VERIFING OF SIMPLIFIED METHODS FOR SEISMIC ASSESEMENT OF STEEL MOMENT FRAMES BY NONLINAER STATIC AND DYNAMIC ANALYSIS

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ABSTRACT

The major guidance documents for seismic assessment of existing buildings are ASCE 41-06 inUS, and NZSEE recommendations in New Zealand.

All of these guidelines have provided simple criteria in terms of simplified rehabilitations on the assessment of existing buildings. 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 moment frames. For this, three different special moment steel frames are assessed under these two guidelines and the accuracy of the results are checked with the nonlinear static and dynamic analysis.

INTRODUCTION

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 the simplified methods is examined on samples of steel moment frames. For this three different special steel moment frames with different number of stories (4- 8 and 12-storey) were assessed under these methods.

At first in order to determine the reliability of the methods, the nonlinear static analysis was used for verifying. 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 METHOD OF NZSEE (SLaMA)

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:

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

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

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

$$V_{bdl} = V_{bgl} + (M_{bl} + M_{br})/L_b \tag{3}$$

$$V_{bdr} = V_{bgr} + (M_{bl} + M_{br})/L_b$$

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:

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

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:

$$M^*_{bl} = (V_{brl} - V_{bgl})/L_b - M_{br} \tag{5}$$

$$M^*_{br} = (V_{brr} - V_{bgr})/L_b - M_{bl}$$

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).

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:

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

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

For column sway:

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

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 overturning moment induced by external forces is according to follow equation.

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

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

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:

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

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

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

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

SIMPLIFIED REHABILITATION METHOD OF ASCE 41-06

Simplified rehabilitation method that proposed by ASCE 41-06, 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 Moment Frame			
Stiff Diaphragm	6	4	3
Flexible Diaphragm	4	4	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 (19).

$$V = S_a CW \tag{19}$$

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 Moment Frame	1.3	1.1	1	1

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

$$F_i = \frac{w_i h_i^k}{\sum_{j=1}^{n} w_j h_j^k} \qquad \begin{array}{l} k = 2 \quad for \quad T \geq 2.5 \quad \sec \\ k = 1 \quad for \quad T \leq 0.5 \quad \sec \\ \text{Linear interpolation for intermediate values of T} \end{array} \tag{20}$$

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 (21).

$$Q_U = Q_G \pm Q_E \tag{21}$$

$$Q_G = 1.1 \times [Q_D + Q_L] \tag{22}$$

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 (23).

$$Q_C \geq \frac{Q_U}{m} \tag{23}$$

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
Columns – flexure		
for $P/P_{CL} < 0.2$		
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	
for $0.2 < P/P_{CL} < 0.5$		
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	

THE STUDIED FRAMES

Three special moment 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 23.25 KN/m for all the 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.

Modeling parameters and acceptance criteria for beams, columns and braces are in accordance with ASCE 41-06.

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 (4).

Table 4- the values of m-factors in columns.

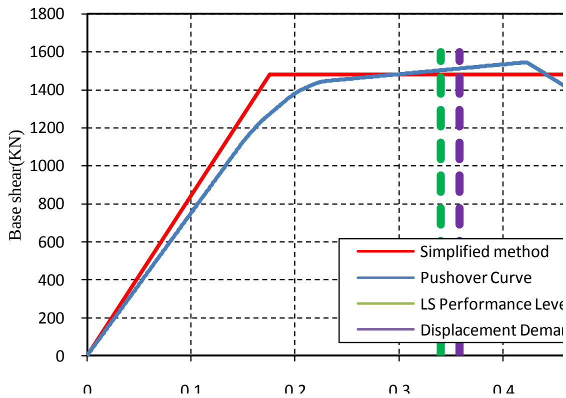
4Storey	8 Storey	12 Storey	NO. Story
1.43	1.48	2.34	1
0.81	1.01	1.68	2
0.73	0.72	1.37	3
0.65	0.17	1.42	4
	0.13	1.32	5
	0.16	1.27	6
	0.24	1.63	7
	0.16	1.11	8
		1.19	9
		1.97	10
		0.69	11
		1.23	12

EVALUATION OF FRAMES USING NZSEE

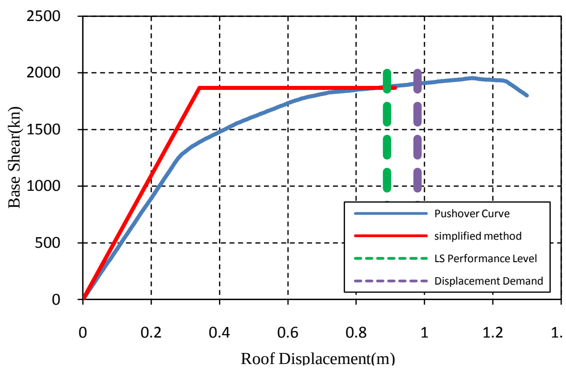
In order to assess the adequacy of the simplified procedures of the NZSEE, comparison with results obtained from pushover analyses have been carried out. Validation of the simplified pushover curves obtained from SLaMA procedure is shown in Figs. 1 (a)-(b)-(c). The green lines show the limit of the Life 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. (24)

$$\delta_t = C_0 C_1 C_2 S_a \frac{T_e^2}{4\pi^2} g \tag{24}$$

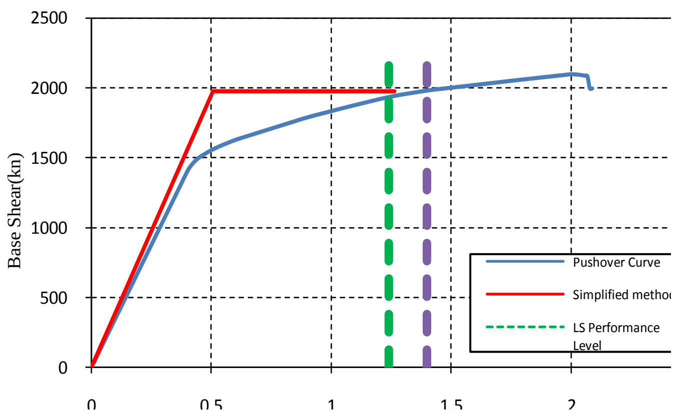
If the life safety performance level be less than the displacement demands, the structure will be failed, but if this limit be more, the structure will be satisfy the life safety performance level.



s(a) 4 story



(b) 8 story



(c)12 story

Figure 1. Comparison between simplified and pushover analyses in moment frames

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:

$$PGA_f = \frac{\frac{V_{prob}}{W_t}.\mu_{sc}}{C(T_1)} \quad (25)$$

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.

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. 2 for simplified method of NZSEE and nonlinear time history analyses.

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.1, with regarding to the pushover curves that obtained by Opensees, in all 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 4, columns have satisfied the relationship in Eq(23) and this means that LS Performance Level is satisfied that do not correspond exactly to reality.

In all frames the result of the ASCE41-06 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).

From the Figs. 1 (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.

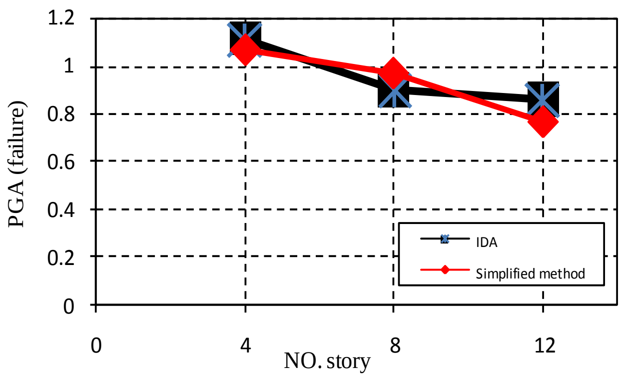


Figure 2. The PGA values cause the collapse in moment frames

As shown in Fig. 2, the results of SLaMA is compatible with the nonlinear dynamic analysis.

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