CHAPTER 2

LITURATURE REVIEW

2.1 INTRODUCTION

There are two components in the structure: primary components also called main components, and secondary components or non-structural components (NSCs). The main components are designed to resist different types of loads. The NSCs even connected to the main components, are not able to transfer vertical or lateral loads. Examples of non-structural components are cladding panels, furniture, transformers, partition walls, etc. Initially, it is assumed that the NSCs do not affect the stiffness and the seismic effect of the building, but recent experimental studies found that during the seismic action, the NSCs also influence the stiffness and the seismic response of the structures [32-33]. The consequences of NSCs are not only financial, but they also have an impact on human lives [34-35]. The functional behaviour of the primary and secondary components is substantially impacted during earthquake activity [36-43]. The 1971 San Fernando earthquake [44], the 2000 Enggano earthquake [45], the 2001 Bhuj earthquake [46], the 2004 Grate Sumatra earthquake [47], the 2010 Chili Earthquake [48], the 2015 Nepal Earthquake [49], and the 2017 Iran-Iraq earthquake [50] are examples of significant earthquakes that have impacted the NSCs. Following the seismic activity, the cost of upgrading NSCs can sometimes exceed the cost of primary structural parts [51-54].

NSCs categorised based on the storey drift sensitivity and acceleration sensitivity. Most of the research has attained the floor acceleration response spectra (FRS) of the structures and proposed some guidelines [55]. FRS is generated from the absolute acceleration response of a floor in a building that is excited by the input ground motion, as shown in Fig.2.1. Many research has been conducted to observe the behaviour of the FRS in relation to the seismic design of NSCs. In contrast to ground acceleration spectra, FRS reflects the dynamic properties of building structures. That is, the supporting structure filters out vibrations with frequencies other than the building's natural frequencies, whereas vibrations with frequencies near to the natural frequencies are amplified [56-58]. Several studies were carried out to develop a generic FRS for the seismic design of NSCs based on structural dynamics fundamentals. These studies showed that the FRS were highly dependent on various parameters related to the building's characteristics and the NSC characteristics, such as the location of the NSCs in the structure [59-65], the ratio of the NSC period to the building's modal periods [66-73], the damping ratio of the supporting structures and the NSCs [74-76], the nonlinear structural behaviour [77-87], and the damping ratio of the supporting structures and the NSCs, the interaction between the NSCs and the supporting structure [88-93], the torsional response of the supporting structures [94] (Qu et al., 2014), the diaphragm flexibility of the supporting structures [95], the type of lateral load resisting system in the supporting building [96-97], the soil-structure interaction (SSI) [98-101], and the nonlinear behaviour of NSCs [102-107].

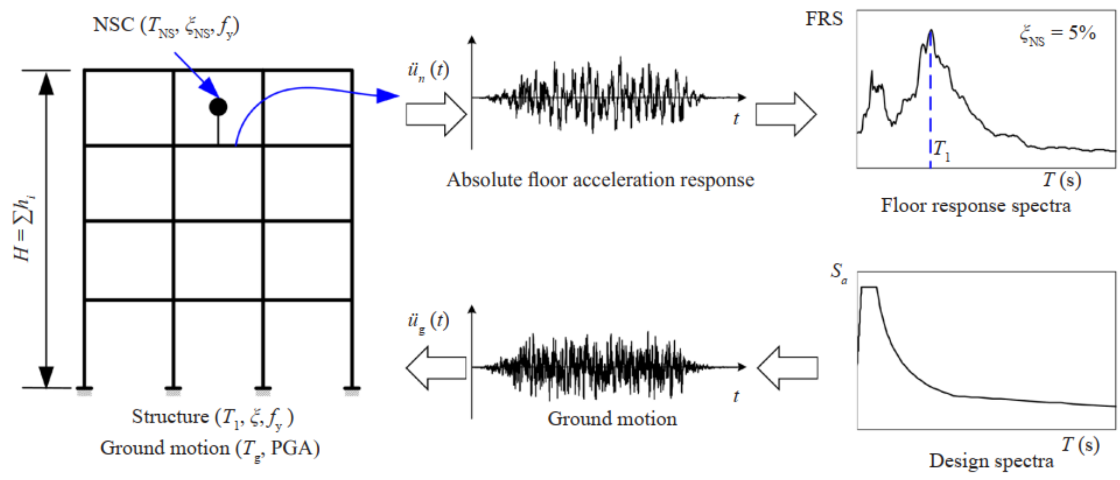

Figure 2.1 Definition and development of FRS: concept and definition of FRS wang (2021) [108]

NSCs are designed with inertia forces in mind, and these inertia forces are created by the floor response spectra (FRS). Different ways for constructing FRS using single-degree-of-freedom (SDOF) and multiple-degrees-of-freedom (MDOF) models are presented first, followed by a discussion of amplification factor methods. Following that, numerous newly developed FRS techniques are summarised and contrasted with those used in current seismic design codes. The connection between structural components and NSCs, such as infill walls, the soil-structure interaction, the damping ratio of NSCs, and the nonlinear behaviour of NSCs, are then investigated in depth. Field observations and experimental research on the FRS and floor acceleration response during earthquakes are discussed. Finally, major knowledge gaps as well as prospective future research challenges, are identified. Field observations and experimental research on the FRS and floor acceleration response during earthquakes are discussed. Finally, major knowledge gaps, as well as prospective future research challenges, are identified. Most of the data presented here was derived for linear NSCs with a damping ratio of 5% unless otherwise stated.

2.2 DEVELOPMENT OF FRS

SDOF models, MDOF models, and amplification factor models are utilised to design FRS. Figure 2.2 and 2.3 illustrates the definition of each parameter. The following section deals with all of the characteristics in further depth.

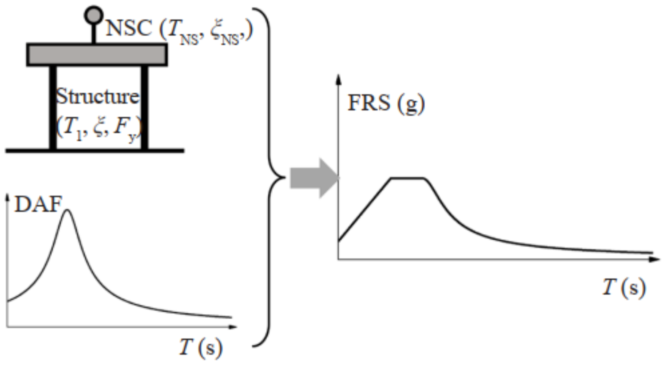

Figure 2.2 FRS based on SDOF Model (wang too 2021) [108]

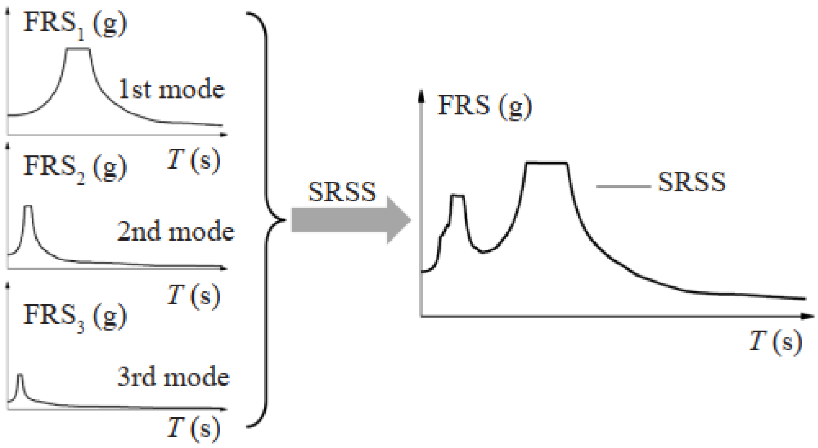

Figure 2.3: FRS based on MDOF Model (wang too 2021) [108]

2.2.1 FRS based on SDOF models

Research on FRS generation methods began in the 1970s. Early methods usually treated the supporting structure and NSC as SDOF systems. The FRS of the supporting structures was created using Time history approaches [109-110]. After that, Yasui et al. [111] developed the direct approach, which they used to execute smooth FRS utilising design response spectra. Due to the Duhamel integration, this method was not required to calculate the empirical dynamic amplification factor (defined as the ratio of spectral acceleration on NSCs to peak floor acceleration of linked structures). The following is a representation of the FRS formula:

$$FRS\ (T_{NS}, \xi_{NS}) = \sqrt{\frac{[(T_S/T_{NS})^2\ S_a(T_S,\xi_S)]^2 + S_a\,(T_{NS},\xi_{NS})^2}{[\,1-(T_S/T_{NS})^2]^2+4(\xi_S+\xi_{NS})^2(T_S/T_{NS})^2}} \tag{2.1}$$

Where T_{NS}, represent the natural period of the NSCs, ξ_{NS} is the damping ratio of the NSCs, respectively. T_s and ξ_s are the fundamental period and the damping ratio of the structures. $S_a(T_S, \xi_S)$ and $S_a(T_{NS}, \xi_{NS})$ are the values at the specific period and damping ratio in the elastic ground acceleration spectrum, $FRS\ (T_{NS}, \xi_{NS})$ are the FRS at the specific period T_{NS} and the damping ratio ξ_{NS}.

In comparison to MDOF structures, the resonance area (a region of the floor spectrum that includes peak and surrounding peak spectrum values) in SDOF structures is smaller. When compared to SDOF structures, MDOF structures have a higher number of natural modes and a larger resonance zone. The outermost of the resonance zone, the FRS determined by the Yasui et al. approach [111], is significantly closer to the FRS values acquired by the time history method, according to Vukobratovic and Fajfar [112-114]. (THM) However, it discovers some differences in the FRS in the significant region; thus, it presented equation (2.2) to calculate the FRS in the resonant zone.

$$FRS\ (T_{NS}, \xi_{NS}) = AMP * \frac{S_a(T_S,\xi_S)}{R_\mu} \tag{2.2}$$

Here, AMP represent the amplification factor in the resonance region; however, the second term ($\frac{S_a(T_S,\xi_S)}{R_\mu}$) denotes the values of the inelastic acceleration spectrum, which can be obtained by decreasing the elastic acceleration spectrum with the help of strength factor (R).

Jiang [115] and Li [116] introduce a concept of tunning response spectrum (TRS) for understanding the behaviour of FRS when the NSCs are tuned to the primary components. Based on the outcomes of the THM to develop the relationship between

TRS and GRS. Sullivan [57-58] gave an equation (2.3) based on the dynamic amplification factor (defined as the ratio between peak acceleration on the NSC to the peak acceleration of the floor on which the NSCs are connected) to determine the FRS in SDOF system.

$$FRS\ (T_{NS}) = \frac{T_{NS}}{T_S}[a_{max}\ (DAF_{max} - 1)], \qquad T_{NS} < T_S \tag{2.3}$$

$$= a_{max}\ DAF_{max}\ , \qquad T_S \leq T_{NS} \leq T_e$$

$$= a_{max}\ DAF, \qquad T_{NS} > T_e$$

Where FRS(T_{NS}) is the spectral acceleration demand for a supported component with period T_{NS}, a_{max} is the maximum acceleration of the supporting structure (obtained for an SDOF system by dividing the structure's lateral resistance by the seismic mass).

2.2.2 FRS based on MDOF models

The behaviour of an SDOF system differs from that of an MDOF system. As a result, Calvi and Sullivan [117-118] expanded the approach developed by Sullivan et al. (2013) for SDOF structures to MDOF structures reacting in the elastic variety. The evaluation process is summarised as follows:

- Calculating the natural period of vibration and the mode shape is the first step in the procedure. Once the natural period and mode shape have been identified, the PFA for each mode can be calculated using the traditional model response spectrum method (see "Dynamics of Structures: Theory and Applications to Earthquake Engineering, 2nd Edition," 2001)[119]. The following is a mathematical expression:

$$a_{max,j,i} = \frac{\emptyset_{j,i}}{\sum \emptyset_{j,i} m_j} m_{e,i}\, S_{a,i}$$

Where $a_{max,j,i}$ is the floor acceleration at degree-of-freedom (i.e. floor level) j from mode i, $\phi_{j,i}$ is the mode shape for level j and mode i, m_j is the seismic mass at level j, and $m_{e,i}$ is the effective modal mass for mode i. The term $S_{a,i}$ is the spectral acceleration value for mode i obtained from the design ground response spectrum.

- The approach for SDOF systems provided by Sullivan [57] can be used to obtain floor spectra for each of the modes once the modal PFA contributions are known. The upper-level floor response spectra are currently created by combining each of the modal floor spectra using a well-known modal combination rule, such as square-root-of-sum-of-squares (SRSS).
- For the lower floors, the FRS is the maximum between the GRS and the spectral acceleration obtained from the SRSS of the modal spectra computed in Step (1).

Only elastic structures were used in the aforementioned technique; non-linear structures were not used. However, using a factor termed model response reduction factors, this method was improved [120-121] for analysing the nonlinear behaviour of supporting structures (it is the ratio between the FRS of a linear response to the FRS of a non linear response).

$$Ri, single\ record = \frac{FRS(T_i)_L}{FRS(T_i)_{NL}}$$

Where, $FRS(T_i)_L$ is represented by the floor spectral acceleration at the period of mode i for linear response and $FRS(T_i)_{NL}$, is the floor spectral acceleration at the period of mode i for a nonlinear response, respectively. This reduction factor depends on the ductility of the connected structures. Merino used the nonlinear regression method [122-123] after the modified concept given by Calvi and Sullivan's [117] to perform the relation between ductility of the corresponding structure and the reduction factors. It created a codeoriented methodology for calculating the FRS of supporting structures nonlinear behaviour. Pan et al. [124-125] provided a method for determining the FRS of an MDOF system using analogous SDOF systems based on push over analysis.

2.3 AMPLIFICATION FACTOR METHODS

The GRS, or design response spectra, is used to generate the FRS of the buildings. The amplification factor refers to the ratio of FRS to GRS. Shooshtari et al. [126] looked at six moment-resisting RC frame buildings and six shear wall buildings of various heights. After the analysis it was found that the amplification of FRS is maximum at the top of the buildings and gradually reduces toward the first floor. Wieser et al. [86] evolved an empirical multi-linear envelope spectral acceleration amplification function. The amplification function was developed based on the incremental dynamic analysis results of four steel structural buildings. The function compares the NSCs period ratio to the supporting structure of the first period. It also takes into account the higher mode effect and relative height. NSCs that are stiff, tuned to the second mode, and tuned to the first mode, respectively, have peaks at period ratios of 0, 0.3, and 1.0. The effective period was substituted for the natural period of the structures for calculating the influence of structural yielding. Surana et al. [127] proposed the floor amplification function to determine the FRS at any height of the building from the GRS. The proposed amplification function depended on the size of the buildings, first two modes of supporting structures, building fundamental period and strength factor of the supporting structures. When the fundamental period of the building is shorter than 0.5T, the author found that the amplification factor is constant for second and higher modes. Peaks of the first two modes, however, were considered when the natural period of the buildings was more than 0.5T. Individual parabolic functions were used to characterise the peak of FRS/GRS for each mode's influence.

2.4 ACCELERATION DEMANDS OF NSCS DEFINED IN SEISMIC DESIGN CODES

The term "acceleration amplification factor" is used to determine the acceleration demand on NSCs. It is defined as the ratio of PFA to PGA. Several codes have guided the seismic design of NSCs. The Applied Technology Council Report (ATC 1978) [128] was the first to mention it. Several codes altered the amplification factor in response to the obtained results. Eurocode 8 (CEN 2004) [129] assumes the linear distribution of PFA/PGA regarding the height of the buildings, and at the top of the building, its maximum value is 2.5. Similarly, Chinese GB 50011-2010 [130] and ASCE 7-16 (2016) discovered a linear relationship between PFA/PGA and building height and recommended maximum values of 2.0 and 3.0, respectively, at the building roof. In the New Zealand NZS 1170.5 code [131], the acceleration amplification factor depends on the floor height coefficient (C_{Hi}), which is a form of the elevation of the building (h). If the building's elevation is less than 12 m, the acceleration amplification factor behaves linearly, with the maximum value being one at the bottom and (1+h/6) at the top. When the building's elevation exceeds 12 m, the acceleration amplification factor is bilinear, with a maximum value of 3 at the top. The NEHRP code (FEMA P-750, 2009) [132] follows the same rules as the ASCE 7-16 code for determining the PFA/PGA value (2016).

ASCE 7-16 (2016)[133], gave a concept of component amplification factor (ap), which is defined as the ratio between FRS/PGA of the buildings. For rigid components (period of the NSCs less than 0.06 sec), the component amplification factor is 1. However, when the period is higher than 0.06 sec (flexible components), ap maximum value is 2.5. GB

50011-2010 [130] assumes the ap value for flexible NSCs is 2.0, respectively. The New Zeeland code [131] determines the ap value based on the NSCs. Although Eurocode 8 [129] and NEHRP [101] determine the ap value based on the NSCs nature and the condition of supporting structures. ASCE 7-16 code (2016)[132] and GB 50011-2010 code [130] gave some guidelines based on past experience and engineering judgment to determine the ap value.

2.5 CURRENT MODEL EQUATIONS

Although many research also evaluated the amplification factor based on different parameters. Drake and Bachman [134] 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 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.

UBC, ASCE and some of the renowned researcher proposed the model based on the different parameters for determine the acceleration amplification factor of the structures. These models are described as follows:

2.5.1 Uniform Building Code 1997 (UBC)

In this code [135], the horizontal force acting on the non-structural components of the floor is given as:

$$F_p = \frac{a_p C_a I_p}{R_p}\left(1 + 3\frac{h_x}{h_n}\right) W_p. \tag{2.4}$$

Where a_p is the structural amplification factor, R_p is the response modification factor,

h_x and h_n are the altitude of the components and the over-all height of the building from the bottom of the structures, I_p represent the important factor of the components, W_p is the total weight of the components respectively. The coefficient of a_p, wary between 1.0 to 2.5 and the response modification factor (R_p) vary between 1.0 to 4.0. In this equation $\left(1+3\frac{h_x}{h_n}\right)$ represent the floor acceleration amplification factor of the secondary elements.

The floor horizontal force F_p

$$0.7C_a I_p W_p < F_p < 4.0C_a I_p W_p \tag{2.5}$$

For determining the horizontal force on the elastic components of the floors, the response amplification and structural amplification factor are considered as 1.0.

2.5.2 ASCE

The lateral seismic force acting on the non-structural components, defined by ASCE/ SEI 7-16 [133] in section 13.3.1 as

$$F_p = 0.4S_{ds}a_p\left(\frac{I_p}{R_p}\right)\left(1+2\frac{z}{h}\right)W_p \tag{2.6}$$

$$0.3S_{ds}a_p W_p \leq F_p \leq 1.6S_{ds}a_p W_p \tag{2.7}$$

Where F_p is the lateral seismic design force, S_{ds} represent the site-specific short period spectral acceleration, a_p is the component amplification factor having a range of 1.0 to 2.5, z and h denote the height of the component and the height of the building with respect to base respectively, Ip is the component important factor, and R_p refers to the component response modification factor which shows the energy absorbed by the component and W_p

is the weight of the component. However, the value of $\left(1 + 2\frac{z}{h}\right)$ express the floor acceleration amplification factor of the non-structural segments.

2.5.3 IITK-GSDM [136]

For the design of non-structural components in RC frame structures, IS 1893-2002 [137] code does not provide clear information. Clause 7.12.2 defines the lateral force acting on the non-structural components as five times the horizontal design acceleration multiplied by the weight of the components. The provision given by the code gave highly inadequate results for estimating lateral force acting on non-structural components. IITK-GSDM [136] proposed an acceleration amplification model for obtaining the amplification factor on the RC frame structures. IITK amplification model is based on the normalized height of the structures which is given as

$$\Omega = \left(1 + \frac{z}{h}\right) \tag{2.8}$$

Where z and h are the height of the components and the height of the building with respect to the base, this model found that the maximum amplification of the non-structural components which occurred is 2 when the z and h are equal.

2.5.4 Akhlaghi And Moghadam [138]

Akhalaghi and Moghadam estimated that the seismic behaviour of rigid acceleration-sensitive secondary elements having a fundamental period lesser than or equal to 0.06 sec. It was observed that the nature of the peak horizontal acceleration of the floor or roof is the same as the nature of the rigid non-structural components along with the height of the building, linked with the main structure. They concluded that the response of the floor or roof during the ground motion was the same as the response of the non-structural components, so they proposed the floor acceleration amplification factor (Ω) equations based on the fundamental time period of the structures.

$$\Omega = 1 + (\alpha - 1)\left(\frac{h_i}{h_n}\right) \tag{2.9}$$

Where Ω is the floor acceleration amplification factor, defined as the ratio between peak horizontal floor acceleration to peak ground acceleration, h_i and h_n are the height of the storey and the total height of the building with respect to the base of the building and α represent the fundamental period dependent factor, which was given as:

$$\alpha = 3 \qquad \text{when } T<0.5$$

$$\alpha = \frac{2.5}{T^{1/4}} \qquad \text{when } 0.5 \leq T \leq 1.0$$

$$\alpha = \frac{2.5}{T^{3/4}} \qquad \text{when } T>1$$

Where T is the fundamental period of the structures.

2.5.5 Fathali and Lizundia

Fathali and Lizundia [139] observed that the floor acceleration amplification factor is not only dependent on the height of the components of the structure but also dependent on the level of the ground motion and proposed a non-linear equation based on it.

$$\Omega = 1 + \alpha\left(\frac{z}{h}\right)^{\beta} \tag{2.10}$$

Where z, h are the height of the non-structural component and height of the storey to the base. α and β are two parameters based on the natural period of the structure and the level of the ground motion respectively. The values of α and β are shown in table 1 and 2, respectively.

Table 2.1 Value of α is suggested for the seismic design of newly constructed NSCs

Natural period	PGA = 0.4SDS < 0.067 g	0.067 ≤ PGA = 0.4SDS < 0.20 g	PGA = 0.4SDS ≥ 0.20 g
T_a < 0.5s	2.120	1.930	1.750
0.5≤ T_a <1.5s	2.610	1.550	1.010
T_a ≥1.5s	2.520	1.530	0.500

Table 2.2 Value of β is suggested for the seismic design of newly constructed NSCs

Natural period	PGA = 0.4SDS < 0.067 g	0.067 ≤ PGA = 0.4SDS < 0.20 g	PGA = 0.4SDS ≥ 0.20 g
T_a < 0.5s	0.780	1.250	0.920
0.5≤ T_a< 1.5s	1.160	0.750	0.690
T_a ≥1.5s	1.640	1.650	3.000

Where SDS ia site-specific short-period spectral acceleration.

2.5.6 Joseph Wiser [86]

The proposed equation for determination of the floor acceleration amplification factor in terms of the period of the structures -

$$\Omega = \frac{PFA}{PGA} = (1+\frac{\mathrm{Tmax}-\mathrm{T}}{T}\frac{z}{h}) \tag{2.11}$$

In equation (2.11), T represents the period of the supporting structure, and T_{max} is the maximum structural period when the roof acceleration is greater than or equal to Peak Ground Acceleration (PGA), and it was considered equal to 2.5 sec.

2.6 Need of the Research

- From the above literature, it was observed that the amplification factor is an important term for determining the inertia force acting on the NSCs of the structures. Limited research has been done for determining the acceleration amplification factor. According to codes, the amplification factor solely depends on normalising building heights; however, it is also influenced by other factors. As described by codes or previous study models, the amplification factor's behaviour is not necessarily linear. Further research, on the other hand, found that it was influenced not only by the size of the structure but also by the structure's basic period. Furthermore, the acceleration amplification factor is discovered to be dependent on other factors such as ground motion range, ductility ratio, effective period, and so on.

2.7 OBJECTIVE OF THE RESEARCH

The broad objective of the research works as follows:

- To develop the Seismic Acceleration Amplification Factor Model for Non-Structural Components in RC Frame Structures
- To develop the Non-linear Seismic Acceleration Amplification Factor Model for Non-Structural Components in RC Frame Structures
- To compare the acceleration amplification models between the fixed and pin-supported RC frame structures.
- To develop the Non-linear Seismic Acceleration Amplification Factor Model for Non-Structural Components in Pin Supported RC Frame Structures
- To develop the Floor Acceleration Amplification Factor in Yielding of Moment Resisting RC frame Structures

2.8 LAYOUT OF THESIS

The thesis is divided into eight chapters. A brief introduction and overview of the thesis are provided in chapter 1. Findings the literature related to the non-structural components, methods for analysis of the structures and the objective of the thesis are discussed in chapter 2. In the next chapter, to analysis the structures using different ground motion data and obtain the acceleration amplification values. To obtained the upper bound acceleration amplification values and these values are compared with the previous models. All the previous models exist below the upper bound amplification factor. So that, to proposed the upper bound acceleration amplification factor. In chapter 4, to analyze the model and obtain the mean+Sd amplification factor. This amplification factor is compared with the previous models. It found that the previous models results are quite conservative to propose the new models based on the mean+Sd amplification factor. Chapter 5 discusses the acceleration amplification values based on different support conditions. It observed that the behaviour of amplification factor changed as changed the support conditions. Based on this, to proposed the amplification factor of the pinned supported condition is discussed in chapter 6. Chapter 7 discusses the amplification factor based on a nonlinear analysis of the structures. The ductility ratio, effective time period, and floor response spectra are important for calculating the acceleration amplification factor. Using these factors to proposed the acceleration amplification factor and compared the previous amplification models. Chapter 8 deals with the conclusion of the research work, respectively