JSEE Vol. 17, No. 4, 2015

Floor Response Modification Factors for Nonstructural Components due to Near-Field Pulse-Like Earthquakes

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Received: 03/08/2015
Accepted: 16/12/2015

ABSTRACT

Floor response modification factor (R) under near-field strong ground motions (SGMs) with directivity pulses are proposed in this paper. The R factor is defined as the floor response spectrum (FRS) for linear elastic primary structures normalized by the FRS for an inelastic primary structure. The terms 'elastic' and 'inelastic' refer to the behavior of the supporting structure while only elastic nonstructural components (NSCs) are used in this study. Considering the lack of comprehensive study on the behavior of NSCs under near-field SGMs with directivity pulses, this study evaluates the dependence of the proposed response modification factor (R) under bunches of near-field records with wide ranges of directivity pulse periods. A statistical analysis of the peak response of NSCs supported on inelastic regular moment-resisting frame structures exposed to near-field pulse-like SGMs is presented. Peak component demands were quantified based on the FRS method with considering dynamic interaction effects. In This paper the main factors affecting the FRS caused by inelasticity in the primary structures represented by parameter R has been evaluated. The results show that FRS values at the initial modal periods of the supporting structure are reduced due to the inelastic action in the primary structures. Comparing the results with the same earthquake events without directivity pulses shows that the reduction factor in near-field pulse-like SGMs is considerably larger than R factor in far-field SGMs.

Keywords:
Floor response spectra; Modification factor; Nonstructural component; Primary structure; Near-field earthquake; Directivity pulse

1. Introduction

Floor response spectra in terms of acceleration are usually used for the seismic design and evaluation of acceleration sensitive equipment installed in buildings. A comprehensive state-of-the-art paper on the seismic design of components and secondary systems, which includes a historical overview of analysis methods, including the floor acceleration spectra, was prepared by Villaverde [1].

Past earthquake events have shown that the Nonstructural Component (NSC) damages and failures can cause substantial economic losses and life-threatening hazards to occupants. Thus, evaluation and reduction of nonstructural damages have gained rising awareness in the recent years [2-5]. Recent researches in NSCs area are focused on developing practical methods for seismic performance assessment of nonstructural systems and accurately estimating the acceleration demands. During the last four decades, several methods have been proposed for seismic analysis of nonstructural components attached to the structures, which are mainly restricted to linear nonstructural components mounted on linear structures [6]. However, they cannot be used in estimating responses of nonstructural components, under severe seismic events, where their supporting structures show nonlinear behavior. According to the codes and standards available for the design of NSCs and their anchorage systems, maximum component absolute acceleration demands are estimated based on the elastic responses and fundamental mode of supporting structures [7-10]. Therefore, engineering demand parameters, used in current seismic design codes, rely heavily on the parameters in which no reliable nonlinear responses of supporting structures are taken into account. Therefore, many researchers have focused on the field of assessing the influence of the nonlinear behavior of the building on the floor response spectra. For primary structures modeled by single degree of freedom (SDOF) systems, different reduction factors have been proposed. These reduction factors enable an estimate of the floor spectra of nonlinear structures based on the floor spectrum of linear structures. In these cases, the influences of the input ground motion, ductility, hysteretic behavior and the natural period of the structure have been considered as well as damping of the equipment [11-17]. The direct spectra-to-spectra, is another method developed for generating floor response spectra in nuclear facilities. It avoids the deficiencies of time history methods. It is proposed based on modal combination rule which considers the correl-ation between modal responses of a structure and equipment, based on random vibration theory [18]. A comprehensive study of several parameters (i.e., stiffness distribution along the height of the frame, amount of inelasticity of the supporting structure, location, damping ratio of the nonstructural component and higher mode effects) under regular far-field SGMs was done [19-25]. Parametric studies on the behavior of linear and nonlinear SDOF components mounted on moment-resisting steel plane frames have shown that, in general, but not always; nonlinear primary structure behavior has a beneficial effect on the component response [26-29]. There is exception in some special structures in which the inelastic demands can be greater than the elastic demands [15-16]; therefore, it is important to quantify the parameters that contribute to increase or decrease of inelastic floor response spectra with respect to the elastic FRS.

As ground motions close to a ruptured fault are significantly different from those observed far from the seismic source, a number of studies have been done to develop predictive relationships for parameters characterizing this special type of ground motions [29-33]. These studies developed relationships based on period and amplitude of forward-directivity pulses. Near-fault recordings from recent earthquakes indicate that forward-directivity pulse is a narrow band pulse whose period increases with magnitude. This magnitude dependence on the pulse period causes the response spectrum to have a peak whose period increases with magnitude, such that the near-fault ground motions from moderate magnitude earthquakes may exceed those of larger earthquakes at intermediate periods [34].

Due to the lack of comprehensive survey on the effects of forward directivity on the FRS, this paper tries to present the effect of forward-directivity pulses on acceleration, velocity and displacement FRS in inelastic moment-resisting frame structures. The findings are based on a comprehensive study of 40 near-fault pulse-like ground motion records. It is important that the previous researches focused only on the acceleration FRS. The proposed modification factors help to estimate the performance of acceleration-, velocity- and displacement-sensitive components in nonlinear supporting structures based on the results of linear time history analysis. In the other words, the advantage of using the proposed parameter is that it can address the variations in elastic FRS values due to yielding of the supporting structure without a need to time-consuming nonlinear time history analysis.

2. Theoretical Background, Structural Models and Ground Motions

Consider an N-degrees-of-freedom primary system with a mass $[M_p]$, damping $[C_p]$, and stiffness matrix $[K_p]$, attached by a single degree of freedom secondary system with a mass (M_s), a damping (C_s), and a stiffness (K_s), to its mth degree of freedom, which have been subjected to a horizontal near-field ground excitation, $\{\ddot{X}_g(t)\}$ the equations of motion of the combined system become:

$$[M]\{\ddot{Y}\}+[C]\{\dot{Y}\}+[K]\{Y\}=-\begin{bmatrix}[M_p]\\ M_s\end{bmatrix}\{\ddot{X}'_g(t)\} \qquad (1)$$

where $\{Y\}$= relative displacement response of combined system and

$$
\begin{aligned}
[M] &= \begin{bmatrix} [M_p] & \{0\} \\ [0] & M_s \end{bmatrix} \\
[K] &= \begin{bmatrix} [K_p] & \{0\} \\ [0] & K_s \end{bmatrix} + [K_c] \\
[C] &= \begin{bmatrix} [C_p] & \{0\} \\ [0] & 0 \end{bmatrix} + [C_c]
\end{aligned}
\qquad (2)
$$

in which $[K_c]$ and $[C_c]$ are the coupling matrices associated with the stiffness and damping matrices, respectively, contain the stiffness and damping coefficient of the secondary system in the m^{th} and N+1$^{\text{th}}$ element.

The process of analysis as shown in Figure (1) consists of performing dynamic simulations in which structural models (single-bay, two-dimensional frames) are exposed to a set of 40 pulse-like near-field ground motions all of which are scaled to $PGA = 0.35\,\text{g}$ in order to be comparable. For a given structural model and ground motion, the acceleration, velocity and displacement responses at selected floor levels are obtained considering primary-secondary system interaction to develop corresponding elastic and inelastic FRS. The damping ratio, ξ, of interest for the NSCs is 5%. This study does not consider the nonlinearity of the component and is valid for light components that do not offer dynamic feedback to the building.

Multi-story structures have been utilized for the dynamic time-history analysis. The building structures have been designed based on Iranian Standard Code-2800 with special steel moment-resisting frames with the same mass at all floor levels. The frames have been extracted such that their dynamic behavior is the same as that of the buildings [35]. Nonstructural components have been considered as single-degree-of-freedom linear systems on one of the floors for each analysis. Figure (2) shows a schematic of the 8-story building interior frame with nonstructural components attached at its floors. The other studied buildings have frames designed for each individual building. Modal periods of supporting systems are shown in Table (1).

The ground motions used in this study were selected from pulse-like near-field PEER databases, have a moment magnitude that varies from 5.7 to 7.6, and closest distances to the fault rupture that vary from 0.1 to 21 km. Detailed information on the ground motions can be seen in Table (2). Individuals and mean acceleration, velocity and displacement response spectra of all records are shown in Figure (3).

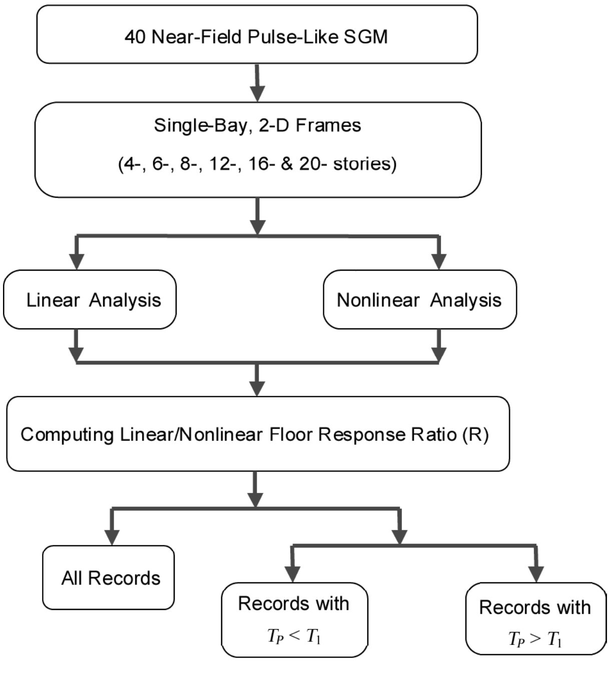


Figure 1. Schematic procedure of analysis.

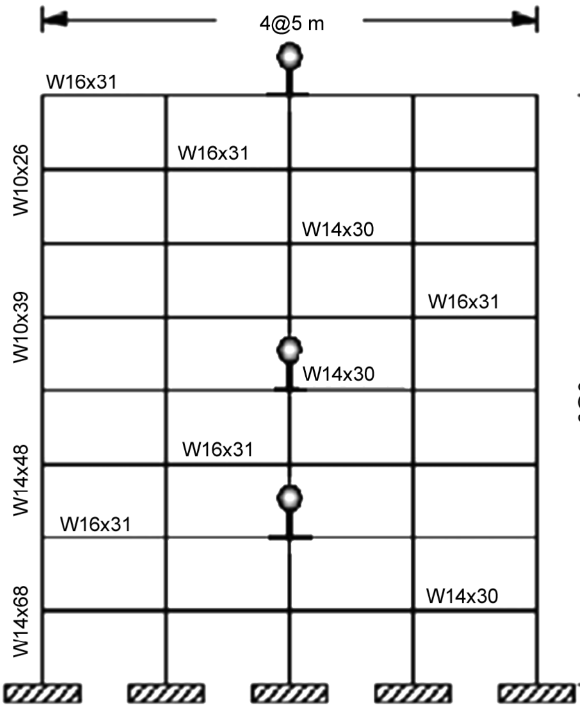
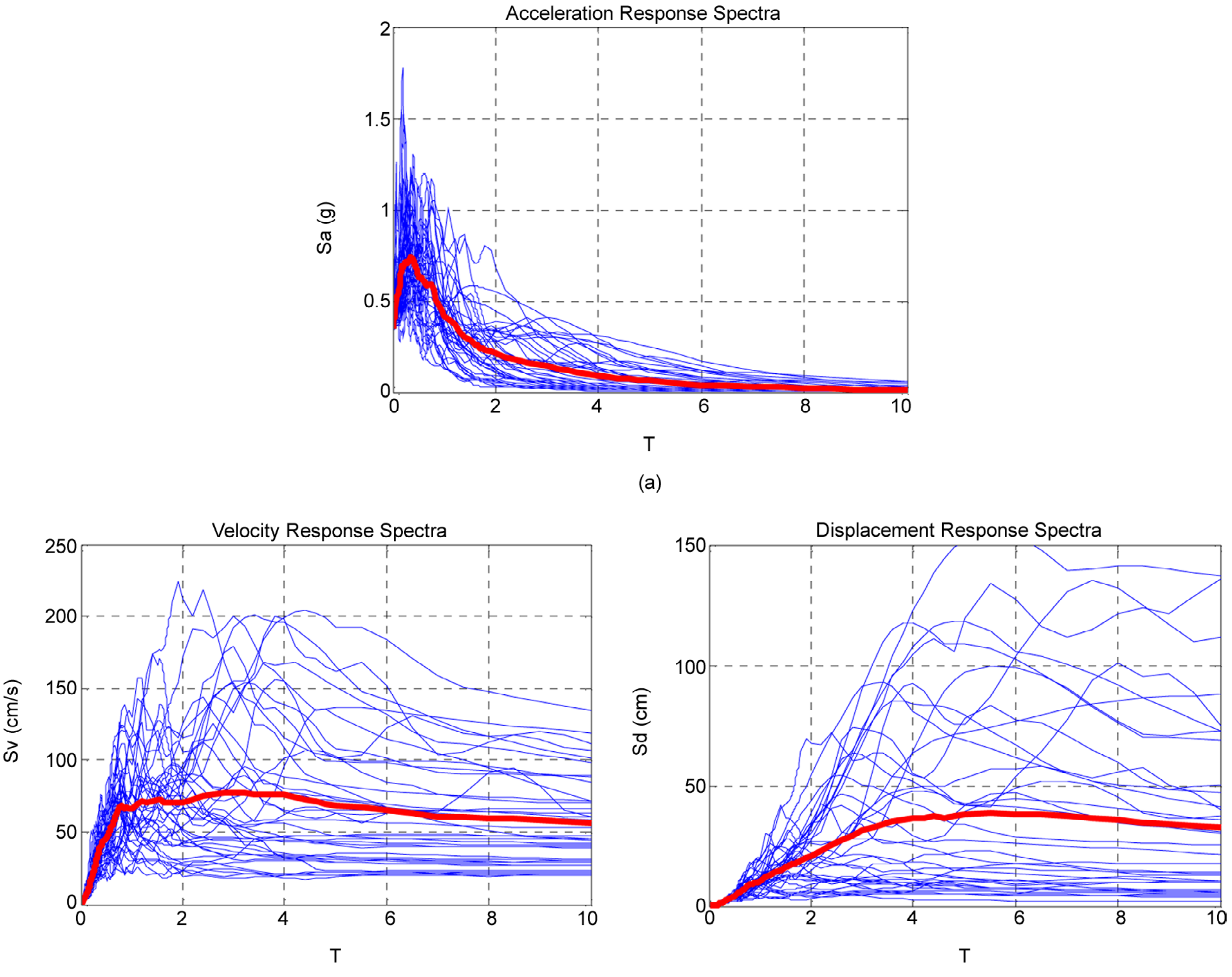


Figure 2. Interior frame 8-story building with nonstructural components attached to its lower, middle or top floor [36].

Table 1. Modal periods of supporting structures.

Mode No.	4 Story	6 Story	8 Story	12 Story	16 Story	20 Story
1	0.97	1.36	1.65	2.16	2.63	3.00
2	0.33	0.48	0.58	0.85	0.99	1.15
3	0.19	0.27	0.34	0.48	0.58	0.67
4	0.14	0.19	0.23	0.33	0.4	0.46
5		0.14	0.18	0.24	0.3	0.35
6		0.10	0.14	0.2	0.23	0.28
7			0.11	0.16	0.19	0.22
8			0.11	0.14	0.17	0.19
9				0.11	0.14	0.17
10				0.09	0.12	0.14
11				0.07	0.1	0.13
12				0.05	0.09	0.11
13					0.08	0.10
14					0.06	0.08
15					0.05	0.07
16					0.04	0.07
17						0.06
18						0.06
19						0.04
20						0.03



(a)

(b)

(c)

Figure 3. Individuals and mean: Acceleration, (b) Velocity and (c) Displacement response spectra of all records.

Table 2. Near-field pulse-like ground motion characteristics.

No.	Earthquake	Station	Year	Mw	R (km)
1	Parkfield	Cholame-Shandon Array#2	1966	6.2	6.3
2	Parkfield	Temblorpre-1969	1966	6.2	16.0
3	San Fernando	Pacoima Dam (Upper left abut)	1971	6.6	1.8
4	Gazli, USSR	Karakyr	1976	6.8	5.5
5	Tabas, Iran	Tabas	1978	7.4	2.1
6	Coyote Lake	Gilroy Array#6	1979	5.7	3.1
7	ImperialValley-06	EC County Center FF	1979	6.5	0.1
8	ImperialValley-06	El Centro Array#4	1979	6.5	7.1
9	ImperialValley-06	El Centro Array#6	1979	6.5	1.4
10	ImperialValley-06	El Centro Array#7	1979	6.5	0.6
11	Coalinga-01	Pleasant Valley P.P.-bldg	1983	6.4	8.4
12	Morgan Hill	Anderson Dam (Downstream)	1984	6.2	3.3
13	Morgan Hill	Coyote Lake Dam (SWA but)	1984	6.2	0.5
14	Morgan Hill	GilroyArray#6	1984	6.2	9.9
15	Morgan Hill	Halls Valley	1984	6.2	3.5
16	Nahanni, Canada	Site1	1985	6.8	9.6
17	Nahanni, Canada	Site2	1985	6.8	4.9
18	N. Palm Springs	Desert Hot Springs	1986	6.1	6.8
19	N. Palm Springs	North Palm Springs	1986	6.1	4.0
20	N. Palm Springs	Whitewater Trout Farm	1986	6.1	6.0
21	WhittierNarrows-01	Bell Gardens-Jaboneria	1987	6	17.8
22	WhittierNarrows-01	Downey-Co Maint Bldg	1987	6	20.8
23	WhittierNarrows-01	Norwalk-Imp Hwy, SG rnd	1987	6	20.4
24	WhittierNarrows-01	Santa Fe Springs-E.Joslin	1987	6	18.5
25	SuperstitionHills-02	El Centro Imp. Co. Cent	1987	6.5	18.2
26	SuperstitionHills-02	Parachute Test Site	1987	6.5	1.0
27	Loma Prieta	Gilroy Array#2	1989	6.9	11.1
28	Loma Prieta	LGPC	1989	6.9	3.9
29	Loma Prieta	Saratoga-W Valley Coll.	1989	6.9	9.3
30	Erzican, Turkey	Erzincan	1992	6.7	4.4
31	Landers	Lucerne	1992	7.3	2.2
32	Northridge-01	Jensen Filter Plant	1994	6.7	5.4
33	Northridge-01	LAD am	1994	6.7	5.9
34	Northridge-01	Sylmar-Converter Sta	1994	6.7	5.4
35	Northridge-01	Sylmar-Olive View Med FF	1994	6.7	5.3
36	Kobe, Japan	KJMA	1995	6.9	1.0
37	Kocaeli, Turkey	Duzce	1999	7.5	15.4
38	Kocaeli, Turkey	Yarimca	1999	7.5	4.8
39	Chi-Chi, Taiwan	TCU075	1999	7.6	0.9
40	Chi-Chi, Taiwan	TCU076	1999	7.6	2.8

3. Floor Response Modification Factor

It is clear that the neglecting of the influence of structural inelasticity on floor accelerations may lead to unrealistic results, since the inelastic response of a building during an earthquake affects the floor motions and the forces to which NSCs are subjected to. In general, significant reductions in peak values of floor response spectra can be achieved if inelastic behavior of the structure and/or equipment is taken into account [12, 17, 21, 35]. Parametric studies are conducted on the influence of structural nonlinearity due to the far-field SGM on equipment response primarily with SDOF elastic equipment mounted on a SDOF structure. In general, it is perceived that this structural nonlinearity will generally reduce the NSC acceleration responses in most situations [11, 14, 17, 37]. Due to the complexities in obtaining an analytical solution for the inelastic case, most of the existing research efforts have been directed towards the development of modification factors by which a linear elastic FRS can be modified to take into account the nonlinearity. Sewell et al. [14-15] used the ratio of FRS for the inelastic primary structure normalized by the FRS of the corresponding elastic primary structure to quantify the nonlinear behavior. This ratio was called floor response spectra ratio (FRSR). A parameter denoted as acceleration response modification factor is introduced by Sankaranarayanan and Medina [21] to quantify the effect of building non-linearity on the peak acceleration (strength) demands of NSCs due to far-field strong ground motions and is equivalent to the inverse of the FRSR factor.

This study mostly tried to show the variations in peak acceleration, peak velocity and peak displacement demands in three regions: long-period, fundamental-period and short-period due to near-field earthquakes containing forward directivity pulses.

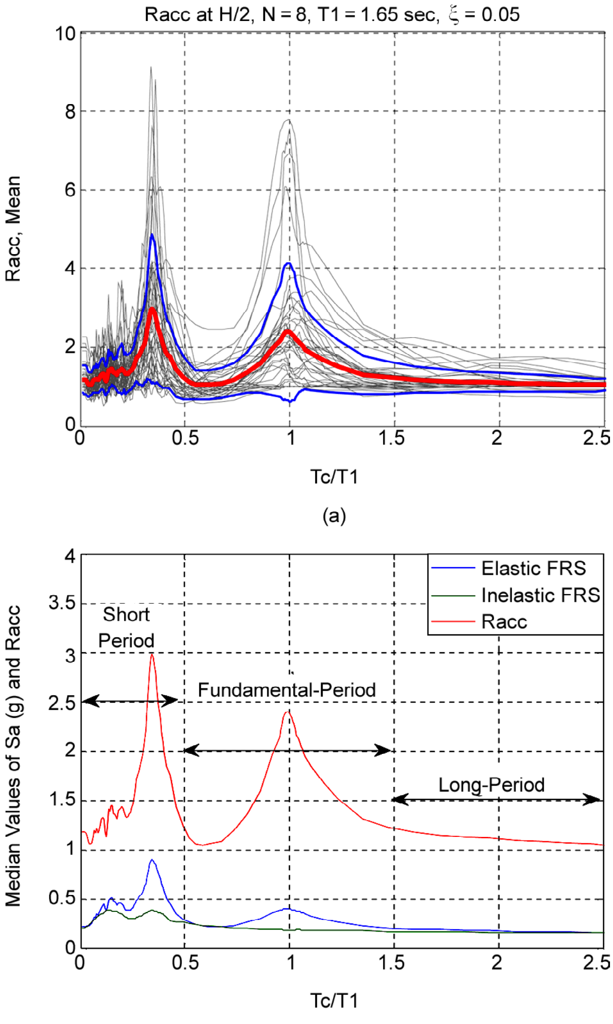
The *R* factor is defined as the FRS for linear elastic primary structures normalized by the FRS for an inelastic primary structure, Eq. (3). The terms 'elastic' and 'inelastic' refer to the behavior of the supporting structure; NSCs are assumed to remain elastic:

$$R_D = \frac{S(elastic)}{S(inelastic)} \tag{3}$$

A typical plot of the variation of R_{Acc} values due to record-to-record variability as well as the elastic and inelastic FRS of R_{Acc} is presented in Figure (4) respectively. According to Figure (4a), R_{Acc} variations along the horizontal axis create three different regions. As it can be seen, a decrease in Peak Component Accelerations (*PCAs*) caused by the inelasticity of the primary structure is different when the period of the NSC is close to one of the modal periods of the primary structure in comparison to the other parts. This observation restates the importance of the parameter T_C / T_i where T_i is the period of vibration of the ith mode in the quantification demands on NSCs mounted on inelastic frames.

Three different period ranges are demarcated:

1. Long-period (low-frequency) region (T_C/T_i>1.5);
2. Fundamental-period region ($0.5 < T_C / T_i < 1.5$);
3. Short-period (high-frequency) region (T_C/T_i<0.5).



(a)

(b)

Figure 4. Floor acceleration response modification factor for 8-story frame: (a) dispersion of R_{Acc} (acceleration) due to record-to-record variability and (b) three different FRS regions.

To compare the differences between *R* values under near-field pulse-like and far-field ordinary SGMs, the same earthquake events with epicentral distance more than 60 km are chosen as far-field inputs. All records are scaled to $PGA = 0.35$g as near-field ones. As it can be seen in Figure (5), the absence of directivity pulse reduced nonlinearity in supporting structure and leads to smaller *R* factors in comparison to near-field inputs.

Representative plots for median R_{Acc} values at various locations in a 4-, 8-, 12- and 16-story frames are presented in Figure (6). R_{Acc} increases with the height of the NSCs attachment point, and larger values of R_{Acc} are observed for NSCs with periods near the periods of the supporting structure. As well as acceleration, median velocity and displacement modification factor values (R_{Vel} and R_{Dis}) at various heights of structural frames are depicted in Figures (7) and (8) respectively.

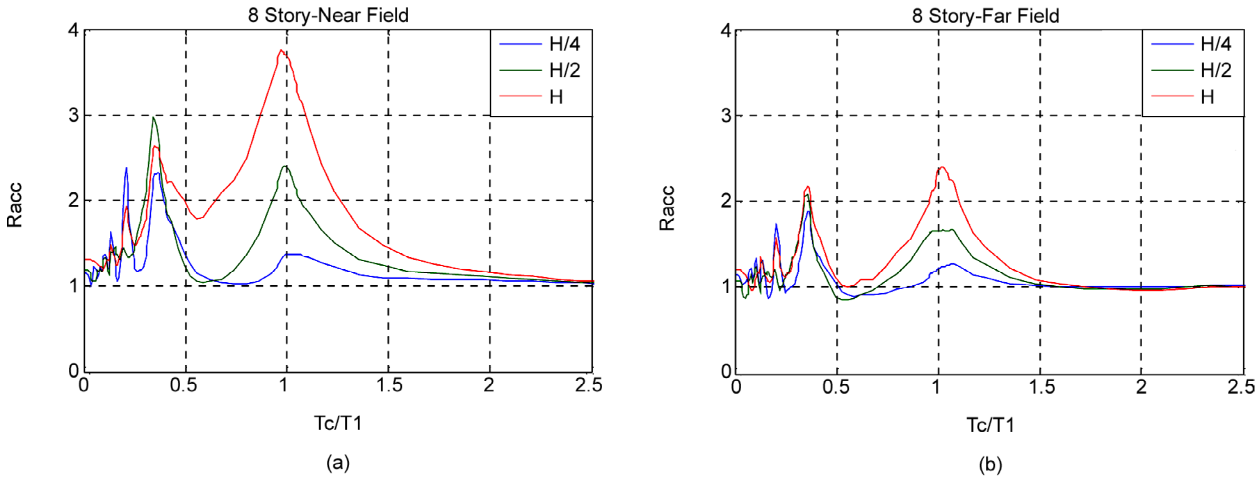


Figure 5. Comparison floor acceleration modification factor for 8-story frame under: (a) near field (b) far field SGMs.

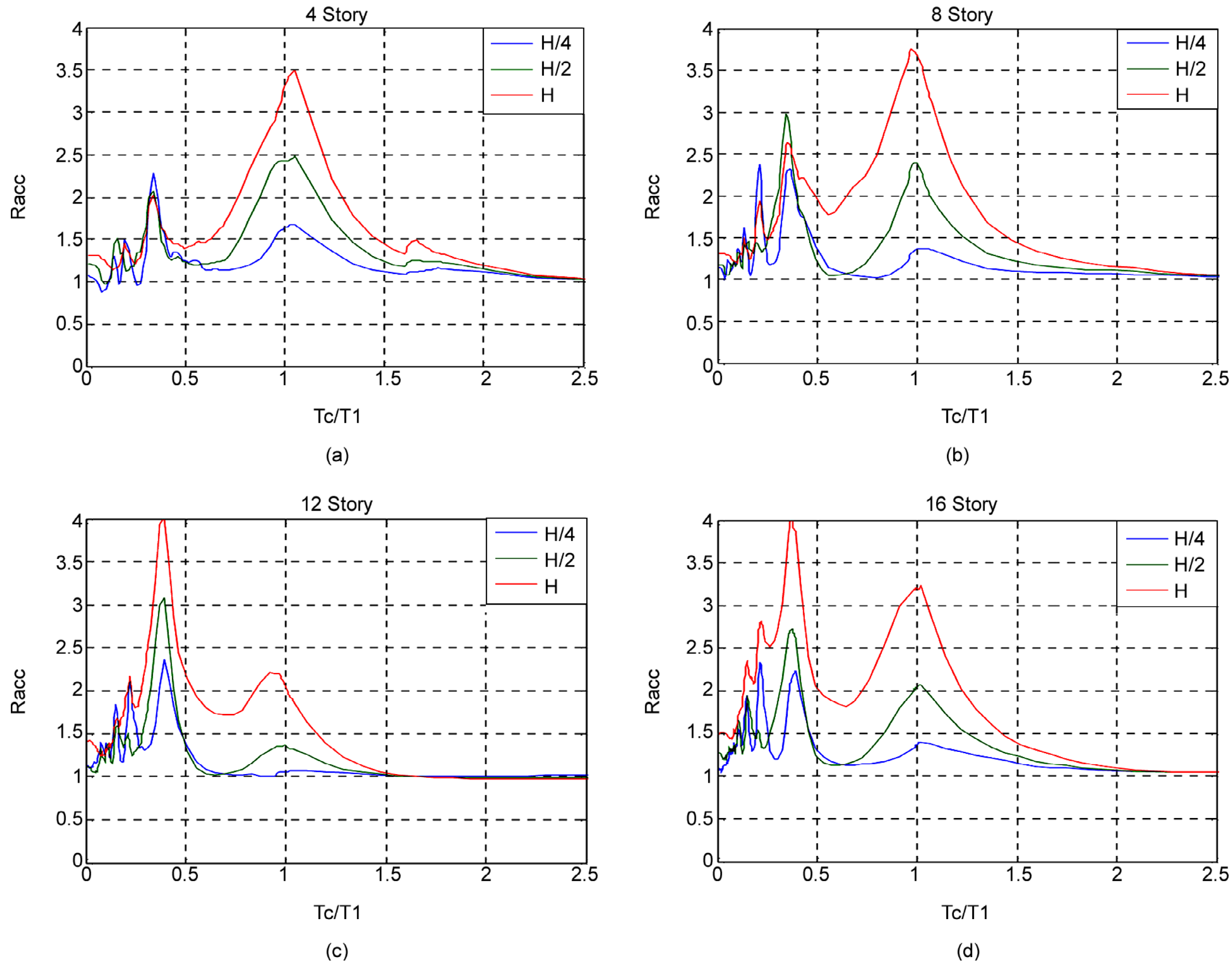


Figure 6. Median R_{Acc} (acceleration) values at various heights in 4-, 8- , 12- and 16-story structures.

3.1. Long-Period Region

In this region as shown in Figure (4a), the median R values generally are slightly greater than one. The R_{Acc} values for individual ground motions tend to fluctuate about one. R_{Acc} values less than one imply that the FRS values for inelastic primary structures at a given period are slightly higher than those corresponding to elastic primary structures. For NSCs, which are more flexible than the primary structure ($T_C > 2T_1$), elastic or inelastic behavior of the supporting structure has no effect on the response of the component.

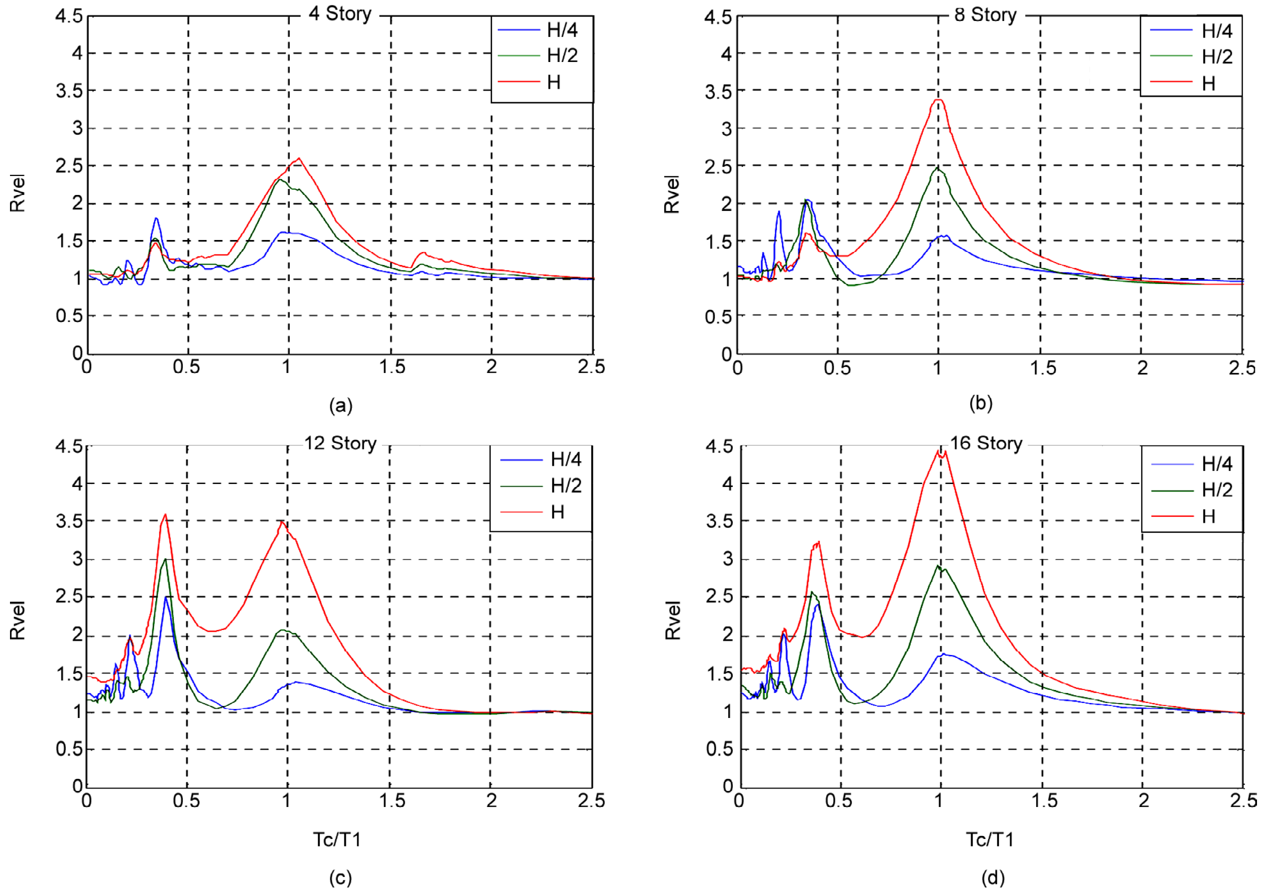


Figure 7. Median R_{Vel} (velocity) values at various heights in 4-, 8- , 12- and 16-story structures.

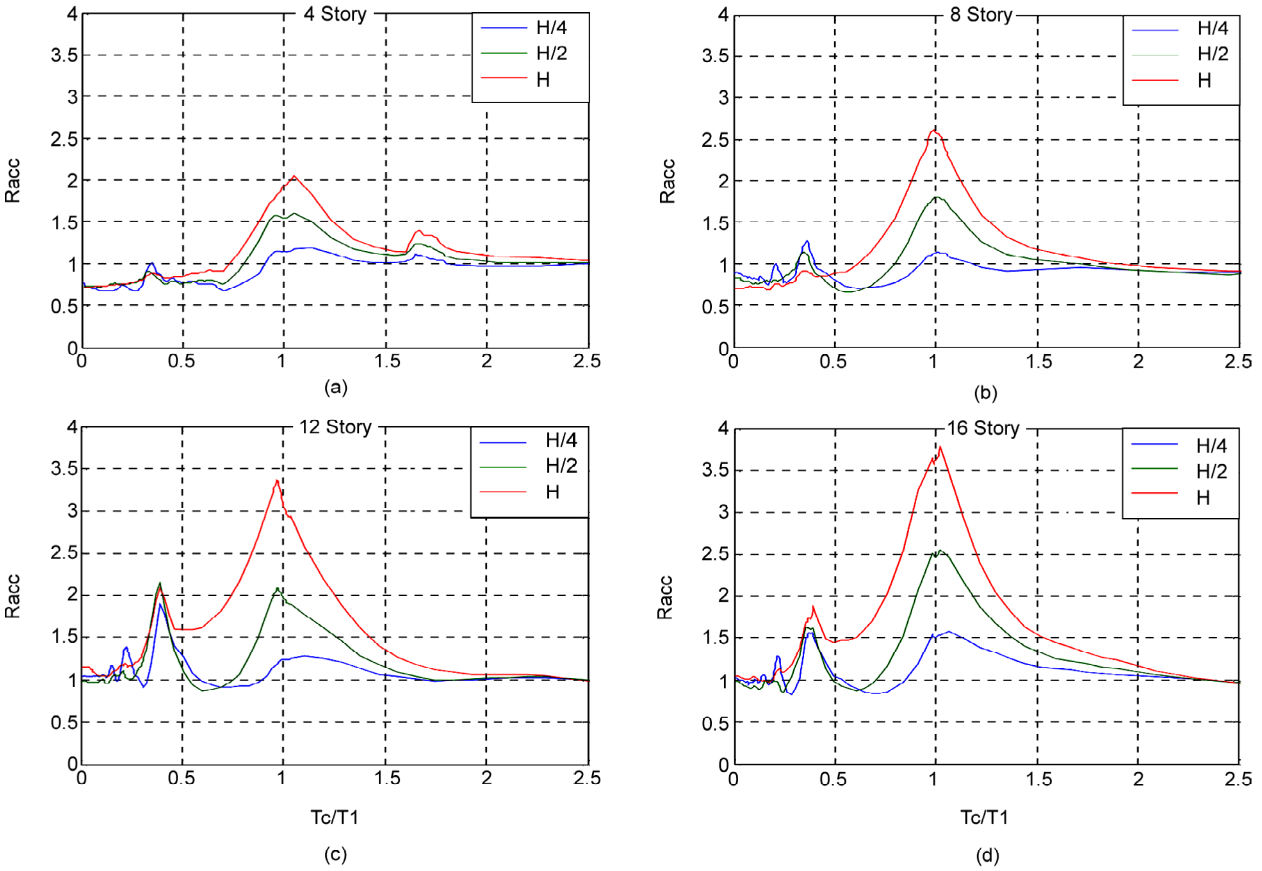


Figure 8. Median R_{Dis} (displacement) values at various heights in 4-, 8- , 12- and 16-story structures.

3.2. Short-Period Region

The behavior of *R* values in this region is dependent on the effect of higher modes and almost it is independent of the location of NSC in the building. It means that the *R* factor is the same in different height of the structure as it can be seen in Figures (6) and (7). Acceleration-sensitive NSCs with periods in tune with the higher modal periods of the primary structure experience a reduction in FRS ordinates when the structure passes from elastic to inelastic behavior under the effect of ground motion. This effect is more intense in higher structures and upper levels, Figure (6). The same trend can be seen in R_{Vel} but in displacement sensitive high-frequency NSCs, median R_{Dis} values less than one are observed. This behavior was observed for stiffer frames. It shows that the displacement of FRS for inelastic primary structures may actually increase in comparison to those obtained when the structure is assumed to remain in the linear elastic range. It should be noticed that we are speaking about median values. For individual records *R* values less than one can be observed. One of the most important sources of *R* values less than one is the modal interaction or internal resonance of the modes of the nonlinear system that causes peak component acceleration, velocity or displacement demands corresponding to inelastic primary structures to increase with respect to the elastic ones.

3.3. Fundamental Period Region

A substantial increase in *R* values can be seen in the vicinity of the elastic fundamental period of vibration of the primary structure. This increase can be attributed to two reasons: (1) increasing in the energy dissipated by the primary structure due to inelastic behavior; and (2) shifting of the fundamental period of the structure away from the period of the NSC. This reduction in FRS values is quantified by higher *R* factors that induce considerable reductions in seismic force demands of NSCs. As it can be seen in Figures (6) to (8), the variation of peak values of *R* is almost linear along the height of the frame.

4. Records Classification Based on Fundamental Period of Structure

Due to the wide dispersion of *R* values based on Figure (4a), records are classified in two categories. Group 1 contain those records whose directivity pulse periods are shorter than the fundamental period of the supporting structure, and group 2 contain records with pulses larger than the fundamental period of the supporting structure. Individual records and mean acceleration response spectra for both groups are demonstrated in Figure (9). Groups are classified based on the fundamental period of 8-story frame.

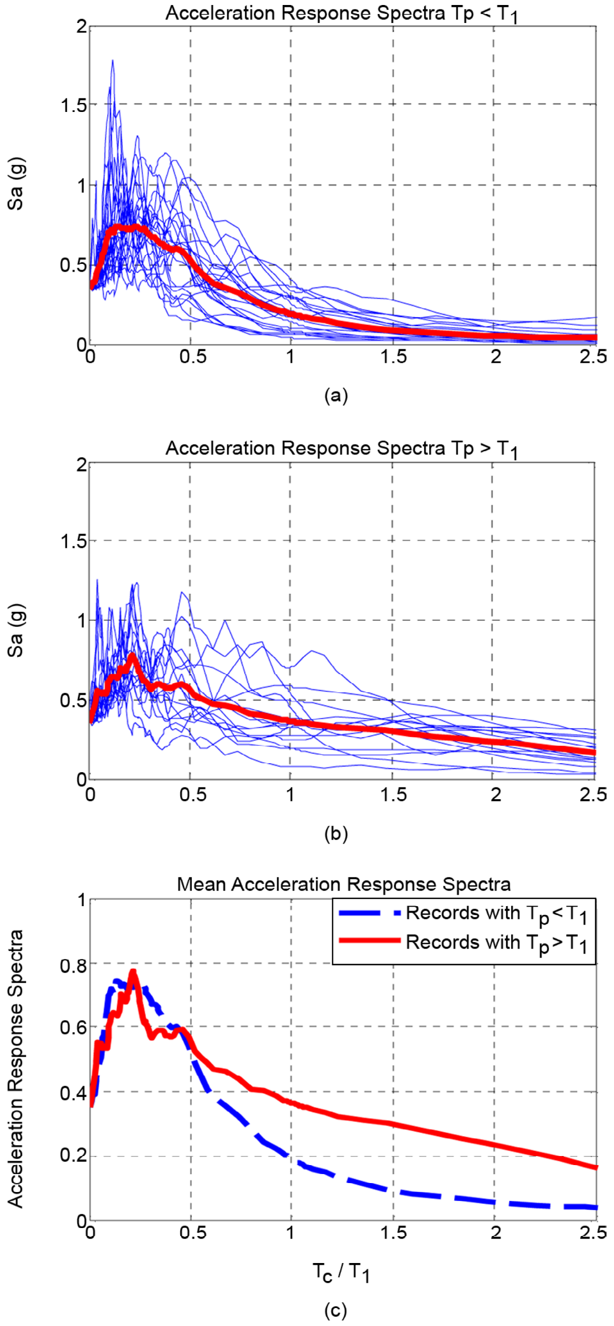


Figure 9. Floor acceleration response spectra for: (a) records with pulse period shorter than T_1, (b) records with pulse period larger than T_1 and (c) comparison of mean values' shape based on fundamental period of 8-story frame.

Based on the classified records, R_{Acc} was plotted in two categories: $T_P < T_1$ and $T_P > T_1$; which T_P and T_1 represent the period of the directivity pulse and the fundamental period of supporting structure. The R_{Acc}, R_{Vel} and R_{Dis} results for 4-, 8- and 16-story are presented in Figures (10) to (12) to compare the acceleration, velocity and displacement floor spectra as high-, mid- and low-frequency responses, respectively.

Considering Figure (10), in records containing

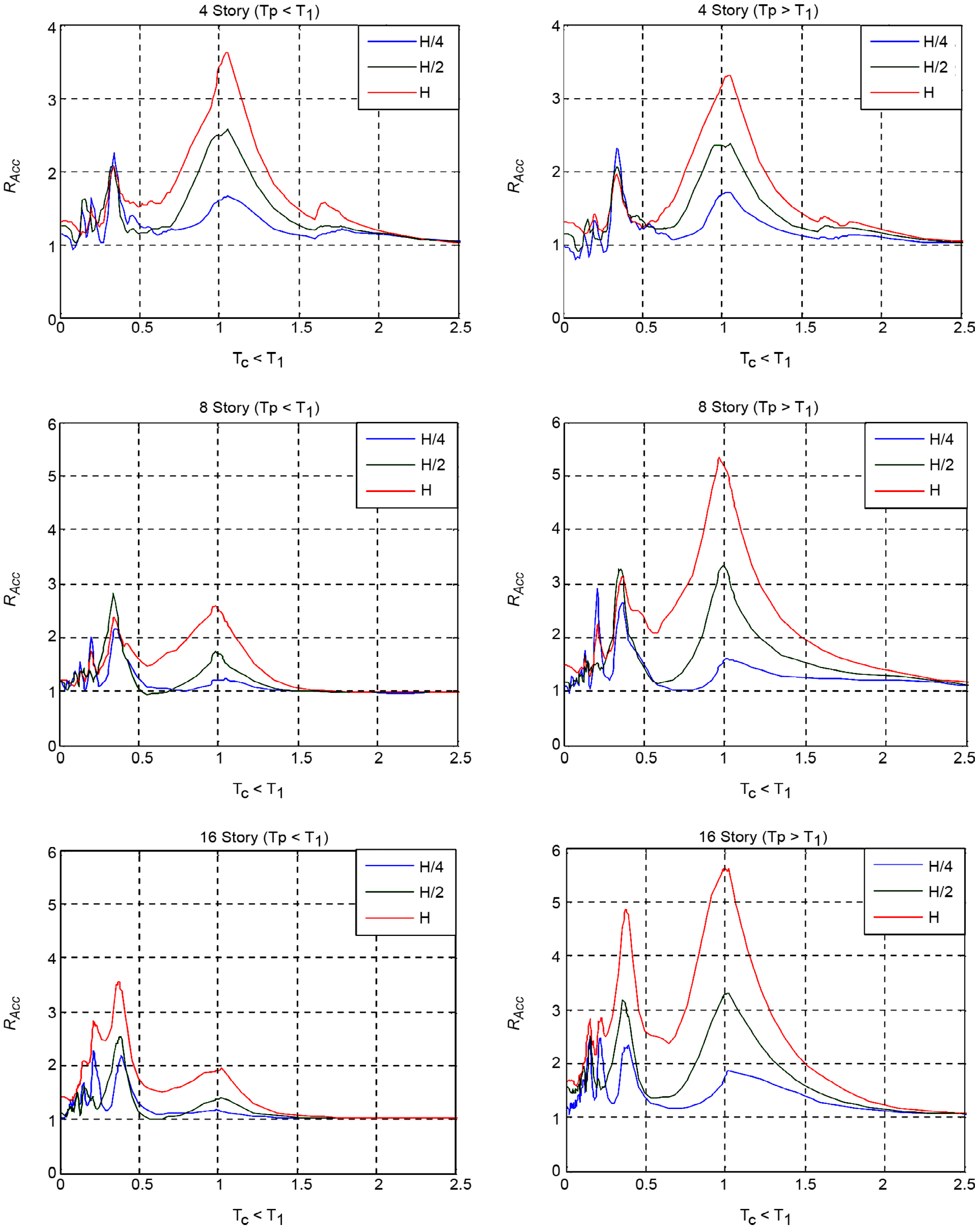


Figure 10. Filtered median R_{Acc} (acceleration) values at various heights in 4-, 8- and 16-story frames due to the pulse period.

directivity pulses longer than first period of the structure, R_{Acc} is noticeably larger than those with pulses shorter than the fundamental period of the structure. The same trend has been detected in R_{Vel} but not in R_{Disp}, Figures (11) and (12).

To summarize the presentation, only the results of 4-, 8- and 16-story frames have been presented in Figures (10) to (12). These frames were selected as the representatives of three categories of structures: structures with fundamental period less than 1 sec,

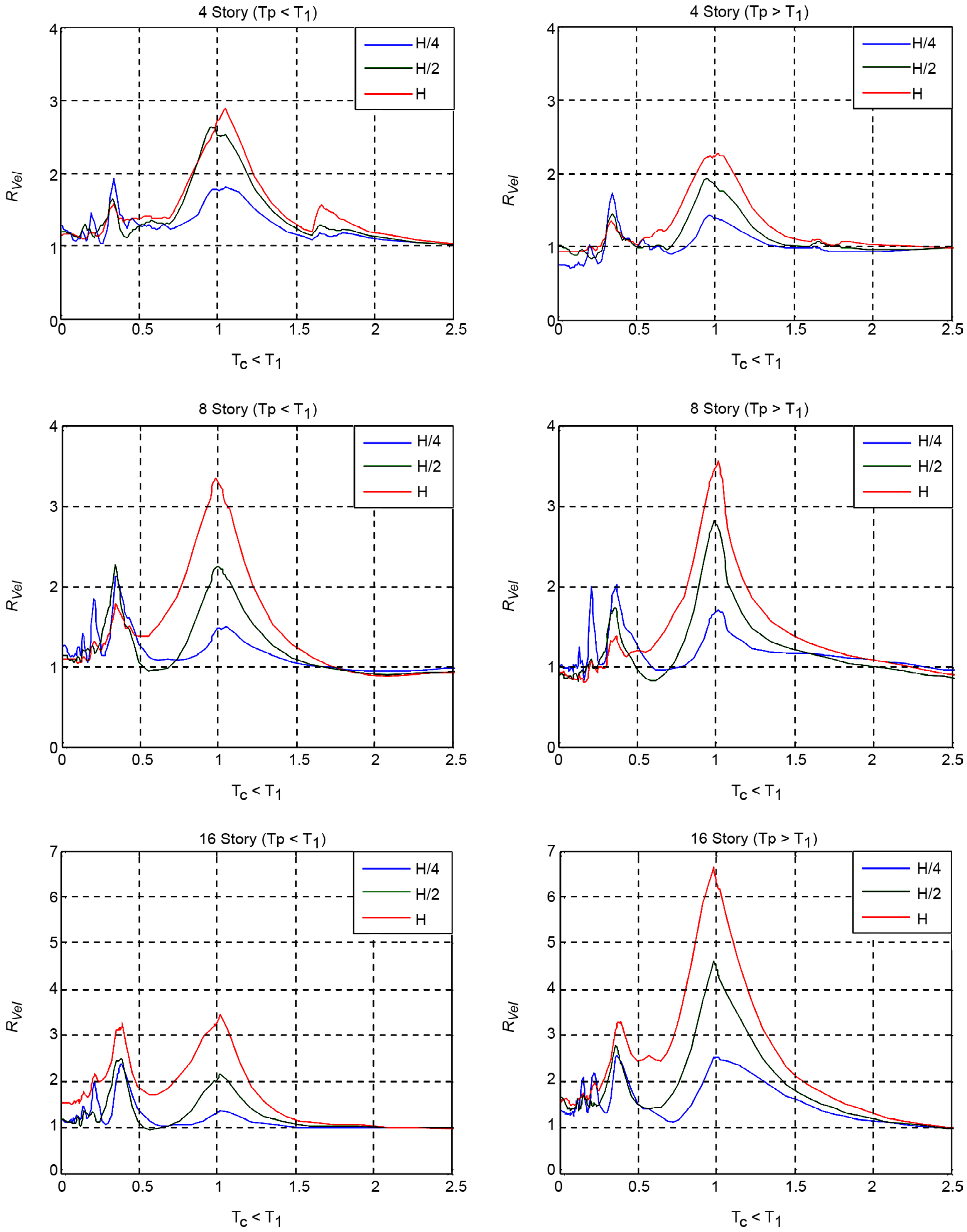


Figure 11. Filtered median R_{Vel} (acceleration) values at various heights in 4-, 8- and 16-story frames due to the pulse period.

between 1 and 2 sec, and structures with fundamental period more than 2 sec. Other structural responses have shown the same pattern of the results. Based on this comprehensive study and engineering judgment, the *R* factors are proposed in Table (3) for two directivity pulse period ranges: period of the directivity-pulse shorter than the fundamental period of the structure ($T_P < T_1$), and period of the directivity-pulse longer than the fundamental period of the structure $T_P < T_1$.

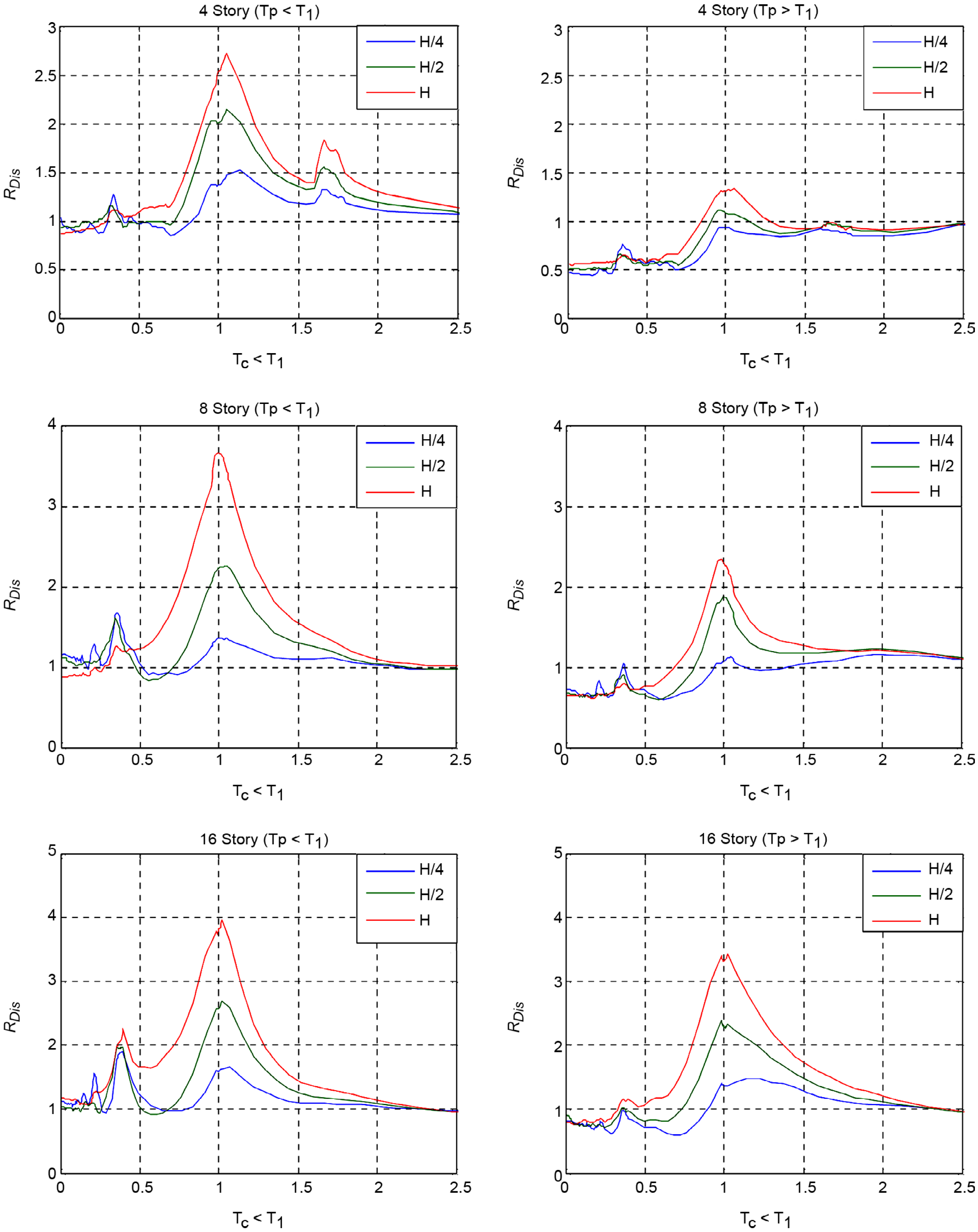


Figure 12. Filtered median R_{Dis} (acceleration) values at various heights in 4-, 8- and 16-story frames due to the pulse period.

Table 3. Maximum allowable reduction factor R at Fundamental and Short-period region.

If $T_P<T_1$

Fundamental Period of Primary Structure	Acceleration		Velocity		Displacement	
	$T_c<0.5T_1$	$0.5T_1<T_c<1.5T_1$	$T_c<0.5T_1$	$0.5T_1<T_c<1.5T_1$	$T_c<0.5T_1$	$0.5T_1<T_c<1.5T_1$
$T_1 < 1$	2.0	1+2.5 z/H	2.0	1+ z/H	1.0	1+1.5 z/H
$1 < T_1 < 2$	2.5	1+1.5 z/H	2.0	1+2 z/H	1.5	1+2.5 z/H
$T_1 > 2$	2.5	1+ z/H	3.0	1+2 z/H	2.0	1+3 z/H

If $T_P>T_1$

Fundamental Period of Primary Structure	Acceleration		Velocity		Displacement	
	$T_c<0.5T_1$	$0.5T_1<T_c<1.5T_1$	$T_c<0.5T_1$	$0.5T_1<T_c<1.5T_1$	$T_c<0.5T_1$	$0.5T_1<T_c<1.5T_1$
$T_1 < 1$	2.0	1+2.5 z/H	1.5	1+ z/H	0.5	1
$1 < T_1 < 2$	2.5	1+4 z/H	2.0	1+2.5 z/H	0.5	1+ z/H
$T_1 > 2$	3.0	1+4 z/H	2.5	1+5 z/H	1.0	1+2 z/H

5. Summary and Results

This study evaluates the variation of the proposed response modification factor (R_{Acc}, R_{Vel} and R_{Dis}) due to near-field pulse-like strong ground motions. This proposed factor has the potential to be used similar to strength-reduction factors for primary structures to scale the elastic FRS to obtain the inelastic FRS. þThe advantage of using the parameter R is that it can address the decrease in elastic FRS values due to yielding of the supporting structure.

The results of this study suggest that in the long-period region; generally, median R values are slightly greater than one. For NSCs with $T_C > 2T_1$, elastic or inelastic behavior of the supporting structure has no effect on the acceleration, velocity and displacement responses of the components. The behavior of R values in short-period region is dependent on the effect of higher modes and almost is independent of the location of NSC in the building. Median R_{Dis} values less than one can be seen in short-period region, especially in stiffer frames. It shows that the displacement of FRS for inelastic primary structures may actually increase in comparison to those obtained when the structure is assumed to remain in the linear elastic range. A substantial increase in R values can be seen in the vicinity of the elastic fundamental period of vibration of the primary structure due to: (1) increasing in the energy dissipated by the primary structure in inelastic range; and (2) shifting the fundamental period of the structure away from the period of the NSC.

The effect of height of the supporting structure can be studied by evaluating the behavior of R in different number of stories of frames. The increasing of R values is close to a straight line variation at different stories. The variation of median values of R_{Acc}, R_{Vel} and R_{Dis} show that increasing the fundamental period of the frames induce increasing R values around modal periods of supporting structure. Floor response modification factors significantly increase due to the inelasticity of the primary structure, i.e. strong ground motions contain larger directivity pulses generally produce a corresponding increase in the peak values of R that signify a reduction in FRS ordinates. To compare the differences between R values under near-field pulse-like and far-field ordinary SGMs, the same earthquake events with epicentral distance more than 60 km are chosen. The results show that the absence of directivity pulses reduce nonlinearity in supporting structures and lead to smaller R factors in comparison to near-field inputs.

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