Height-Wise Distribution of Peak Horizontal Floor Acceleration (PHFA)

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ABSTRACT :

Peak Horizontal Floor Acceleration (PHFA) has often been used to estimate the vulnerability of acceleration sensitive nonstructural elements. To estimate the force imposed on nonstructural elements such as architectural, mechanical and electrical components, PHFA is assumed to vary along the height of building. Since the value of PHFA at a particular level of building depends on its dynamic properties; the level of nonlinearity induced; and the ground excitation intensity, the PHFA cannot be formulated as a function of height without considering these aspects. In this study, the distribution of absolute acceleration amplification factor Ω (PHFA normalized by peak ground acceleration) along the height of buildings with different dynamic parameters is developed through Nonlinear Time History Dynamic Analyses (NTHDA). A total of five 2D steel moment-resisting frame buildings are considered. An ensemble of 28 different ground motions recorded on four soil types are used as earthquake excitation input for NTHDA. Resulting distributions are compared with code-recommendations, then a simplified distribution of Ω is proposed. Although simplified, the suggested distribution shows promising results in estimating the vulnerability of rigid acceleration sensitive nonstructural components.

KEYWORDS : Peak Horizontal Floor Acceleration, Nonstructural Components

1. INTRODUCTION

It has been recognized that the effect of failure of nonstructural components and equipments is significant during earthquakes. Infact, economic and life losses due to nonstructural components and equipment damages during recent earthquakes in the Iran (Hosseini 2003) and other earthquake prone countries (Ayers 1973; Reitherman 1995) has been quite considerable. These losses are sometimes far greater than those resulting from structural damages. After the 1971 San Fernando earthquake, it was recognized that the damage of nonstructural components and equipments may not only result in major economic loss but also poses a threat to life (Ayers 1973). Nonstructural components and equipments built in important buildings such as hospitals, fire and police stations, power generation facilities, water supply and water treatment facilities, generally have high vulnerability to functionality loss. Moreover, fire hazards resulting from sliding, toppling and breaking of chemical storage containers or glassware resting on furnishings are very serious. During the 1994 Northridge earthquake, several major hospitals had to be evacuated, not due to structural damage but because of the damage caused by failure of water lines and water supply tanks; the failure of emergency power systems and heating; ventilation; and air conditioning units; damage to suspended ceilings and light fixtures; and some broken windows (Chaudhuri et al. 2004). The nonstructural components and systems are classified as either deformation-sensitive or acceleration-sensitive. Components such as: interior partitions; stairs; doors and exit routes are generally classified as deformation-sensitive. Components such as : suspended ceilings; bookshelves; emergency power generation systems; air conditioning units; cable trays and control panels; chemical glasswares; parapet walls and piping are generally classified as acceleration sensitive. As a large number of nonstructural components are classified as acceleration sensitive, the determination of appropriate floor acceleration in a building when subjected to an earthquake seems as the most important parameter in estimating the vulnerability of nonstructural components.

Although the significance of the survival of nonstructural components is well understood, limited research has been conducted to understand and mitigate their vulnerability. Present seismic design provisions (IBC 2006; NEHRP 2003) recommend a linear variation of acceleration along the building height to estimate the design force induced on nonstructural components. The design philosophy adopted by IBC (2006) and NEHRP (2003) provisions seeks to ensure these components are able to withstand design earthquake load without collapse, toppling or shifting. Such a philosophy is common in seismic design of building structures. Using these codes, an equivalent lateral load is determined as a function of weight of element, anticipated ground acceleration, location of the element within the building, the dynamic amplification of the element, and its ability to absorb inelastic deformations. The inelastic behavior of the support structure has not been included in codified formulas yet, because it is believed that: (1) the extent of inelastic behavior is usually minor for structures designed by modern building codes, as their design is, in many cases, governed by drift limits or other loads; (2) nonstructural components are often designed without knowledge of the structure composition; and (3) it is a conservative consideration. In NEHRP (2003) and IBC (2006) code recommendations, one notes that the calculation of forces applied to nonstructural components assumes a linear distribution of acceleration, varying with the PGA (Peak ground acceleration) at the ground level to three times the PGA at the roof level. The provisions used in IBC (2006) and NEHRP (2003) were developed empirically on the basis of floor acceleration data recorded in buildings during California earthquakes. Codified formula recommend the same distribution along the building height, regardless of the number of stories in the building, its lateral resisting system or expected nonlinear behavior. As a consequence, it is not known whether or not a nonstructural component designed with these formulas will be able to resist a large earthquake (Chaudhuri et al. 2004). Kehoe (1998) and Searer (2002) concluded that the intensity and distribution of floor accelerations over the height of a building is influenced by the predominant period of vibration of the building, the mode shapes and their relative contributions. However, their conclusions are based on earthquakes not strong enough to induce nonlinear deformations. Miranda (2003, 2005) presented a simplified method for estimating floor acceleration distribution of elastic buildings when subjected to a particular ground motion. However, this work also did not consider nonlinear behavior of the building, which is very common when subjected to large earthquakes. Several investigations have pointed out that the nonlinear behavior of a building and nonstructural system may greatly affect the response of nonstructural systems, either by significantly reducing or substantially amplifying the response, as compared with the corresponding linear response (e.g. Sewell 1989; and Singh 1993).
In this study, the distribution of the peak horizontal floor acceleration (PHFA) along the height of building structures is investigated assuming 28 ground motions, with a broad range of seismic hazard levels recorded on four soil types. For this purpose, five 2D steel moment-resisting frame buildings are considered and analytical models constructed using SAP 2000 nonlinear ver 9.1.6 (2005). The nonlinearity occurred in beam and column elements are kept limited to immediate occupancy performance acceptance criteria. Although there is significant scatter in the resulting acceleration amplification distribution, a nonlinear distribution on a per floor basis is shown able to reasonably estimate the ensemble floor distributions and compared with the NEHRP code recommendations (2003). Finally, a proposed amplification of relative acceleration distribution is presented. The proposed eqation can be used to estimate the vulnerability of rigid acceleration sensitive nonstructural components.

2. THE BUILDING MODELS

For this study, five steel moment-resisting frame (SMRF) buildings with four, eight, twelve, sixteen and twenty stories each with three 5 m span are considered. The five structural models, have the same story height of 3 m (Fig. 1), and have a uniform mass distribution over their height and a non-uniform lateral stiffness distribution (Table 2.1). They were designed using the lateral load distribution specified in the Iranian Building Codes and Standards (2005). Results of eigenvalue analyses of the different models, are provided in Table 2.2. The fundamental periods of these structures is ranging from 0.5-1.8 second. Numerical models were developed for analyses, using a representative 2D frame of the buildings along the transverse direction. It is assumed that the beam members can develope flexural plastic hinges. The flexural plastic hinges in column are considering the axial load-flexural moment interaction. The nonlinear direct integration time history analyses is used with a 5% damping coefficient for the first and the second modes of structural models.

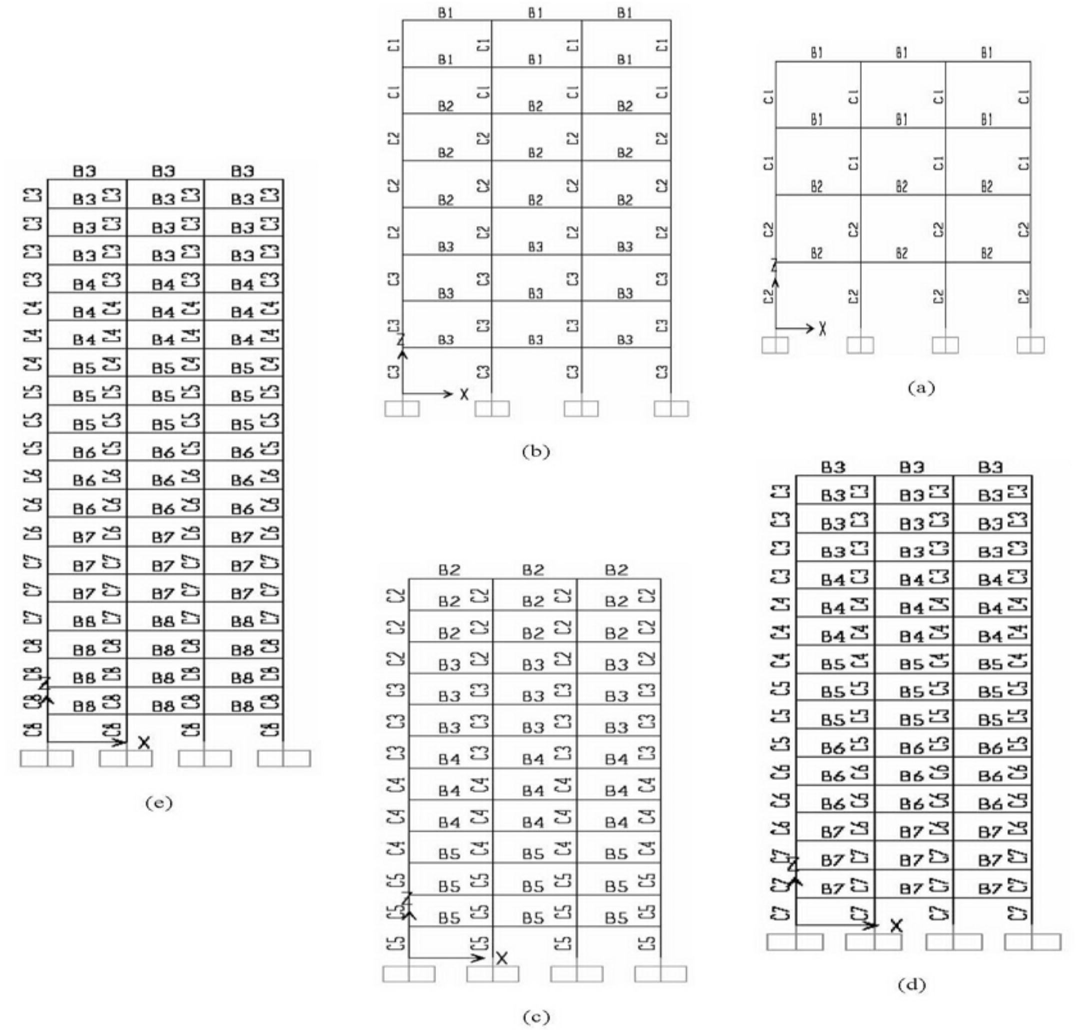


Figure 1 The five SMRF buildings with (a) four, (b) eight, (c) twelve, (d) sixteen and (e) twenty stories

Table 2.1 The flexural stiffness of elements

Member	Section	Moment Inertial (cm4)
Beam	B 1	39340
	B 2	51900
	B 3	73890
	B 4	93820
	B 5	112700
	B 6	133800
	B 7	153800
	B 8	180900
Column	C 1	57380
	C 2	75400
	C 3	106800
	C 4	121300
	C 5	152000
	C 6	176300
	C 7	191000
	C 8	237800

Table 2.2 Dynamic parameters of structures

Model	Period (sec)		Modal Participating Mass Ratio - Mode 1	Modal Participating Mass Ratio - Mode 2
	Mode 1	Mode 2		
4 Story	0.50	0.16	81.4%	12.5%
8 Story	0.91	0.32	76.5%	13.0%
12 Story	1.19	0.42	75.4%	12.7%
16 Story	1.48	0.51	74.1%	13.7%
20 Story	1.83	0.62	71.9%	14.7%

3. GROUND MOTIONS CONSIDERED

For this study, 28 ground motions recorded on four soil types (classified by shear wave velocity in soil layers) are used. These ground motions were generated for the UC Science building as a part of PEER (Pacific Earthquake Engineering Research Center) test bed project (2007). The ground motions are derived from actual ground motion records considering their magnitude and distance from the fault to site. The list of the ground motions used along with some of their parameters is provided in Table 3.1. The peak ground acceleration (PGA) for these motions varies from 0.26g to 0.821g. The range of peak ground velocity (PGV) is 12.2 – 120.7 cm/sec, and the range of peak ground displacements (PGD) is 1.9 – 41.3 cm.

Table 3.1 Earthquake motions used for input in this study

Earthquake	Station	PGA (g)	T (sec)	Tp (sec)	Significant Duration
Kobe 1995/01/16	KJMA	0.82	48	0.34	8.36
Landers 1992/06/28	Lucerne	0.79	48	0.08	13.76
Loma Prieta 1989/10/18	Corralitos	0.64	40	0.30	6.88
Northridge 1994/01/17	LA - Univ. Hospital	0.49	40	0.38	10.44
Parkfield 1966/06/28	Temblor pre-1969	0.36	30	0.38	4.35
San Fernando 1971/02/09	Lake Hughes #12	0.37	37	0.16	10.73
Victoria, Mexico 1980/06/09	Cerro Prieto	0.62	24	0.06	8.57
Cape Mendocino 1992/04/25	Rio Dell Overpass - FF	0.55	36	0.42	10.86
Landers 1992/06/28	Coolwater	0.42	28	0.34	8.23
Whittier Narrows 1987/10/01	LA - Obregon Park	0.45	40	0.18	8.01
Loma Prieta 1989/10/18	Saratoga - Aloha Ave	0.51	40	0.16	9.36
Morgan Hill 1984/04/24	Anderson Dam (Downstream)	0.42	28	0.44	6.83
Northridge 1994/01/17	Beverly Hills - 12520 Mulhol	0.62	24	0.26	7.59
N. Palm Springs 1986/07/08	North Palm Springs	0.69	20	0.18	5.15
Imperial Valley 1979/10/15	SAHOP Casa Flores	0.51	16	0.18	7.47
Loma Prieta 1989/10/18	Capitola	0.53	40	0.28	11.92
Northridge 1994/01/17	Glendale - Las Palmas	0.36	30	0.20	9.49
Parkfield 1966/06/28	Cholame #5	0.44	44	0.36	6.45
Coyote Lake 1979/08/06	Gilroy Array #2	0.34	27	0.16	4.23
Morgan Hill 1984/04/24	Gilroy Array #4	0.35	40	0.24	12.55
Chi-Chi, Taiwan 1999/09/20	TCU072	0.40	70	0.74	24.00
Imperial Valley 1979/10/15	Centro Array #3	0.27	40	0.18	11.87
Chi-Chi, Taiwan 1999/09/20	CHY041	0.64	70	0.40	22.08
Loma Prieta 1989/10/18	APEEL 2 - Redwood City	0.27	36	1.06	8.41
Kobe 1995/01/16	Takatori	0.62	41	0.18	9.93
Kobe 1995/01/16	Takarazuka	0.69	41	0.48	3.68
Kobe 1995/01/16	Nishi-Akashi	0.51	41	0.46	9.72
Kobe 1995/01/16	Kakogawa	0.35	41	0.16	12.86

The Seismosignal computer program (2007) is used to estimate the Ground Motion Parameters of Table 3.1. In this table, the effective duration is based on the significant duration concept but both the start and end of the strong shaking phase are identified by absolute criteria. Also, the predominant period (Tp) is the period at which the maximum spectral acceleration occurs in a 5% damped acceleration response spectrum.

4. NONLINEAR TIME HISTORY DYNAMIC ANALYSES RESULTS

Nonlinear time history dynamic analyses (NTHDA) are performed for all models using the SAP 2000 ver 9.1.6, Structural Analyses Program (CSI 1997) and considering the 28 ground motions previously described.
Detail results of all models are displayed in Figures 2(a)-(e) in terms of the PHFA amplification (i.e. relative acceleration = PHFA/PGA) versus normalized height (h_0). The h_0 is taken as the floor height divided by the total height of building. These plots show the actual data of relative acceleration obtained from the NTHDA and their mean value at each floor level. Also in this Figures, the comparison of results of the relative acceleration average distribution for models and NEHRP (2003) are presented.

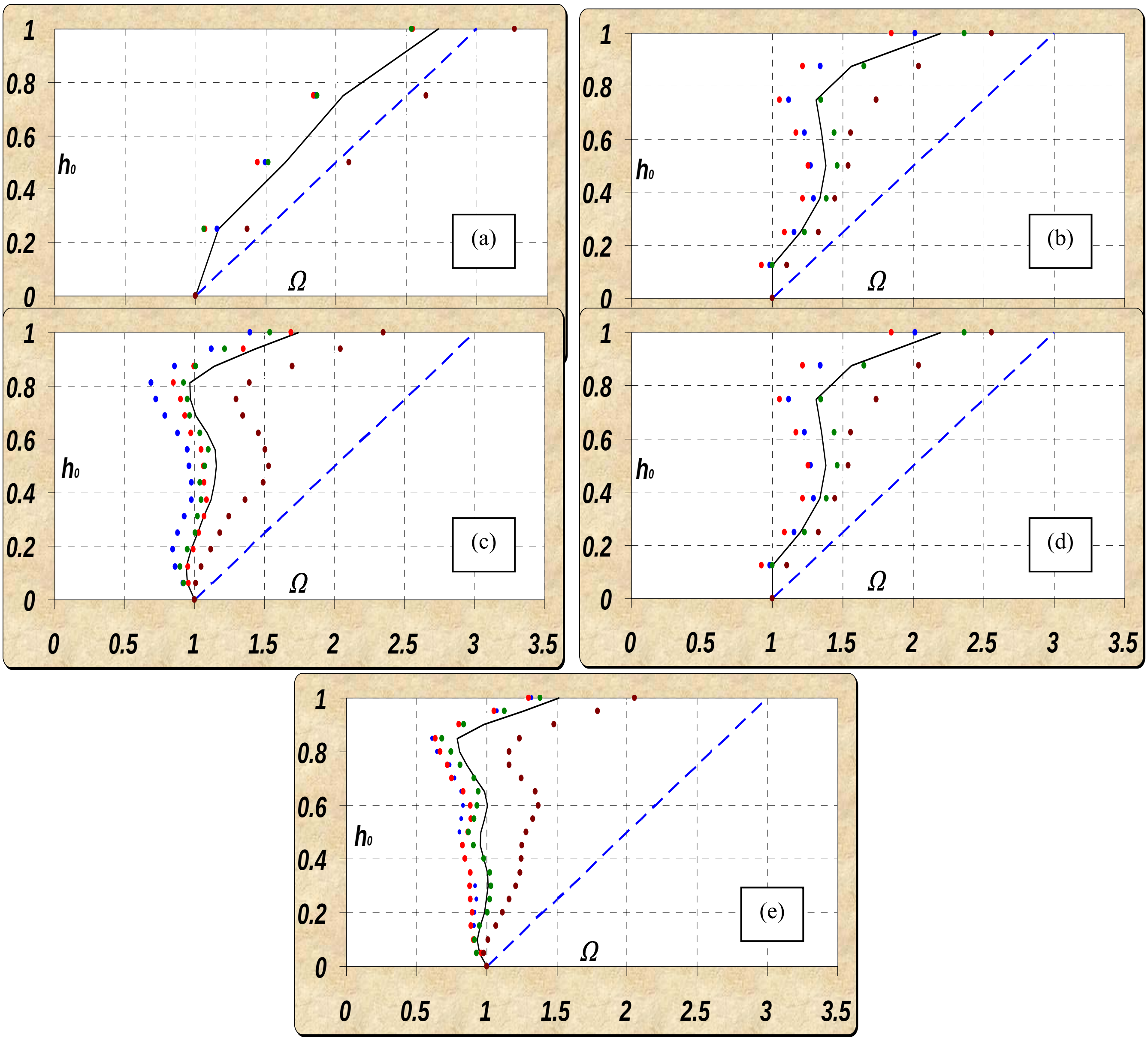


Figure 2(a)-(e). Comparison of NEHRP (2003) and relative acceleration average distribution for all models

According to the Figures 2(a)-(e), the forms of the relative acceleration average distribution are nonlinear (approximately S shape for higher models) and the code recommendation relation to relative acceleration average distribution, are over-conservative. So, to increase the period of model (or to increase the flexibility of model), the amplification of relative acceleration average distribution obtained in this study is decreasing. For example according to the Fig. 2(e), the amplification of relative acceleration average distribution until 90% of the 20 story

model height is approximately constant and equal to one (equal to the PGA). But in all models, the maximum intensity of relative acceleration always occurs at the uppermost story.
These figures illustrate that the data does not follow any significant trend and more importantly, the code provisions do not provide a good estimate of the amplification of relative acceleration and NEHRP provision (2003) is more useful for short and rigid buildings with low fundamental period.

5. PROPOSED DISTRIBUTION

Figures 2(a)-(e) illustrate that the amplification of relative acceleration average distribution along the height for all of models follows a nonlinear shaped curve. This shows that for all frames, lower amplification is observed at the bottom floors and higher amplification is observed at the upper floors. For these buildings, their fundamental mode shape resembles a type of shear-flexural behavior. Shear behavior is observed at the bottom floors and flexural behavior is observed at the upper floors. It is evident in Figures 2(a)-(e) that, performance of relative acceleration average distribution, is depending to the behavior of structures, rigidity and flexibility of buildings and the fundamental period of buildings. However, the resulting analytical form may not be justified in light of the uncertainty associated with the ground motion records (input data), the building characteristics and other modeling assumptions.
In this study, to estimate the seismic vulnerability of rigid acceleration sensitive nonstructural components with a fundamental period less than or equal to 0.06, the distribution of Peak Horizontal Floor Acceleration (PHFA) along height of structures as the behavior of these rigid acceleration sensitive nonstructural components are the same as behavior of floor or roof that they have mounted on it. So response of floor or roof to earthquake excitation is approximately similar to these nonstructural components response and their both acceleration response spectra are approximately the same.
Therefore, the following simplified Equation is proposed to obtain A_i at any floor in the building:

$$A_i = \Omega_i A_0 \tag{5.1}$$

Where A_i is the Peak Horizontal Floor Acceleration (PHFA), A_0 is the Peak Ground Acceleration (PGA) and Ω_i is the factor of floor acceleration amplification. This factor is given by,

$$\Omega_i=1+(\alpha-1)(h_i/h_n) \tag{5.2}$$

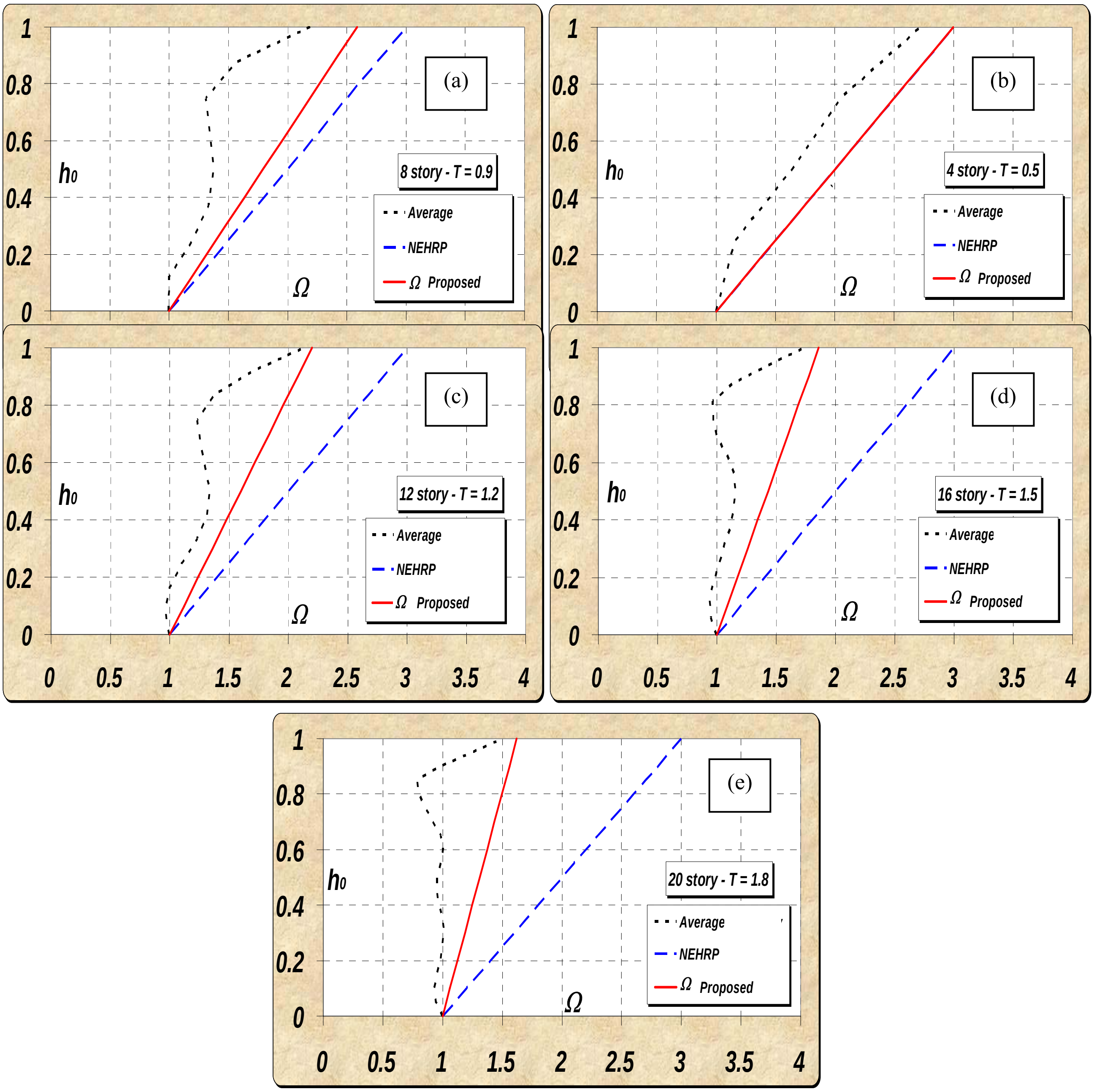
Where h_i is the height of the floor in consideration, and h_n is the height of the uppermost level of the building both measured from the base. And α is period-dependent factor for the building. This factor is given by,

$$T<0.5;\ \alpha=3 \tag{5.3}$$

$$T>1;\ \alpha=2.5/T^{3/4} \tag{5.4}$$

$$0.5<=T<=1;\ \alpha=2.5/T^{1/4} \tag{5.5}$$

Where T is the fundamental period of building.
The final results of investigation are shown on Figures 3(a)-(e). This Figures compares the floor relative acceleration distribution obtained from NEHRP (2003), also from the NTHDA of average of four soil types and finally from the simplified method (Ω_i) for 4, 8, 12, 16 and 20 story models.
According to Fig. 3 (a) and Eq. (3), for fundamental period (T) less than 0.5, the α factor is equal to 3. It means that the floor relative acceleration distribution obtained from proposed simplified method (Ω_i) is equal to NEHRP (2003) proposed distribution, therefore the proposed distribution of NEHRP (2003) is useful. But for fundamental period (T) higher than 0.5, the α factor is lower than 3 and according to Figures 3 (b)-(e), the proposed distribution of NEHRP (2003) is conservative.



Figures 3(a)-(e). Comparison of the floor relative acceleration distribution obtained from NEHRP (2003), from the NTHDA of average of four soil types and from the simplified method (Ω_i) for 4, 8, 12, 16 and 20 story models

6. SUMMARY AND CONCLUSIONS

The paper presents a simplified method for calculating the input acceleration for rigid acceleration sensitive nonstructural components. In this paper, the distribution of the peak horizontal floor acceleration (PHFA) along the height of structural models is investigated using a large number of ground motions recorded on four soil types. Results of NTHDA are compared for five models of different heights and periods.

Finally, a proposed floor acceleration amplification distribution (Ω_i) is presented. According to this investigation, the proposed distribution of NEHRP (2003) is useful for rigid buildings with short fundamental period (less than 0.5 seconds). It is evident that, for semi-rigid and flexible buildings with moderate and high fundamental period (higher than 0.5 seconds), the proposed distribution of NEHRP (2003) is conservative, as the simplified method (Ω_i) is proposing.

Furthermore, The proposed distribution can directly be used to estimate the vulnerability of rigid acceleration-sensitive nonstructural components housed within similar types of buildings and considering the range of fundamental periods from this study.

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REFERENCES

1. American Society of Civil Engineers (ASCE). (2005). Minimum design loads for buildings and other structures, *SEI/ASCE 7-05*, Reston, Va.
2. Ayers, J.M., Sun, T.Y., and Brown, F.R. (1973). Nonstructural damages to buildings. The Great Alaska Earthquake of 1964: Engineering, National Academy of Science, Washington, D.C.
3. Building Seismic Safety Council (BSSC). (2003). NEHRP recommended provisions for the development of seismic regulations for new buildings. 2003 Ed., Parts 1 and 2, FEMA 450, Federal Emergency Management Agency, Washington, D.C.
4. Chaudhuri, S.R., and Hutchinson, T.C. (2004). Distribution of peak horizontal floor acceleration for estimating nonstructural element vulnerability. Proc., 13th World Conference on Earthquake Engineering, Vancouver, B.C., Canada, 1721-1735.
5. Hosseini, M. (2005). On the behavior of nonstructural elements in the Bam, Iran earthquake of December 26, 2003. *Earthquake Spectra* **21:S1**, S 439-S 453.
6. International Code Council (ICC). (2006). International Building Code (IBC), Whittier, Calif.
7. Iranian Building Codes and Standards. (2005). Iranian code of practice for seismic resistant design of buildings. Standard No. 2800-05, 3th Ed., BHRC Publication No. S-253, Building and Housing Research Center, Tehran, Iran.
8. Kehoe, B.E., and Freeman, S.A. (1998). A critique of procedures for calculating seismic design forces for nonstructural elements. Proc., ATC-29-1 Seminar on Seismic Design, Retrofit and Performance of Equipment and Nonstructural Components, Applied Technology Council, San Ferancisco, Calif., 57-70.
9. Miranda, E., and Taghavi, S. (2003). Estimation of seismic demands on acceleration sensitive nonstructural components in critical facilities. Proc., ATC-29-2 Seminar on Seismic Design, Retrofit and Performance of Nonstructural Components, Applied Technology Council, Newport beach, Calif., 347-360.
10. Miranda, E., and Taghavi, S. (2005). Approximate floor acceleration demands in multistory buildings. I: Formulation. *Journal of Structural Engineering* **131:2**, 203-211.
11. Reinoso, E., and Miranda, E. (2005). Estimation of floor acceleration demands in high-rise buildings during earthquakes. *The Structural Design of Tall and Special Buildings* **14**, 107-130.
12. Reitherman, R., and Sabol, T.A. (1995). Northridge earthquake of January 17, 1994: Reconnaissance report nonstructural damage. *Earthquake Spectra* **11**, 453-514.
13. Searer, G.R., and Freeman, S.A. (2002). Unintended consequences of code modification. Proc., 7th U.S. National Conf. on Earthquake Engineering (CD-ROM), Earthquake Engineering Research Institute, EERI, Boston, Ma.
14. SeismoSignal, ver 3.2.0. (2007). On-line documentation at: http://www.seismosoft.com.
15. Singh, M.P., Moreschi, L.M., Suarez, L.E., and Matheu, E.E. (2005). Seismic design forces. I: Rigid nonstructural components. *Journal of Structural Engineering* **132:10**, 1524-1532.
16. Singh, M.P., Suarez, L.E., Matheu, E.E., and Maldonado, G.O. (1993). Simplified procedure for seismic design of nonstructural components and assessment of current code provisions. Technical Rep. No. NCEER-93-0013, National Center for Earthquake Engineering Research, State Univ. of New York, Buffalo, N.Y.
17. Structural Analyses Program (SAP). (2006). Linear and Nonlinear Static and Dynamic Analyses and Design of Three-Dimensional Structures, Advanced ver 9.1.6. Computers and Structures, Inc., CSI, Berkeley, Calif.