BUILDING HEIGHT EFFECTS ON NONLINEAR TIME HSITORY RESPONSES OF MOMENET RESISTING STEEL FRAME USING FAM

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Abstract

This paper presents the effects of building heights on seismic responses of two dimensional mid-rise moment resisting steel frames by performing Nonlinear Time History Analysis (NTHA) using Force Analogy Method (FAM). 6-storey and 10-storey moment resisting steel frames subjected to seven earthquake ground motion records having different earthquake characteristics (*viz*., Peak Ground Velocity (PGV), dominant period of ground excitation and duration of strong motion) have been considered for the study. Based on the time history analysis, it has been observed that although, both PGV and dominant period of ground excitation have been found to affect the seismic responses of both the building frames; it is seen that long period ground excitation has more effect on the higher (long period) building frame (10-storey), in comparison to the lower (6-storey) height building frame, wherein, the effect of short period ground excitation is more. Thus, seismic response of building frame has been found to increase as the height of the building increases in case of long period ground velocity excitation; whereas seismic response of building frame has been found to decrease as the height of the building increases in case of short period ground velocity excitation.

Keywords: Force Analogy Method, nonlinear time history analysis, ground velocity excitation period, story drift ratio.

1. Introduction

Seismic responses of the building frames depend on both earthquake characteristics *viz*., peak ground acceleration (PGA) and peak ground velocity to acceleration ratio (V/A) [1], peak ground velocity (PGV) [2], dominant period of ground motion and effective response duration [1, 2]; and building properties *viz*., dominant frequency content and fundamental periods. Thus, building frames of different heights will respond differently when excited by the same earthquake; and also the same building will respond differently when excited to different ground motions having different earthquake characteristics. In this paper, an attempt has been made to assess the effect of building height on seismic response of building frames. As far as seismic analysis of building frame is concerned, there are broadly four different methods of seismic analysis [3] e.g., i) Linear Static Procedure, LSP (see e.g. [4, 5]) ii) Linear Dynamic Procedure, LDP (see e.g. [6, 7]), iii) Nonlinear Static Procedure, NSP (see e.g. [8], 9, 10]), and iv) Nonlinear Dynamic Procedure, NDP (see e.g., [2, 11]). Among the four aforementioned methods of seismic analysis, NDP (more specifically known as Nonlinear Time History Analysis, NTHA) is generally considered as the most accurate, robust and detailed method of seismic analysis [11], since it can not only takes care of the nonlinear behaviours (material as well as geometric) but also the dynamic behaviour of building frames. In the early days, NTHA was not widely adopted due to the requirements of lot of computational efforts. However, with the advancement in computer technology, it became relatively easier to conduct Nonlinear Time History Analysis (NTHA). In 1997, Meyer [12] performed NTHA on a frame structure, for the first time, to the best of author's knowledge, using a conventional (updated or variable stiffness matrix approach) Finite Element (FE) based computer program/software called NLDYN. Subsequently, in a similar line, researchers like, Uang *et al*., (1997) [13]; Ventura and Ding (2000) [14]; Mulas (2004) [15]; Krishnan and Muto (2013) [2]; Hariri-ardebili *et al*. (2013) [16] and Mokarram and Banan, (2018) [17] used conventional FE based programs/softwares like DRAIN-2DX; CANNY-E; STEFAN; FRAME3D; PERFORM-3D and OpenSees respectively, to perform

NTHA of moment resisting steel frames. Although, this method (i.e. NTHA) is the preferred choice, for a complete and detailed seismic analysis of frame structures; in practice, the use of this NTHA method is very limited; owing due to the requirement of large computational efforts that may arise in solving non-linear dynamic equations (see e.g. [11, 17]). This may be linked to the use of conventional Finite Element (FE) based solution technique in the Conventional Structural Dynamic Analysis (CSDA) method, wherein the stiffness matrix needs to be updated as the solution progresses, to capture the force reduction beyond attainment of material yielding. However, with the introduction of Force Analogy Method (FAM) by Wong and Yang (1999) [18], the computation workload for Nonlinear Time History Analysis (NTHA) can now be substantially reduced. In a case study [19], it has been reported that the computational time efficiency of FAM is reduced as much as by 70% as compared to that of the solution by SAP2000 (2011) which employs the CSDM. Thus, in this paper, an attempt has been made, based on FAM methodology, to assess the effect of building heights on seismic responses of building frames. For the investigation, two building frames of different heights (i.e. 6-storey and 10-storey) have been considered. FAM based NTHA considering both material and geometric nonlinearities has been conducted using seven earthquake ground motion records (*viz*., Kobe, 1995; Northridge, 1994; El_Centro, 1940; Imperial Valley, 1979; Loma Prieta, 1989; Fruili, 1976; and Kocaeli, 1999) having different earthquake characteristics.

2. Force Analogy Method (FAM)

FAM is an algorithm to study the inelastic behaviour of structural system considering only the initial stiffness matrix throughout the entire nonlinear analysis. The algorithm has been based on inelastic displacement concept proposed by Lin in 1968 [21] where inelastic behaviour of structural member is determined by changing the structural displacement field which is different from conventional method of changing stiffness approach. The algorithm has also been seen to achieve a high degree of accuracy due to incorporating state space numerical integration technique in structural dynamic analysis; as well as, it became numerically stable algorithm because it can analyse not only strain-hardening problems, but also elastic-perfectly plastic and strain-softening problems. FAM has been used in different areas of research works; for example, RCC works [22, 19], and performed energy based seismic analysis [25, 26].

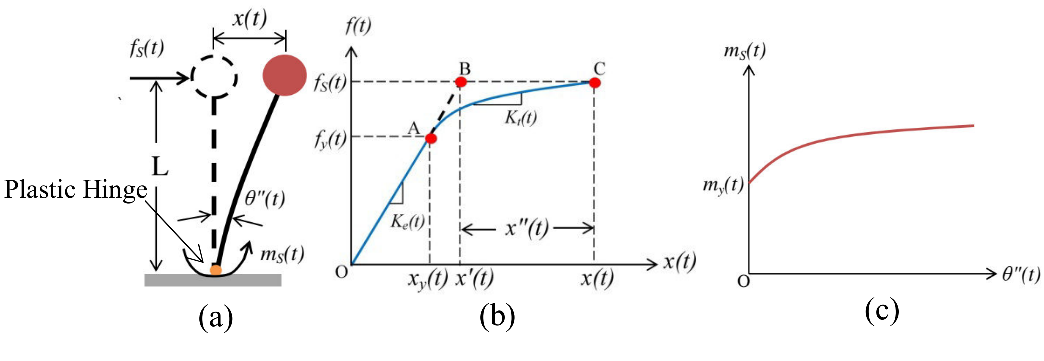


Fig. 1 – Framework of the FAM: (a) SDOF system, (b) force-displacement relationship and (c) moment-plastic rotation relationship

The basic concept of FAM is briefly described as per Fig. 1. It has been seen that when lateral force ($f_S(t)$) excites a Single Degree of Freedom (SDOF) system, lateral deformation ($x(t)$) and support moment ($m_S(t)$) are formed as shown in Fig. 1(a). Plastic hinge is observed to locate at the support of SDOF system which will measure the plastic rotation ($\theta''(t)$) caused by the moment demand in excess of the yield moment capacity of the member. Fig. 1(b) shows force-deformation curve where, $K_e(t)$ and $K_t(t)$ represent the initial elastic stiffness and the varying post-yield stiffness respectively. Point C in Fig.1 (b) indicates the lateral deformation ($x(t)$) of SDOF system. The fundamental concept of FAM is to extend the initial stiffness line OA until it reaches the force $f_S(t)$ level at point B (see Fig. 1(b)). The displacement corresponding to point B is termed as elastic displacement ($x'(t)$) and the difference between the total displacement ($x(t)$) and elastic displacement ($x'(t)$) is known as the inelastic or plastic displacement ($x''(t)$). Fig. 1(c) indicates moment rotation relationship.

For a moment-resisting frame with n degrees of freedom (DOFs) and p plastic hinge locations (PHLs), the total displacement $x(t)$ and total moment $m_S(t)$ at each DOF and PHL are respectively given by the summations of elastic displacement $x'(t)$ and inelastic displacement $x''(t)$; and elastic moment $m_S'(t)$ and inelastic (residual) moment $m_S''(t)$.

$$x(t) = x'(t) + x''(t) \tag{1}$$

$$m_S(t) = m_S'(t) + m_S''(t) \tag{2}$$

The displacements in Eq. (1) and moments in Eq. (2) are inter-related by the following expressions [25].

$$m_S'(t) = K'(t)^T + x'(t) \tag{3}$$

$$m_S''(t) = -[K''(t) - K'(t)^T K(t)^{-1} K'(t)]\theta''(t) \tag{4}$$

where, $K(t)$ is $n \times n$ global stiffness matrix, $K'(t)$ is $n \times p$ stiffness matrix formed by relating plastic rotations at the PHLs with the restoring forces at the DOFs, $K''(t)$ is the $p \times p$ stiffness matrix formed by relating plastic rotations with corresponding residual moments at the PHLs and $\theta''(t)$ is the plastic rotation at each PHL (refer [18]).

Putting Eqs. (3) and (4) into Eq. (2) and rearranging the terms gives the first governing of FAM for dynamic analysis as:

$$m_S(t) = m_S'(t) + m_S''(t) \tag{5}$$

The second governing equation of FAM relates the inelastic displacement $x''(t)$ with the plastic rotation $\theta''(t)$.

$$x''(t) = K(t)^{-1} K'(t)\theta''(t) \tag{6}$$

In FAM, plastic energy (PE) dissipation is determined by multiplying elastic moment (m') with change in plastic rotation (θ'') [26].

$$PE = \int_0^t x_d'(t)\bar{K}_L dx_d'' = \int_0^t x_d'(t) K_p d\theta'' = \int_0^t m'(t) d\theta'' = \sum_{k=1}^{t_k} m_{i,k}(\theta_{i,k}'' - \theta_{i,k-1}'') \tag{7}$$

3. Problem Description

A 10-storey moment resisting steel frame (see Fig. 2b) referred in [27] has been adopted as first frame in this paper. All the frame members have been considered axially rigid, giving a total of 50 degree of freedoms (DOFs) (i.e. from x_1 to x_{50}). Plastic hinges have been assigned at both the ends of each member, thereby giving a total of 140 plastic hinge locations (PHLs). Mass moment of inertias at the rotational DOFs (i.e. from x_{11} to x_{50}) have been ignored which helps to condense the size of the matrices by static condensation method. All the 40 rotational DOFs (i.e. from x_{11} to x_{50}) have been eliminated thereby reducing to only 10 translational DOFs (i.e. from x_1 to x_{10}). Structural member section properties have been provided as per ASTM A6/A6M–07 and section details are represented in Table 1. Damping of 3% has been assumed for these steel sections. The storey mass for each floor has been considered as 218.9 Mega-gram (Mg) which gives total mass of 2189 Mg for the entire building frame.

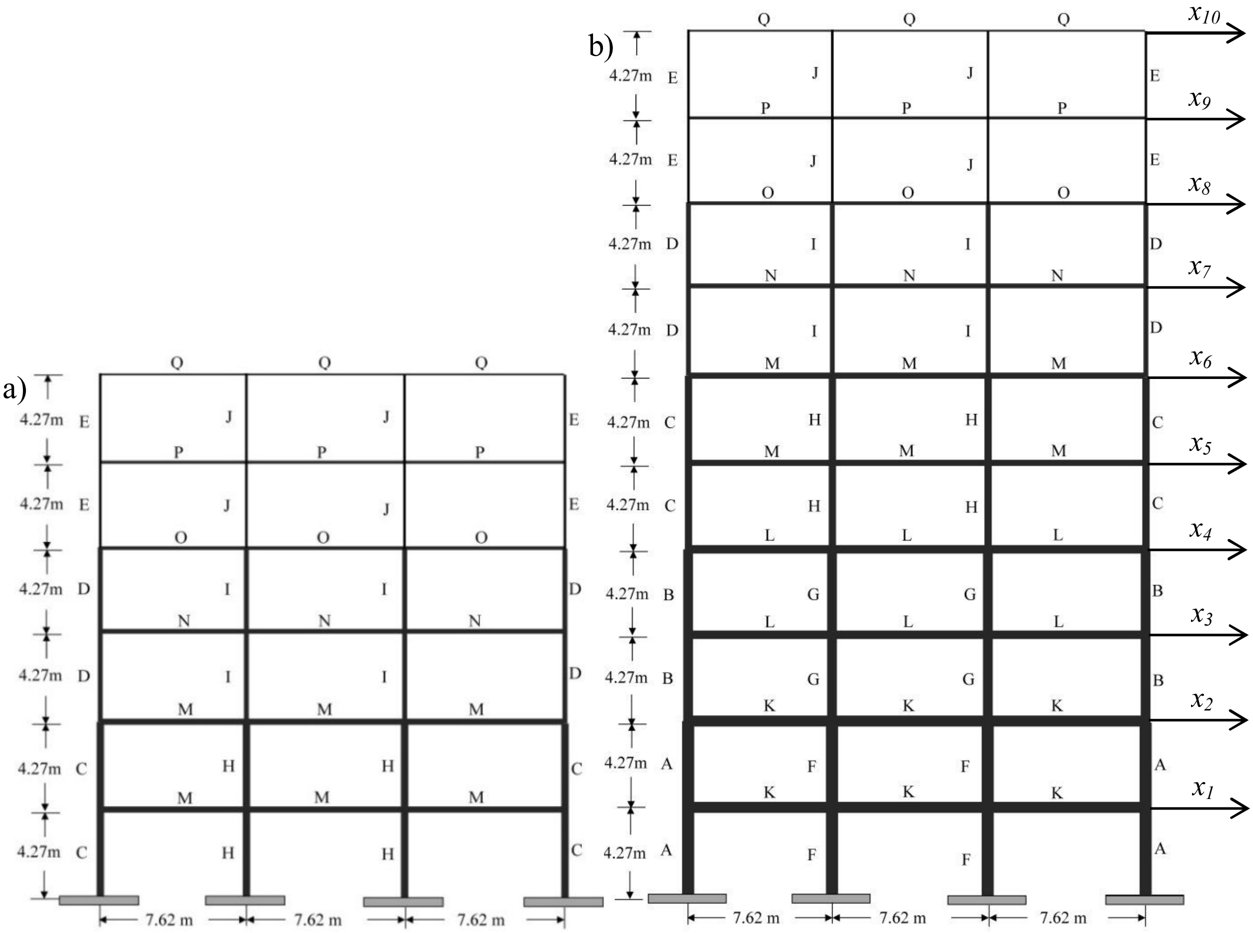


Fig. 2 – a) 6-Sotey and b) 10-Storey moment resisting steel frame showing sections

Table 1 – Section properties used in 6 and 10-storey moment resisting steel frames

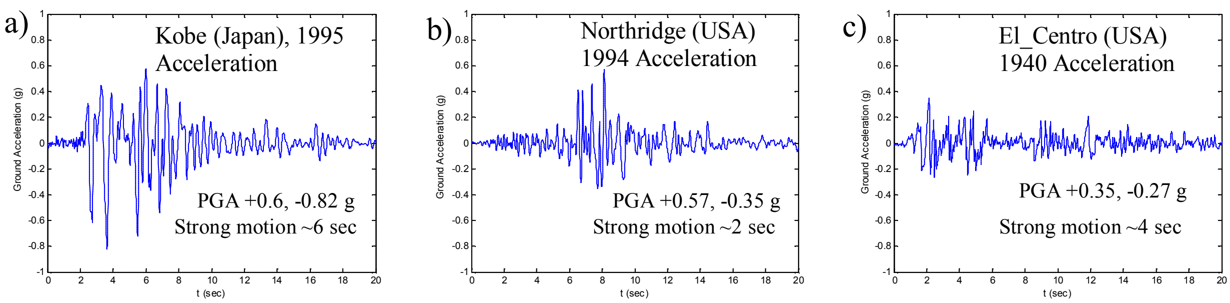
Type	Sections	Moment of Inertia, (I) × 10^{-3} m^4	Plastic Section Modulus, (Z) × 10^{-3} m^3	Yield Moment, (M_y) KN-m
A	W14 × 370	2.26	12.06	3015.21
B	W14 × 311	1.80	9.88	2470.34
C	W14 × 283	1.59	8.88	2220.44
D	W14 × 257	1.41	7.98	1995.12
E	W14 × 193	0.99	5.81	1454.35
F	W14 × 605	4.49	21.63	5407.73
G	W14 × 550	3.92	19.33	4834.18
H	W14 × 455	2.99	15.33	3834.57
I	W14 × 426	2.74	14.24	3560.08
J	W14 × 342	2.03	11.01	2753.02
K	W36 × 260	7.20	17.43	4358.95
L	W36 × 230	6.24	15.25	3814.08
M	W36 × 210	5.49	13.65	3412.60
N	W36 × 182	4.70	11.76	2941.47
O	W36 × 150	3.76	9.52	2380.22
P	W36 × 135	3.24	8.34	2085.25
Q	W27 × 94	1.36	4.55	1138.90

In order to investigate the effect of building height on seismic responses, the second frame i.e. 6-storey moment resisting steel frame, has been developed from the first frame (i.e. 10-storey) by eliminating the bottom four storeys as shown in Fig. 2a. Rest of the building's member sections and floor-wise loading remains same. Building properties i.e. time periods and frequency contents of both 10-storey and 6-storey building frames are given in Table 2.

Table 2 – Building periods and frequencies for different modes of 6 and 10-storey steel frames

	6-storey		10-storey	
Modes	Periods (Sec)	Frequency (H_Z)	Periods (Sec)	Frequency (H_Z)
1	1.05	0.95	1.54	0.64
2	0.38	2.61	0.55	1.78
3	0.22	4.43	0.33	2.95
4	0.15	6.37	0.23	4.22
5	0.11	8.48	0.18	5.52
6	0.09	10.5	0.14	6.76
7			0.12	8.15
8			0.10	9.48
9			0.09	10.58
10			0.08	12.12

Seven earthquake ground motion records having different earthquake characteristics (*viz*., Kobe, 1995; Northridge, 1994; El_Centro, 1940; Imperial Valley, 1979; Loma Prieta, 1989; Friuli, 1976; and Kocaeli, 1999) have been considered for the analysis. Ground acceleration and ground velocity records for all the seven earthquakes are shown in Figs. 3 and 4. Details of the seven earthquakes (i.e. PGA, PGV and dominant time periods,) are also shown in Table 3.



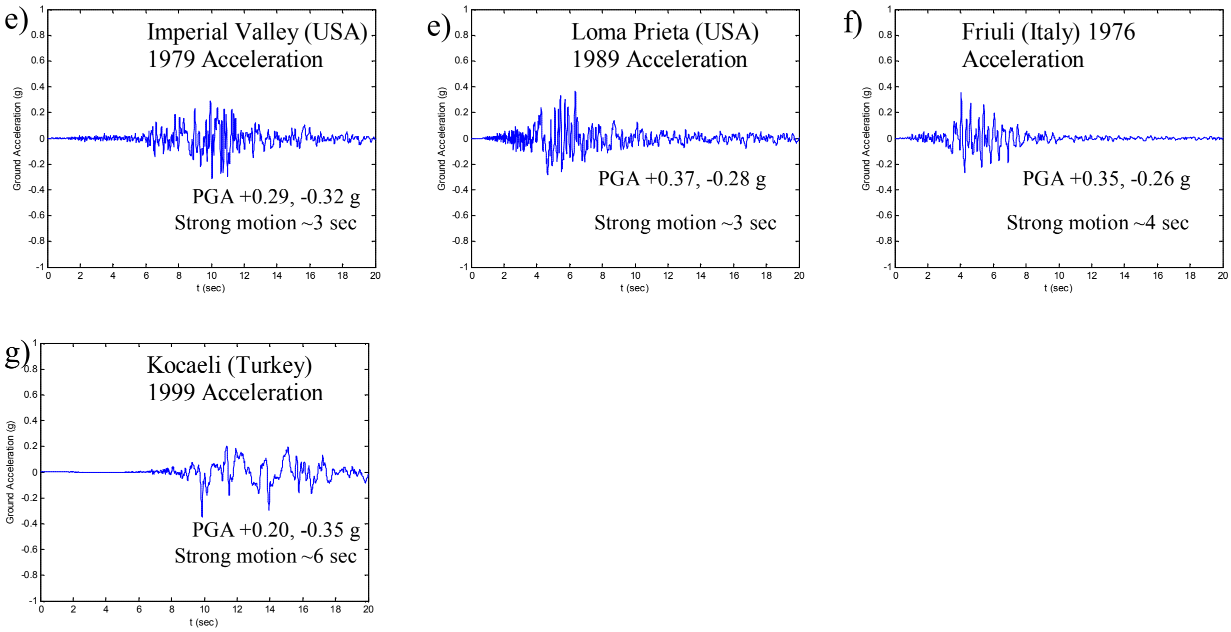


Fig. 3 – Seven earthquake ground acceleration records [28]

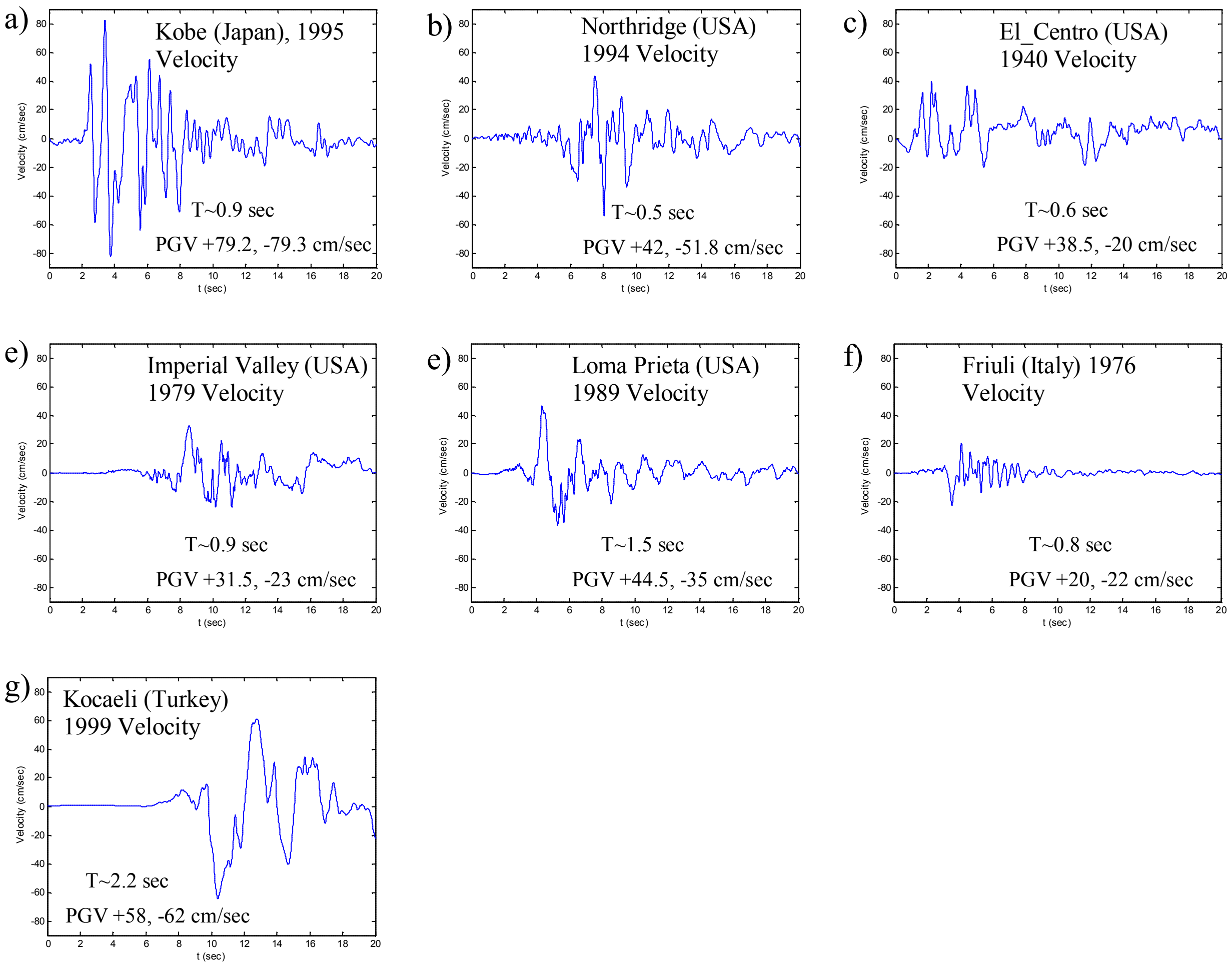


Fig. 4 – Seven earthquake ground velocity records [28]

Table 3 – Earthquakes record data (Courtesy: Strong-motion virtual data center [28])

	Earthquakes	Recording station:	PGA (g)	PGV (m/sec)	Dominant periods T (sec)
1	The Kobe (Japan) January 16, 1995.	KJMA, Japan Meteorological Agency	+0.6, -0.82	+79.2, -79.3	0.9
2	The Northridge (USA) January 17, 1994.	090 CDMG STATION 24278	+0.57, -0.35	+42, -51.8	0.5
3	El_Centro (USA) May 19, 1940.	USGS STATION 0117	+0.35, -0.27	+38.5, -20	0.6
4	The Imperial Valley (USA) October 15, 1979.	USGS STATION 5115	+0.29, -0.32	+31.5, -23	0.9
5	The Loma Prieta (USA) October 18, 1989.	090 CDMG STATION 47381	+0.37, -0.28	+44.5, -35	1.5
6	The Friuli (Italy) May 06, 1976.	TOLMEZZO(000)	+0.35, -0.26	+20, -22	0.8
7	The Kocaeli (Turkey) August 17, 1999.	YARIMCA(KOERI330)	+0.20, -0.35	+58, -62	2.2

4. Effect of earthquake ground motion on 10-storey steel frame

The story drift ratio and plastic energy dissipation of the 10-storey steel frame for the seven earthquakes are shown in Fig. 5. It has been observed that among the seven earthquakes, three earthquakes (i.e. El_Centro, 1940; Imperial Valley, 1979; and Friuli, 1976) resulted in linear responses as indicated by the absence of plastic energy dissipation for these three earthquakes (see Fig. 5b, Table 4); whereas the remaining four earthquakes (i.e. Kobe, 1995; Northridge, 1994; Loma Prieta, 1989; and Kocaeli, 1999) resulted in nonlinear responses as indicated by plastic energy dissipation (see Fig. 5b, Table 4). The non-linear responses may be related to the critical characteristics of these four earthquakes (i.e. peak ground acceleration (PGA) and peak ground velocity to acceleration ratio (V/A) [1], peak ground velocity (PGV) [2], dominant period of ground motion and effective response duration [1, 2]. Among the seven earthquakes, Kobe earthquake has been found to give the maximum seismic responses since Kobe earthquake has highest PGA (0.82g) (refer Table 3), highest PGV (79.3 m/sec) (refer Table 3), dominant period of ground excitation (0.9 sec) (refer Table 3) close to 10-storey fundamental period (1.54 sec) (refer Table 2), and effective longer duration of strong motion (0.6 sec) (see Figure 3a). It has also been observed that Kocaeli earthquake has found to give higher seismic response than Northridge earthquake for 10-storey steel frame (see Fig. 5) although PGA of Northridge (i.e. 0.57 g) is more than that of Kocaeli (i.e. 0.35 g) (refer Table 3). It may relate with the findings of [29] that PGA which is dominated by the higher frequencies has been found to have little effect on the seismic behaviour of long period (e.g. 10-storey steel frame) building since the amplitudes of high frequency ground acceleration are likely to have had attenuated significantly due to long propagation distance. Hence seismic behaviours of long period building frame has been found to have better correlation with PGV rather than PGA. Further long period building frames have been generally observed to be more responsive with long period seismic excitation [2]. Thus, the observed higher seismic responses of relatively longer period (1.54 sec) 10-storey steel frame due to Kocaeli earthquake may be associated with larger PGV (62 m/sec) and longer period (2.2 sec) of seismic excitation as compared to that of Northridge earthquake (PGV 51.8 m/sec and period of seismic excitation 0.5 sec). It has also been observed that PGV and period of ground motion of Loma Prieta earthquake are found to be 44.5 cm/sec and ~1.5 sec respectively (see Table 3). Thus, the sequence of earthquakes (giving nonlinear responses) in order of decreasing criticality for 10-storey steel frames is found to be as Kobe, Kocaeli, Northridge and Loma Prieta (see Fig. 5, Table 4). It has also been seen that maximum storey drift ratio and maximum plastic energy dissipation are found to occur within 30% to 40% building height for 10-storey steel frame for all seven earthquakes (see Fig. 5).

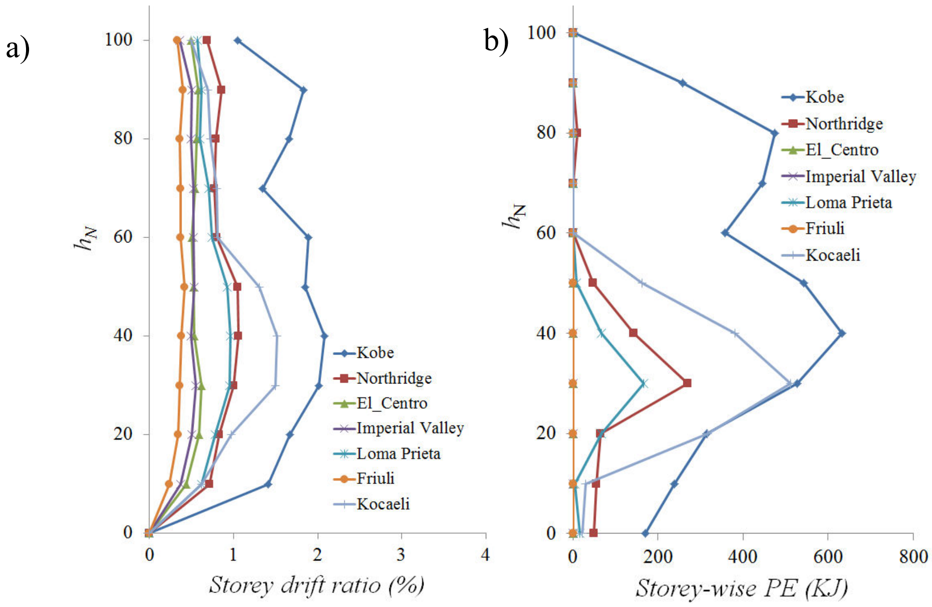


Fig. 5 – a) Storey drift ratio and b) plastic energy dissipation for 10-storey steel frame

Table 4 – Plastic energy dissipated in Kilo-Joules (KJ) for 10-storey steel frame

	Kobe	Northridge	El-Centro	Imperial Valley	Loma Prieta	Friuli	Kocaeli
Gr Floor	169.66	49.94	0.00	0.00	16.90	0.00	21.36
1st Floor	237.71	56.36	0.00	0.00	3.85	0.00	30.15
2nd Floor	314.88	64.53	0.00	0.00	66.71	0.00	316.01
3rd Floor	526.41	269.22	0.00	0.00	166.06	0.00	511.38
4th Floor	631.79	142.86	0.00	0.00	66.34	0.00	380.99
5th Floor	542.74	47.45	0.00	0.00	8.58	0.00	162.05
6th Floor	357.79	0.00	0.00	0.00	0.00	0.00	0.21
7th Floor	445.01	0.12	0.00	0.00	0.00	0.00	0.00
8th Floor	474.80	10.16	0.00	0.00	0.00	0.00	0.00
9th Floor	258.19	0.00	0.00	0.00	0.00	0.00	0.00
10th Floor	1.86	0.00	0.00	0.00	0.00	0.00	0.00
Total	**3960.86**	**640.63**	**0.00**	**0.00**	**328.44**	**0.00**	**1422.14**

5. Effect of earthquake ground motion on 6-storey steel frame

For 6-storey steel frame, it has been observed that among seven earthquakes, two earthquakes (i.e. Imperial Valley, 1979 and Friuli, 1976) resulted in linear responses as there is no plastic energy dissipation for these two earthquakes (see Fig. 6b, Table 5); whereas remaining earthquakes (i.e. Kobe, 1995; Northridge, 1994; El_Centro, 1940; Loma Prieta, 1989; and Kocaeli, 1999) resulted in nonlinear responses as indicated by plastic energy dissipation (see Fig. 6b, Table 5). Among seven earthquakes, Kobe earthquake has been found to give the maximum responses since Kobe earthquake has maximum PGA of 0.82g, PGV of 79.3 m/sec with dominant period of ground motion T of ~0.9 sec (see Table 3) which is close fundamental time period of 6-storey steel frame (i.e. 1.05 sec, refer Table 2). In case of 6-storey steel frame, Kocaeli earthquake has been found to give lesser seismic response than Northridge earthquake (see Fig. 6); it may be due to the reason that dominant period of ground motion of Northridge earthquake (i.e. 0.5 sec, refer Table 3) is much closer to the fundamental period of 6-storey steel frame (i.e. 1.05 sec, refer Table 2) than that of Kocaeli earthquake (i.e. 2.2 sec, refer Table 3). Thus, it has been observed that long period ground excitation has been seen to have less effect on short period building frame. The sequence of earthquakes (giving nonlinear responses) in order of decreasing criticality for 6-storey steel frames is found to be as Kobe, Northridge, El_Centro, Kocaeli and Loma Prieta (see Fig. 6, Table 5). Unlike 10-storey steel frame, it has also been observed that maximum storey drift ratio occurs at 20% building height whereas maximum plastic energy dissipation occur within 0% to 20% building height for 6-storey steel frame for all seven earthquakes (see Fig. 6).

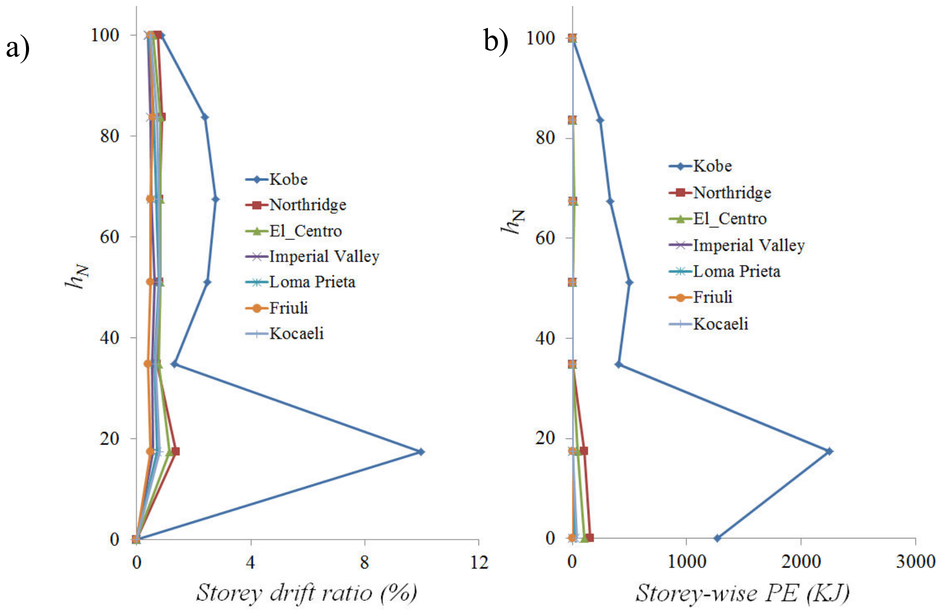


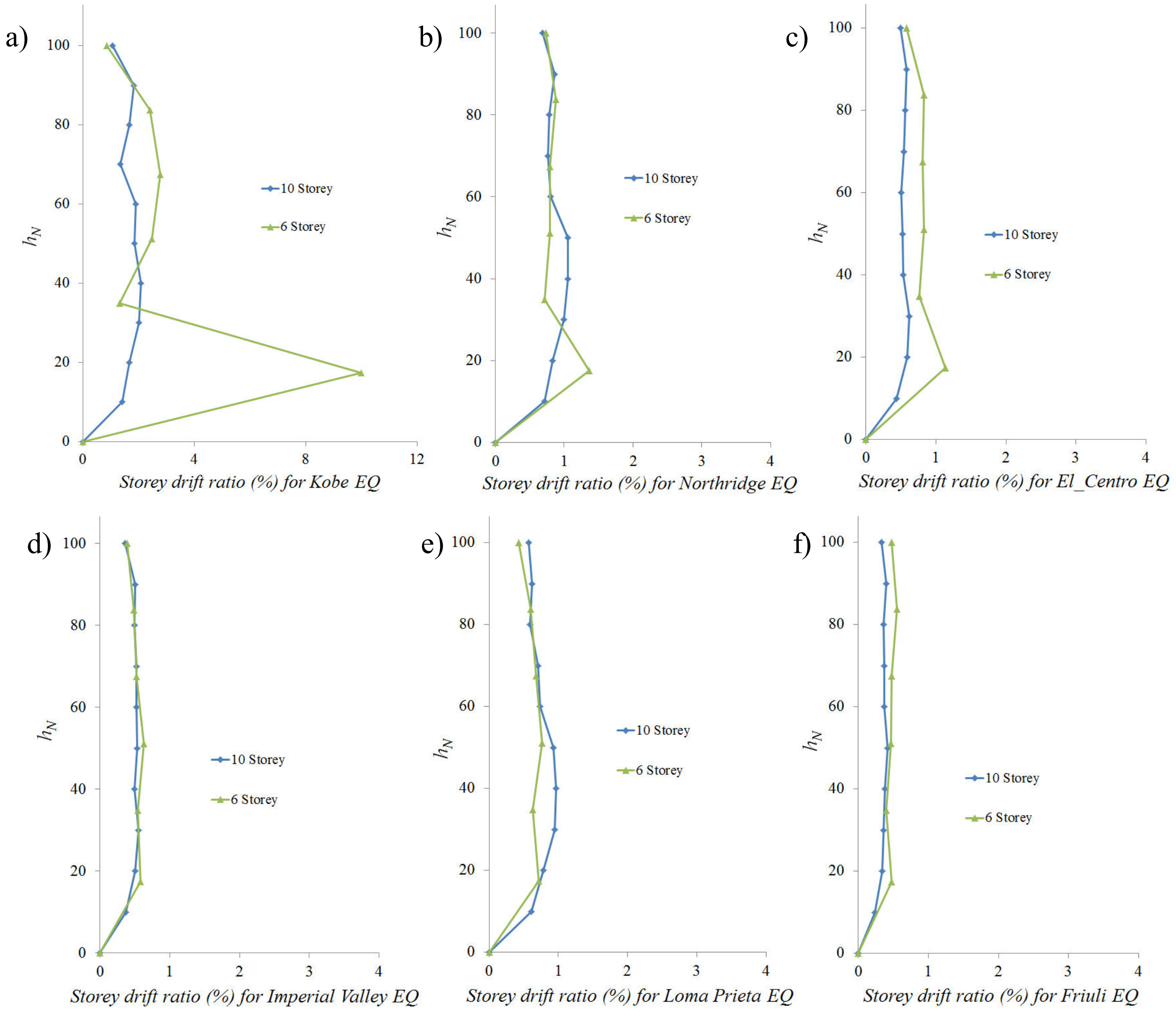
Fig. 6 – a) Storey drift ratio and b) plastic energy dissipation for 6-storey steel frame

Table 5 – Plastic energy dissipated in Kilo-Joules (KJ) for 6-storey steel frame

	Kobe	Northridge	El-Centro	Imperial Valley	Loma Prieta	Friuli	Kocaeli
Gr Floor	1265.32	159.46	108.50	0.00	22.41	0.00	42.77
1st Floor	2248.15	107.57	47.92	0.00	0.00	0.00	0.00
2nd Floor	404.55	0.00	0.00	0.00	0.00	0.00	0.00
3rd Floor	503.01	0.00	5.70	0.00	0.00	0.00	4.94
4th Floor	333.86	13.38	14.06	0.00	0.00	0.00	0.00
5th Floor	245.30	3.23	0.00	0.00	0.00	0.00	0.00
6th Floor	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Total	**5000.18**	**283.64**	**176.18**	**0.00**	**22.41**	**0.00**	**47.71**

6. Effect of building height on seismic responses of mid-rise moment resisting steel frames

The effect of storey height on seismic responses of mid-rise moment resisting steel frames is presented in the form of variation of storey drift ratio as shown in Fig. 7, for all seven earthquakes. It has been observed that seismic responses (i.e. storey drift ratio) increases as the height of the building frame decreases (i.e. from 10-storey to 6-storey) when subjected to Kobe earthquake (see Fig. 7a). It is because Kobe earthquake has shorter period of ground excitation (~0.9 sec) and hence shorter periods building frames, e.g. 6-storey (1.05 sec) is likely to respond more than the longer period 10-storey (1.54 sec) steel frame. Thus it has been seen that seismic responses increase as the height of building frame decreases for short period ground excitations (i.e. Kobe (T ~0.9 sec), Fig. 7a; Northridge (T ~0.5 sec), Fig. 7b; El_Centro (T ~0.6 sec), Fig. 7c; Imperial Valley (T ~0.9 sec), Fig. 7d; and Friuli (T ~0.8 sec), Fig. 7f); whereas seismic responses decrease as the height of building frame decreases for long period ground excitations (i.e. Loma Prieta (T ~1.5 sec), Fig. 7e; and Kocaeli (T ~2.2 sec), Fig. 7g).



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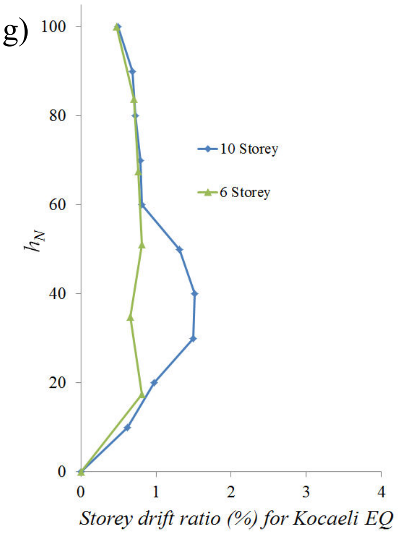

Fig. 7 – Storey drift ratio for a) Kobe EQ. b) Northridge EQ. c) El_Centro EQ. d) Imperial Valley EQ. e) Loma Prieta EQ. f) Friuli EQ. and g) Kocaeli EQ.

6. Conclusion

This paper presents the effects of building heights on seismic responses of two dimensional mid-rise moment resisting steel frames by performing NTHA using implemented FAM code in Matlab considering both material and geometric nonlinearities. Seven earthquake ground acceleration records have been considered in the analysis. The results are presented in the form of storey drift ratio and plastic energy dissipation. The following conclusions are drawn from the above analysis.

1. Maximum amount of plastic energy has been found to dissipate at the location where there is maximum storey drift ratio along the building height.

2. The seismic response of the building frame is found maximum when the input ground motion has critical characteristics like maximum PGA and PGV values, periods of ground excitation close to building fundamental period and large duration of strong motion.

3. Seismic response of the building frame has been found to increase when the height of the building frame decreases for short period ground excitation; on the other hand, the seismic response has been found to decrease when the height of the building frame decreases for long period ground excitation.

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