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Peak Horizontal Floor Accelerations due to Near-Fault Ground Motion

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ABSTRACT: This study uses representative numerical models of four reinforced concrete frame buildings and several types of ground motion for incremental dynamic analysis and investigates the distribution of peak horizontal floor acceleration along the height of buildings in response to slight and major damage. Ground motions in a near-fault region have special characteristics, particularly when these motions contain forward directivity effects such as intense velocity pulses. The seismic design of structures located close to active faults must account for these characteristics. This study compares peak horizontal floor acceleration from two near-fault data sets; one containing 30 pulse-type motions with forward directivity effects and the other containing 20 motions without forward directivity effects. The results show that current seismic code provisions do not provide an adequate characterization of peak component accelerations. This study also confirms that peak horizontal floor acceleration distribution changes in response to changes in peak ground acceleration.

1. Introduction

The failure of nonstructural components after the 1971 San Fernando earthquake was recognized as critical for two reasons: (1) the damage of nonstructural components resulted in major economic loss and; (2) it posed a threat to life (Ray-Chaudhuri and Hutchinson 2004). Special design requirements should be considered where the partial or total collapse of structural and non-structural components (NCs) must meet life safety and collapse prevention performance levels. It is also very important to prevent damage to structural and non-structural components at the immediate occupancy and operational performance levels. Hirakawa and Kanda (1997) reported that, as a result of the 1995 Hyogo-ken Nanbu earthquake, 40% of 210 reinforced concrete (RC) buildings and 40% of structural components had sustained damage. Consequently, mitigation of nonstructural damage will reduce economic loss.

In ground motions like those produced by the 1989 Loma Prieta, 1994 Northridge, and 2001 Nisqually earthquakes (Shephard et al. 1990; Hall 1995; Filiatrault et al. 2001), economic loss from nonstructural components generally exceeded that from structural components. Several studies have reported that economic loss from nonstructural components are substantially greater than those resulting from structural damage (Ayers et al. 1973; Reitherman and Sabol 1995).

To estimate the design force for acceleration-sensitive NCs, several recent building codes recommend a linear variation of peak horizontal floor acceleration (PHFA) along the height of a building (Singh et al. 2006). Uniform building code recommendations (UBC 1997) and the building seismic safety council (NEHRP 94) suggest that PHFA varies from one at ground level to four times the peak ground acceleration (PGA) at roof level (trapezoidal distribution). In contrast, NEHRP 2003 and the International Building Code (IBC) (ICC 2006) assume a linear variation where the PGA at roof level is three times that of the PGA at ground level. The provisions used in these codes were developed empirically on the basis of floor acceleration data recorded in buildings during California earthquakes (Kehoe and Freeman 1998).

Many researchers have estimated the PHFA distribution for buildings, including Kehoe and Freeman (1998), Miranda and Taghavi (2005), Singh et al. (2006), Medina et al. (2006), Kumari and Gupta (2007), Rodriguez et al. (2002) and Ray-Chaudhuri and Hutchinson (2011). Some structural engineers have criticized current provisions by showing that they do not adequately estimate PHFA distribution (Kehoe and Freeman 1998; Horne and Burton 2003; Reinoso and Miranda 2005; Miranda and Taghavi 2005; Ray-Chaudhuri and Hutchinson 2011).

Recent studies have used incremental dynamic analysis (IDA) analysis to investigate ground motion characteristics (Kumar et al. 2013; Jager and Adam 2013). The influence of far-field ground motion characteristics on drift demands in steel moment frames were investigated by Kumar et al. (2013). It was shown that the salient parameters influencing global drift are the ratio of fundamental period to mean period and the behavior factor. The influence of near-field effects on collapse capacity spectra were investigated by Jager and Adam (2013), who derived collapse capacity spectra based on different definitions of collapse (near collapse and excessive ductility collapse).

The present study investigated the responses of a variety of concrete framed structures (3, 7, 10, 20 stories) subjected to forward directivity (FD) and non-FD effects of a set of 50 near-fault ground motions and the PHFA distribution along the height of the representative buildings. Four code-designed RC frame buildings were considered and IDA was conducted for each model. PHFA distribution using failure criteria were investigated at initial damage level (IDL). This level were achieved by increasing the PGA at each step of the IDA and analyzing the structures. The response acceleration of each floor was investigated and PHFA distribution of each structure for IDL was obtained. This level of damage is very important for engineers because despite of negligible damage on structures, ground motions can cause crucial damage on nonstructural elements.

Two types of near-fault ground motion were used to study the FD effect on PHFA distribution. The structures was tested using a variety concrete structures of varying stories and their PHFA distributions were compared with NEHRP 94, UBC 1997, NEHRP 2003 and IBC ICC 2006.

This study demonstrates that current seismic code provisions do not provide an adequate characterization of peak component accelerations. The new PHFA distributions are particularly useful for estimating seismic forces in non-structural components and, in general, to verify the seismic performance of non-structural components at immediate occupancy and operational performance levels.

2. Methods

In this study, four RC framed structures were designed using seismic force levels obtained from the Iranian Seismic Code (2005). They were proportioned using the ACI 318 building code (1999) for soil profile (S_2) and the highest seismic zone. The configuration was regular in elevation. The floor elevations and the span lengths of the frames were 3 and 4 m, respectively. Representative values of strength parameters are: unconfined compressive strength of concrete, 210 MPa; yield strength of steel, 300 MPa; ultimate strength of steel, 420 MPa. The four failure criteria (Monavari and Massumi, 2012; Massumi and Monavari 2013) were: inter-story drift (ID), Park-Ang damage index (DI), stability index (θ), and member curvature (φ).

2.1. Definitions of the four failure criteria

To evaluate the PHFA, a number of response criteria are needed to define the collapse limit states of a structure. Four failure criteria are utilized here. These are classified into two groups, local and global criteria (Louzai and Abed, 2014).
A local criterion is defined based on the limitation of plastic hinge rotation of different elements (beams, columns) to the ultimate rotation, θ_u (stability index(θ)). The Iranian Seismic Code defines θ as (BHRC, 2005):

$$\theta_i = Interstorydrift \frac{VerticalLoads}{StroyShear} \tag{1}$$

and

$$\theta_{\max} = \frac{1.25}{R} \leq 0.25 \tag{3}$$

The adopted global failure criteria are:

(a) An upper limit of the inter-storey drift, ID, equal to 3% of the story height (h_e). This limit is also specified in (Mwafy and Elnashai 2002; Massumi and Monavari 2013), and closed to those adopted by seismic design codes Eurocode 8 (2004) and UBC 97 (1997), which vary between 2 and 3%,

(b) An upper limit of the Park-Ang damage index, DI, equal to 1. Damage indices have been investigated by many researchers such as Park *et al.* (1988), De Guzman and Ishiy Ama (2004), Sadeghi and Nouban (2010), Ghosh *et al.* (2011).

The Park-Ang damage index developed base on experimental studies and observed damages in actual building. As defined in Park and Ang (1985):

$$DI_{P\&A} = \frac{\delta_m}{\delta_u} + \frac{\beta \int dE_h}{\delta_u P_y} \tag{4}$$

For more information see Park and Ang (1985). β is model constant parameter, and also DI is dimensionless, whereas DI>0.4 represents damage beyond repair, and DI> 1 represents total collapse (Park and Ang, 1985). The works of Park *et al.* (1988) and de Guzman and Ishiyama (2004) were utilized to verify these parameters.

(c) An upper limit of the member curvature, φ, equal to the final member curvature. Valles *et al.* (1996) considered the ultimate deformation capacity of a section to be its ultimate curvature.

2.2. Numerical Modeling

For simplicity, it presumed that the dynamic behaviour of the representative frames is able to capture the dynamic behaviour of the eight structures. Numerical models of frames (Fig. 1) were developed using IDARC 2D (Reinhorn *et al.*, 2009). It was assumed that all frames were fixed at the base and masses could be lumped at the floor nodes. Rayleigh proportional damping was used and damping coefficient was considered to be 5%. The nonlinear beam-column elements were fiber-based, allowing the spread of plasticity along the member length. A bilinear material model with 3% kinematic material hardening was assumed for the nonlinear beam-column elements. Newmark-Beta integration and the pseudo-force methods were used for nonlinear dynamic analysis.

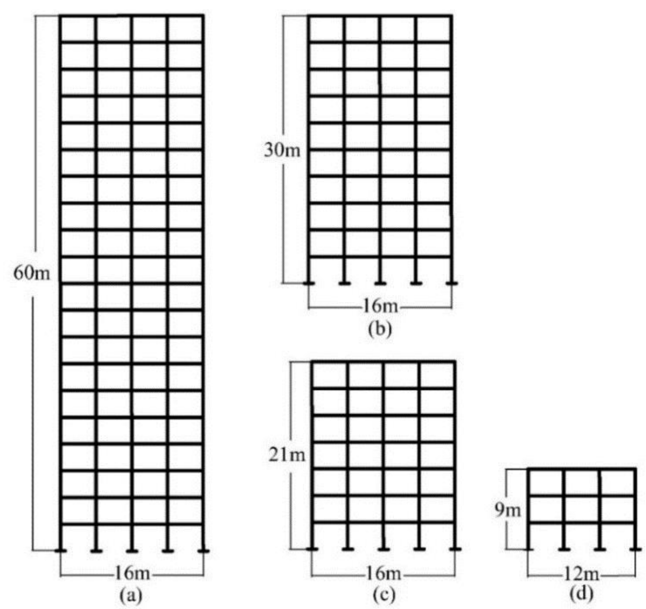

Fig. 1 – Test frames

2.2.1 Computer program for numerical models

IDARC 2D (Reinhorn *et al.*, 2009) is a two-dimensional analysis program for studying the non-linear response of multistory RC buildings. This program is a platform for nonlinear structural analysis in which various aspects of concrete, steel and other materials behaviour can be modeled and tested (Reinhorn *et al.*, 2009).

2.2.2 Eigenvalue and dynamic characteristics

Modal properties of each linear frame are obtained by performing an eigenvalue analysis of the associated numerical model, as shown in table 2.

Table 2 – Dynamic characteristics of the building frames considered in this study

Type of Building	First natural period (s)	second natural period (s)	Third natural period (s)	Fourth natural period (s)	Fifth natural period (s)
3 story	0.57	0.17	0.09	---	---
7 story	1.2	0.41	0.23	0.15	0.12
10 story	1.73	0.59	0.34	0.23	0.17
20 story	3.25	1.1	0.63	0.43	0.32

3. Calculating the Distribution of PHFA

PHFA distribution along the height of the representative buildings was investigated using a large number of near fault ground motions (30 FD and 20 non-FD effect records). Proposed structures and numerical models of their two-dimensional representative frames were constructed. IDA was conducted for each model using a set of 50 recorded ground motions. PHFA distribution was investigated using failure criteria as the initial damage level (IDL). IDL is defined as ID = 1%, DI<0.4, and the absence of SI, θ, φ.

The structures were analyzed using IDA. In the first step of IDA, the PGA was 0.02 g; the structure was analyzed and the responses recorded. The PGA was then increased to 0.04 g and the structure was analyzed again. PGA was increased until the two levels of failure occurred and the PHFA distributions of the two failure levels were then investigated. This procedure was repeated for all structures and motions (200 IDA). Two types of FD and non-FD effect records were used to investigate the effects of pulse-type records on PHFA distribution. Significant results on the effects of the number of floors, and pulse-type records on PHFA distribution were obtained.

3.1. Earthquake ground motions

In this study two near-fault ground motion sets for investigating the forward directivity (*FD*) effects are considered, one containing 30 motions with *FD* effects (pulse-type motions) and the other containing 20 motions without *FD* effects (ordinary ground motions) that were selected from a database by Baker (2007) and Sehhati *et al.* (2011) respectively (Tables 3 and 4). They were obtained from the NGA ground-motion library and Pacific Earthquake Engineering Research Center database. All ground motions were taken from stations within 25 km of the fault rupture. These ground motions are of engineering interest because of their large amplitude and having *FD* effects.

Table 3 – Ground motions with FD effects (Baker, 2007)

No.	Event	Year	Station	T_P (s)	PGV (cm/s)	M_W	R (Km)
1	San Fernando	1971	Pacoima Dam (upper left abut)	1.6	116.5	6.6	1.8
2	Coyote Lake	1979	Gilroy Array #6	1.2	51.5	5.7	3.1
3	Imperial Valley-06	1979	Aeropuerto Mexicali	2.4	44.3	6.5	0.3
4	Imperial Valley-06	1979	Agrarias	2.3	54.4	6.5	0.7
5	Imperial Valley-06	1979	Brawley Airport	4	36.1	6.5	10.4
6	Imperial Valley-06	1979	EC County Center FF	4.5	54.5	6.5	7.3
7	Imperial Valley-06	1979	EC Meloland Overpass FF	3.3	115	6.5	0.1
8	Imperial Valley-06	1979	El Centro Array #10	4.5	46.9	6.5	6.2

Table 3 – Ground motions with FD effects (Baker, 2007) (Cont.)

No.	Event	Year	Station	T_P (s)	PGV (cm/s)	M_W	R (Km)
9	Imperial Valley-06	1979	El Centro Array #11	7.4	41.1	6.5	12.5
10	Imperial Valley-06	1979	El Centro Array #3	5.2	41.1	6.5	12.9
11	Imperial Valley-06	1979	El Centro Array #4	4.6	77.9	6.5	7.1
12	Imperial Valley-06	1979	El Centro Array #5	4	91.5	6.5	4
13	Imperial Valley-06	1979	El Centro Array #6	3.8	111.9	6.5	1.4
14	Imperial Valley-06	1979	El Centro Array #7	4.2	108.8	6.5	0.6
15	Imperial Valley-06	1979	El Centro Array #8	5.4	48.6	6.5	3.9
16	Imperial Valley-06	1979	El Centro Differential Array	5.9	59.6	6.5	5.1
17	Imperial Valley-06	1979	Holtville Post Office	4.8	55.1	6.5	7.7
18	Mammoth Lakes-06	1980	Long Valley Dam (upper left abut)	1.1	33.1	5.9	----
19	Irpinia, Italy-01	1980	Sturno	3.1	41.5	6.9	10.8
20	Westmorland	1981	Parachute Test Site	3.6	35.8	5.9	16.7
21	Coalinga-05	1983	Oil City	0.7	41.2	5.8	----
22	Coalinga-05	1983	Transmitter Hill	0.9	46.1	5.8	----
23	Coalinga-07	1983	Coalinga – 14th & Elm (old CHP)	0.4	36.1	5.2	----
24	Morgan Hill	1984	Coyote Lake Dam (southwest abut)	1	62.3	6.2	0.5
25	Morgan Hill	1984	Gilroy Array #6	1.2	35.4	6.2	9.9
26	Taiwan SMART1(40)	1986	SMART1 C00	1.6	31.2	6.3	----
27	Taiwan SMART1(40)	1986	SMART1 M07	1.6	36.1	6.3	----
28	N. Palm Springs	1986	North Palm Springs	1.4	73.6	6.1	4
29	Whittier Narrows-01	1987	Downey – company maintenance building	0.8	30.4	6	20.8
30	Whittier Narrows-01	1987	LB – Orange Ave.	1	32.9	6	24.5

Table 4 – Ground motions without *FD* effects (Sehhati *et al.*, 2011)

No.	Event	Year	Station	T_P (s)	PGV (cm/s)	M_W	R (Km)
1	Loma Prieta	1989	BRAN #13	0.49	53.34	7	10.7
2	Loma Prieta	1989	Capitola #47125	0.64	34.56	7	15.2
3	Loma Prieta	1989	Corralitos #57007	0.75	45.48	7	3.9
4	Loma Prieta	1989	UCSC Lick Observatory #15	0.36	17.69	7	18.4
5	Loma Prieta	1989	UCSC #58135	0.16	11.61	7	18.5
6	Loma Prieta	1989	WAHO #14	0.23	25.42	7	17.5
7	Loma Prieta	1989	N Hollywood–Coldwater Can. #90009	1.2	22.89	7	12.5
8	Loma Prieta	1989	Sunland–Mt Gleason Ave. #90058	1.04	19.25	7	13.4
9	Loma Prieta	1989	Burbank–Howard Rd. #90059	0.64	8.14	7	16.9
10	Loma Prieta	1989	Simi Valley–Katherine Rd. #90055	0.62	51.4	7	13.4
11	Loma Prieta	1989	Sun Valley–Roscoe Blvd. #90006	1.01	25.86	7	10.1
12	Loma Prieta	1989	Santa Susana Ground #5108	0.69	20.31	7	16.7
13	Loma Prieta	1989	Big Tujunga, Angeles Nat F. #90061	0.64	6.67	7	19.7

Table 4 – Ground motions without *FD* effects (Sehhati *et al.*, 2011) (Cont.)

No.	Event	Year	Station	T_P (s)	PGV (cm/s)	M_W	R (Km)
14	Chi-Chi	1999	CHY028	0.62	72.86	7.6	3.1
15	Chi-Chi	1999	CHY029	0.67	30.35	7.6	11
16	Chi-Chi	1999	CHY035	1.28	45.61	7.6	12.7
17	Chi-Chi	1999	CHY080	0.88	107.61	7.6	2.7
18	Chi-Chi	1999	CHY006	1.81	55.44	7.6	9.8
19	Duzce	1999	Bolu	0.79	56.51	7.1	17.6
20	Duzce	1999	Duzce	5.5	59.99	7.1	8.2

4. Results and Discussion

4.1. FD effect of near-fault ground motion

Two types of near-fault records were used to investigate the effects of pulse-type ground motions on PHFA distribution. Four frames were analyzed using IDA and the results were investigated for IDL. The results can be seen in Fig. 2. In this figure the circles denote the PHFA values in each floor, and the lines and dash-lines denote their means and means plus one standard deviation, respectively.

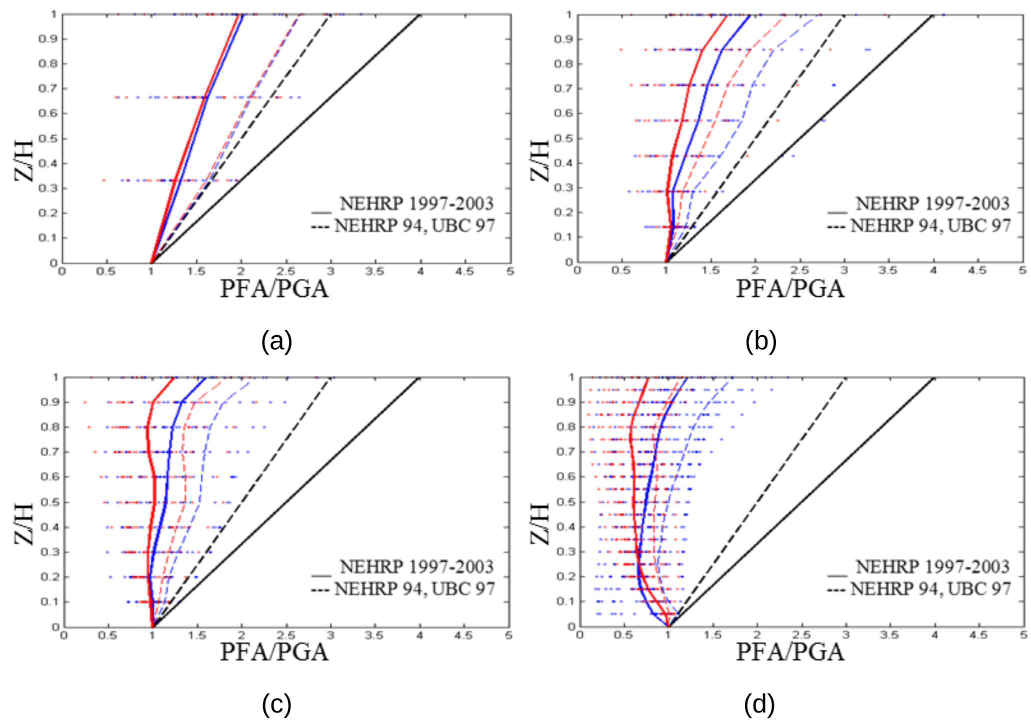

Fig. 2. PHFA distribution for IDL: (a) 3-story, (b) 7-story, (c) 10-story, (d) 20-story.

The figure show that, for all structures, PHFA means from the FD records are the largest, and in each structure, approximately, the greater the number of floors, the larger the means of PHFA distributions.

For the considered failure level (IDL), the structures behaved in a linear or near- linear fashion, resulting in a large PHFA. Analysis showed that the current provisions do not adequately estimate the variation of floor accelerations along the height. The values of the PHFA distributions adopted by the current seismic design code are overestimated, especially for high-rise frame structures. Studies indicate that large amplifications occur in the top 10% of the height of a building, where the contribution of higher modes can introduce large amplifications. These amplifications on average depend on the number of floors and differ from about 1.5 to 2.5. (Reinoso and Miranda, 2005).

4.2. PHFA Distribution

Fig. 3 shows the average and average plus one standard deviation of all 200 PHFA distributions (50 analyses for 50 records for 4 frames) due to FD and non-FD records for all frames. Solid lines indicate FD and dashed lines indicate non-FD records.

The average plus one standard deviation of all PHFA distributions is the curve that logically covers the most distributions in each level. Using these curves assures approximations of the PHFA distributions.
Chaudhuri and Hutchinson (2011) concluded that current code recommendations produce adequate estimations of the variation in acceleration demand along the height of tall buildings, however, Reinoso and Miranda (2005) concluded that current code recommendations may not produce adequate estimations of the variation in acceleration demands along the height of tall buildings nor of component amplification factors.
The results in this study confirm that the current provisions may do not adequately estimate the variation of floor accelerations along the height, especially for high-rise buildings. It suggests that code-specified profiles overestimated peak floor acceleration at different fundamental periods. It was also shown that the variation in acceleration demand along the height of structures can differ significantly from that currently recommended in US seismic provisions for anchoring building nonstructural components.

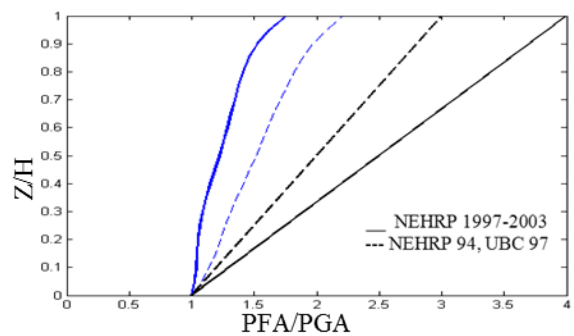

Fig. 3. Average and average plus one standard deviation for all PHFA distributions

5. Conclusion

In this study, two critical PHFA distributions were obtained for considered failure levels, IDL. The results show that PHFA distributions produced larger amplification in the top 10% of the height of the buildings. The new approximations for IDL can be expressed as:

$$PFA = 1 + a\left(\frac{z}{H}\right) \tag{5}$$

where z/H is the normalized height that differs from 0 to 1 and *a* = 1.2. The approximations suggest that PHFA varies from one time at ground level to 2.2 times the PGA at roof level. See Fig. 4. Blue line indicates PHFA distributions for the most critical distributions of the FD and non-FD records, and red dashed line indicates approximate PHFA distributions.

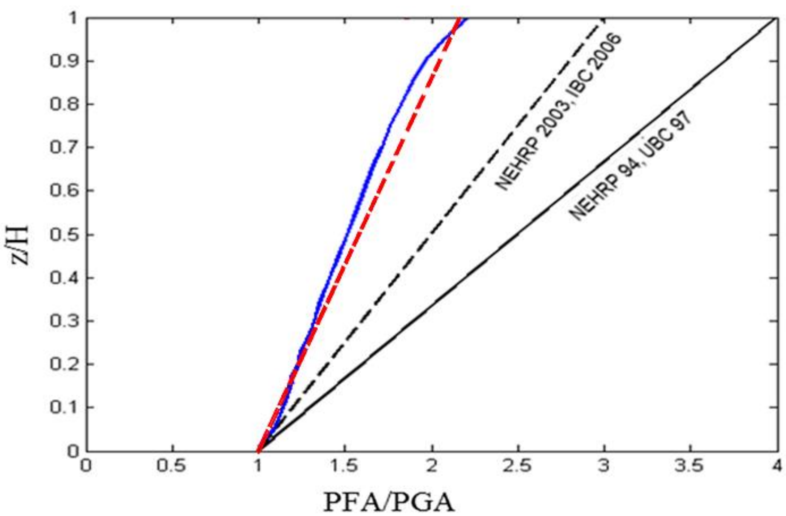

Fig. 4 –Normalized peak floor acceleration demands

Although further research is needed, the results show that the distributions for NEHRP (2003) and IBC ICC (2006) are significantly greater than those calculated in this study (Fig. 4). It is important to note that these results were obtained by investigating structures with regular and 2D frames and may differ for irregular and actual structures. It is strongly recommended that this study be repeated using far-field ground motions.

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