and braces, are presented in Tables 6, 7, 8.



(a) Moment frame(4 story) (b) Braced frame(**4** story)

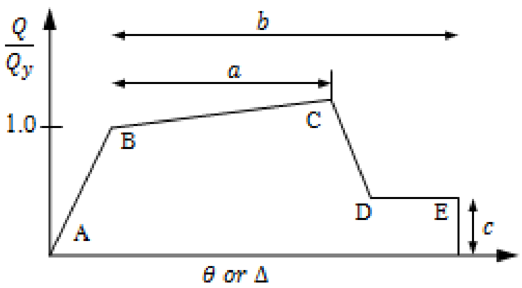
Fig. 5 Elevations, spans and hinge properties frames

Table 4 Summary of design results of three different moment frames

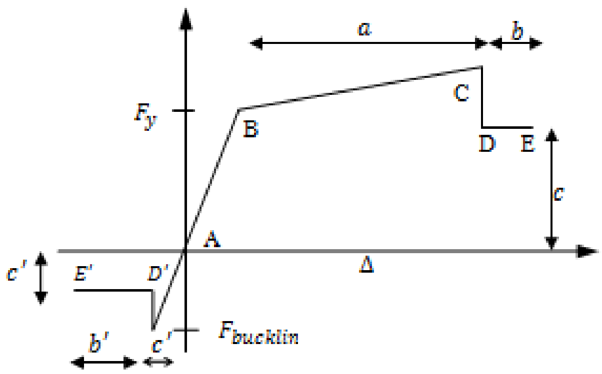
	Story	Beams	Interior cols	Exterior cols
4-Stories	1,2	W18×35	W24×162	W24×162
	3,4	W16×36	W24×117	W24×117
8-Stories	1,2,3	W18×60	W24×279	W24×279
	4	W18×60	W24×207	W24×207
	5,6	W18×46	W24×207	W24×207
	7,8	W18×46	W18×46	W18×46
12-Stories	1,2,3	W21×73	W24×335	W24×335
	4,5,6	W21×73	W24×279	W24×279
	7,8,9	W18×60	W24×229	W24×229
	10	W16×45	W24×229	W24×229
	11,12	W16×45	W24×104	W24×104

Table 5 Summary of design results of three different braced frames

	Story	Beams	Interior cols	Exterior cols	Braces
4-Stories	1,2	W18×40	W24×68	W24×68	W12×26
	3,4	W18×35	W21×44	W21×44	W10×22
8-Stories	1,2,3	W18×46	W24×207	W24×104	W12×40
	4	W18×46	W24×104	W21×48	W12×40
	5	W18×40	W24×104	W21×48	W12×35
	6	W18×40	W21×48	W21×44	W12×35
	7,8	W18×40	W21×48	W21×44	W12×26
12-Stories	1,2,3	W18×55	W27×539	W27×114	W12×45
	4,5	W18×55	W24×229	W24×84	W12×40
	6	W18×40	W24×229	W24×84	W12×40
	7,8,9	W18×40	W24×117	W24×68	W12×35
	10,11,12	W18×35	W24×55	W21×44	W12×26



(a)



(b)

Fig. 6 Generalized force–deformation relation for steel elements

Table 6 Modeling parameters and acceptance criteria for beams

Modeling parameters			Plastic rotation angle
a	b	c	Acceptance criteria Collapse Prevention
9θy*	11θy	0.6	8θy

*θy: the rotation at yield

Table 7 Modeling parameters and acceptance criteria for columns

P/PCL*	Modeling parameters			Plastic rotation angle
	a	b	c	Acceptance criteria Collapse Prevention
P/PCL<0.2	9θy	11θy	0.6	9θy
0.2≤P/PCL≤0.5	$11\left(1-\frac{5}{3}\frac{P}{P_{CL}}\right)\theta_y$	$17\left(1-\frac{5}{3}\frac{P}{P_{CL}}\right)\theta_y$	0.2	$11\left(1-\frac{5}{3}\frac{P}{P_{CL}}\right)\theta_y$

*P: The axial force of column; PCL: The lower-bound axial compressive strength of column

Table 8 Modeling parameters and acceptance criteria for braces

	Modeling parameters			Plastic Deformation	
	a	b	c	Acceptance criteria	
				LS	CP
Braces in Compression	$0.5\Delta_C$*	$8\Delta_C$	0.2	$5\Delta_C$	$7\Delta_C$
Braces in Tension	$11\Delta_T$	$14\Delta_T$	0.8	$7\Delta_T$	$9\Delta_T$

* Δ_T : The axial tensile deformation; Δ_C : The axial compressive deformation

Table 9 The values of m-factors in buildings

12 Storey		8 Storey		4Storey	
Braced frame	Moment frame	Braced frame	Moment frame	Braced frame	Moment frame
4.82	2.34	3.46	1.48	3.39	1.43
4.38	1.68	3.12	1.01	2.89	0.81
4.25	1.37	2.91	0.72	2.78	0.73
4.54	1.42	2.79	0.17	1.48	0.65
4.37	1.32	3.21	0.13		
4.10	1.27	2.70	0.16		
5.28	1.63	2.67	0.24		
4.66	1.11	1.30	0.16		
3.97	1.19				
4.21	1.97				
2.90	0.69				
1.46	1.23				

4. Validation of simplified methods

4.1 Evaluation of frames using ASCE41-06

For simplified seismic assessment by ASCE41-06 a liner static analysis should be done that has been presented in section (2-2). For simplicity this analysis was done by SAP2000 software. After performing a linear static analysis, the flexural strength of columns in moment frames and the axial tensile strength of brace elements were calculated and design efforts were obtained using the described method. The assessment results for all frames are presented in Table 9.

4.2 Evaluation of frames using NZSEE

4.2.1 failure mechanism

For all moment frames, the beam sideway mechanism was diagnosed as the probable failure mechanisms.In braced frames, using the demand ratio analysis, the failure mechanisms were predicted in second floor of 4-story frame, fifth floor of 8-story frame and forth floor of 12-story frame.

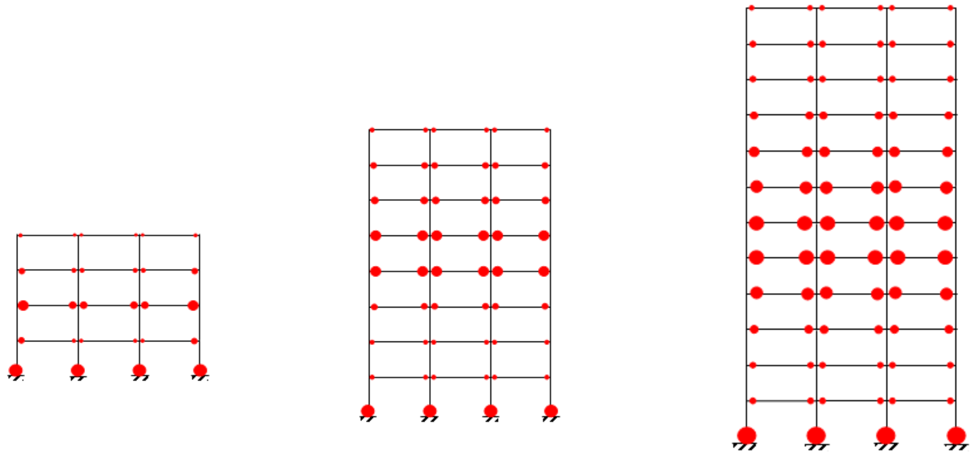
To verify the accuracy of this method in prediction of the failure mechanism, the plastic hinge distribution at the pushover analysis was identified (as mentioned in section 3, Opensees was used for performing nonlinear static and dynamic analyses).

Table 10 The failure mechanisms of frames

No.Story	12 Storey		8 Storey		4Storey	
	Moment Frames	Braced Frames	Moment Frames	Braced Frames	Moment Frames	Braced Frames
	*sway potential index, *Si*	**demand-capacity ratio, *DCR*	sway potential index, *Si*	demand-capacity ratio, *DCR*	sway potential index, *Si*	demand-capacity ratio, *DCR*
1	0.379	4.29	0.429	2.890	0.259	1.876
2	0.405	1.92	0.435	1.321	0.153	**0.923**
3	0.168	1.28	0.208	1.245	0.132	1.131
4	0.259	1.36	0.215	**0.990**	0.118	1.004
5	0.267	1.14	0.303	1.039		
6	0.276	1.00	0.216	0.944		
7	0.367	1.10	0.222	0.892		
8	0.370	1.03	0.228	0.869		
9	0.381	**0.97**				
10	0.264	1.06				
11	0.270	1.04				
12	0.278	1.03				

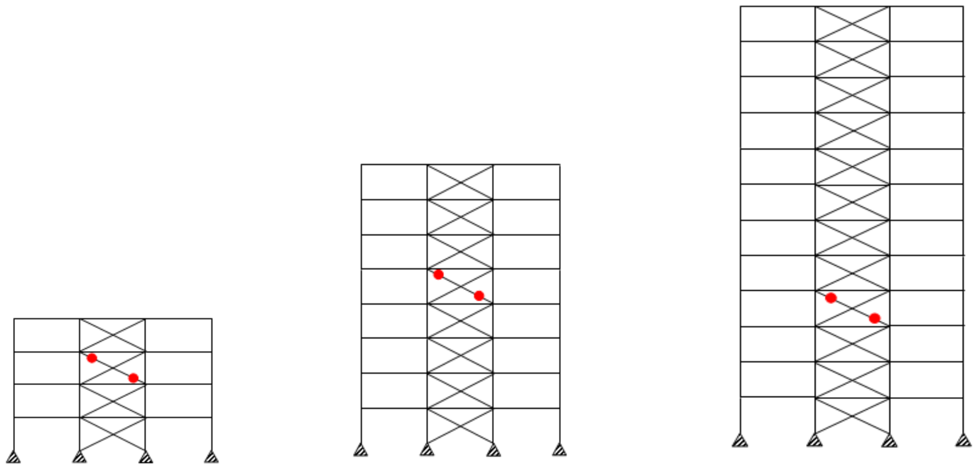
*If S_i<0.85, plastic hinges would develop in the beams and at the top and bottom of the column bases, otherwise they would develop in the columns

**If DCR_i<1, implies failure at that level. But this ratio should be lower than the value for upper and bottom stories. If in calculations more than one floor have this property, the story that has the greatest difference with upper and bottom stories choose as the critical floor



(a) 4 story (b) 8 story (c) 12 story

Fig. 7 Plastic hinge formation in different types of moment frames at the pushover analysis



(a) 4 story (b) 8 story (c) 12 story

Fig. 8 Plastic hinge formation in different types of braced frames at the pushover analysis

4.2.2 Capacity curve

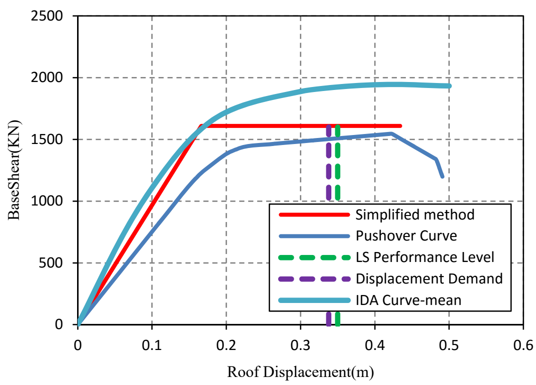
In order to assess the adequacy of the simplified procedures of the NZSEE, comparison with capacity curves obtained from dynamic and static analyses have been carried out. Capacity curves are usually defined in a base shear versus top displacement diagram for the structures, and they can be obtained using both dynamic and static analysis. In their dynamic form, each point of the diagram is defined through finding the maximum of base shears and top displacements in their corresponding response time history. An IDA procedure consists of a series of time-history analyses, so it can result a dynamic capacity curve (Shafiee *et al.* 2015).

For performing the IDA, the near-field record set recommended in FEMA P695, (2009) consisting of 28 records (56 individual components) from the strong ground motion database of the Pacific Earthquake Engineering Research Centre (PEER) (http://peer.berkeley.edu) has been selected. The 28 records are taken from 14 events that occurred between 1976 and 2002. The selected near-fault ground motion records correspond to locations which were at most 10km from a rupturing fault. The record set includes both pulse-like and non-pulse-like near-fault records. Event magnitudes range from M6.5 to M7.9 with an average magnitude of M7.0. More information has been stated in FEMA P695.

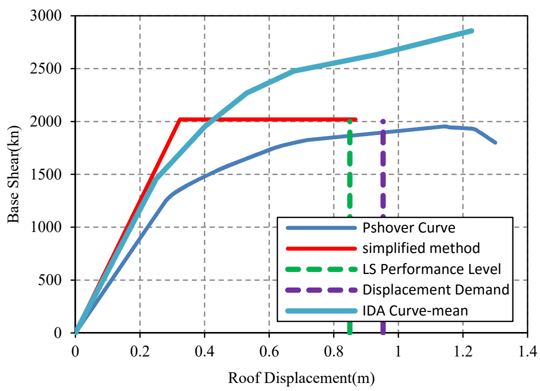
Validation of the simplified pushover curves obtained from SLaMA procedure is shown in Figs. 9-10(a), (b), (c). The green lines show the limit of the Life Safety Performance Level of structures and the purple lines indicate the position of the structures in the displacement demand. In accordance to ASCE 41-06, the displacement demand is determined as Eq. (30)

$$\delta_t = C_0 C_1 C_2 S_a \frac{T_e^2}{4\pi^2} g \tag{30}$$

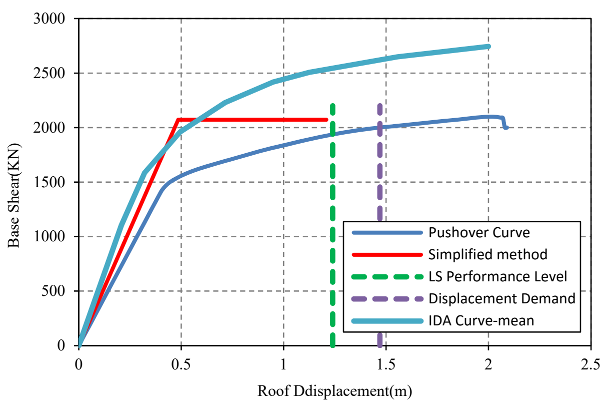
If the life safety performance level be less than the displacement demands, the structure will be failed, but if this limit be more, the structure will be satisfy the life safety performance level.



(a) 4 story

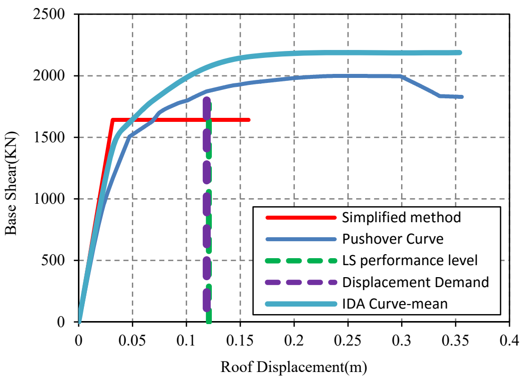


(b) 8 story

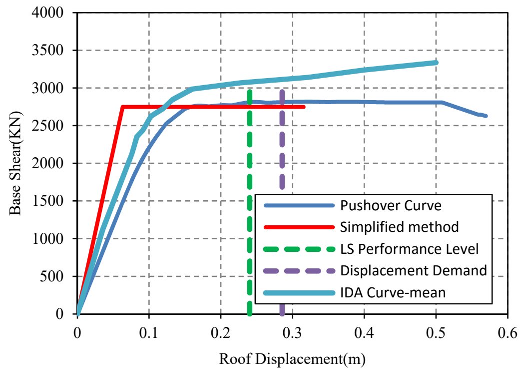


(c) 12 story

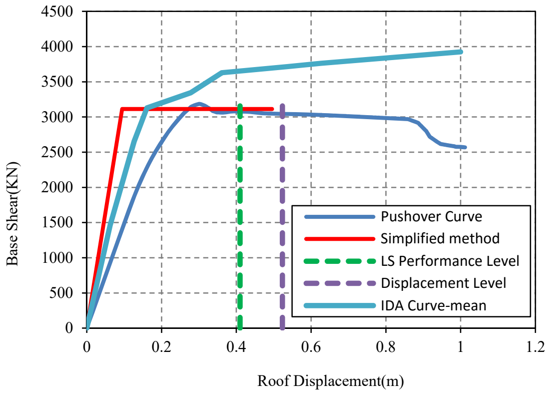
Fig. 9 Comparison between simplified and pushover analyses in moment frames



(a) 4story



(b) 8 story



(c) 12 story

Fig. 10 Comparison between simplified and pushover analyses in braced frames

4.2.3 peak ground acceleration

Another comparison of the SLaMA with nonlinear dynamic analysis was made in terms of the PGA_f value that causes the collapse of the structures. The PGA_f has been arbitrarily related to the spectrum of Standard No. 2800 for the soil type II; it is believed that the results of the comparisons would not change to any significant extent if a different reference spectrum were selected. The PGA_f values for the results of the SLaMA procedure were determined as follow

$$PGA_f = \frac{\frac{V_{prob}}{W_t}.\mu_{sc}}{C(T_1)} \tag{31}$$

Where V_{prob} is the base shear capacity of structure, μ_{sc} is the structural ductility, W_t is total seismic weight of structure and $C(T_1)$ is the ordinate of 5% damped elastic acceleration spectrum for T_1 (fundamental period of structure). Summary of calculations is presented in Tables 11 and 12 for moment frames and braced frames respectively.

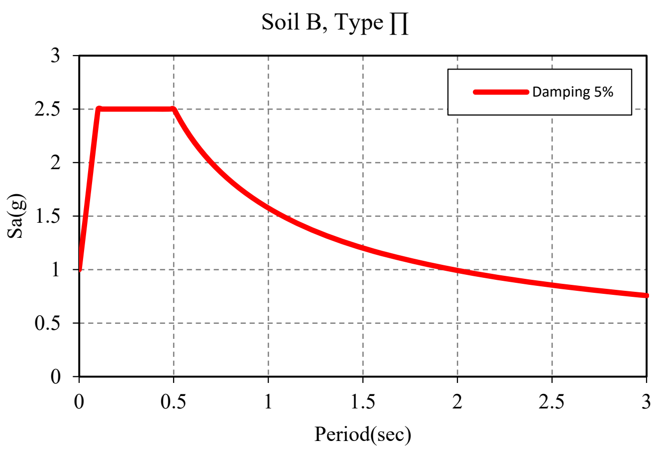


Fig. 11 Standard No. 2800-05 acceleration spectrum for soil Type ∏

Table 11 Summary of calculations in the SLaMA for moment frames

$V_{prob}(kg)$	$W(t)$	$\Delta_u(m)$	$\Delta_d(m)$	μ_{sc}	T_1 (sec)	$C(T_1)$	PGA
160835.9	159300	0.166	0.434	2.6	0.579	2.267	1.16
198037.8	329940	0.323	0.867	2.68	0.931	1.603	0.97
211150.9	572580	0.485	1.208	2.49	1.402	1.308	0.73

Table 12 Summary of calculations in the SLaMA for braced frames

$V_{prob}(kg)$	$W(t)$	$\Delta_u(m)$	$\Delta_d(m)$	μ_{sc}	T_1 (sec)	$C(T_1)$	PGA
164111.5	279979	0.031	0.157	4.99	0.353	2.5	1.16
274279.3	587599	0.063	0.315	4.99	0.608	2.192	1.06
309399	701748	0.094	0.495	5.23	0.848	1.757	1.32

To check the reliability of the SLaMA, the comparison between the obtained results and the corresponding ones deduced from nonlinear dynamic analyses should be carried out. So after performing incremental dynamic analysis for all records that mentioned in section 4.2.2, capacity curves in terms of seismic intensity versus the demand parameter were plotted. The Intensity Measure (IM) and Damage Measure (DM) in this study were the peak ground acceleration and the maximum inter story drift ratio respectively (Fig. 12). The CP point on capacity curves was defined according to FEMA356 guidelines, which is not exceeded on the IDA curve until the final point where the local tangent reaches 20% of the elastic slope or θ_{max}=10%, whichever occurs first in IM terms (Vamvatsikos *et al.* 2003).

After finding PGAf's for each record, Minitab as a software was used to fit best probabilistic distribution on 56 data's. The variability in the PGA_f is best described by a lognormal distribution so present study used average of natural log dates instead of simply average (Vamvatsikos *et al.* 2002) (see Figs. 13,14);

The PGAf values that cause the collapse in the first element of the frames are reported in Table 13 and shown in Figs. 15-16 for simplified method of NZSEE and nonlinear dynamic analyses.

Figs. 17, 18 illustrates percent of error among SLaMA and IDA in estimation of base shear and PGA_f.

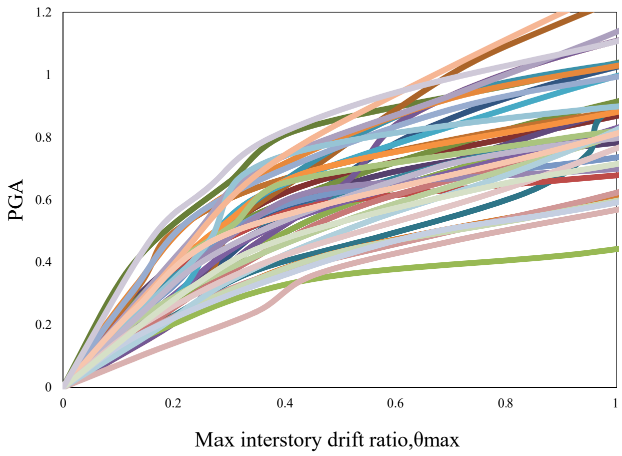


Fig. 12 Incremental dynamic analysis response curves

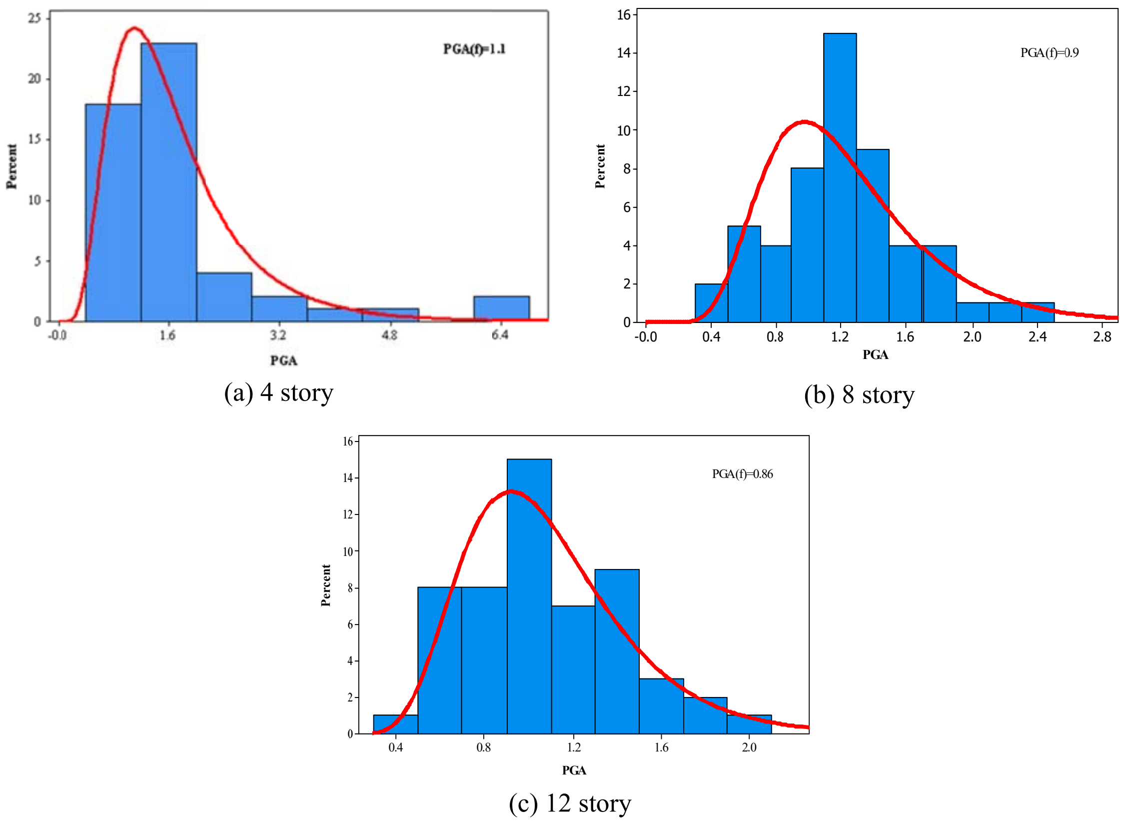
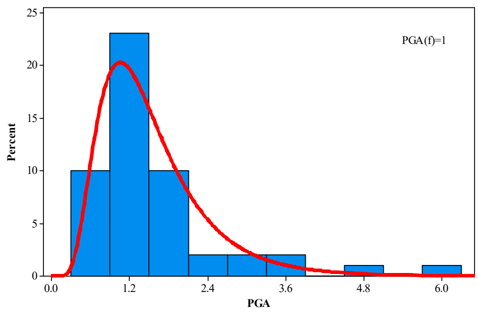
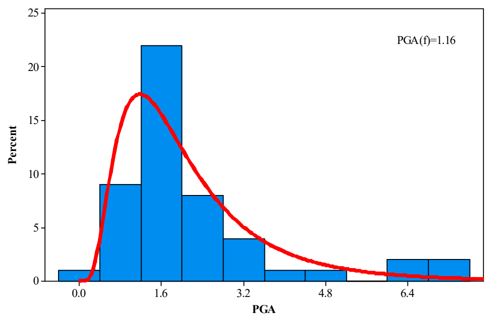


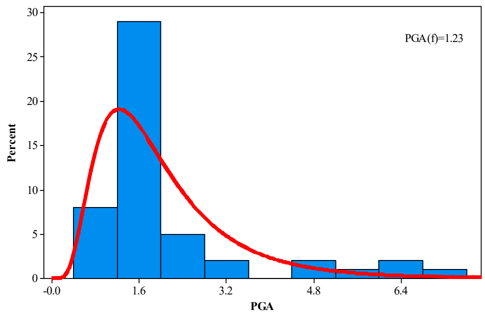
Fig. 13 The lognormal distribution of PGA failure values form IDA in moment frames



(a) 4 story



(b) 8 story



(c) 12 story

Fig. 14 The lognormal distribution of PGA failure values in braced frames

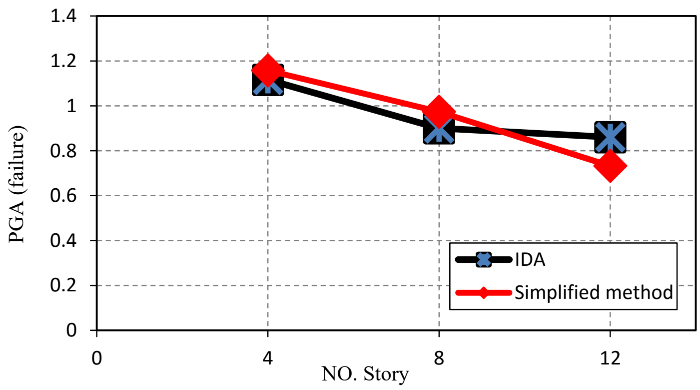


Fig. 15 The PGA values cause the collapse in moment frames

Table 13 Collapse Peak Ground Acceleration (PGA) in units of g

	Moment Frames		Braced Frames	
	Simplified method	Nonlinear Dynamic Analysis	Simplified method	Nonlinear Dynamic Analysis
4	1.16	1.11	1.16	1
8	0.97	0.9	1.06	1.16
12	0.73	0.86	1.32	1.23

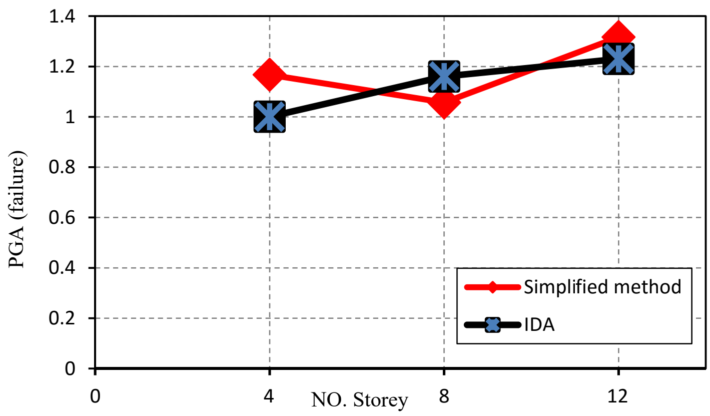
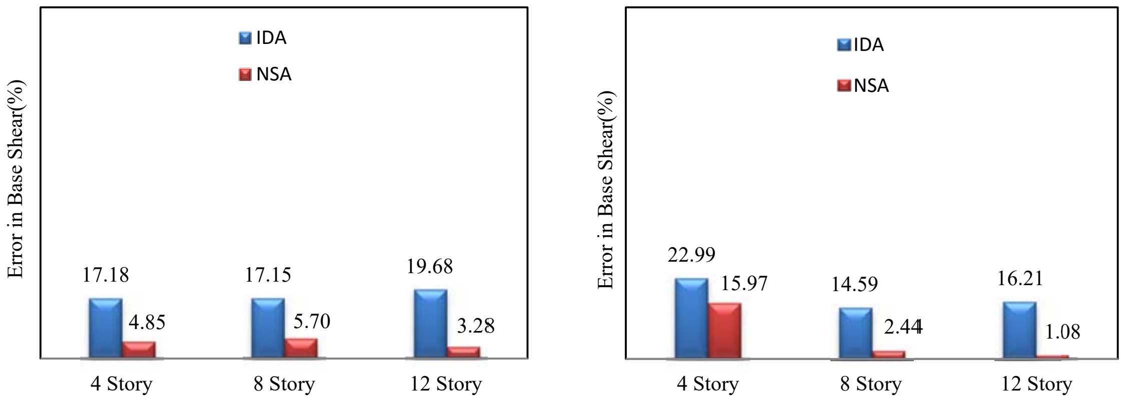


Fig. 16 The PGA values cause the collapse in braced frames



(a) The results of Moment frames (b) The results of Braced frames

Fig. 17 Observed error in base shear in SLaMA

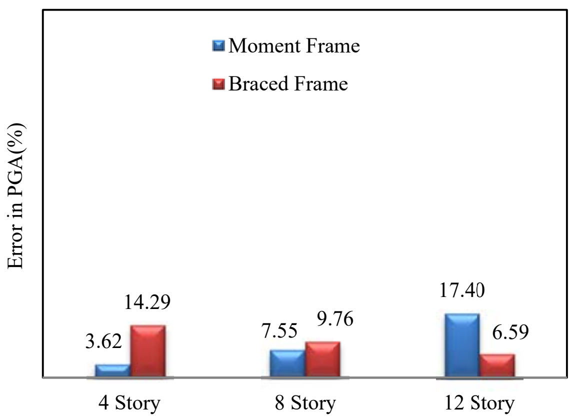


Fig. 18 Observed error in PGA $_{failure}$ in SLaMA

5. Conclusions

In the present study effectiveness of the simplified methods that have proposed by the NZSEE and ASCE 41-06 guidelines was investigated. NZSEE has presented a simple lateral mechanism analysis that is a hand static analysis for determining of the capacity curve and ASCE 41-06 has introduced a simple method for assessment of certain buildings based on maximum building height in stories. According to Table 1, in regions with high seismic risk, the simple method of the ASCE 41-06 is applied for structures with the number stories less than 3 for both types of frames but we did this method for all frames to examine the results of the assessment. As shown in Figs. 9, 10, with regarding to the capacity curves that obtained by Opensees, in 4 story frames, the LS Performance Level and displacement demand are approximately equal whereas in other frames the LS Performance Level is less than the displacement demand, so the structures could not satisfy the life safety performance level and will be failed but according to the results of ASCE 41-06 that were presented in Table 9, the flexural strength of columns in moment frames and the axial tensile strength of brace elements have satisfied the relationship in Eq. (29) and this means that LS Performance Level is satisfied that do not correspond exactly to the reality.

Results are given in following:

• In 4 story frames the result of assessments by the ASCE41-06 partially is closer to the results of the nonlinear dynamic analysis so maybe we can use this method also for 4 story frames. But in other frames the results don't have agreement with the reality. For this reason this method only is applied to a select group of simple buildings that represented in Table 1.

• According to the plastic hinge distribution at pushover analysis witch shown in Figs. 7-8, the failure mechanism was predicted correctly by this method in both types of frame.

• From the Figs. 9-10(a), (b), (c) and Fig. 17(a), (b) it can be concluded that the results of the SLaMA of NZSEE has a good agreement in estimation of the base shear capacity with the results of the nonlinear static analysis and the initial stiffness and the elastic displacement are close to the results of dynamic analysis but in moment frames with increasing the height of building and in braced frames this method gives the lower bound value of ductility so it can be say SLaMA is a conservative approach.

• In braced frames with increasing the height of building the initial stiffness was estimated less than the capacity curve of the IDA. To overcome this weakness, we need to model more frames to modify the empirical relationship for the elastic displacement of this frames.

• As shown in Figs. 13-16 the results of the SLaMA is compatible with the nonlinear dynamic analysis so this method in evaluation of existing structures is effective.

At the end it becomes clear from the study that further research should be carried out in order to improve the assessment procedures prescribed in this article.

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