COMPARISION OF DIFERENT SIMPLIFIED METHODS FOR SEISMIC ASSESMENT OF CONCENTRIC BRACED FRAMES

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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. In this study the accuracy of simplified methods is examined on samples of steel concentric braced frames. Next the PGA values that cause their collapse are determined and these results are compared with the corresponding values determined by Incremental Dynamic Analysis. After comparison of obtained results, suggestions are presented to improve seismic retrofit criteria.

INTRODUCTION

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.

Another guidance document for seismic assessment of existing buildings is NZSEE2006 recommendations in New Zealand. This guideline has proposed a hand analysis to determine the probable collapse mechanism, lateral strength and displacement capacity with simplified consideration of capacity issues, so this method was named simple lateral mechanism analysis (SLaMA). The behavior of the structure is reduced to that of an equivalent single-degree-of freedom system.

In this study, a hand static analysis for seismic evaluation of steel braced frames is presented by using of New Zealand guideline and ASCE 41-06 criteria .Three different special steel braced frames with different number of stories (4- 8 and 12-storey) were assessed with the SLaMA approach.

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

SIMPLIFIED REHABILITATION METHOD OF ASCE 41-06

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).

Table 1- Limitations on Use of the Simplified Rehabilitation Method

Model Building Type	Maximum Building Height in Stories by Seismic Zone1 for Use of the Simplified Rehabilitation Method		
	Low	Moderate	High
Steel Braced Frame			
Stiff Diaphragm	6	4	3
Flexible Diaphragm	3	3	3

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 (1).

$$V = S_a C W \tag{1}$$

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 2- C-coefficient factor

Model Building Type	Number of stories			
	1	2	3	≥4
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 (2).

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

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 in members due to gravity loads and earthquake loads, Q_U , is calculated in accordance with Eq (3).

$$Q_U = Q_G \pm Q_E \tag{3}$$

$$Q_G = 1.1 \times [Q_D + Q_L] \tag{4}$$

Where Q_G is action due to design gravity loads and Q_E is action due to design earthquake loads.

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

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

$$Q_C \geq \frac{Q_U}{m} \tag{5}$$

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

Table 3- Acceptance Criteria for Linear Procedures—Structural Steel Components

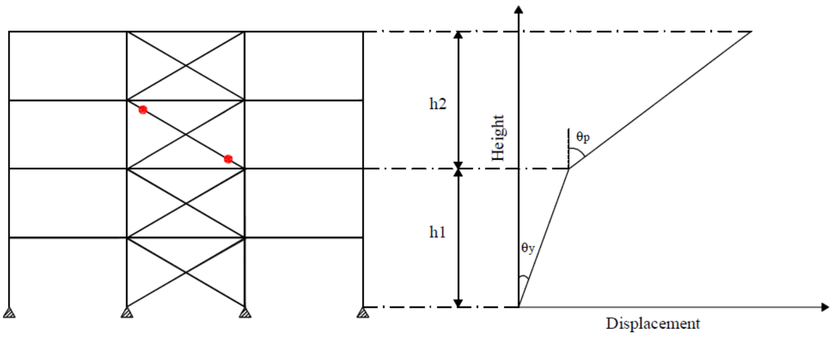
Component/Action	*m*-factors for Linear Procedures	
	LS	CP
Braces in Compression (except EBF braces)		
W or I shape	6	8
Braces in Tension (except EBFbraces)	6	8

SIMPLE LATERAL MECHANIZM ANALYSIS (SLaMA) OF NZSEE

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.

$$DCR_i = \frac{F_{Di}}{F_{Ci}} \tag{6}$$

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. 10 .In its simplest form, a ratio greater than 1, implies failure at that level. The Demand Ratios is calculated for the vertical distribution of the seismic forces which proposed by the ASCE 41-06 accordance with Eq. (2) at each floor;



(a) Plastic mechanism (b) Displacement profiles

Fig. 1 Determining limit state displacement profile from mechanism

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

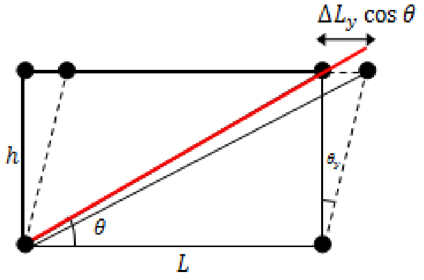


Fig. 2 Deformed shape of a story

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. 2);

$$\theta_y = \frac{\Delta L_y \cos\theta}{h} = \frac{\frac{F_{ye} L_{brace}}{E}\cos\theta}{h} = \frac{\varepsilon_y L_{bay}}{h} \tag{7}$$

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).

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

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 4). Tensile plastic deformation of braces for CBFs in different building performance levels is according to Table 4. The plastic rotation angle can be calculated by the following equation;

$$\theta_p = \frac{\Delta L_P \cos\theta}{h} \tag{9}$$

Table 4 Modeling parameters and acceptance criteria for braces.

	Modeling parameters			Plastic Deformation Acceptance criteria	
	a	b	c	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

With the use of Eqs.(8)-(9) and according to Fig. 1 (a)-(b) the ultimate roof displacement is obtained as Eq.(10):

$$\Delta_d = h_1\theta_y + h_2\theta_p \tag{10}$$

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:

$$V_{base} = P_{Tx} + P_{Cx} = (P_T + P_C)\cos\theta \tag{11}$$

That P_{Tx} is the horizontal brace tensile force component and P_{Cx} is the horizontal brace compressive force component in plastic. Where θ is the angle between the brace and horizontal line.

THE STUDIED FRAMES

Three special steel concentric braced frames with 4, 8 and 12 stories were considered in this study. Analytical models of buildings were developed using nonlinear finite element program OpenSees which is capable of performing nonlinear static and dynamic analyses. The frames have three bays with the width of 6 m and the height of 3.5m. The gravity load containing both dead and live load was assumed to equal 29KN/m for all levels. 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 but the braces were modeled as nonlinear beam column elements. The interaction of the axial force and bending moment was considered in brace elements.

Modeling parameters and acceptance criteria for beams, columns and braces are in accordance with ASCE 41-06 that for brace elements was presented in Table 4.

EVALUATION OF FRAMES USING ASCE 41-06

All frames were modeled in SAP2000 software. After performing a linear static analysis that has been presented, the axial tensile strength of brace elements in braced frames were calculated and design efforts were obtained using the described method. The assessment results for all frames are presented in Table (5).

Table 5- the values of m-factors in tensile braced elements

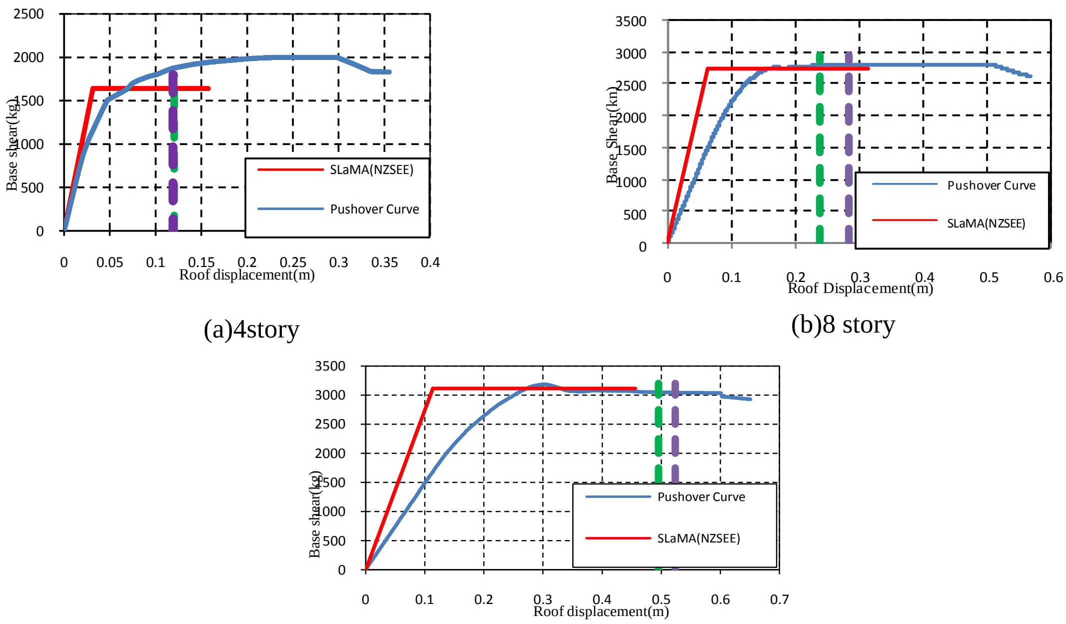
4Storey	8 Storey	12 Storey	NO. Story
3.39	3.46	4.82	1
2.89	3.12	4.38	2
2.78	2.91	4.25	3
1.48	2.79	4.54	4
	3.21	4.37	5
	2.70	4.10	6
	2.67	5.28	7
	1.30	4.66	8
		3.97	9
		4.21	10
		2.90	11
		1.46	12

EVALUATION OF FRAMES USING NZSEE

In order to assess the adequacy of the simplified procedures of the NZSEE, comparison with results obtained from pushover analyses have been carried out. Validation of the simplified pushover curves obtained from SLaMA procedure is shown in Figs. 4 (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. (12)

$$\delta_t = C_0 C_1 C_2 S_a \frac{T_e^2}{4\pi^2} g \tag{12}$$

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)4story (b)8 story

(c)12 story

Fig. 3 Comparison between simplified and pushover analyses

Using the demand ratio analysis of NZSEE, 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.

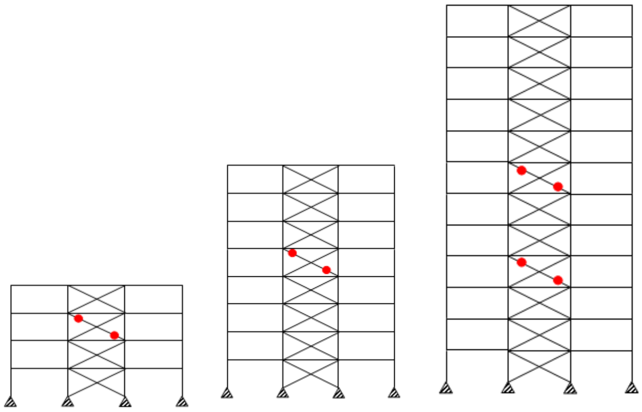


Fig. 4 Plastic hinge formation in different types of braced frames at the pushover analysis

NONLINEAR DYNAMIC ANALYSIS

The comparison of the NZSEE method (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. 2800-05 for the soil type II. The PGA_f values for the results of the SLaMA procedure were determined as follow:

$$PGA_f = \frac{\frac{V_{prob}}{W_t}.\mu_{sc}}{C(T_1)} \tag{13}$$

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).

To estimate the response of the frames under earthquake, nonlinear dynamic analysis was done using 56 Near-Field records that have been listed in FEMA-P695. These records are between 1976to2002 with magnitudes range from M6.5 to M7.9. After performing incremental dynamic analysis for all above mentioned records, 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. The CP point on capacity curves was defined according to ASCE 41-06 guidelines, which is not exceeded on the IDA curve until the final point where the local tangent reaches 20% of the elastic slope or $\theta_{\max} = 10\%$, whichever occurs first in IM terms.

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 PGAf is best described by a lognormal distribution so present study used average of natural log dates instead of simply average.

The PGAf values that cause the collapse in the first element of the frames are shown in Fig. 5 for simplified method of NZSEE and nonlinear time history analyses.

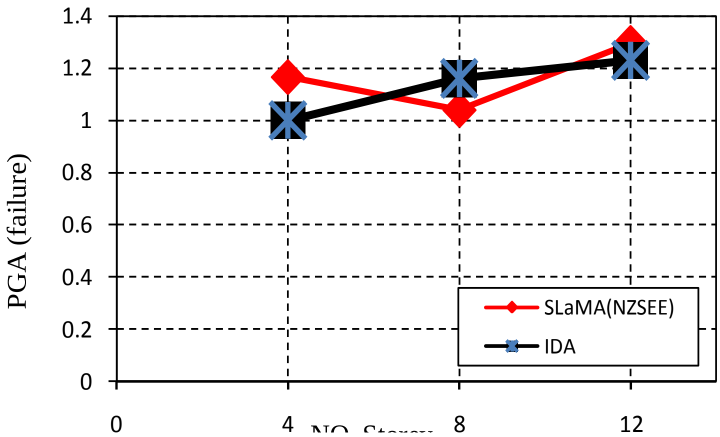


Fig. 5 The PGA values cause the collapse in braced frames

CONCLUSION

According to Table (1), in regions with high seismic risk, the simplified method of the ASCE 41-06 is applied for structures with the number stories less than 3 but we did this method for all frames to examine the results of the assessment. As shown in Fig.3, with regarding to the pushover curves that obtained by Opensees, in 4 story frame, the LS Performance Level and displacement demand are 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 5, all bracing elements have satisfied the relationship in Eq(5) and this means that LS Performance Level is satisfied that do not correspond exactly to reality.

Although in 4 story frame the result of the ASCE41-06 partially is closer to the results of the nonlinear static analysis but in other frames the results don't have agreement with the nonlinear static analysis. 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 Fig. 4, the failure mechanism was predicted correctly by SLaMA method in frames.

From the Figs. 3 (a)-(b)-(c), it can be concluded that the results of SLaMA has a good agreement with the results of the nonlinear static analysis especially in estimation of the base shear capacity but the initial stiffness was estimated less than the pushover results. 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 Fig. 5, the results of SLaMA is compatible with the nonlinear dynamic analysis.

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