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Elastic floor response spectra of nonlinear frame structures subjected to forward-directivity pulses of near-fault records

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Abstract. This article presents the statistical characteristics of elastic floor acceleration spectra that represent the peak response demand of non-structural components attached to a nonlinear supporting frame. For this purpose, a set of stiff and flexible general moment resisting frames with periods of 0.3- 3.6 sec. are analyzed using forty-nine near-field strong ground motion records. Peak accelerations are derived for each single degree of freedom non-structural component, supported by the above mentioned frames, through a direct-integration time-history analysis. These accelerations are obtained by Floor Acceleration Response Spectrum (FARS) method. They are statistically analyzed in the next step to achieve a better understanding of their height-wise distributions. The factors that affect FARS values are found in the relevant state of the art. Here, they are summarized to evaluate the amplification and/or reduction of FARS values especially when the supporting structures undergo inelastic behavior. The properties of FARS values are studied in three regions: long-period, fundamental-period and short-period. Maximum elastic acceleration response of non-structural component, mounted on inelastic frames, depends on the following factors: inelasticity intensity and modal periods of supporting structure; natural period, damping ratio and location of non-structural component. The FARS values, corresponded to the modal periods of supporting structure, are strongly reduced beyond elastic domain. However, they could be amplified in the transferring period domain between the mentioned modal periods. In the next step, the amplification and/or reduction of FARS values, caused by inelastic behavior of supporting structure, are calculated. A parameter called the response acceleration reduction factor (R_{acc}), has been previously used for far-field earthquakes. The feasibility of extending this parameter for near-field motions is focused here, suggested repeatedly in the relevant sources. The nonlinearity of supporting structure is included in (R_{acc}) for better estimation of maximum non-structural component absolute acceleration demand, which is ordinarily neglected in the seismic design provisions.

Keywords: Floor Acceleration Response Spectrum (FARS); absolute acceleration modification factor; seismic design; stiff and flexible moment resisting frames; near-field strong ground motions

1. Introduction

After earthquakes, the operation of essential facilities relies heavily on the non-structural components (NSCs) behavior. These components are more susceptible, compared to the structural elements, and their damages (if any) can cause substantial economical and functional losses. In the recent earthquakes, great economical losses were due to the weak performance of non-structural

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components. In the Northridge earthquake (1994), the estimated repair cost was \$5.2 and \$1.1 billion for non-structural and structural damages, respectively (Kircher 2003). According to some reports 82%- 92% of investments in structures are related to the non-structural components and building contents (Taghavi 2003). The case has been studied widely after the strong earthquake of Alaska, 1964 (Ayers 1973).

During the last four decades, several methods have been proposed for seismic analysis of non-structural components attached to the structures. These methods are mainly restricted to linear non-structural components mounted on linear structures (Viti 1981). However, they cannot be used directly in estimating maximum responses of non-structural components, under severe seismic events, where their supporting structures show nonlinear behavior.

According to the formula proposed by UBC (1997) and other similar codes, maximum component absolute acceleration demands are estimated based on the elastic responses and fundamental mode of supporting structures (Miranda and Taghavi 2005 and Singh *et al.* 2006). 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. Response modification factor (R_p) is the only nonlinear parameter in the seismic design codes. This factor represents only nonlinearity of non-structural component or its attachments. It means that the nonlinearity effect of supporting structure is neglected in the estimation of maximum component absolute acceleration demand which cannot be reliably computed by current seismic design provisions.

Nonlinear behaviors of structures and non-structural components may significantly affect the response of non-structural components, Kawakatsu *et al.* (1979), Viti *et al.* (1981), Lin and Mahin (1985), Aziz and Ghobarah (1988), Segal and Hall (1989), Toro *et al.* (1989), Sewell *et al.* (1989), Igusa (1990), Schroeder and Bachman (1994), Singh *et al.* (1996), Adam and Fotiu (2000), Adam (2001), Chaudhuri and Villaverde (2008). In 1985, Lin and Mahin showed that in certain situations the ordinates of nonlinear FARS values may be higher than those of linear ones. Villaverde (1987) proposed a new procedure for seismic analysis of the equipment attached to the elasto-plastic structures. In 1989, Sewell *et al.* found that nonlinear behavior of supporting structures would amplify the acceleration responses of their attached linear single-degree-of freedom non-structural components. They found that such effects are more significant where the nonlinearity of supporting structure is localized. Also, the amplifications of FARS values depend on the frequency contents of earthquake motions as well as the location of non-structural component. According to Singh *et al.* (1996), the increase of acceleration response observed by Sewell *et al.* was due to the inter-modal resonance, the well-known phenomenon in the field of nonlinear oscillations presented by Nayfeh (2000). Singh *et al.* also studied the first floor of a nonlinear ten-story shear building, subjected to an ensemble of synthetic broadband ground motions. They found that some ordinates might be higher in the nonlinear floor acceleration response spectra comparing to their corresponding linear ones. The theoretical achievements became more important when Villaverde (2000) proposed his Design-oriented approach for seismic nonlinear analysis of non-structural components. Chaudhuri and Villaverde (2008) studied a set of nonlinear steel frames to achieve a better understanding of height-wise absolute acceleration distribution. Although their study was comprehensive, they recommended to be followed for different type of structures.

According to the literature, the nonlinearity effect of supporting structure is mainly in the form of corresponding linear response reduction. On the contrary, the nonlinearity effects are seen in the form of amplification in the cases where: (1) a linear non-structural component is tuned to the higher mode of structure; (2) the natural frequency of structure in higher mode is odd integer multiple of its fundamental natural frequency; (3) the nonlinearity of supporting structure is localized; (4) the seismic input is narrow-banded and centered around the fundamental natural frequency of supporting structure. In such conditions, the inelastic effects are misled using absolute acceleration reduction factor. To distinguish the effect of nonlinearity on the important facilities, comprehensive analysis is strongly recommended (Singh *et al.* 1993).

Most non-structural components, widely used in the industrial facilities, are rigidly connected to the supporting structure. These components, which are highly sensitive to the acceleration, are pressure vessels, boilers, heat exchangers and machineries such as pumps, turbines and generators. In order to assure the performance reliability of such components, the researchers have focused simultaneously on: 1) estimating properly the absolute acceleration demand 2) developing more reliable estimation on their capacities. Meeting these goals is necessary in the process of performance-based engineering of non-structural components.

Here a few moment resisting frames are studied for height-wise distribution of maximum floor absolute accelerations as well as the affecting factors. Accordingly, the approaches proposed by Chaudhuri and Villaverde (2008) and Sankaranarayanan and Medina (2006) have been regarded. The considered factors are natural period, height-wise location and damping ratio for non-structural components and modal periods, height and ductility level for supporting structures.

Twelve steel moment resisting frames are subjected to forty-nine near-field strong ground motions. The non-structural components are considered as elastic-linear single degree of freedom systems, rigidly attached to the supporting structures. Several well-known approaches, mentioned above, have been used for to study absolute acceleration modification factors (R_{acc}) in far-field earthquakes. This research attempts to find out whether or not the obtained results can be generalized to near-field earthquakes. The effects of near-field earthquakes have been studied from two different points of views: 1) to distinguish the validity of effective parameters proposed for far-field situation; 2) to compare the values and distribution of absolute acceleration modification factors obtained for far-field and near-field events. The ductility of supporting structure can be considered for better estimation of absolute acceleration demand of non-structural components.

2. Methodology and analysis assumptions

This article studies the statistical characteristics of peak floor acceleration response spectra where supporting structure is overwhelmed and exceeds its elastic response range. For this purpose, first the effective parameters are re-evaluated for near-field events, and then their influences on the peak floor acceleration response spectra are assessed. In this regard, a set of supporting frame structures with different ductilities are subjected to an ensemble of near-field earthquakes. Direct-integration time-history procedure, widely recommended in the literature, is used to estimate the results more properly. Then absolute floor acceleration time-history function is calculated in each story of supporting structures. This function is the base excitation input of elastic variably damped single degree of freedom (SDOF) which is the representative of non-structural components. By the way, the interaction between primary and secondary systems is ignored; i.e. that the mass ratio of secondary to primary system is negligible. The inelasticity is limited to the supporting structure. The assumed critical damping ratios of equivalent SDOF system are 0.01%, 1%, 2% and 5%.

2.1 Supporting structures

Table 1 Modal periods of supporting structures (in seconds)

		Stiff frames			Flexible frames		
		1st mode	2nd mode	3rd mode	1st mode	2nd mode	3rd mode
Number of Stories	3	0.300	0.094	0.045	0.600	0.187	0.091
	6	0.600	0.203	0.110	1.200	0.406	0.221
	9	0.902	0.312	0.177	1.800	0.623	0.354
	12	1.200	0.419	0.244	2.400	0.839	0.487
	15	1.500	0.527	0.310	3.000	1.054	0.619
	18	1.800	0.634	0.375	3.600	1.268	0.751

Supporting shear frame structures are similarly studied by Medina *et al.* (2006), Chaudhuri and Hutchinson (2004) and Chaudhuri and Villaverde (2008). The single bay planar shear frames have 3, 6, 9, 12, 15 and 18 stories with uniform height-wise mass distribution. The frames are designed based on strong column- weak beam concept to assure the plastic hinge formation sequence. In this way, the plastic hinges are formed at both ends of the beams as well as the columns of first storey. Bilinear-rotational springs, with about 3% strain hardening of the initial stiffness, have been considered to represent the hysteresis behavior. Panel zone deformations are considered regarding relative rigidity of beam-column connections. The critical damping ratios of the first and last modes are calibrated as 5% ($\zeta = 0.05$) .

Two groups of stiff and flexible frames, described by Santa Ana and Miranda (2000), are developed and their first modal periods (T_{B1}) are $0.1N$ and $0.2N$ respectively; where, N is the number of stories in the supporting structures. The modal periods of supporting frames are summarized in Table 1.

All frames are analyzed in both elastic-linear and inelastic-nonlinear domains. In the former, the supporting structure is assumed to be elastic, while in the latter, eight different values of ductility are used. As proposed by Sankaranarayanan and Medina (2006), base shear coefficient ($\gamma = \frac{V_y}{W_e}$) is calibrated to show 1 to 8 integer values of ductility and referred as relative intensity (RI) in the below formula

$$RI = \frac{V_{elastic}}{V_y} = \frac{\frac{S_a(T_{B1})}{g} W_e}{\gamma . W_e} = \frac{\frac{S_a(T_{B1})}{g}}{\gamma} = \frac{S_a(T_{B1})}{\gamma . g} \tag{1}$$

where, V_y is base shear strength and W_e is effective seismic weight.

2.2 Selected strong ground motions

The effects of far-field earthquakes on the floor absolute acceleration spectra are well assessed quantitatively in the literature. However, there are few references on the effects of near-field motions (Sewell *et al.* 1989) and therefore strongly recommended by the researchers, Roesset, 1998.

Here, forty-nine near-field earthquake records, previously categorized by Fu (2005), have been used and some of their specifications are as follows:

- All are of the western coast of the United States;
- All are recorded in stiff soils or medium rocks. The soft soils and their effects have been excluded;
- The moment magnitude (M_w) of corresponding earthquakes are limited to 6.0- 6.9 in order to exclude the effects of large earthquakes;
- The epicentral distances are 2.3- 17.0 km;
- The horizontal time-history records are normal to the fault direction. It means that the horizontal records, parallel to the fault direction, are excluded;
- The selected earthquakes correspond to the frequency content of IBC-2003 design spectrum.

The current strong ground motion ensemble is widely addressed as near fault. However, lengthier fault rupture, which is corresponded to larger moment magnitude, increases the directivity effects. Therefore, the results of forward directivity effects could not be generalized to large earthquakes. The main characteristics of earthquake records are summarized in Table 2.

Table 2 Main characteristics of earthquake records

T_{pulse}(sec.)	Station	(M_w)	Earthquake name
1.26	North Palm Springs		
1.38	Desert Hot Springs	6.0	N. Palm Springs
0.63	Whitewater Trout Farm		
0.71	Bell Gardens–Jaboneria	6.0	Whitter Narrows
0.7	Santa Fe Springs–E Joslin		
1.88	Station 2 (Cholame#2)	6.1	Parkfield
0.39	Temblor pre-1969		
0.49	Anderson Dam		
1.04	Gilroy Array#6	6.2	Morgan Hill
0.76	Coyote Lake Dam		
0.7	Pleasant Valley P.P.yard	6.4	Coalinga
3.43	Brawley Airport		
4.1	EC County Center FF		
2.93	EC Meloland Overpass FF		
4.55	El Centro Array#3		
4.18	El Centro Array#4		
3.66	El Centro Array#5		
3.63	El Centro Array#6	6.5	Imperial Valley
3.57	El Centro Array#7		
4.67	El Centro Array#8		
4.01	El Centro Array#10		
4.22	El Centro Differential Array		
4.33	Holtville Post Office		

Table 2 Continued

1.38	Pacoima dam	6.6	San Fernando
2.41	El Centro Imp. Co. Cent	6.7	Superstition Hills
2.12	Parachute Test site		
2.02	Canoga Park-Topanga Can	6.7	Northridge
1.89	Canyon Cty-W Lost Cany		
2.83	Jensen Filter Plant		
0.93	Newhall – Fire Station		
1.16	Rinaldi Receiving		
2.99	Sepulveda VA		
2.88	Sylmar Converter		
3.05	Sylmar Converter East		
2.53	Sylmar Olive View		
2.18	Newhall-W.Pico Canyon		
0.48	Pacoima Dam Downstreet		
0.72	Pacoima Ragel Canyon		
1.42	LA Dam		
1.54	Gilroy-Historic Bldg.	6.9	Loma Prieta
4.24	Gilroy Array#1		
1.43	Gilroy Array#2		
1.79	Gilroy Array#3		
1.37	Gilroy Array#4		
1.77	Gilroy-Gavilan Coll.		
2.25	Saratoga-Aloha Ave.		
2.16	Saratoga-W Valley Coll.		
3.21	Los Gatos		
1.81	Lexington Dam		

3. Absolute acceleration modification coefficient

Nonlinear analysis of the structures, excited by earthquakes, is a time consuming procedure and normally not attracted by designers. Therefore, the equalization coefficients are used in the seismic design codes in order to compensate the neglected post-yielding energy consumption capacity. The results obtained by linear-elastic analysis are calibrated by these coefficients to meet the corresponding inelastic responses. The equalization can be achieved by modification coefficients such as reduction factors, proposed in current seismic design codes. Such coefficients are used to calibrate base shear and lateral displacements.

Several researchers have attempted to extend the concept of modification coefficients to the field of non-structural components. Lin and Mahin (1985) proposed an amplification factor conceptually the inverse of current modification factors. Sewell and *et al.* (1986, 1987) developed floor response spectral ratio. Singh *et al.* (1993) proposed a response reduction factor (R-factor), obtained from dividing elastic absolute acceleration ratio by inelastic one. The modification coefficients can amplify or reduce elastic responses. However, their reducing effects have been focused in the literature to achieve higher economical advantages due to the lower demands. Sankaranarayanan and Medina (2007) extended the concept of modification coefficient more quantitatively and practically for far-field earthquakes. Here it is studied for near-field motions. Absolute acceleration modification coefficient is the spectral ordinates of elastic floor absolute acceleration response, which is normalized to its corresponding inelastic one, shown in, the below formula

$$R_{acc}(T_C) = \frac{S_{aC,elastic}(T_C)}{S_{aC,inelastic}(T_C)} \tag{2}$$

Where, T_C is natural period of SDOF non-structural component.

Maximum absolute acceleration response at the top of elastic SDOF system, rigidly attached to the floor of an elastic supporting structure, is related to its corresponding inelastic value by R_{acc}. The parameters, affecting response modification coefficient, have been discussed properly by Chaudhuri and Villaverde (2008) and Sankaranarayanan and Medina (2006).

Sewell *et al.* (1986) attempted to get better understanding of near-field effects on the absolute acceleration modification coefficient. However, their results cannot be generalized due to the limited used data. Therefore, in this research a broader range of structures and near-field time-history records are used to better quantify the modification coefficient corresponding to near-field earthquakes.

4. General specifications of response modification coefficient

Absolute acceleration time-history records of the floors have been selected in the mentioned cases. They are used as the base excitation input of an elastic SDOF non-structural component in order to generate their absolute acceleration response spectra. For this purpose, the incremental step is assumed as 0.01 sec. ($\Delta T = 0.01\ sec.$). In general, time-period is utilized as horizontal axis; here, abscissa is normalized by fundamental period of supporting structure to meet better resonance phenomena. Response modification coefficient (R_{acc}) is obtained by dividing absolute acceleration response spectrum ordinate, corresponded to the elastic supporting structure, by that of inelastic one. Fig. 1 represents the median response modification coefficient function of 9-storey-stiff frame subjected to near-field earthquakes. The conclusions, inferred from the figure, are as follows:

- The elastic response is more reduced if relative intensity (RI) increases in the supporting structure;
- More reduction in the elastic response is expected around the fundamental period of supporting structure.

These observations are similar to the results obtained by other researchers regarding far-field earthquakes; with the exception of slightly higher reduction value and more dispersion. Regardless some minor amplifications, observed in the long period region, great economical benefit is expected in counting on the inelastic behavior of supporting structure. The amount of reduction depends on different parameters, well presented in the literature. These parameters are: fundamental period of supporting structure; natural period of Non-Structural Component; quotient of higher modal frequency of supporting structure to its corresponding fundamental one; inelasticity level in the supporting structure; inelastic hysteresis model; local mechanism in supporting structure; NSC's damping ratio; total height of supporting structure and height-wise location of NSC.

The concept of period domain demarcation which is previously proposed by Chaudhuri and Villaverde (2008) and Medina *et al.* (2006) could be extended to the near-field excitations. In this way, the abscissa time-period could be divided into three regions:

- Large period ratios$\left(1.5 \leq \frac{T_C}{T_{B1}}\right)$. The median values of R_{acc}, corresponded to all RI, will converge to 1. However, $R_{acc} < 1$, seen in a few points, can be attributed to the nonlinear period of supporting structure. This amplification is limited and could be ignored in the engineering purposes. These properties could be interpreted as the independency of R_{acc} values to any other parameters.
- Resonance period ratios$\left(0.5 \leq \frac{T_C}{T_{B1}} \leq 1.5\right)$. This region represents the resonance condition in which the natural period of non-structural component and fundamental period of supporting structure are tuned to each other. R_{acc} considerably increases in this region due to the energy mitigation adjacent to fundamental period of supporting structure. The maximum response modification coefficient observed in this region defined as $R_{acc,1}$

$$R_{acc,1} = max\left(R_{acc}(T_C, T_{B1})\right); While\ 0.95 \leq \frac{T_C}{T_{B1}} \leq 1.05 \tag{3}$$

This could be interpreted as maximum achievable reduction in the vicinity of predominant period of supporting structure, Lin and Mahin (1985).

- Low period ratios$\left(\frac{T_C}{T_{B1}} \leq 0.5\right)$. In this region, response modification coefficient (R_{acc-HF}) is defined as the maximum response modification coefficient and observed within higher mode regions.

$$R_{acc-HF} = max\left(R_{acc,i}(T_C, T_{Bi})\right); While\ 0 \leq \frac{T_C}{T_{B2}} \leq 1.05 \tag{4}$$

Here two important behaviors are observed: increasing the values of R_{acc} adjacent to the higher modal periods of supporting structure and the chaotic behavior in the intra-modal areas.

Non-structural damping has a crucial role in this region; more particularly, lower damping values are potentially resulted in higher amplifications. The amplification is a well-known phenomenon discussed previously by the researchers (Sewell *et al.* 1987); however, the quantification of response amplification is ignored there. Also, R_{acc-HF} plays a governing role in determining the maximum acceleration reduction of upper floors, affected more significantly by higher modes.

Better estimation of earthquake induced forces is very important in designing and/or retrofitting of any non-structural component. It is economically reasonable to account on response reduction when supporting structure is overwhelmed by a great earthquake. Therefore, quantifying the amount of response modification would be of a great interest. The amplification and/or reduction of peak floor acceleration response should be quantified along with reliable assessment of non-structural components. The acceleration response reductions of stiff/flexible frames, subjected to near-field earthquakes, are quantified in the following.

5. Parameter efficiency evaluation

Quantifying the response modification factor is contingent on the understanding of effective parameters. Some of these parameters were previously discussed by other researchers for far-field earthquakes. Sewell *et al.* (1986) have studied:

- The quotient of higher modal period of supporting structure to its corresponding fundamental one;
- Inelastic hysteresis model;
- Local mechanism in supporting structure.

Other important parameters, discussed by Chaudhuri and Villaverde (2008) and Medina *et al.* (2006) for far-field earthquakes are:

- Fundamental and modal period of supporting structure;
- Effective height of supporting structure;
- Ductility of supporting structure and its distribution;
- Height-wise location of non-structural component;
- Damping ratio of non-structural component.

In order to assess the effectiveness of these parameters in the near-field earthquakes a brief discussion is presented here based on median values of response modification factor. In this way, the feasibility of extending far-field results to near-field situations is controlled.

5.1 Fundamental and modal period of supporting structure.

Ordinarily, the maximum absolute acceleration modification coefficient (R_{acc}) is discussed around the fundamental and modal periods of supporting structure, Sankaranarayanan & Medina (2006). They showed the importance of fundamental period of supporting structure (T_{B1}) and the quotient of secondary natural period to the primary fundamental period ($\frac{T_C}{T_{Bi}}$). Fundamental period of supporting structure (T_{B1}), plays a governing role on the period domain demarcation, discussed in the following. However, $\frac{T_C}{T_{B1}}$, is a better parameter to investigate the non-structural resonance in the vicinity of modal periods of supporting structure.

5.1.1 Fundamental period of supporting structure.

Response modification coefficients, R_{acc1} and R_{acc-HF}, as functions of the fundamental period of supporting structure (T_{B1}), are shown in Fig. 2. Excluding the 3-storey stiff frame with $T_{B1} = 0.3$, other supporting structures behave similarly.

Regarding the frames with medium to large-periods, the modification coefficients corresponded to the fundamental mode (R_{acc1}) weakly depend on the fundamental period (T_{B1}). However, in the higher modes, the coefficients (R_{acc-HF}) increase linearly with T_{B1}.

5.1.2 Secondary-primary natural period quotient.

Response modification coefficient variation is presented in Fig. 3 as a function of secondary-primary natural period quotient, $\frac{T_C}{T_{B1}}$. It is evident that non-structural components tuned to modal periods will experience resonance. On the other hand, the response modification factor, corresponded to large values of secondary-primary natural period quotient, $\frac{T_C}{T_{B1}} > 1.50$, is converged to unity.

These observations are special characteristics of secondary-primary natural period quotient parameter, $\frac{T_C}{T_{B1}}$, in the separation of period domain. The mentioned facts are used in developing formulation of response modification factor and discussed in the next section.

5.2 Effective height of the supporting structure

As expressed earlier, two different types of supporting frames were chosen as the representatives of flexible and rigid structures. The fundamental period of flexible frame is twice that of the stiff one as per certain number of stories. In other words, a specific fundamental time-period can be attributed to a stiff frame with the height twice as its similar flexible counterpart. This specification allows the study of height effect on the response modification factor. The flexible and stiff frames with different number of stories and equal time-periods are compared and shown in Fig. 4.

The flexible and stiff frames with fundamental periods of 0.3 and 1.8 sec. are presented in Figs. 4(a) & 4(b), respectively. According to the figure, the response modification factors of both types of structures are equal in the fundamental and high-period quotient regions but different in low-period due to different modal participating ratios. This could be attributed to fewer effective modes of flexible frames resulting in lower response modification factor peaks.

According to the Fig. 4(b), response modification factors follow similar trends and values, as expected, in both types of supporting structures. It should be noted that, in this case the number of modes and modal participating ratios are somehow similar. Finally, the convergence of response modification factor, corresponded to the rigid and rigidly supported non-structural components, should be considered in all cases. It means that, peak floor acceleration ratios would be the same, regardless the height of supporting structure.

5.3 The ductility of supporting structure

According to the current seismic design codes, nonlinear behaviors of engineering structures should be conserved during severe earthquakes due to their major economical investments. Nonetheless, this will affect the seismic response of non-structural component supported by the floors.

Regarding the mentioned codes, the accelerations will decrease and the displacements increase when the structure experiences nonlinear behavior. Although this generalization has been refused by some researchers (Sewell *et al.* 1987), it works in many cases. Therefore, it is generally expected that increasing the ductility in supporting structure will be resulted in lower floor acceleration. In other words, increasing the ductility level of supporting structure often ends in the increase of response modification factor. This fact, shown in Fig. 5, is in accordance with the current literature (Sankaranarayanan and Medina 2006).

According to the above figure, the acceleration response modification factor is a function of supporting structure's ductility, which can offer more economical advantages in case of being increased. Floor acceleration response reduction is more pronounced in non-structural components tuned to fundamental period of supporting structure. Therefore, the $R_{acc,1}$ values are much greater than their corresponding R_{acc-HF} values, as expected. It should be mentioned that lower ductility values are resulted in the acceleration response reduction factors, which are relatively constant along the height of supporting structure. Therefore, ductility can play a key role in developing the formulations of predicting response modification factor.

5.4 The height-wise location of non-structural component

The installing location of NSC is one of the well-known parameters commonly addressed by the researchers (Sankaranarayanan and Medina 2006). This parameter affects peak component acceleration (PCA) through modifying peak floor acceleration (PFA). According to the current seismic design codes, non-structural component accelerations increase linearly with their installation heights.

Fig. 5 depicts the height-wise distribution of response modification factor. It is obvious that $R_{acc,1}$ follows a general trend regardless of the total height of supporting structure; however, R_{acc-HF} has a disturbed pattern. The fundamental response modification factor exceeds its corresponding higher frequency response modification factor, $R_{acc,1} > R_{acc-HF}$, approximately along the total height of the supporting structure.

It should be mentioned that $R_{acc,1}$ has a linear ascending relationship with the relative height of supporting structure (RH) up to its mid-height. This relationship is descending beyond mid-height. Therefore, the maximum value of $R_{acc,1}$ occurs at the upper half part of supporting frame, excluding the 3-storey one in which its higher modal participation is not similar to those of the others.

5.5 Damping ratio of non-structural component

Considering non-structural component responses, two different types of damping, supporting structure damping and non-structural component damping, should be taken into account. The effects of structural damping on the acceleration have been well studied by the researchers (Chopra and Chintanapakdee 2001). However, there are few studies about the effects of non-structural component damping on the floor acceleration response spectra (Sankaranarayanan and Medina 2006).

The increase of damping is generally resulted in FARS decreasing. This phenomenon is more pronounced in the areas adjacent to modal periods. The acceleration response modification factor is a function of period quotient for different values of non-structural component damping, shown in Fig. 6. According to the figure, this factor is amplified and its pattern will become more jagged because of NSC's damping reduction. These observations are consistent with those found for the effects of damping on the floor acceleration response spectrum.

Based on the above mentioned figure, period quotient demarcation can be advantageous again. The damping effect is negligible in the long-period quotient region, as expected. This could be attributed to the reduction in the efficiency of damping as an important factor because of corresponding natural softness of the region. Regarding fundamental period quotient region, economical benefit can be achieved as the NSC's damping ratio decreases. As mentioned earlier, floor acceleration response modification factor is defined as the elastic acceleration spectrum divided by inelastic one. Based on this definition, inelastic acceleration spectrum is not as much affected by NSC’s damping as its corresponding elastic one. It should be noted that the lower the

NSC's damping values, the lower the response modification factor values are in the short period quotient region, $R_{acc-HF} < 1.00$.

Fig. 7 represents height-wise variation of response modification factors for different values of NSC's damping ratios. According to this figure, the effects of NSC's damping ratio are significantly stronger on $R_{acc,1}$ compared to R_{acc-HF}.

Based on the above mentioned observations, NSC's damping ratio is one of the most effective parameters in developing the statistical formulations of response modification factor and therefore cannot be neglected. What is observed so far for near-field earthquakes are qualitatively in accordance with those of far-field events, mentioned in the literature.

6. Estimating the acceleration response modification

Developing a mathematical model for estimating non-structural component responses is a time consuming activity and met only at the final steps of structural design. In practice, to utilize relationships for predicting response acceleration factor is more expeditious than to analyze directly the non-structural component mounted on the supporting structure. Concerning the latter, the comprehensive understanding of effective parameters is needed and cannot be attained by a limited database.

It has been attempted to propose reliable relationships for estimating the acceleration response of non-structural components supported by nonlinear structures. Accordingly, Chaudhuri and Hutchinson (2004) have proposed a general relationship regardless the specifications of ground motions; Sankaranarayanan and Medina (2006) have presented a more specific formula limited to the far-field excitations. It is strongly recommended to assess the feasibility of extending far-field results to near-field situation by Roesset (1998), Sewell *et al.* (1986) and Sankaranarayanan and Medina (2006). This recommendation has been followed in the present study. It should be noted that limited data have been used in this research.

6.1 Main characteristics of database

The supporting structures have been precisely analyzed; however, the analysis procedure sometimes encountered dynamic instability, originating from P-Delta effects, and/or progressive failure corresponding to vast nonlinearity. This would be the case for flexible frames that widely suffer from lack of adequate stiffness. These frames tend to encounter instability for $RI \geq 5$. Also, more than a quarter of flexible supporting structures experience dynamic instability for $RI = 4$. The results of 3-storey supporting structures have been eliminated from the database to achieve a more convenient relationship between the acceleration response modification factor and the governing parameters.

Therefore, a limited set of results has been used to develop statistical relationships between acceleration modification factor and dominant parameters. The scope of stiff and flexible frames are the same in their ductility levels ($RI = 2, 3\ and\ 4$) and non-structural damping ($\zeta = 0.01\%, 1\%, 2\%\ and\ 5\%$). However, the numbers of stories are 6, 9, 12, 15 and 18 in stiff frames and, 6 and 9 in the flexible ones.

Other results are excluded for reducing the standard error in the proposed relationship. As it was discussed earlier, the reduction can be replaced by amplifying the bottom parts of the supporting structure. It should be noted that this study neglected such amplifications and focused on the acceleration response reduction. In the next section, the proposed response modification formulas have been presented based on period quotient demarcation.

6.2 Proposing the relationships for acceleration response modification factor

Period quotient parameter is used here to differentiate the response modification behavior and its dependency on the effective parameters. In this regard three distinct regions are addressed to simplify the proposed relationships:

6.2.1 Long-period quotient region $\left(1.5 \le \frac{T_C}{T_{B1}}\right)$:

In this region R_{acc} is converged to unit value; however, the amplifications are observed at a few points and are more prominent in lower values of non-structural damping $(\zeta < 1\%)$. Therefore, as it is recommended in the researches, response modification factor could be taken as 1.00 in long-period quotient region (Sankaranarayanan and Medina 2006).

6.2.2 Fundamental-period quotient region $\left(0.5 \le \frac{T_C}{T_{B1}} \le 1.5\right)$:

Many regression patterns could be used to develop statistical relationship between response modification factor and the effective parameters. Two well-known patterns are presented in the literature. The first one is limited to far-field earthquakes, Sankaranarayanan and Medina (2006). The second is based on a direct combination of shear and flexural modes, Chaudhuri and Hutchinson (2004) and Razaghi and Mahmoudzadeh kani (2009). The first pattern has been selected here to develop statistical relationship and assess the feasibility of developing far-field results to near-field circumstances. The selected relationship is a single parameter (RI) pattern presented here as:

$$R_{acc}\left(\zeta_{NSE}, RH, RI, \frac{T_C}{T_{B1}}\right) = R_{acc,1}\left(\zeta_{NSE}, RH, RI, \frac{T_C}{T_{B1}}\right) = A \times (RI)^B; While\ 0.5 \le \frac{T_C}{T_{B1}} \le 1.5 \quad (5)$$

Where, A and B are constant coefficients, presented in Table 3. According to the results of statistical analyses, the correlation factor of more than 0.67 shows a good correlation between response modification factor and relative intensity. It should be mentioned that increase in ductility will be resulted in the decrease of correlation coefficient. Therefore, the above mentioned relationship is very limited and could be interpreted just as the feasibility of far-field result extension to near-field situation.

6.2.3 Small-period quotient region $\left(0 \le \frac{T_C}{T_{B1}} \le 0.5\right)$:

Here, two-parameter (RI and T_{B1}) relationship has been developed to include the effect of fundamental period of supporting structure as follows:

$$R_{acc}(\zeta_{NSE}, RH, RI, T_{B1}, T_C) = R_{acc-HF}(\zeta_{NSE}, RH, RI, T_{B1}, T_C) = C \times (RI)^D \times (T_{B1})^E;$$

$$While\ 0 \le \frac{T_C}{T_{B1}} \le 0.5 \quad (6)$$

The constant parameters C, D and E, presented in Table 4, depend on the non-structural damping and height-wise location of non-structural component. Despite developing two-parameter relationship, the correlation coefficient expectedly decreases to 0.60 which is lower than that of one-parameter relationship developed for fundamental period quotient region.

6.3 Comparing the results

Table 3 The Constant Parameters (A and B) proposed to estimate R_{acc} values in the fundamental period quotient region

RH	ζ	A	B
1.0	0.01%	1.154	1.468
1.0	1.00%	1.066	1.349
1.0	2.00%	0.981	1.208
1.0	5.00%	0.904	1.058
0.5	0.01%	1.179	1.541
0.5	1.00%	1.084	1.412
0.5	2.00%	0.990	1.276
0.5	5.00%	0.924	1.128
0.33	0.01%	1.303	1.365
0.33	1.00%	1.177	1.230
0.33	2.00%	1.076	1.116
0.33	5.00%	1.014	0.976

Table 4 The Constant Parameters (C, D and E) proposed to estimate R_{acc} values in the short period quotient region

RH	ζ	C	D	E
1.0	0.0001	1.227	1.119	0.602
1.0	0.01	1.104	0.998	0.498
1.0	0.02	1.012	0.890	0.432
1.0	0.05	0.952	0.775	0.348
0.5	0.0001	1.258	1.015	0.603
0.5	0.01	1.108	0.929	0.512
0.5	0.02	1.019	0.831	0.464
0.5	0.05	0.990	0.711	0.382
0.33	0.0001	1.288	1.069	0.685
0.33	0.01	1.120	0.955	0.584
0.33	0.02	1.036	0.855	0.540
0.33	0.05	0.951	0.776	0.482

In this section, some important specifications, inferred from comparing far-field studies -Sankaranarayanan and Medina (2006), Chaudhuri and Hutchinson (2004) and Razaghi and Mahmoudzadeh Kani (2009) - and near-field observations are presented. Accordingly, similar relationships are developed in both cases while multipliers and exponents have their own values.

Moreover, the correlation factors of near-field results are lower than those of far-field ones, showing weaker statistical relationships. This could be attributed to the size of strong ground motion ensemble as well as the effect of near-field velocity pulse which causes the failure of some supporting structures.

It could be deduced that the relationship proposed for far-field excitations can be extended to near-field situations. However, this conclusion is limited to the assumptions of this study.

7. Conclusions

In this study, the effects of near-field excitations have been assessed on the acceleration response modification factor. Moreover, the feasibility of developing any relationship similar to what proposed for far-field excitations has also been studied. In this regard, well-known parameters which affecting response modification factors in far-field situations have been investigated once more for near-field strong ground motions.

The general properties of response modification factor in different period quotient regions are very close to what has been observed by other researchers for far-field situation. Here, the period quotient abscissa can be demarcated again; however, the results obtained in low-period region are more jagged comparing to far-field situation. The acceleration response is reduced more, when natural period of non-structural component is tuned to the structural modal periods. On the other hand, amplification can occurs in the transitional period quotient between modal periods of supporting structure, which is not in the scope of current study. It should be mentioned that the controlling parameters that have play important roles in the far-field excitations can be addressed to the near-field earthquakes as well.

These observations were followed by studying the possibility of developing statistical relationships. The statistical studies confirmed the feasibility of this idea; however, less correlation values can be achieved and more limitations are imposed on the near-field relationships.

It should be mentioned that the proposed statistical relationship is limited to the assumptions of this study.

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