Nonlinear Amplification Model in RC Frame Structures: Case Study for Chi-Chi Earthquake

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Abstract For the seismic design of non-structural components (NSCs), more awareness is required not only for prevention of catastrophic impact of human life but also for saving the economical crises. Due to the seismic ground motion, an inertia force acts upon the primary structure as well as non-structural components. The inertia force which affects the NSCs is related to the acceleration amplification factor. Various researchers presented the amplification factor as a function of the normalized height of the structures, but it is found that the amplification factor depends on other factors as well. In this paper, non-linear acceleration amplification models are proposed, which not only depends on the normalized height of the structure but also on the range of ground acceleration and the natural period of the structures. For this, five moments resisting RC frame models of 2,4,6,8 and 10 storeys are considered. A large number of ground motion data, having the range of less than 0.067g, twenty-nine ground motion data in the range of 0.067g to 0.2g and twenty-four ground motion data in the range of 0.2g to 0.3g respectively are accounted. Linear time history method is used for the analysis of all models. Based on the results, the non-linear amplification model is proposed and compared with the previously reported models.

Keywords Time History Analysis, Non-Structural Component, Floor Acceleration, Acceleration Amplification Factor

1. Introduction

Taiwan, situated on the western circum Pacific seismic belt region, is regularly affected by the seismic hazard. As reported by the central weather bureau (CWB), every year nearly 18500 earthquake events occur in Taiwan [1]. In 1999, Chi-chi earthquake caused more than 9000 buildings collapsed. Major losses were occurred not only in terms of social or economic but also affected the life of the people [2]. In terms of damages of structures, some buildings were RC structures and brick masonry buildings. Massive damages were observed on account of NSCs [3].

All the major earthquakes in the world such as Alaska [4], Loma prieta [5] [6], San Fernando [7], Northridge [8] [9], and Darfield [10] have shown that the non-structure elements of the structure were more affected during the seismic action. Sometimes the repairing costs of the non-structural elements are more than the primary components of the structures [11] [12] [13] [14] after the seismic hazard.

The non-structural component of the structures is divided based on the acceleration sensitivities and storey drift sensitivities [15]. Masonry walls, doors, windows partitions etc. are the example of the NSCs based on the storey drift sensitive, similarly parapets, boilers, suspended ceilings etc. are the examples of the NSCs based on the acceleration sensitive [16]. For the design of the secondary elements of the structures, floor amplification factor, is an important parameter for the estimation of maximum inertia force on the structure. Some of the pioneering literature, reported on the

behaviour of the secondary elements of the structures are explained.

ASCE7-10 [17] code describes the design process of the seismic force act on the non-structural components. Drake and Bachman [18] used the concept of the ASCE code and gave the relationship between the peak ground acceleration (PGA) and the peak floor acceleration (PFA). Darke and Bachman, were considered many recorded time history data sets. This large number of data set compiled and obtained the relationship from 16 California earthquake ground motion data. These data sets were derived by taking the average of the PFA in each direction. To obtain the relationship between the PFA and PGA, the average values of PFA was divided by the average value of PGA in each direction. Finally, results were compared with the NEHRP provisions.

Gillengerten and Bachman [19] gave a concept of the acceleration amplification factor considering the major changes of the NEHRP provisions observed in the 1997 edition. The equation of the acceleration amplification factor is the same as given in ASCE 7-05 [20] and ASCE 7-10 [17]. They observed that the acceleration amplification factor “[1+2(z/h)]” was bounded between the mean plus standard deviation of the peak acceleration.

Saeed Fathali et al. [21], investigated the acceleration amplification factor and the component amplification factor, with the concept of mean plus standard deviation (Mean + SD). They considered 73 different ground motion data with different ground motion range of less than 0.067g, 0.067g to 0.2g and more than 0.2g. Finally, they proposed the non-linear acceleration amplification formula and compared with the provisions given by ASCE code.

In another approach, Crescenzo Petrone et al. [22], considered the five different heights of moment-resisting RC frame models, designed by Euro code 8. The linear and non-linear methods were used to analyze of these models. It was found that the Euro code formula for analysis of amplification factors gave conservative results compared to the analytical results.

Surana et al. [23], examined the peak floor acceleration and compared with the different codes (ASCE, EN 1998, NZS 1170.4 and IS 1893) and observed that for low rise buildings (up to 4 storeys) the amplification values performed linear behaviour. After that its behaviors were nonlinear. It was also found that the peak floor acceleration demand at the top of the building is decreases as the natural period of the structures increases.

In this paper, non-linear acceleration amplification models are proposed for moment-resisting RC frames buildings having the natural fundamental periods between 0.1 to 1.5 second. For the analysis, the model mass participation ratio higher than 90% are considered as per ASCE provisions. The seismic ground motion data are considered in the range of 0.01g to 0.32g. Based on the Mean + SD acceleration amplification results, to propose the amplification model and compared with the previous proposed models.

2. Existing Models

ASCE/SEI 7-05 [19] section 13.3.1, defines the lateral seismic force on the non-structural component as-

$$F_p = 0.4S_{ds}a_p \left(1 + 2\frac{z}{h}\right) \left(\frac{I_p}{R_p}\right) W_p \quad (\text{ASCE/SEI 7-05 Equation 13.3-1})$$

$$F_p \leq 1.6S_{ds}I_pW_p \quad (\text{ASCE/SEI 7-05 Equation 13.3-2})$$

$$F_p \geq 0.3S_{ds}I_pW_p \quad (\text{ASCE/SEI 7-05 Equation 13.3-3})$$

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

ASCE gave the acceleration amplification factor (Ω) in terms of normalized height (z/h) of the structure. It gives a linear relation between the PFA and PGA. The codal provisions for amplification formula are given as:

$$\Omega = 1 + 2\left(\frac{z}{h}\right). \quad (1)$$

Saeed Fathali [20] gave the non-linear relationship between the acceleration amplification factor and the normalized height. It [20] considered two constants α and β for obtaining the non-linear amplification factor formula to change the seismic ground motion. The observed mathematical model is given as:

$$\Omega = \frac{PFA}{PGA} = 1 + \alpha \left(\frac{z}{h}\right)^\beta, \quad (2)$$

where α and β are constants based on the range of the ground motion. The values of α and β are given in Table 1 and 2.

Table 1. Values of α for acceleration sensitive non- structural components

Time	PGA=0.04S _{DS} <0.067g	0.067g≤PGA=0.04S _{DS} <0.2g	PGA=0.04S _{DS} ≥0.2g
T _a = 0.5 sec.	2.12	1.93	1.75
0.5s ≤ T _a < 1.5s	2.61	1.55	1.01
T _a ≥ 1.5s	2.52	1.53	0.5

Table 2. Values of β for acceleration sensitive non- structural components

Time	PGA=0.04S _{DS} <0.067g	0.067g≤PGA=0.04S _{DS} <0.2g	PGA=0.04S _{DS} ≥0.2g
T _a = 0.5 sec.	0.78	1.25	0.92
0.5s ≤ T _a < 1.5s	1.16	0.75	0.69
T _a ≥ 1.5s	1.64	1.65	3.00

3. Proposed Mathematical Model

ASCE formula is based on the normalized height of the structure only and does not depend upon the natural period of the structure. However, Fathali's model observed the acceleration amplification factor depends not only on the normalized height of the structure but also on seismic ground motion range. It was also marked that, the acceleration amplification factor was depend on the period of the structure [24]. However, it notifies that the shape of the amplification factor is nonlinear in the present case whereas the previous model performed was linear behaviour which gave conservative results. Based on these factors to propose a mathematical model shown as:

$$\Omega = 1 + \left(\frac{T_{max}-T}{T*a} \right) y(y^5)^{y^4}, \quad (3)$$

where T_{max} is the maximum structural period, and the recommended value is 2.5 second [24]. T is the period of supporting structure for which the peak roof acceleration is not less than PGA. "y" is represented as the normalized height (z/h) of the structure, and "a" represent constant which depends upon the period of supporting structure and the seismic ground motion range. Values of "a" given in Table 3.

Table 3. Values of "a" based on ground motion range and period of supporting structure

Ground motion acceleration	"a"	Period of supporting structure
PGA=0.04S _{DS} <0.067g	2.25	0<T<0.5sec
	1.1	0.5sec≤T<1.0sec
	0.8	1.0sec≤T<1.5sec
0.067g≤PGA=0.04S _{DS} <0.2g	2.25	0<T<0.5sec
	1.25	0.5sec≤T<1.0sec
	0.85	1.0sec≤T<1.5sec
0.2g≤PGA=0.04S _{DS} <0.31g	2.25	0<T<0.5sec
	1.25	0.5sec≤T<1.0sec
	0.75	1.0sec≤T<1.5sec

The proposed mathematical model results non-linear behaviour. The non-linear general exponential function is

obtained with the help of Euraqa Pro [25] software which is an artificial intelligence based power modelling engine. Euraqa pro software is mainly used to determine the mathematical equations that describe sets of data in their simplest form. For the proposed mathematical equation, first step is to determine the natural period of the structures. Next step is to obtain the non-linear equations using Euraqa pro software. Finally, is to determine the constant "a" based on intensity of ground motion and period of the supporting structures using with trials and error method and to obtain the satisfactory values given in table 3, respectively.

4. Building Configuration

In this paper, five RC moment-resisting frame structures of two, four, six, eight, and ten stories are considered. The supports of the models are fixed. The first storey height is 4m, and subsequent storey height is 3.4m. The 2D models are shown in Figure 1 and the size of the beams and columns are given in Table 4. The fundamental period of the structures lies in the ranges of 0.1 to 1.5 second and the damping ratio is 5%. For the analysis of all the models, Etabs [26] software has been used.

Table 4. Size of beams and columns

	Configuration	Size in mm
Beam	B1	300x400
	B2	300x450
	B3	450x500
	B4	450x600
	B5	450x650
	B6	450x675
Column	C ₀	300x400
	C1	300x450
	C2	450x500
	C3	525x550
	C4	550x600
	C5	600x700
	C6	650x850

Figure 1. Moment resisting frame models (a) 2 (b) 4 (c) 6 (d) 8 and (e) 10 stories

5. Ground Motion Selection

Many researches worked with the higher range of the ground acceleration (higher than 0.3g), but most of the cases, the buildings are damaged at a low degree of the ground motion. Therefore, this paper is to study the behaviour of the structures with a lower range of ground acceleration (0.01g to 0.32g). The selection of the ground motion was obtained from the strong ground motion

virtual data centre [27]. For the analysis of the models, time history data having ranges between 0.01g to 0.31g is taken, and it divided (0.01g to 0.067g, 0.067g to 0.2g and more than 0.2g) based on Fathali [21]. In this study, 28 recorded ground motion data are between 0.01g to 0.067g, 29 ground motion data in the range of 0.067g to 0.2g and 24 ground motion data are between 0.2g to 0.31g. Details of the ground motion data are given in Table 5, 6 and 7 as below.

Table 5. Recorded ground motion data having ranges 0.01g to 0.67g

Ground motion name	PGA (g)	T (sec)	T _p (sec)
Chi-chi 1	0.066	64.992	16.696
Chi-chi 2	0.057	52.98	17.005
Chi-chi 3	0.044	47.975	15.66
Chi-chi 4	0.047	47.975	16.115
Chi-chi 5	0.0199	45.988	17.14
Chi-chi 6	0.027	45.988	16.112
Chi-chi 7	0.028	63.98	22.875
Chi-chi 8	0.0237	63.98	26.69
Chi-chi 9	0.0372	53.98	20.615
Chi-chi 10	0.066	70.97	15.56
Chi-chi 11	0.0516	70.97	16.23
Chi-chi 12	0.0441	50.98	18.42
Chi-chi 13	0.0352	50.98	17.14
Chi-chi 14	0.0592	60.98	18.1
Chi-chi 15	0.0503	60.98	13.73
Chi-chi 16	0.0467	56.985	14.305
Chi-chi 17	0.0492	56.985	16.775
Chi-chi 18	0.0361	62.98	25.205
Chi-chi 19	0.065	62.98	17.275
Chi-chi 20	0.0662	74.98	14.69
Chi-chi 21	0.066	62.98	17.275
Chi-chi 22	0.0604	59.98	15.656
Chi-chi 23	0.0574	59.98	18.694
Chi-chi 24	0.0511	62.988	19.476
Chi-chi 25	0.0514	62.988	20.66
Chi-chi 26	0.024	49.992	16.004
Chi-chi 27	0.0185	49.992	15.872
Chi-chi 28	0.0439	56.992	20.424

Table 6. Recorded ground motion data having ranges 0.067g to 0.2g

Ground motion name	PGA (g)	T (sec)	T _p (sec)
Chi-chi 1	0.1374	60.98	17.74
Chi-chi 2	0.1348	60.98	15.93
Chi-chi 3	0.1217	56.985	13.905
Chi-chi 4	0.1179	56.985	15.08
Chi-chi 5	0.1092	57.975	15.99
Chi-chi 6	0.147	65.045	14.58
Chi-chi 7	0.1215	65.045	14.475
Chi-chi 8	0.1167	70.97	16.555
Chi-chi 9	0.127	70.97	17.265
Chi-chi 10	0.1913	65.045	16.81
Chi-chi 11	0.1565	65.045	16.545
Chi-chi 12	0.1654	65.045	15.6
Chi-chi 13	0.1692	65.045	16.0
Chi-chi 14	0.1439	63.985	14.275
Chi-chi 15	0.1624	65.045	14.45
Chi-chi 16	0.1517	65.045	14.73
Chi-chi 17	0.1322	66.005	13.45
Chi-chi 18	0.1234	66.005	16.125
Chi-chi 19	0.1866	74.98	17.235
Chi-chi 20	0.155	74.98	15.04
Chi-chi 21	0.157	61.98	16.72
Chi-chi 22	0.197	74.985	15.47
Chi-chi 23	0.1344	96.985	11.85
Chi-chi 24	0.1863	96.07	16.635
Chi-chi 25	0.1563	93.985	12.685
Chi-chi 26	0.1883	96.985	12.02
Chi-chi 27	0.1831	99.03	10.715
Chi-chi 28	0.1699	61.99	9.66
Chi-chi 29	0.1829	71.0	14.38

Table 7. Recorded ground motion data having ranges 0.2g to 0.32g

Ground motion name	PGA (g)	T (sec)	T _p (sec)
Chi-chi 1	0.2296	47.99	8.015
Chi-chi 2	0.2167	74.985	11.94
Chi-chi 3	0.2061	86.485	17.615
Chi-chi 4	0.2252	124.06	10.04
Chi-chi 5	0.2779	65.005	5.8
Chi-chi 6	0.2347	68.03	15.22
Chi-chi 7	0.2678	68.03	15.39
Chi-chi 8	0.2818	74.98	12.15
Chi-chi 9	0.2164	77.49	19.62
Chi-chi 10	0.2465	63.985	14.28
Chi-chi 11	0.2504	139.98	36.02
Chi-chi 12	0.2021	122.97	37.19
Chi-chi 13	0.2449	149.97	37.53
Chi-chi 14	0.2206	143.97	31.98
Chi-chi 15	0.201	139.98	20.14
Chi-chi 16	0.2515	149.97	46.14
Chi-chi 17	0.2828	149.97	33.97
Chi-chi 18	0.2820	149.97	34.05
Chi-chi 19	0.2656	60.03	10.965
Chi-chi 20	0.2598	149.97	47.37
Chi-chi 21	0.2229	149.97	29.39
Chi-chi 22	0.2270	149.97	37.37
Chi-chi 23	0.201	119.976	31.64
Chi-chi 24	0.2596	119.976	36.688

In these tables, T represents the total recorded period and T_p represents the time when peak acceleration occurs.

6. Analysis based on the Finite Element Method

The analysis of all RC frame models has been done by finite element method (e.g. Etabs v2016). It is a numerical technique to solve the frame models into elements models. Linear time history method is used in different models explained in Agrahari *et al.* [28]. The fixed support condition is assigned at the joint base of the structures, and the soil type is hard. A large number of near field time history data (0.01g to 0.32g) is considered and also the fundamental natural period of the structure is taken up to 1.5 sec.

7. Results and Discussion

7.1. Floor Spectral Acceleration

For performing the dynamic analysis, selected horizontal time history is applied at a different level for each model. For obtaining the floor spectral acceleration, ground acceleration with 5% damping is considered in the present work. The mean spectral acceleration of each model for the ground motion range of 0.2g to 0.32g is shown in Figure 2. From the figure, it is seen that, as the height of the building increases, the amplification value decreases. It is also observed that for the building period is less than 0.5 sec, the spectral acceleration values are higher.

Figure 2. Mean floor spectral acceleration for ground motion range 0.2g to 0.31g (a) 2 storey (b) 4 storey (c) 6 storey (d) 8 storey (e) 10 storey

Figure 3. Behavior of floor acceleration for ground motion 0.06g (a) 2 (b) 4 (c) 6 (d) 8 and (e) 10 stories

Figure 4. Behavior of floor acceleration for ground motion 0.16g (a) 2 (b) 4 (c) 6 (d) 8 and (e) 10 stories

Figure 5. Behavior of floor acceleration for ground motion 0.32g (a) 2 (b) 4 (c) 6 (d) 8 and (e) 10 stories

7.2. Comparison of Peak Floor Acceleration

The peak floor acceleration is used to define the seismic demands on rigid or semi-rigid (when frequencies of vibration more than 20 Hz) acceleration sensitive NSCs. However, for flexible NSCs, the acceleration is different from the peak floor acceleration. Figures 3-5 shows the comparison of the 5% damped floor acceleration, for the various heights of buildings during the Chi-Chi earthquake. The PGA of Chi-Chi earthquake time histories are 0.06g, 0.16g and 0.32g, respectively. For the fundamental period of the building is less than 0.5 sec. the peak floor acceleration is approximately 3.5, 2 and 1.5 times higher than the PGA for 0.06g, 0.16g and 0.32g, respectively. However, the increment is 3, 1.8 and 1.3 times the peak ground acceleration when the building period lies in the range of 0.5 to 1 sec. For higher building period (more than 1 sec), the peak floor acceleration is approximately 2, 1.5 and 1.3 times the peak ground accelerations having PGA of 0.06g, 0.16g, and 0.32g, respectively. Moreover, when the PGA of the ground motion increases, the peak floor acceleration decreases. It is also observed that when the building period increases, the peak floor acceleration decreases.

7.3. Effect of Building Period to Amplification Factor

Based on linear time history method, the acceleration amplification factor depends on the fundamental period of the building. Figure 6 illustrates the mean and Mean + SD

of PFA/PGA for each of the five buildings. From comparison of the PFA/PGA values with ASCE code, it can observe that PFA/PGA values are constant for various natural periods of structures for ASCE model. However, Fathali's model showed that the PFA/PGA values are not consistent, and its values are different for different building period.

Figure 6. Comparison of building period to PFA/PGA with different models

7.4. Comparison of the Amplification Factor with Different Models

Comparison with the proposed model, ASCE, Fathali's non-linear mathematical model and the Mean + SD results, are shown in Figure 7, 8 and 9 respectively.

Figure 7. Comparisons between acceleration amplification factor to normalized height when ground motion ranges less than 0.067g (a) 2 (b) 4 (c) 6 (d) 8 and (e) 10 stories

ASCE amplification model shows the linear behaviour as the natural period of the structure increases. For the natural period of the structure less than 0.5 sec, ASCE results observed that its values are 11.0% less than mean + standard deviation (Mean + SD). However, Fathali's model resulted 7.0% less than the amplification factor for Mean + SD results. As the height of the building increases, the ASCE and Fathali's acceleration amplification models achieved conservative results. The proposed amplification model provided the values similar to the Mean + SD results. The shape of the curve from proposed model is also similar to the Mean + SD results as the height of the building increases.

Figure 8. Comparisons between acceleration amplification factor to normalized height when ground motion ranges are 0.067g to 0.2g (a) 2 (b) 4 (c) 6 (d) 8 and (e) 10 stories

For the ground motion range from 0.067g to 0.2g, the Fathali's model overestimated the amplification factors when the normalized height of the building is higher than 0.5. The difference between Mean + SD amplification and ASCE model's amplification values increases as the fundamental period of the building increases. Fathali's

and ASCE amplification models are approximate 28% and 50% higher than the Mean + SD results as the natural period of the building increases up to 1.5 sec. Though, the proposed amplification model obtained values 15% higher than Mean + SD results. The performance of the proposed model is similar to the Mean + SD amplification results.

Figure 9. Comparisons between acceleration amplification factor to normalised height when ground motion ranges are 0.2g to 0.32g (a) 2 (b) 4 (c) 6 (d) 8 and (e) 10 stories

When the ground motion range is between 0.2g to 0.32g, the amplification values of the Fathali's model is around 15% higher than the Mean + SD results. For this ground motion range (0.2g to 0.32g) Fathali's model gave better results, but the deviation from Mean + SD results increases as the normalized height goes higher than 0.5. As the fundamental period of the building rises, the amplification values of Mean + SD falls. However, ASCE

model observed that amplification factor varies linearly as the height of the building increases and its maximum values at the top of the building is 3. The amplification factor results obtained by ASCE code are approximate 60% higher than the Mean + SD results. The results observed by the proposed model are approximately 10% to 20% higher than Mean + SD results. The behavior of the proposed model is very close to the behavior of Mean +

SD results, respectively.

7.5. Comparison of Component Amplification Factors

Figure 10. Comparison of actual and ASCE a_p spectra based on 30 floor acceleration histories recorded in 4 Storey building under earthquakes with $PGA > 0.2$ g.

The component amplification factor (a_p) refers to the dynamic amplification of NSCs compared to PFA. The component amplification factor is defined as the ratio between FRS/PFA. ASCE code defines a_p is 1 for rigid components (those components having a natural period less than 0.06 sec.) and 2.5 for flexible components (those with a natural period higher than 0.06 sec). From figure 10, it can observe that the codal values gave conservative results when the component period is higher than 0.25 sec. For flexible components, a_p values observed from the Mean + SD component amplification results are approximately 2 to 2.5 times higher than the ASCE codal results.

From the comparison of the proposed model and the previous research model, it is found that the ASCE amplification factor formula gives significant results when the period of the structure is less than 0.5 second. ASCE formula provided conservative results when the period of the structure higher than 0.5 seconds. Fathali's model notify better results as compared to the ASCE code when the ground motion acceleration range is between 0.067g to 0.32g. The difference of amplification values between the Mean + SD and Fathali's model results are increasing between the ground motion range of 0.01g to 0.067g, respectively. However, the proposed model gives more realistic results compared to the other models.

8. Conclusions

In this paper, five moment-resisting RC frame models having different heights are considered. Building frames are analyzed using linear time history method, with an observed large number of time history data having ranges less than 0.067g, 0.067g to 0.2g and 0.2g to 0.32g respectively. To get the actual PFA and PGA values and

with the help of these data, a non-linear acceleration amplification formula is proposed and compared with the previously reported models. It is found that:

- ASCE formula provide better results when the period of the structure is less than 1.0 second. After that, it performed conservatively.
- The Fathali's model showed promising results to ASCE. It gave obscure results when ground motion lies 0.01g to 0.067g.
- In many cases, the Fathali's model provide better results at the roof of the building, but for other storeys, it is less satisfactory.
- The shape of the amplification factor obtained by the proposed model is close to the shape of the Mean + SD results.
- The a_p values given by the ASCE is conservative and its values are 2 to 2.5 times lower than Mean + SD results at sometimes.
- On comparing the previous mathematical model, propose non-linear amplification models give more realistic results to Mean + SD results.

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