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COMPARISON OF DIFFERENT NON-LINEAR STATIC ANALYSIS USED FOR SEISMIC ASSESSMENT OF EXISTING BUILDINGS

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Abstract. *The present study is focused on the comparison of non-linear static analysis (Pushover) which is used by two major guidance documents, the New Zealand guideline and the US FEMA 440, on the assessment of existing buildings currently available for moment resisting concrete frames. The main purpose of the study is to trace the differences in the results produced by these two guidelines. For this, three different moment resisting concrete frames are assessed under these two guidelines to determine the PGA value that causes their collapse. In the next step, these are compared by their similar values that are determined from the non-linear dynamic analysis in which Park and Ang damage index is used as acceptance criteria for components. As is found, the result of force based approach which is proposed by New Zealand guideline is more compatible with the nonlinear dynamic analysis.*

1 INTRODUCTION

The disastrous effects observed in recent seismic events, in terms of loss of lives as well as immediate and long-term economic damage has prompted the need to produce documents in the area of assessment and improvement of the structural performance of existing buildings in times of an earthquake. One of the earliest guidelines which has been published for the evaluation and retrofit of concrete buildings is ATC-40 [1]. After it, FEMA 273 and FEMA 356 have been published respectively as a guideline and pre-standard for the seismic rehabilitation of buildings [2,3]. In 2005, FEMA 440 has been published. The purpose of the FEMA440 has been to evaluate current non-linear static procedures (NSPs), as described in FEMA 273/FEMA 356 and ATC-40, and to develop improvements where feasible. The primary objectives were, to develop guidelines for practicing engineers on how to apply the procedures to new and existing buildings.

The first step of the assessment process in FEMA440 is definition of “Rehabilitation Objectives”; where each goal shall consist of a “Target Building Performance Level” and an “Earthquake Hazard Level”. Building performance is a combination of the performance of both structural and nonstructural components. Three performance levels are considered such as: Immediate Occupancy (IO), Life Safety (LS) and Collapse Prevention (CP) for structural components and the nonstructural performance level of a building shall be selected from five discrete performance level consisting of Operational (N-A), Immediate Occupancy (N-B), Life Safety (N-C), Hazard Reduced (N-D), and Not Considered (N-E) [4]. The seismic hazard can be represented either by an acceleration response spectrum or by acceleration time history.

Four different analysis procedures are allowed for the evaluation of the response of the buildings: the linear static (LSP), the linear dynamic (LDP), the non-linear static (NSP) and the non-linear dynamic (NDP). The two linear procedures are permitted only for buildings with “regular” structural configuration. As the non-linear dynamic procedure is the most complex, using non-linear static analysis is more common for its simplicity and ability to estimate components and system deformation demands with an acceptable accuracy.

In nonlinear static procedure the demand of the buildings is calculated by means of a pushover analysis. For this, the target displacement, which is intended to represent the maximum displacement likely to be experienced during the designed earthquake, shall be determined at first. According to this standard, the control node shall be located at the center of the roof of a building and its target displacement can be calculated by the Coefficient Method as below:

$$\delta_t = C_0 C_1 C_2 S_a \frac{T_e^2}{4\pi^2} g \qquad (1)$$

Where C_0 is a modification factor to relate spectral displacement of an equivalent single degree of freedom (SDOF) system to the roof displacement of the building, C_1 is a modification factor to relate expected maximum displacements to displacements calculated for linear elastic response, C_2 is a modification factor to represent the effect of pinched hysteresis shape, cyclic stiffness degradation, and strength deterioration on maximum displacement response, S_a is the response spectrum acceleration at the effective fundamental period and damping ratio, and T_e is the effective fundamental period of the building.

Acceptance criteria for nonlinear procedures are presented by the plastic rotation angle for the concrete components. One of the conditions which can affect the acceptance criteria of components is the conditions of transverse reinforcement. A component is conforming if, within the flexural plastic hinge region, hoops are spaced at ≤ d/3, and if, for component of moderate and high ductility demand, the strength provided by the hoops is at least threefourths of the design shear. Otherwise, the component is considered nonconforming. Where d is effective depth of the cross section.

In 1996, a draft was published for the assessment and improvement of the structural performance of earthquake risk building in New Zealand titled NZSEE 1996 [5]. This draft has been reviewed twice till now and is now available as NZSEE 2002 and NZSEE 2006 [6, 7]. The New Zealand documents concentrate only on matters relating to life safety, i.e. collapse which leads to loss of life.

Three possible approaches for performing the assessment are indicated in the 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 the document it is stated that "the displacement based approach is generally considered to produce more rational and less conservative assessment outcome, the force based one is more familiar to designers" [7].

Five analysis methods are proposed for the evaluation of the structural response: equivalent static analysis, modal response spectrum analysis, simple lateral mechanism analysis, lateral pushover analysis and inelastic time history analysis. In this study, the proposed lateral pushover analysis is compared with the similar procedure of FEMA440.

The demands of the buildings are dependent upon the analysis method applied. In the force based approach, the acceleration response spectra are used to model the earthquake action and the demand is stated by the structural ductility factor which can be found as below:

$$k_\mu = \frac{C(T_1)S_P W_t (\%NBS)_t}{V_{prob}} \qquad (2)$$

Where $C(T_1)$ is the ordinate of the elastic site hazard spectrum for T_1 and for the site, W_t is total seismic weight of the structure, S_P is structural performance factor and $(\%NBS)_t$ is target percentage of new building standard which is considered equal to one in this study.

In the displacement based approach, the displacement response spectra are used to model the earthquake action and the demand is stated by the displacement at the effective height of building which is found from the spectra using the effective period and the equivalent viscous damping.

The ultimate curvature is considered as the acceptance criteria of concrete components. To calculate the ultimate curvature, having the ultimate concrete strain is inevitable. It is stated in the document that for unconfined concrete the ultimate concrete strain, ε_{cu} =0.004 and for the confined concrete, Mander model can be assumed as below [8]:

$$\varepsilon_{cu} = 0.004 + \frac{1.4\rho_s f_{yh}\varepsilon_{su}}{f_{cc}} \qquad (3)$$

Where ε_{cu} is ultimate concrete strain; ρ_s is the volumetric ratio of transverse reinforcement and can be approximated by:

$$\rho_s = 1.5A_v/b_c s \qquad (4)$$

Where A_v is total area of transverse reinforcement in a layer, s is spacing of layers of transverse reinforcement, b_c is the width of concrete core, f_{yh} is the yield strength of the transverse reinforcement, ε_{su} is the steel strain at maximum stress, and f_{cc} is the compressive strength of the confined concrete.

The document presents conditions corresponding to confined and unconfined sections. One of these conditions which are dealt with in this study is spacing of hoops. According to NZSEE guideline, when the spacing of hoops or stirrups sets in potential plastic hinge greater than or equal to d/2 or $16d_b$, the section shall be assumed as "unconfined", where d is the effective depth of section and d_b is the diameter of longitudinal reinforcement. However the sections are assumed as confined in this study.

Lupoi et al. presented a comparison of the practical applicability, the relative ease of use and the degree of agreement on the results of the methods proposed by FEMA 356, NZSEE 2002 and Japanese Standards. The PGA_f value that causes the collapse of three structures is determined based on the above mentioned documents and in comparison with each other. As it is mentioned in the paper, from the small number of cases examined, all studied structures have three stories, thus it is not possible to systematically trace the differences in the results produced by the different approaches [9]

In this study, 5, 10 and 15 story moment resisting concrete frames are assessed by the NZSEE 2006 and FEMA440 to determine the PGA_f values that cause the collapse. Then the frames are analyzed against twenty two earthquakes with the use of nonlinear dynamic analysis to determine the PGA_f value. To find, the performance of frames in the latest analysis, Park and Ang Damage Index has been used as is explained in the following paragraphs [10].

2 COMPARATIVE STUDY

This section presents the choices and assumptions made by using the above mentioned documents in order to compare results.

2.1 The studied frames

Three moment resisting concrete frames with 5, 10 and 15 stories are considered in this study. Figure 1 illustrates five-story moment resisting frame. As is shown, the frame has four bays with the width and the height of 6 and 3.2m, respectively. The design details of other frames are revealed on Table 1, 2 and 3. The gravity load containing both dead and live load is assumed 28.86 kN/m for all the levels.

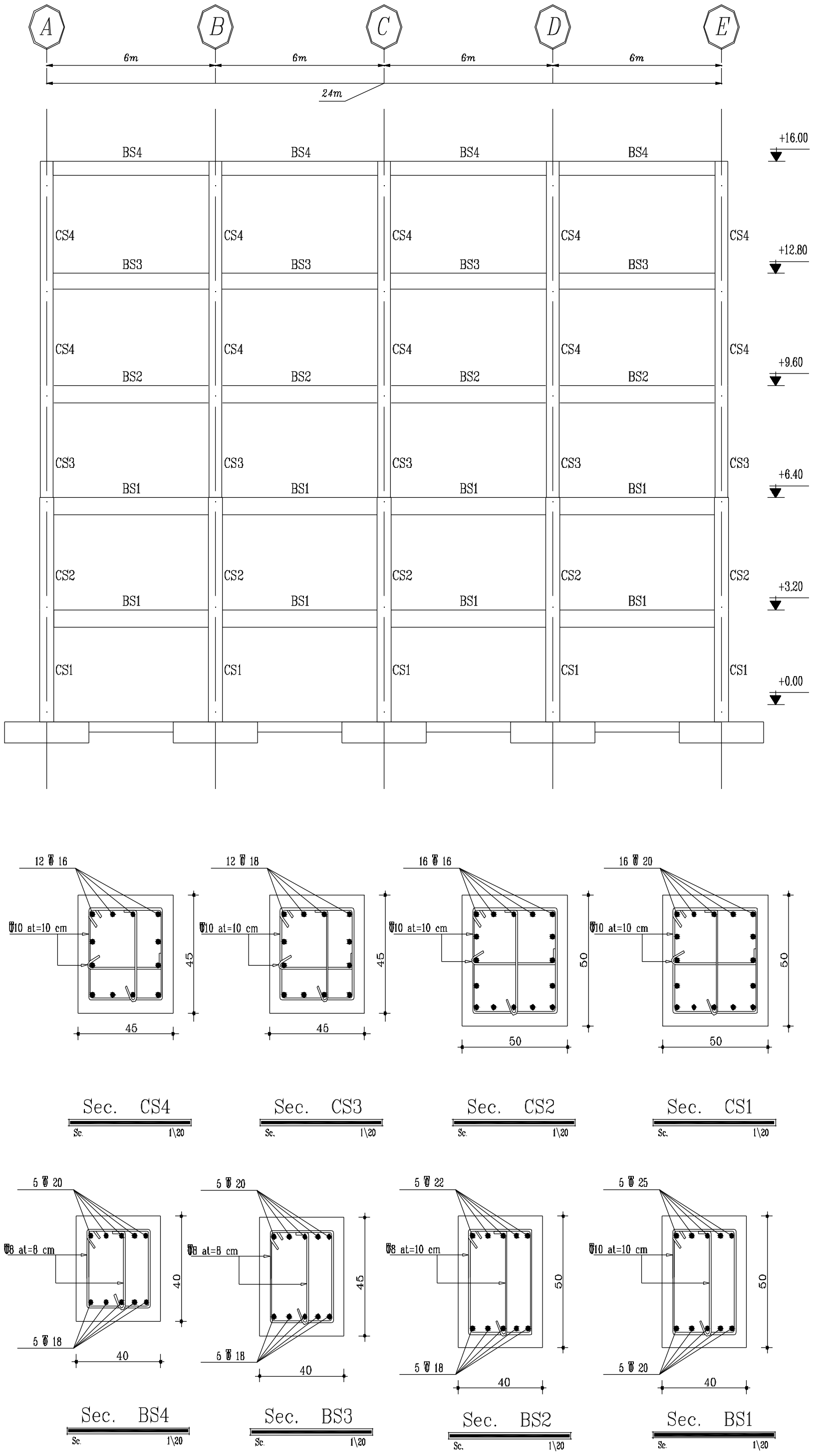

Figure 1. Details for the 5-story reinforced concrete special moment frame

Element	Story	b(mm)	h(mm)	ρ^{*}	ρ_p^{**}	ρ_{sh}	S(mm)
Beams	st-1	400	500	0.013	0.009	0.0014	100
	st-2	400	500	0.013	0.009	0.0014	100
	st-3	400	500	0.011	0.008	0.0009	100
	st-4	400	450	0.011	0.009	0.001	80
	st-5	400	400	0.012	0.01	0.0011	80
Columns	st-1	500	500	0.02	-----	0.0013	100
	st-2	500	500	0.013	------	0.0013	100
	st-3	450	450	0.015	------	0.0016	100
	st-4	450	450	0.012	------	0.0016	100
	st-5	450	450	0.012	------	0.0016	100

* Top Reinforcement Ratio

** Bottom Reinforcement Ratio

Table 1. Elements Properties of 5-srory Reinforced Concrete special Moment Frame

Element	Story	b(mm)	h(mm)	ρ	ρ_p	ρ_{sh}	S(mm)
Beams	st-1	500	600	0.010	0.007	0.0012	125
	st-2	500	600	0.013	0.009	0.0012	125
	st-3	500	550	0.012	0.009	0.0013	120
	st-4	500	550	0.012	0.009	0.0013	120
	st-5	500	550	0.012	0.009	0.0013	120
	st-6	500	500	0.013	0.009	0.0009	100
	st-7	500	500	0.013	0.009	0.0009	100
	st-8	500	500	0.011	0.008	0.0009	100
	st-9	500	450	0.011	0.009	0.0011	80
	st-10	500	400	0.011	0.008	0.0012	80
Columns	st-1	600	600	0.021	0.021	0.0016	100
	st-2	600	600	0.014	------	0.0011	100
	st-3	550	550	0.017	------	0.0014	100
	st-4	550	550	0.017	------	0.0014	100
	st-5	550	550	0.017	------	0.0014	100
	st-6	500	500	0.016	------	0.0013	100
	st-7	500	500	0.016	------	0.0013	100
	st-8	500	500	0.015	------	0.0013	100
	st-9	500	500	0.012	------	0.0013	100
	st-10	500	500	0.012	------	0.0013	100

Table 2. Element Properties of 10-srory Reinforced Concrete special Moment Frame

2.2 Term of Comparison

The comparison of selected procedures is 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 (Iranian code of practice for seismic resisting design of building) for the soil type 2; 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 different approaches are determined as follow:

Element	Story	b(mm)	h(mm)	ρ	ρ_p	ρ_{sh}	S(mm)
Beams	st-1	500	700	0.007	0.005	0.0010	125
	st-2	500	700	0.009	0.006	0.0010	125
	st-3	500	650	0.011	0.008	0.0011	120
	st-4	500	650	0.011	0.008	0.0011	120
	st-5	500	650	0.012	0.008	0.0011	120
	st-6	500	600	0.013	0.009	0.0009	100
	st-7	500	600	0.013	0.009	0.0008	100
	st-8	500	600	0.013	0.009	0.0008	100
	st-9	500	500	0.013	0.009	0.0009	100
	st-10	500	500	0.013	0.009	0.0009	100
	st-11	500	500	0.014	0.009	0.0009	100
	st-12	500	500	0.012	0.009	0.0009	100
	st-13	500	500	0.010	0.007	0.0009	100
	st-14	500	400	0.012	0.009	0.0012	80
	st-15	500	400	0.011	0.009	0.0012	80
Columns	st-1	700	700	0.020	------	0.0014	100
	st-2	700	700	0.017	------	0.0014	100
	st-3	700	700	0.016	------	0.0014	100
	st-4	650	650	0.023	------	0.0017	100
	st-5	650	650	0.012	------	0.0017	100
	st-6	600	600	0.014	------	0.0012	100
	st-7	600	600	0.014	------	0.0012	100
	st-8	600	600	0.014	------	0.0012	100
	st-9	550	550	0.010	------	0.0010	100
	st-10	550	550	0.010	------	0.0010	100
	st-11	500	500	0.020	------	0.0018	100
	st-12	500	500	0.013	------	0.0013	100
	st-13	500	500	0.010	------	0.0013	100
	st-14	500	500	0.010	------	0.0013	100
	st-15	500	500	0.010	------	0.0013	100

Table 3. Element Properties of 15-srory Reinforced Concrete special Moment Frame

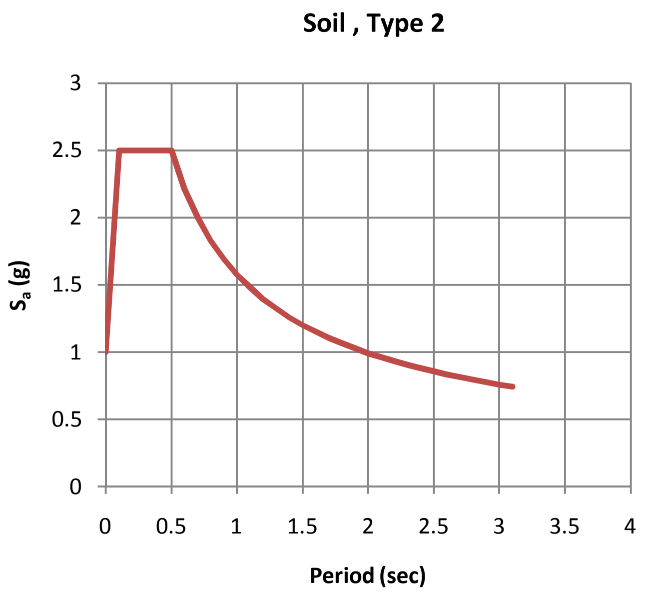

Figure 2: Standard No. 2800-05 acceleration spectrum for soil Type 2

1. The nonlinear static approach of FEMA440 (Displacement Modification):

$$(PGA_f)_{FEMA440} = \frac{(V_{prob}/W_t).(\delta_u/\delta_t)}{C(T_1)} \tag{5}$$

2. The force-based NZ procedure:

$$(PGA_f)_{NZ,force} = \frac{(V_{prob}/W_t).\mu_{sc}}{C(T_1)} \tag{6}$$

3. The displacement-based NZ procedure:

$$(PGA_f)_{NZ,displacement} = \frac{U_{el}}{C(T_{eff}).({T_{eff}}^2/4\pi^2).g\,.k(\xi_{eff})} \tag{7}$$

Where :

$$T_{eff} = 2\pi\sqrt{\frac{M_e}{K_{eff}}} \tag{8}$$

$$K_\xi = \sqrt{\frac{7}{2+\xi}} \tag{9}$$

3 NON-LINEAR DYNAMIC

To estimate the response of the frames under earthquake, nonlinear dynamic analysis is done using twenty-two acceleration time histories as described in Table 4. Records are selected from the PEER-NGA strong motion database which recommended by FEMAP695 [11].

Park and Ang damage index is used as acceptance criteria for components as below [10]:

$$\text{DamageIndex} = \frac{\delta_{max}}{\delta_u} + \frac{\beta}{F_u.\delta_u}\int dE \tag{10}$$

Where δmax is the peak deformation, δu is the ultimate deformation capacity under monotonic loading, Fu is the calculated yield strength, β is the calibration parameter for cyclic damage and is considered to equal 0.2 in this study, and E is the dissipated energy. Maximum rotation and yield moment under monotonic loading are selected as ultimate deformation and yield strength, respectively. DI is considered equal to one for the severe damage regards to CP levels based on the classification suggested by Park et al. [12].

After doing nonlinear dynamic analysis for all above mentioned records and finding PGA_f's for each one, Minitab [13] as a software was used to fit best probabilistic distribution on 22 data's. The variability in the PGA_f is best described by a lognormal distribution so present study uses average of natural log dates instead of simply average.

ID No	Record Seq. No.	Earthquake		File Names	
		Year	Name	Component 1	Component 2
1	953	1994	Northridge	MUL009	MUL279
2	960	1994	Northridge	LOS000	LOS270
3	1602	1999	Duzce,Turkey	BOL000	BOL090
4	1787	1999	Hectot Mine	HEC000	HEC090
5	169	1979	Imperial Valley	H-DLT262	H-DLT352
6	174	1979	Imperial Valley	H-E11140	H-E11230
7	1111	1995	Kobe,Japon	NIS000	NIS090
8	1116	1995	Kobe,Japon	SHI000	SHI090
9	1158	1999	Kocaeli,Turkey	DZC180	DZC270
10	1148	1999	Kocaeli,Turkey	ARC090	ARC000
11	900	1992	Landers	YER270	YER360
12	848	1992	Landers	CLW-LN	CLW-TR
13	752	1989	Loma Perieta	CAP000	CAP090
14	767	1989	Loma Perieta	GO3000	GO3090
15	1633	1190	Manjil,Iran	ABBAR--L	ABBAR—T
16	721	1987	Superstition Hills	ICC000	ICC090
17	725	1987	Superstition Hills	POE270	POE360
18	829	1992	Cape Mendocino	RIO270	RIO360
19	1244	1999	Chi-Chi, Taiwan	CHY101-E	CHY101-N
20	1485	1999	Chi-Chi, Taiwan	TCU045-E	TCU045-N
21	68	1971	San Fernando	PEL090	PEL180
22	125	1976	Friuli,Italy	A-TMZ000	A-TMZ270

Table 4. Selected records for the nonlinear dynamic analysis

4 NON-LINEAR NUMERICAL ANALYSIS

Available element models generally do not accurately represent the full range of behavior (low level, frequent ground motions which contribute most to damage and financial loss as well as high level, rare ground motions which contribute most to collapse risk). Therefore, plastic hinge model to capture strength and stiffness deterioration and collapse, are used. The plastic hinge model also includes the beam-column element lumps the bond-slip and beam column yielding response into one concentrated hinge. Due to modern capacity design provisions for RC SMRF buildings, shear failure is not expected for the elements of RC SMRF buildings, so only flexural damage is modeled.

OpenSees was used for the structural analyses. P-Delta effects are accounted for using a combination of gravity loads on the lateral-resisting frame and gravity loads on a leaning column element. The model includes 5% Rayleigh damping anchored to the first and third modal periods [14]. As shown in Figure 3, plastic hinge models for beam columns have a trilinear backbone curve described by five parameters (M_y , θ_y , K_s , $\theta_{cap,pl}$, and K_c). The model captures the four important modes of cyclic strength and stiffness degradation; this is based on an energy dissipation capacity and a term that describes how the deterioration rate changes as damage accumulates. Model parameters (for initial stiffness and deformation capacity) of RC beam columns are based on recommendations from Fardis et al. [15] and Haselton calibrations to test data using the PEER structural performance database [16].

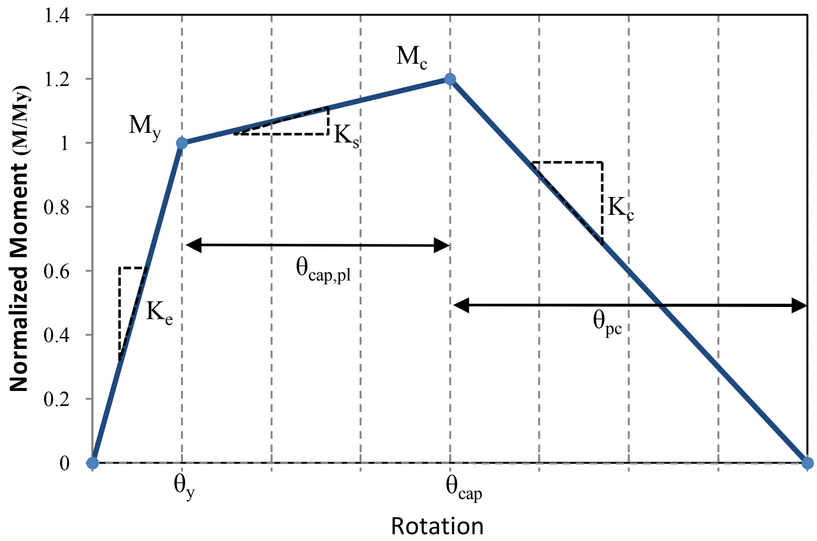

Figure 3: Monotonic backbone curve

5 DISCUSSION AND CONCLUSION

The PGA_f values that cause the collapse in the first element f the frames are reported in Table 5 and shown in Figure 4 for all the approaches. Figure 5 illustrates percent of error among each nonlinear static procedures and nonlinear time history analyses. As shown, the result of nonlinear dynamic analysis is more compatible with the New Zealand force approach and also has lower error.

No. Story	Collapse Peak Ground Acceleration (PGA_f) in Units of g				
	New Zealand		FEMA440		Time History
	Force Based	Displacement	Displacement Modification	Equivalent Linearization	
5	0.55	0.77	0.75	0.72	0.48
10	0.59	0.72	0.76	0.71	0.51
15	0.62	0.68	0.68	0.77	0.54

Table 5: The PGA_f values that cause the collapse

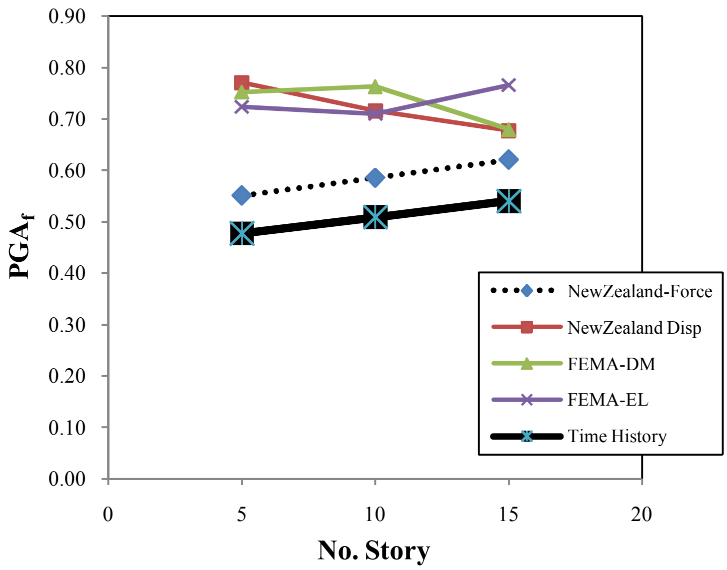

Figure 4: The PGA values cause the collapse from different approaches

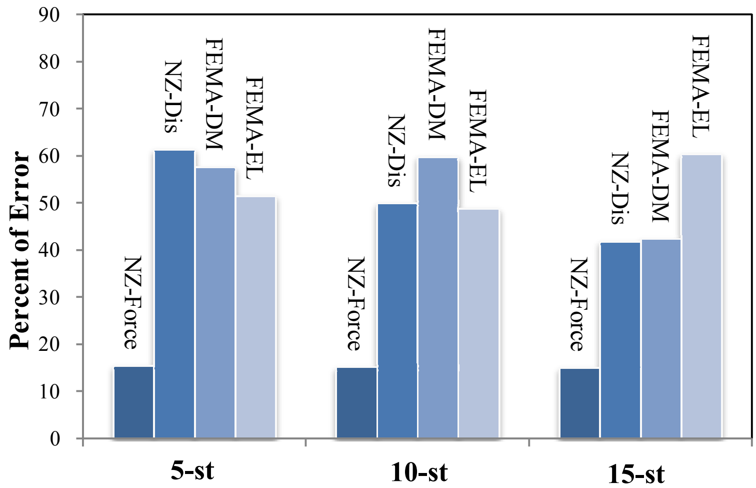

Figure 5: Error Percentage

From the figure 5, it can be concluded that the New Zealand force approach is most compatibility with the nonlinear dynamic analyses in all cases. The New Zealand displacement approach gives appropriate results for all frames but its results are not conservative.

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