Comparative Study of Seismic Acceleration Amplification Models for RC Frame Structures

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Abstract. As the current aspect, the nonstructural components (NSCs) linked with the structures are more affected during the seismic motion. It causes not only loss of the economy but also affected life. The various codal provision has been available for minimizing the damages of primary components, but for NSCs, a minimal requirement is functional. So that more investigation is required for understanding the behavior of NSCs during the seismic motion. The research aims to understand the behavior of acceleration demand on NSCs in a building. Structures subjected to inertia forces due to earthquakes experience damage of nonstructural components (NSC). The inertia force acting the NSCs are related to acceleration amplification factor. For obtaining the peak horizontal floor acceleration with respect to tectonic ground motion, these factors are used. In this paper, mathematical models of the acceleration amplification factor defined as the peak floor acceleration with respect to peak ground acceleration, given by previous researchers, has been compared. For this 2, 4, 6, 8 and 10 storey moment-resisting frame models considering 29 ground motion data ranging between 0.1g to 0.2g, is analyzed using linear time history method. The supports of the models are considered fixed. The ETABS software is used for the analysis of the models. To analyses the models, the modal mass participation ratio plays a significant role. ASCE 7-05 defines that the structure should be investigated and designed when the model mass participation ratio is equal to or more than 90 per cent. Based on the results, a comparison of the reported models is made. There is a strong need for further research to refine the models for the realistic prediction of acceleration amplification factor.

Keyword- Time history data; Nonstructural component; Peak horizontal floor acceleration; acceleration amplification factor.

1. Introduction

Under earthquake motion, the behavior of the structures is changed which cause damages. For minimizing the effect of earthquake on structures, researches have been carried out theoretically and
experimentally world-wide. Earthquakes like Valdivia (Chile) 1960, Alaska (USA) 1964, Bhuj (India) 2001, Sumatra (Indonesia) 2005, Maule (Chile) 2010 and Kathmandu (Nepal) 2015 have caused significant damages to the structures. During seismic ground motion, damage occurs in main components as well as non-structure components lying inside the building [1,2]. The losses of NSCs are more during the earthquake in important buildings such as hospitals, schools, government offices etc. Sometime this loss may exceed the cost of construction [3].

Nonstructural components are those components, connected with the floor, roof and the wall of the structure and the main load does not act on it [4]. Sometimes it is also called secondary components. Non-structure components are categorized on the basis of storey drift conscious and acceleration conscious [5][6]. The partition wall, windows, infill wall etc. are the example of non-structures components based on storey drift conscious and ducts, boilers, parapets etc., are the example of non-structures components based on acceleration conscious [7]. The maximum cladding panel damage occurred in Emilia Earthquake 2012 [8].

The damages of NSCs depend upon the inertia force on the structure during seismic ground motion. The inertia force on the structures depends upon the floor acceleration. A large number of experimental [9] and analytical [10] studies are reported to study the behavior of the components during seismic motion.

ASCE7-10 code [11], proposed an empirical equation to determine the inertia forces, acting on the nonstructural component of the structures. Drake and Bachman [12] proposed a mathematical model to determine the peak floor acceleration using 16 different time history data and applied it on large number of the buildings.

Crescenzo Petrone et al. [14] studied six RC moment resisting models as one, two, three, five and ten stories designed by Eurocode 8. To study the performance of the models, 7 different ground motion data was considered and analyzed by linear as well as non-linear time history method. It was found that the formula given by Eurocode 8 for designing the non-structures component was not accurate as found by actual data.

In another approach, Akhlaghi and Moghadam [15], represented the 2D steel structure models with different storey and obtained the behavior of acceleration amplification factor with the help of non-linear time history analysis. The obtained results were compared with IBC 2006 code and NEHRP 2003 code and observed that some modification was required when the structure height increased.

In this paper, five different RC frame models of different storey height are analyzed using linear time history method. The obtained acceleration amplification factors have been compared with previous repeated models.

2. Building Models

In this paper, two, four, six, eight and ten stories RC moment resisting frame models are considered. Each model has fixed base. The first-floor height is 4 m and above stories are 3.4 m high. Plans of these models are given in Figure 1 and the size of column and beams are given in Table 1. These models were analyzed by linear time history method considered damping ratio as 5%. 
Figure 1. Moment resisting frame models (a) 2 (b) 4 (c) 6 (d) 8 and (e) 10 stories
Table 1: Size of beams and columns

| Beam | Size in mm |
|------|------------|
| B1   | 300x400    |
| B2   | 300x450    |
| B3   | 450x500    |
| B4   | 450x600    |
| B5   | 450x650    |
| B6   | 450x675    |

Column

| Column | Size in mm |
|--------|------------|
| C₀     | 300x400    |
| C1     | 300x450    |
| C2     | 450x500    |
| C3     | 525x550    |
| C4     | 550x600    |
| C5     | 600x700    |
| C6     | 650x850    |

3. Considered Ground Motions
In this study, 29-time history recorded data ranging from 0.1g to 0.2g are considered. These ground motion data were taken from the website of Strong ground motion site [16]. The list of the different recorded time history data is given in Table 2.

Table 2: Time History Data for Peak Ground Acceleration between 0.1g to 0.2g

| Earthquake Station | PGA (g) | T (sec) | Tₚ (sec) |
|--------------------|---------|---------|----------|
| Chi-Chi-I          | 0.135   | 150     | 40.89    |
| Chi-Chi-II         | 0.142   | 146     | 45.58    |
| Chi-Chi-III        | 0.113   | 144     | 49.57    |
| Chi-Chi-IV         | 0.150   | 150     | 48.52    |
| Chi-Chi-V          | 0.193   | 150     | 34       |
| Location     | $T$  | $T_p$ | $A_p$ |
|--------------|------|-------|-------|
| Chi-Chi-VI   | 0.124| 150   | 34.77 |
| Chi-Chi-VII  | 0.162| 150   | 55.02 |
| Chi-Chi-VIII | 0.146| 120   | 36.82 |
| Chi-Chi-IX   | 0.160| 150   | 38.21 |
| Chi-Chi-X    | 0.113| 144   | 49.57 |
| Chi-Chi-XI   | 0.136| 150   | 49.54 |
| Coalinga-I   | 0.133| 60    | 11.56 |
| Coalinga-II  | 0.192| 65    | 6.94  |
| Coalinga-III | 0.131| 21.40 | 2.08  |
| Coalinga-IV  | 0.124| 65    | 6.96  |
| Coalinga-V   | 0.164| 21.02 | 4.62  |
| Costa Rica-I | 0.105| 72    | 11.84 |
| Costa Rica-II| 0.114| 80    | 14.8  |
| Mammoth Lake-I| 0.121| 44.66 | 10.14 |
| Mammoth Lake-II| 0.196| 65    | 10.70 |
| Mammoth Lake-III| 0.163| 65    | 3.14  |
| Mammoth Lake-IV| 0.155| 67.78 | 2.44  |
| Northridge-I | 0.183| 60.01 | 15.34 |
| Northridge-II| 0.106| 60    | 11.38 |
| Northridge-III| 0.120| 65.02 | 7.44  |
| Bhuj         | 0.106| 133.53| 46.94 |
| Camarillo    | 0.124| 65    | 10.52 |
| Elizabeth Lake| 0.114| 60.01 | 10.87 |
| Pomona       | 0.160| 79    | 28.08 |

In Table 2, $T$ represents the total Recorded time period and $T_p$ represents the time of peak acceleration which is shown in Figure 2 for Bhuj Earthquake.
The floor spectral acceleration is used when the weight of the building is lower than the weight of the main structural components. However, when weight of the main structural components is higher than peak floor acceleration demand is used. Figure 3, presented the mean floor acceleration of each floor with the Peak ground acceleration is 0.2g to 0.31g. It observed that, as the natural period of the building increases the amplification value is decreases.

5. Seismic Analysis and Evaluation of Models

ASCE/SEI 7-05 [17] section 13.3.1, defines the lateral seismic force on the nonstructural components:

\[ F_p = 0.4S_{ds} a_p (1+2(z/h)^{0.05}) * W_p \]  

(ASCE/SEI 7-05 Equation 13.3-1)

\[ F_p \leq 1.6 * S_{ds} * I_p * W_p \]  

(ASCE/SEI 7-05 Equation 13.3-2)

\[ F_p \geq 0.3 * S_{ds} * I_p * W_p \]  

(ASCE/SEI 7-05 Equation 13.3-3)

Where \( F_p \) represent the lateral seismic design force, \( S_{ds} \) is the site-specific short period spectral acceleration, \( a_p \) denote the component amplification factor having ranges from 1.0 to 2.5, \( z \) is the height of the structure with respect to base, \( h \) is the total height of the building with respect to base, \( I_p \) refer the component important factor, \( R_p \) is the component response modification factor which represents the energy absorbed by the component and \( W_p \) is the total weight of the component.

For comparisons of building models in terms of the acceleration amplification factor, two mathematical models given by Drake and Bachman [12] and Joseph Wiser [18] are considered. These mathematical models are explained below.
Figure 3. Mean floor spectral acceleration in the ground motion range 0.2g to 0.3g (a) 2 (b) 4 (c) 6 (d) 8 and (e) 10 stories. Drake and Bachman [12] gave an empirical equation for finding out the floor acceleration amplification factor from field observation which is given as

\[ \Omega = \frac{PFA}{PGA} = (1+2z) \]

Joseph Wiser et al [18] proposed another equation for determining the floor acceleration amplification factor based on the period of the structures -
In Equation (2), \( T \) is the period of the supporting structure and \( T_{\text{max}} \) is maximum period of the structure. When the roof acceleration is more than or equal to Peak Ground Acceleration (PGA), and it should be taken as 2.5 sec.

\[
\Omega = \frac{PFA}{PGA} = \left(1 + \frac{m_{\text{max}}}{\pi^2} \right) \tag{2}
\]

Figure 4. Comparison between PFA/PGA with respect to normalize height for ground motion range 0.1g to 0.2g (a) 2 (b) 4 (c) 6 (d) 8 and (e) 10 stories.

Using ETABs software [19], all building frame models are analyzed considering a linear time history method. For analysis of these models, large seismic ground motion data, given in Table 2 are considered. The actual acceleration amplification factor and its comparison with different mathematical acceleration amplification factor models [12, 18] are shown in Figure 3 (a) to (e). All these figures represent the nature of the building with regard to Peak acceleration amplification factor (\( \Omega \)) with respect to normalized height. The normalized height is described as the ratio between floor height and total height of structure with respect to the base.
It is found that when the ground motion ranges between 0.1g to 0.2g, the nature of the structure in terms of acceleration amplification factor and normalized height are nonlinear (S shape). Considering Drake [12] and Wiser [18] models, large numbers of acceleration amplification data exceeded the actual acceleration amplification values. The numbers of exceeded data are given in Table 3 and shown in Figure 4 respectively.

### Table 3: Actual and exceeded Ω data for different mathematical models

| Stories Models | Actual Data Ω | Exceed Ω Data |
|----------------|---------------|---------------|
|                | Drake Model   | Wiser Model   |
| 2 Storey       | 276           | 43            | 5             |
| 4 Storey       | 455           | 67            | 34            |
| 6 Storey       | 637           | 86            | 132           |
| 8 Storey       | 819           | 64            | 129           |
| 10 Storey      | 1001          | 60            | 274           |

![Figure 5. Number of exceeded Ω data for different models](image-url)
It can be observed that Wiser’s model is suitable for low rise building but give large errors when the height of the building increases. Drake model gives high exceedance but it almost constant. As actual variation is non-linear, there is strong need to develop realistic models for more accurate prediction of Ω.

6. Conclusions

It this paper, different RC frame models are analyzed using linear time history method. Large numbers of ground motion data having range between 0.1g to 0.2g are used. The results of acceleration amplification factor are compared with Drake and Wiser models. The salient conclusions are summarized below.

- The shape of acceleration amplification factor with respect to normalized height is nonlinearsuch as S shape.
- Drake model does not consider the natural time period of the structure. It gives constant errorfor all the storey buildings.
- Wiser’s model considers the natural time period of the structure but give more error as storeyheight increases.
- For realistic prediction of acceleration amplification factor, there is strong need of furtherresearch to refine the models.

Acknowledgements

For this study, all ground motion data found from strong ground motion sitehttps://strongmotioncenter.org/vdc/scripts/default.plx.

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