16TH EUROPEAN CONFERENCE ON EARTHQUAKE ENGINEERING THESSALONIKI 18 - 21 JUNE 2018

ASSESSMENT OF SEISMIC ROOFTOP ACCELERATION DEMANDS IN HIGH-RISE BUILDINGS

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ABSTRACT

The evaluation of seismic performance of acceleration-sensitive nonstructural components in buildings requires reliable prediction of the floor acceleration distribution along the height of the buildings. An accurate estimation of seismic acceleration demands is required in order to evaluate the potential seismic induced damage of anchorage, supports and bracing of these components and the corresponding probability of resulting financial losses. High-rise buildings located in seismic prone regions are particularly sensitive to amplified acceleration demands generated by long period ground motions and due to the significant contribution of higher modes to their response. Existing equations in the literature were mainly developed based on recorded accelerations in low to medium-rise instrumented buildings located in California. This paper examines the amplification of seismic rooftop accelerations of high-rise buildings with emphasis on their response to long period ground motions. Peak horizontal rooftop acceleration amplifications were assessed using recorded accelerations in 18 instrumented buildings having between 19 to 43 stories and subjected to the 2011 Tohuku earthquake in Japan. The average recorded peak rooftop acceleration amplification factors (AAF) defined as the average ratio of the peak floor acceleration (PFA) divided by the peak ground acceleration (PGA) range from 2.0 to 6.5 and exceed the values suggested in the NBCC 2015, ASCE 7-10, Eurocode and an equation suggested in the literature. This study showed the significance of considering the seismo-tectonic origin including the magnitude, distance and the level of input PGA for reliable seismic evaluation of acceleration-sensitive nonstructural components in buildings.

Keywords: High-rise buildings; seismic floor acceleration; height acceleration amplification factor, long period ground motion, nonstructural components.

1. INTRODUCTION

A building is made up of components that can be divided into two groups: structural components and nonstructural components (NSCs) known in Canada as operational and functional components (OFCs) (CSA 2014). A large number of NSCs are classified as acceleration-sensitive; i.e. accelerations govern their response. Examples of dominantly acceleration-sensitive components are the mechanical and electrical systems for fire protection, heating-ventilation-air conditioning (HVAC) and water distribution. Estimating the peak floor acceleration (PFA) is only the first step in calculating the seismic demand of acceleration-sensitive components in order to: 1) assess whether the targeted performance level of NSCs could be achieved, 2) estimate the vulnerability of NSCs through the fragility curves (probability of reaching a degree of damage for a given seismic demand), 3) reduce the risk of failure of NSCs by providing adequate restraints.

NSCs account for 75-85% of the original construction cost in buildings (Taghavi and Miranda 2005) and costs associated with their seismic damages could be as high as 92% of the total repair costs (e.g. for a hospital)

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(Taghavi 2003). Therefore, there is a need to understand and quantify horizontal and vertical PFA demands, and propose simplified yet accurate code design-oriented equations. Estimating the horizontal PFA response profile along the building height is still controversial since the profile proposed in most building codes (ASCE 2010; NBCC 2015) was developed empirically based on floor acceleration data recorded in low and medium-rise buildings located in California (ATC 2012). The horizontal floor accelerations and their amplification at the rooftop as proposed in the current editions of National building code of Canada (NBCC), the ASCE (ASCE 2010) and Eurocode-8 (CEN 2004) are presented in Table 1. According to these codes, the peak floor acceleration is obtained by multiplying the design input acceleration at ground level, $0.3F_aS_a(0.2)$, $0.4S_{DS}$ and a_gS/g, respectively, by the height amplification factor that is function of the ratio of the position above the base of the building where the component is located (z and h_x) and the total height of the building (h_n); F_a is an acceleration-based site coefficient; S_{DS} and $S_a(0.2)$ are the spectral accelerations at short periods and 0.2s ; S is the soil factor; T_a and T_1 are the fundamental periods of the nonstructural component and of the building, respectively. For comparison purposes, the ratio of T_a/T_1 will be assumed equal to zero. Therefore, the profile of acceleration demand along the height is assumed linear, regardless of the number of stories, the type of lateral resisting system, the effects of torsion, the influence of higher modes, the nonlinear behavior of the buildings, and the ground motion characteristics (Soong et al. 1993; Searer and Freeman 2002; Assi et al. 2017).

Table 1. Horizontal floor accelerations and rooftop height acceleration amplification factors.

Code	**Design horizontal peak floor acceleration**	**Horizontal height acceleration amplification factors *(Rooftop)***
NBCC	$0.3F_aS_a(0.2)(1+2\frac{h_x}{h_n})$	3
ASCE	$0.4S_{DS}(1+2\frac{h_x}{h_n})$	3
Eurocode	$\frac{a_g}{g}S\left\{3\left(1+z/h_n\right)\Big/\left(1+\left(1-T_a/T_1\right)^2\right)-0.5\right\}$	2.5 for $T_a/T_1=0$

To overcome some of the previously mentioned shortcomings, the FEMA-P58 (ATC 2012) proposed a predictive equation of the horizontal acceleration demand in buildings considering the natural period, the lateral load resisting system and the effects of nonlinear response of the building. However, this equation is only valid for low and medium-rise buildings with height less than or equal 15 stories Huang and Whittaker 2012; Huang et al. 2011); therefore, it will not be considered in this study.

Recently, Fathali and Lizundia (2011) addressed the horizontal acceleration amplification profile from a large database of recorded accelerations in 169 instrumented buildings in California including low, medium and high-rise buildings. Their database includes 16 high-rise buildings with more than 15 stories. The researchers addressed some of the aforementioned drawbacks of the current code equations by proposing an equation for the prediction of the height acceleration amplification factor (AAF) in high-rise buildings considering the fundamental period of the building and the severity of the earthquake through the peak ground acceleration PGA (Equation 1). The AAF is defined as the ratio of the PFA divided by PGA. The regression parameters (α and β) were developed as a function of the (PGA) and the range of the buildings fundamental periods. For the evaluation of the AAF at the rooftop (i.e. z/H =1, where H is the total height of the building), the equation simply becomes a function of the parameter α given in Table 2.

$$AAF = 1 + \alpha\left(\frac{z}{H}\right)^{\beta} \quad (1)$$

Table 2. Recommended values of parameter α of Equation 1 for seismic evaluation of acceleration-sensitive nonstructural components in existing buildings (Fathali and Lizundia 2011).

Period	**PGA < 0.067g**	**0.067g < PGA < 0.2g**	**PGA > 0.2g**
T < 0.5s	1.26	1.04	0.99
0.5s < T < 1.5s	1.52	1.02	0.65
T > 1.5s	0.9	0.72	0.00

Unlike the linear profiles proposed in the aforementioned codes, the proposed profile is nonlinear and results in lower amplification with the increase in building period and lower PGA. Therefore, it is becomes important to validate the applicability of this equation to other regions with different geological and seismo-tectonic settings. This paper presents an evaluation the AAF at the rooftop of instrumented high-rise buildings with fundamental periods exceeding 1.3s. To this end, a database of 18 instrumented buildings subjected to long period ground motion from the 2011 Tohoku earthquake in Japan (Nango and Hida 2012) are studied. This earthquake, known as the Great East Japan Earthquake, caused a large tsunami and enormous damage to eastern Japan (Okawa et al. 2012). The rooftop AAF was computed and compared to AAF as proposed in NBCC, ASCE 7-10, Eurocode 8 and the equation proposed by Fathali and Lizundia (2011) (see Equation 1 and Table 2).

2. SEISMIC RESPONSE OF HIGH-RISE BUILDINGS

2.1 Description of the instrumented buildings used in this study

This section presents the details of a database of 18 studied buildings having recorded seismic accelerations from the 2011 Great East Japan Earthquake which epicenter is presented in Figure 1. The studied buildings are located in two regions of Japan: 13 buildings at Kanto region (which includes the city of Tokyo) located approximately 300 km away for the epicenter and 5 buildings at Kansai region located approximately 700 km away from the epicenter. These buildings are built on thick sedimentary layers that are sensitive to long period ground motions (Nango and Hida 2012). The buildings height varied from 19 to 43 stories and the corresponding fundamental periods varied from 1.32 s to 3.02 s as listed in Table 2. The database consists of reinforced concrete buildings (RC) and steel buildings (S) with the exception of one 25-story building constructed with steel-reinforced concrete frames (SRC). The RC buildings height range is 24 to 37 stories and the S buildings height range is 19 to 29 stories. Table 2 shows the location, ID, structural types, number of stories, the fundamental periods of the buildings based on the information provided in the literature (Nango and Hida 2012; Saito et al. 2012; Pu et al. 2012, Nagano et al. 2016). The buildings were subjected to peak ground accelerations that range from 0.005 g to 0.153g.

Figure 1. Map of Japan regions and the epicenter of the 2011 Tohoku earthquake (photo source: https://en.wikipedia.org)

Table 3. Basic information on the studied buildings.

Building ID and location	**Number of Stories**	**Structural System**	**Fundamental Period (sec)**
A-Kanto	30	RC	1.92
B-Kanto	25	RC	1.39
C-Kanto	30	RC	1.67
D-Kanto	24	RC	1.32
E-Kanto	32	RC	2.27
F-Kanto	33	RC	1.96
G-Kanto	28	RC	1.79
H-Kanto	24	RC	1.33
I-Kansai	33	RC	1.75
J-Kansai	36	RC	2.00
K-Kansai	25	SRC	1.54
L-Kansai	43	RC	2.63
N-Kansai	31	RC	1.92
1-Tokyo	29	S	3.03
2-Tokyo	20	S	1.92
3-Tokyo	19	S	1.89
BRI-Tokyo-C	20	S	1.6
BRI-Tokyo-E	37	RC	1.8

2.2 Recorded rooftop acceleration amplification factor at building sites

As mentioned previously in section 2.1, the acceleration data and basic information related to the studied buildings and the earthquake were derived from literature. The PGA range from 0.005 g to 0.153g as shown in Table 4 and the recorded AAF at the rooftop of each building was computed as the ratio of the average recorded peak rooftop accelerations (PRA) in two orthogonal directions divided by the average PGA recorded at the first floor or the basement level.

Table 4. List of high-rise buildings in Japan and their recorded accelerations from the 2011 Tohoku earthquake

Building ID and location	**Number of Stories**	**Structural System**	**PRA (cm/s^2)**	**PGA (cm/s^2)**	**Recorded AAF**
A-Kanto	30	RC	307	90	3.4
B-Kanto	25	RC	353	70	5.0
C-Kanto	30	RC	343	85	4.0
D-Kanto	24	RC	301	70	4.3
E-Kanto	32	RC	327	110	3.0
F-Kanto	33	RC	294	60	4.9
G-Kanto	28	RC	367	90	4.1
H-Kanto	24	RC	463	150	3.1
I-Kansai	33	RC	25	5	5.0
J-Kansai	36	RC	39	6	6.5
K-Kansai	25	SRC	26	6	4.3
L-Kansai	43	RC	26	5	5.2
N-Kansai	31	RC	72	11	6.5
1-Tokyo	29	S	275.5	90	3.1
2-Tokyo	20	S	180	88	2.0
3-Tokyo	19	S	337.5	67.5	5.0
BRI-Tokyo-C	20	S	178	88	2.0
BRI-Tokyo-E	37	RC	180	92.5	1.9

The average PGA for Kanto and Kansai region buildings are 0.1g (approximately 300 km away for the epicenter) and 0.007g (approximately 700km away from the epicenter), respectively. The level of the PGA at Kansai region is much smaller due to the attenuation of the seismic waves with larger distance. On the other hand, the recorded AAF in the Kanto region varied from 1.9 to 5.0 while the AAF in Kansai region varied from 4.3 to 6.5. Despite these large distances from the earthquake epicenter, the AAF was significant for the high-rise buildings due to the dominant long-period characteristics of the ground motion generated from this earthquake (Saito et al. 2012). Due to this amplification, damage to non-structural members such as falling of gypsum board and ceiling panels were observed in some of these buildings (Saito 2012).

3. COMPARISON OF RECORDED AAF WITH PREDICTIVE EQUATION

As noted in the introduction section, current North American and European Codes (NBCC-2015, ASCE 7-10, CEN 2008) predict a constant rooftop acceleration amplification factor (AAF = 3, 2.5) for all buildings. Therefore, the recorded AAF of the instrumented buildings subjected to the 2011 earthquake in Japan are compared with the corresponding prediction using the equation proposed by Fathali and Lizundia (2011), referred to (FL-2011) and the codes provisions (Table 1) as presented in Table 5.

Table 5. Comparison of the recorded and predicted AAF for the instrumented buildings.

Building ID	Number of stories	Recorded AAF	Period (sec)	PGA (g)	α	Predicted AAF (FL-2011)	(Predicted/ Recorded) AAF
A-Kanto	30	3.4	1.92	0.092	0.72	1.72	0.50
B-Kanto	25	5.0	1.39	0.071	1.02	2.02	0.40
C-Kanto	30	4.0	1.67	0.087	0.72	1.72	0.43
D-Kanto	24	4.3	1.32	0.071	1.02	2.02	0.47
E-Kanto	32	3.0	2.27	0.112	0.72	1.72	0.58
F-Kanto	33	4.9	1.96	0.061	0.72	1.72	0.35
G-Kanto	28	4.1	1.79	0.092	0.72	1.72	0.42
H-Kanto	24	3.1	1.33	0.153	1.02	2.02	0.65
I-Kansai	33	5.0	1.75	0.005	0.90	1.9	0.38
J-Kansai	36	6.5	2.00	0.006	0.90	1.9	0.29
K-Kansai	25	4.3	1.54	0.006	0.90	1.9	0.44
L-Kansai	43	5.2	2.63	0.005	0.90	1.9	0.37
N-Kansai	31	6.5	1.92	0.011	0.90	1.9	0.29
1-Tokyo	29	3.1	3.03	0.092	0.72	1.72	0.56
2-Tokyo	20	2.0	1.92	0.090	0.72	1.72	0.84
3-Tokyo	19	5.0	1.89	0.069	0.72	1.72	0.34
BRI-Tokyo-C	20	2.0	1.60	0.090	0.72	1.72	0.85
BRI-Tokyo-E	37	1.9	1.80	0.094	0.72	1.72	0.88
Mean±St.Dev.		**4.1±1.4**				**1.82±0.1**	**0.50±0.19**
NBCC and ASCE				**3**			
Eurocode				**2.5**			

It can be observed that the AAF predicted by FL-2011 provides estimates that are within the limits of the North American and European codes with an average value of 1.82 (less than 3.0 and 2.5). However, FL-2011 equation underestimates the rooftop AAF observed from the instrumented buildings by 50% on average. Since the FL-2011 equation is based on the assumption that the two main factors that affect the AAF are the level of PGA and the fundamental period of the buildings, the influence of these two parameters on the AAF in light of the observed data will be presented and discussed in the following sections.

3.1. Influence of PGA on the AAF

In this section, the influence of the level of the input PGA on the amplitude of the AAF is presented and compared with the predictions provided by the FL-2011 equation. Figure 2 shows the relationship between the input PGA for each instrumented building and the corresponding AAF. A regression equation was fitted to the data and the coefficient of determination (R^2) is computed in order to quantify how well the regression line fits the real data points.

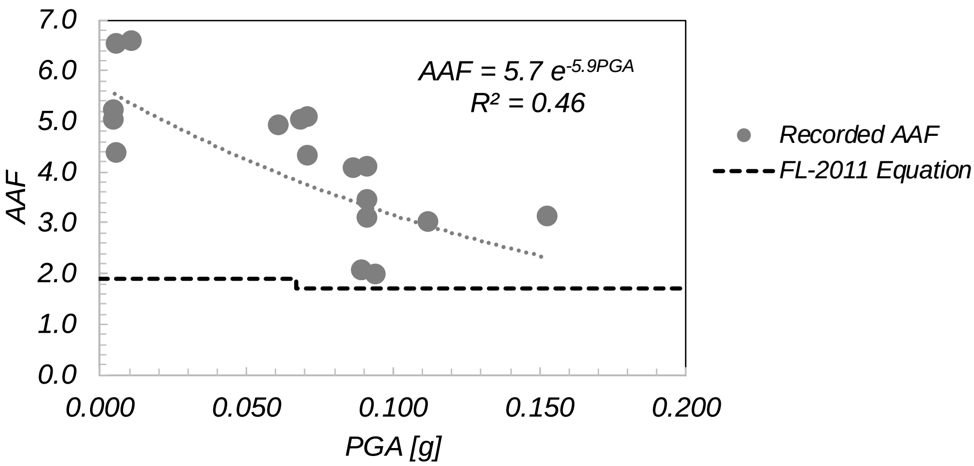

Figure 2. The relationship between the AAF and the PGA for the instrumented buildings

The following observations can be made from Figure 2:

- Despite an R^2 value equal to 0.46, the regression equation clearly shows a close correlation between the AAF and the input PGA where the AAF decreases with an increase in the PGA in buildings subjected to long period ground motion. This confirms the dependency of the AAF on the PGA as proposed in FL-2011 equation (see Table 1).

- The comparison of the AAF of buildings located in Kanto and Kansai regions shows that a higher PGA level (Kanto region) induced lower AAF compared to the Kansai region. This can be attributed to the increased structural damping for buildings in Kanto where they experienced concrete cracking in the structural elements (Nango and Hida 2012). On the other hand, buildings in Kansai were subjected to low-intensity ground shaking with low damping response that results in higher AAF.

- The discrepancy between the predicted and observed amplification is significant. This could be attributed to the fact that the FL-2011 equation was developed based on ground motion records from California and observations are not necessarily compatible with other regions seismicity.

- In contrast to the FL-2011 prediction equation that assumes constant discrete values of the parameter α for a range of PGA (see Table 2 and Figure 2), the recorded AAF shows continuous variation with the input PGA as illustrated with the regression equation in Figure 2.

The above observations confirm the importance of considering the frequency content of the ground motion and the difference in geological conditions for providing reliable estimate of the seismic floor acceleration demands in high-rise buildings.

3.1. Influence of the fundamental building period on the AAF

In this section, the influence of the fundamental building period on the AAF is investigated. Figure 3 shows the plot of the recorded average AAF for the instrumented buildings as a function of their fundamental period and the corresponding prediction using the FL-2011 equation.

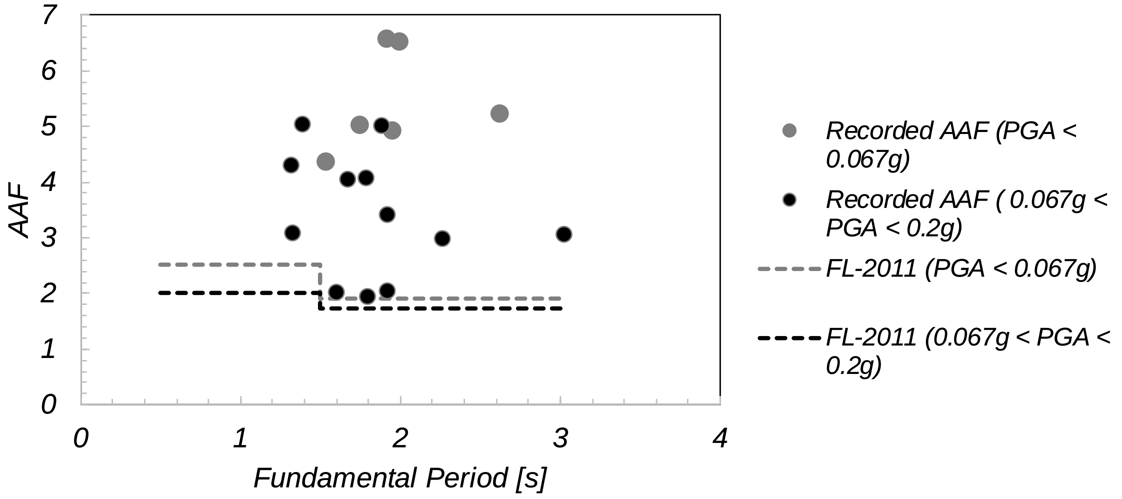

Figure 3. The relationship between the AAF and the fundamental period for the instrumented buildings

The recorded AAF does not show any apparent dependency on the fundamental period of the studied high-rise building. Most of the amplification is noted in the period range of 1.0s to 3.0s; with maximum values close to 7.0 at periods close to 2.0s. This observation does not comply with FL-2011 equation that yields a maximum amplification of 2.0 for periods larger than 1.5s. This high amount of rooftop AAF at the long building period range could be explained by the long-period component of the input ground motion (Saito et al. 2012). Unlike the FL-2011 prediction equation that assumed that the AAF is constant for buildings with fundamental period greater than 1.5s, Figure 3 shows that this assumption is true for only three cases of the studied buildings.

4. CONCLUSIONS

The evaluation of seismic performance of acceleration-sensitive nonstructural components in buildings requires reliable prediction of the floor acceleration distribution along the height of the buildings. High-rise buildings located in seismic prone regions are particularly sensitive to amplified acceleration demands when subjected to long period ground motions and due to the significant contribution of higher modes to their response. This paper examined the amplification of recorded seismic rooftop accelerations of high-rise buildings with emphasis on their response to long period ground motions in comparison to current code recommendations and an equation proposed in the literature. Peak horizontal rooftop acceleration amplifications were assessed using recorded accelerations in 18 instrumented buildings subjected to the 2011 Tohuku earthquake in Japan. The buildings height ranges from 19 to 43 stories. The average peak rooftop height acceleration amplification factors (AAF) defined as the average ratio of the peak rooftop acceleration (PRA) divided by the peak ground acceleration (PGA) was computed for each building. The main conclusions can be summarized as follows:

1) The computed AAF at the rooftop level varied from 2.0 to 6.5, which an average of 4.1±1.4. This implies that current codes provisions underestimate the acceleration amplification in most of the studied high-rise buildings. More studies are needed to further investigate the effect of different parameters such as the frequency content of ground motion, the effect of higher modes and the non-linearity of the structure on the amount of horizontal acceleration amplification in high-rise buildings.
2) The study showed the influence of the input PGA on the amplitude of the AAF. More rooftop acceleration amplification was noticed with smaller input ground acceleration. Therefore, lower values of AAF associated with higher PGA can be attributed to structural damping and observed nonlinearity from cracking in the structural members.
3) On the other hand, there was no apparent relationship between the AAF and the building fundamental period (in the range of 1.0s to 3.0s) based on recorded accelerations.
4) The recorded AAF for the Japanese buildings exceed the values predicted based on FL-2011 equation. Such discrepancy showed the importance of considering difference in geological conditions for reliable assessment of the acceleration demand on high-rise buildings. Therefore, it is suggested to include the effect of the earthquake magnitude, distance and the level of input PGA on the acceleration demand in further studies.

5. ACKNOWLEDGMENTS

The financial support provided by ETS and Natural Sciences and Engineering Research Council of Canada (NSERC) for this project is gratefully acknowledged.

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