NONLINEAR ANALYSIS OF ASYMMETRIC RC BUILDING UNDER REPEATED GROUND MOTIONS

By

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Dissertation submitted in fulfillment of the requirements

For degree of

Master of Science (Structural Engineering)

School of Civil Engineering

Universiti Sains Malaysia

June 2011

ACKNOWLEDGEMENT

I would like to express my thankfulness and appreciation to my supervisor Associate Professor Dr. Taksiah A. Majid, for accepting me under her supervision, for her smile, kindness and her constant guidance during the development of this work.

Also I would like to thanks Mr. Ade Faisal for his help regarding the use of the analysis tools used in this work, for his patient, advices and supplying with everything he can.

Furthermore, I'm grateful to Professor Matjaz Dolsek from University of Ljubljana for providing the results of the experimental work used in this study.

Beside, my greatest thanks goes to Universiti Sains Malaysia, School of Civil Engineering for providing a convenient environment to conduct this work, for my previous university, Sudan University of Science and Technology and for my friends, in Sudan and Malaysia who kept believing in me and for their friendship.

My deepest gratitude goes to my beloved parents, Salwa and Abdallah; my brothers, Mohammed, Anas and Omer; my sister Areej; my grandmothers Um-Alhasan and Bakheeta; and for my big family for their love and constant support throughout my life.

Last but not least, my endless deepest thanks to Allah.

ABSTRACT

This study deals with the nonlinear response of an asymmetrical reinforced concrete building under single and repeated earthquake ground motions. Two main categories of ground motions, namely near-fault and far-fault, have been taken into consideration in this study. A full scale of a four-storey reinforced concrete building is experimentally tested by European Laboratory for Structural Assessment (ELSA) in Ispra, and the results from experimental test is compared with the numerical results conducted by this study. Then, many ground motions records, recorded in (stiff soil), are assigned to the building followed by the repeated ground motions. The repetition cases are created by considering two cases, the first case considers main-shock and after-shock while the second case considers fore-shock, main-shock and after-shock. The results obtained by far-fault are compared with near-fault results, also the repetition cases are compared with the single ground motion case in terms of top floor displacement, lateral displacement, rotation and interstorey drift ratio. The main finding of this study is from a qualitative point of view the sequence of ground motions lead to higher responds compared with the single case, consequently more damage will occur under repeated cases which should be considered when evaluating structures performance under seismic loads.

ANALISIS BUKAN LELURUS BANGUNAN KONKRIT BERTETULANG TIDAK SIMETRI OLEH GERAKAN GEMPA BERULANG

ABSTRAK

Penyelidikan ini mengkaji tindakbalas bukan lelurus untuk bangunan konkrit bertetulang yang tidak simetri oleh gerakan gempa tunggal dan berulang. Dua kategori utama gerakan gempa telah dipertimbangkan dalam kajian ini, iaitu kegelinciran-dekat dan kegelinciran-jauh. Keputusan makmal berskala penuh untuk bangunan empat tingkat konkrit bertulang telah diuji oleh European Laboratory for Structural Assessment (ELSA) di Ispra, dan digunakan sebagai perbandingan dengan analisis berangka yang dilakukan oleh kajian ini. Kemudian, pelbagai rekod gerakan gempa untuk tanah kaku, dikenakan keatas bangunan diikuti oleh kes berulang. Dua kes berulang dihasilkan dengan mempertimbangkan kes pertama: menganggap gempa utama dan gempa susulan sedangkan kes kedua: menganggap gempa awalan, gempa utama dan gempa susulan. Keputusan yang diperolehi dari kegelinciran-jauh dibandingkan dengan kegelinciran-dekat, juga keputusan kes gempa berulang berbanding dengan kes gempa tunggal dalam nilai anjakan tingkat teratas, anjakan mengufuk, putaran dan nisbah anjakan tingkat. Penemuan utama kajian ini adalah dari sudut pandangan kualitatif urutan gerakan gempa yang menyebabkan tindakbalas lebih tinggi berbanding dengan kes gempa tunggal. Akibatnya, lebih banyak kerosakan akan berlaku dalam kes gempa berulang yang harus dipertimbangkan ketika menilai prestasi struktur dibawah beban gempa.

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CHAPTER 1

INTRODUCTION

1.1 Background

An earthquake is a sudden release of energy that causes vibration, which travels through the earth's crust. Earthquakes caused by many things: volcanic eruptions, meteor impacts, underground explosions (an underground nuclear test, for example) collapsing structures (such as a collapsing mine). However, the majority of naturally-occurring earthquakes are caused by movements of the earth's plates. According to the United States Geological Survey, more than three million earthquakes occur every year, however, the vast majority of these 3 million earthquakes are extremely weak.

Earthquake is one of the natural phenomena that can cause considerable loss of life and damage to property around the world. Usually, it's not the shaking ground itself that causes losses of lives but the associated destruction of man-made structures and the instigation of other natural disasters, such as tsunamis, avalanches and landslides. The development of building practices around the world will lead to better and effective methods to mitigate these risks and reduce the overall losses.

Buildings designed as earthquake-resistant structures should be able to resist frequent, minor earthquakes without any significant damage to the non-structural components. Such structures should resist moderate earthquakes without significant structural damage. In the case of severe seismic action, the structure should be able to resist earthquakes without a major failure of the structural system to maintain life and to minimize major economical and cultural losses and to obtain this a satisfactory behaviour of a structure in the inelastic range should be achieved, in other word, the members and connections should have adequate ductility and energy dissipation capacity.

The assessments of structural performance during past earthquakes demonstrates that plan irregularity is one of the most frequent sources of severe damage, since it results in floor rotations (torsion response) in addition to floor translations. Real structures are almost always irregular as “perfect” regularity is an idealization that very rarely occurs. Even in cases where the building is designed to be completely symmetric, factors beyond the designer’s control like unpredictable eccentricities or rotational components of motion could also induce torsion (Stefano and Pintucchi, 2008).

Earthquakes usually don’t occur as a single event but as a series of shocks. Strong earthquakes have more and larger aftershocks, sometimes foreshocks, and the sequences can last for years or even longer. This repetition can be of a large magnitude which could collapse buildings that are damaged from the main shock, in other word, if the building is not prepared to undergo this kind of events, which mean after the shock the building is already at the edge of its ductility limits, the risk of these secondary shocks could be very high.

Seismic assessment is the first step within the strategy to reduce the seismic risk. A good understanding of the weak point of a structure under seismic loading could allow achieving the most appropriate retrofitting solution to reduce the seismic vulnerability.

The assessment of new designed and constructed structures could be obtained either by studying the behaviour of members and structures experimentally or by research tools such as, nonlinear static and dynamic analysis.

1.2 Problem Statement

The design practice of reinforced concrete structures advanced significantly around the world since the 1970's, mainly in the understanding of the seismic hazard. Current seismic design provisions require a structure to have adequate reinforcement detailing to provide an adequate ductile behaviour necessary to resist a targeted level earthquake. However, the current practice of structural earthquake engineering only consider single earthquake event, which is not fully accurate for the assessment of buildings existed in high seismic regions, since the repetition of seismic event at short time one after the other, produces an accumulation of damage on the structure.

This study illustrates the need of considering multiple earthquake events when doing the assessment of new and existing buildings in regions that are likely to undergo the repetition phenomena.

1.3 Objectives

The main objectives of this study are:

i. To determine the nonlinear response of an asymmetric RC building in terms of displacement, rotation and interstorey drift for far and near fault ground motions under single and repeated ground motions.
ii. To study the structural damage indices induced by far and near fault ground motions under single and repeated ground motions.

1.4 Scope of Study

The scope of this study is limited to the followings:

- The studied building is a four storey RC building with a uni-directional eccentricity that tested experimentally at European Laboratory for Structural Assessment (ELSA) in Ispra.
- Slabs are modelled as a rigid diaphragm and masses and moment of inertia of each floor are lumped at the corresponding centre of gravity.
- Beams and columns flexural behaviour is modelled by one-component lumped plasticity elements, composed of an elastic beam and two inelastic hinges.
- Beams are modelled with rectangular sections due to software limitation.
- Ground floor columns are fixed at the base.
- P - Delta effect is not considered.
- Ground motions considered are recorded from B soil type (stiff soil).
- Maximum number of earthquake repetition is three and the time gap taken is 100 seconds. The repeated earthquakes have been assembled randomly.

1.5 Outline

This thesis consists of six chapters, a part from this first introductive chapter, which could be briefly introduced as follows:

Chapter 2 gives a review from relative studies dealing with ground motion, nonlinear analysis and asymmetric buildings to get more comprehensive understanding of the seismic performance evaluation.

Chapter 3 presents an overview of the modelling approaches existing in literature which have been adopted to model structural elements and its components which constitute the building. Particular attention is rotational hinges which have been adopted to model the beam-column joints.

Chapter 4 describes the validation of the analytical model carried out by comparison of numerical and experimental results obtained from tests on the 3D reinforced concrete frame structure. Also it presents the results of numerical analyses performed on the validated three dimensional building with different combination of earthquake repetition. At first pushover analyses were performed to achieve better understanding of the global behaviour of the structure subjected to seismic loading. Then the structural response is investigated by mean of non-linear time history analysis on the structural model subjected to the repeated earthquakes.

Chapter 5 summarize the conclusions reached in this study and gives suggestions for further development and future research investigations.

CHAPTER 2

LITERATURE REVIEW

The topic of seismic assessment of existing and designed reinforced concrete frame structures and the investigation of seismic response of asymmetric frame systems is a topic that has gathered the interest of many researchers. However, few studies have been reported in the literature regarding the assessment of structures under repeated ground motions. In this chapter an overview of a series of studies performed by different authors is presented.

2.1 Ground Motion

Earthquake-induced ground motion seems to be the most unpredictable and has a significant impact on the variability observed in the structural response (Padgett and Desroches, 2007). In fact, ground motions appear random in space and time, due to the inherent complexity of the path that seismically induced waves follow as they travel from the fault-plane source through bedrock and finally through the soil layers to reach the foundation level of a structure (Manolis, 2002).

Chen and Scawthorn (2003) defined near-field as the site within one source dimension of epicentre, where source dimension refer to the width or length of faulting, whichever is shorter, while far-field is as site beyond near-field.

Katsanos et al. (2010) presented a review on currently available methods for selecting and scaling ground motion records, which could be used for dynamic analysis of structural systems in the context of performance-based design. They reviewed and evaluated the codes-based selection criteria beside other selection methods, which are based on specific parameters like magnitude (M) and distance (R), soil profile, strong motion duration and other geophysical/seismological parameters.

Ambraseys and Douglas (2003) stated that strong ground motions from close (near-field) to large magnitude earthquakes are the most severe earthquake loading that structures undergo.

According to Krinitzsky (2002), the earthquake ground motions that ultimately are selected for engineering design depend chiefly on the criticality of a site, structure and the engineering analyses to be performed. The selection of appropriate motions for requirements in design has to consider thresholds at which motions become significant for engineering and to make decisions on specifying appropriate earthquake ground motions for sizes of earthquakes, distances from sources, the structures, sites, and testing to be done.

According to Elghazouli (2009) the ground acceleration time-history frequency content should match the design spectrum beside it is important that earthquake time-histories should be chosen whose time-domain characteristics are appropriate to the regional seismicity and local ground conditions.

There are few studies reported in the literature regarding the multiple earthquake phenomena. Figure (2.1) shows several ground motion repetition recorded by the same station. It shows that the repetition may occur either as a foreshock or an aftershock to the main shock. Also in many cases earthquake repetition may occur as a combination of both fore- and after- shock or sometimes as a series of secondary shocks.

Aftershock is a smaller earthquake that occurs after the main shock in the same area. Aftershocks are usually unpredictable and can be of a large magnitude which could collapse buildings that are damaged from the main shock. Large earthquakes have more and larger aftershocks and the sequences can last for years or even longer. Båth (1979) noted that in many instances the largest earthquake aftershock is about 1.2 less in magnitude than that of the main shock.

Foreshock activity has been detected for about 40 % of all moderate to large earthquakes (National Research Council (U.S.), 2003), and up to 70% for events of M>7.0 (Kayal, 2008). They occur from a matter of minutes to days or even longer before the main shock. However, some large earthquakes show no foreshock activity at all.

Amadio et al., (2003) examined the effect of repeated earthquake ground motions on the nonlinear response of single degree of freedom (SDOF) systems they examined only one natural and two artificial ground motions. Recently, Hatzigeorgiou (2010) examined the influence of multiple earthquakes in numerous (SDOF) systems and found that seismic sequences lead to increased displacement demands in comparison with the 'design earthquake'.

Hatzigeorgiou and Liolios (2010) studied the nonlinear behaviour of RC frames under repeated strong ground motions. They found that the sequences of ground motions have a significant effect on the response and, hence, on the design of reinforced concrete frames. Furthermore, it is concluded that the ductility demands of the sequential ground motions can be accurately estimated using appropriate combinations of the corresponding demands of single ground motions.

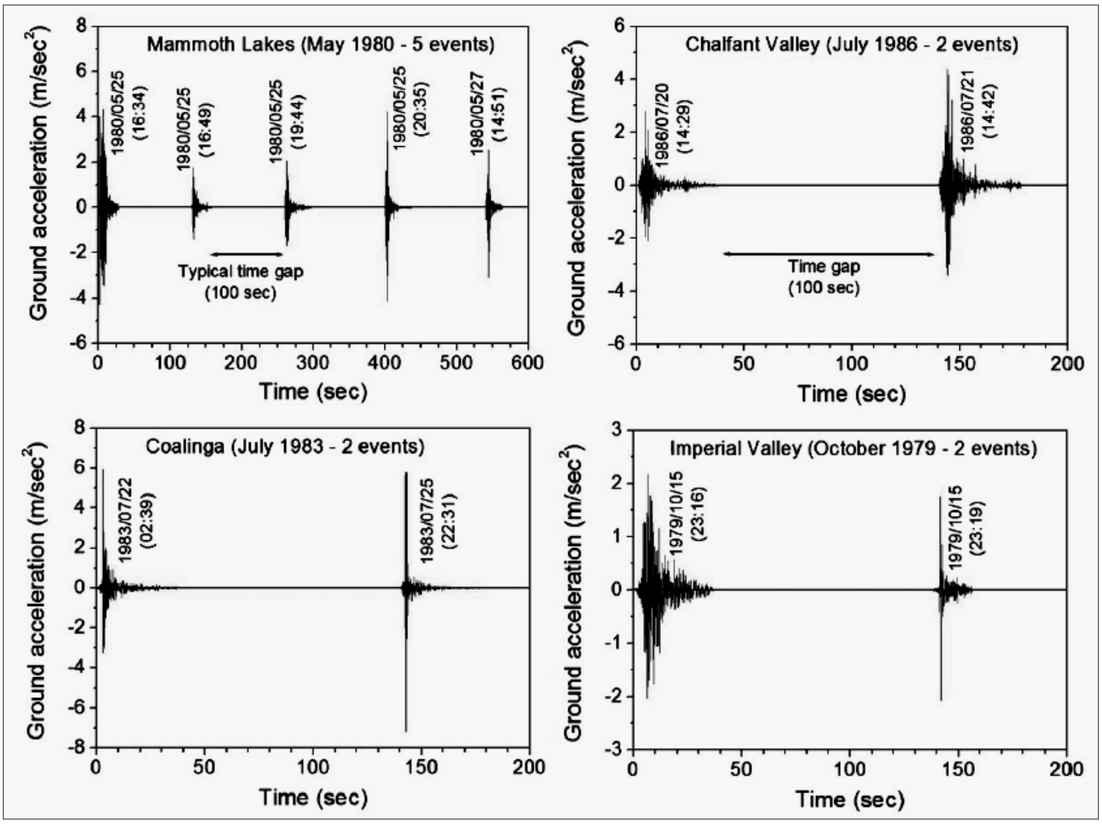

Figure 2.1 Ground motion repetitions (Hatzigeorgiou and Liolios, 2010)

2.2 Nonlinear Analysis

When the load acting on a structure and the resulting deflections are small enough, the load-deflection relationship for the structure is linear. This permits forming the equilibrium equations using the original un-deformed geometry of the structure. However, the equilibrium equations should actually refer to the geometry of the structure after deformation. This type of nonlinearity is known as geometric nonlinearity. Another type of nonlinearity is material nonlinearity. It happened when a material is strained beyond its proportional limit; where the stress-strain relationship is no longer linear. Material nonlinearity may affect the load-deflection behaviour of a structure even when the equilibrium equations for the original geometry are still valid. Simple nonlinear analysis may consider the formation of plastic hinges in the structure by considering material nonlinearity (CSI, 2005).

The modern seismic codes, such as Eurocode 8 and IBC, allow the designer to use different analysis methodologies, in particular: lateral force and multi-modal "elastic" ones and static and dynamic "non-linear" ones. Their level of reliability decreases from the nonlinear dynamic to the elastic lateral force and, consequently, the safety margin with respect to the same limit state should increase according to the same order.

According to Guner (2008), it may be necessary, in some situations, to analyze a structure by considering the nonlinear behaviour to get more accurately predicts of its structural behaviour. Such an analysis may be required for:

- Strength, safety and integrity assessment of a damaged or deteriorated structures, or structures which were designed and built 20 to 30 years ago based on previous codes, standards or practices considered deficient today.

- Performance assessment of planned structures,
- Accurate assessment of large, a typical or unique structures such as nuclear containment structures and offshore platforms,
- Assessing the expected behaviour of retrofitted structures,
- Investigating and selecting a rational retrofit or repair alternative among several alternatives,
- Addressing questions or problems that arise after construction of a new building, or due to the change of use or function of the existing structure,
- Forensic analyses in cases of structural failure or collapse.

For these cases, structural engineers may need to assess the maximum load capacity, ultimate displacement capacity, ductility, deficient members/parts and failure mechanism of the structure. Such an analysis can be performed using nonlinear analysis procedures which typically require specialized software.

2.2.1 Lumped Nonlinearity Models

According to Guner (2008), the nonlinear behaviour of reinforced concrete frames tends to be concentrated at the ends of beams or columns in the case of seismic loading conditions and at the mid-spans in the case of static loading conditions. Therefore, an early means of modelling this behaviour was through the use of zero length plastic hinges as nonlinear springs located at the critical locations and connected by linear-elastic elements. Depending on the formulation, these models may incorporate a number of springs connected in series or in parallel.

Clough and Johnston (1966) introduced the earliest parallel component model allowing for a bilinear moment-rotation (M-φ) relation. As depicted in Figure 2.2(a), this element consists of two parallel elements: one elastic-perfectly plastic to simulate yielding and the other perfectly elastic to represent strain-hardening.

Giberson (1967) formally introduced the series model although it had been reportedly used earlier. As shown in Figure 2.2(b), this model consists of a linear-elastic element with one equivalent nonlinear rotational spring attached to each end in which the inelastic deformations of the member are lumped. This model is more versatile than the original Clough model because more complex hysteretic behaviour can be described.

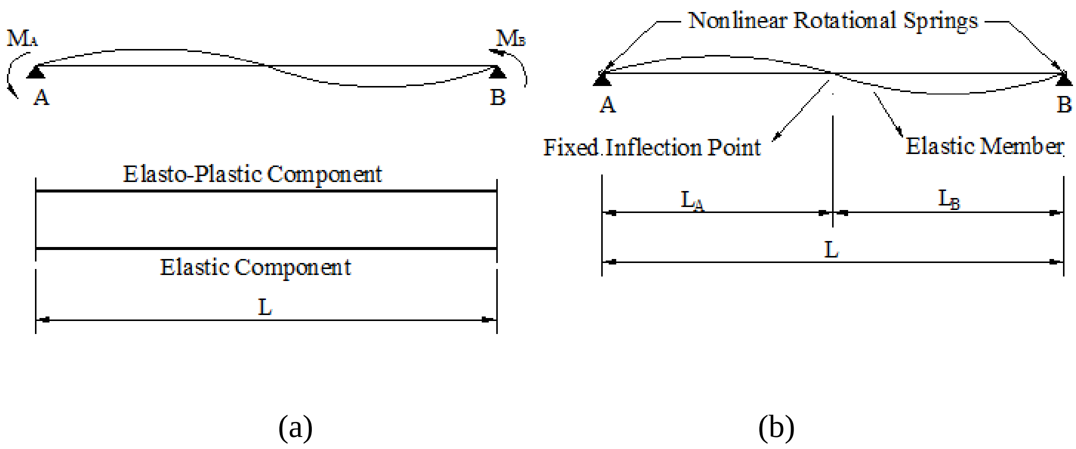

Figure 2.2 Lumped plasticity elements: (a) Parallel model (Clough and Johnston, 1966); (b) Series model (Giberson, 1967) (Figure adopted from Taucer et al. (1991))

Several lumped plasticity constitutive models have been proposed to date. Such models include cyclic stiffness degradation in flexure and shear, pinching under reversal and fixed-end rotations at the beam-column joint interface due to bar pull-out. More details about the other plasticity models and their limitations can be found in Taucer et al. (1991).

Although practical and computationally effective, oversimplification of certain important aspects of hysteretic behaviour of reinforced concrete limits the applicability of the lumped plasticity models proposed to date. Some of the limitations are:

(1) Their inability to consider gradual spread of inelastic deformations into the member as a function of loading.

(2) Their restrictive assumptions for the determination of the spring parameters prior to the analysis.

(3) Their inability to adequately consider the deformation softening behaviour typical of reinforced concrete members.

(4) Their applicability to only well-detailed flexure-critical members with large inelastic deformation capacity at the critical regions.

2.2.2 Nonlinear Time History Analysis

Nonlinear time–history analysis is a powerful tool for the study of structural seismic response. However, there are still some reservations about the dynamic nonlinear analysis, which are mainly related to its complexity and suitability for practical design applications (Mwafy and Elnashai, 2001).

The time-history is specified as a series of data points at time intervals of the order of 0.01s, and the analysis is performed using a stepwise procedure usually referred to as direct integration (Elghazouli, 2009).

In this approach a nonlinear model of structure is analyzed under a ground acceleration time-history. The time-dependent response of the structure may be obtained through direct numerical integration of its differential equations of motion, using the accelerograms to represent the ground motions.

When using this approach, a set of carefully selected ground motion records can give an accurate evaluation of the anticipated seismic performance of structures because of the sensitivity of the outcome to the choice of input ground motions. Therefore, the response should be obtained from at least 7 nonlinear time-history analyses (Eurocode 8-1, 2004).

2.2.3 Nonlinear Static Analysis

Nonlinear static analysis commonly known as Pushover Analysis, this analysis is carried out under conditions of constant gravity loads and monotonically increasing horizontal loads, while the increasing lateral loading applied on the masses of the structural model. This type of analysis is applied to verify the structural performance of newly designed and existing buildings. “Pushover” analysis is essentially the extension of the “lateral force procedure” of static analysis into the nonlinear regime (Eurocode 8-1, 2004).

The nonlinear static pushover analysis is a simple option for estimating the strength capacity in the post-elastic range. It may be also used to highlight potential weak areas in the structure.

Unless the structure is symmetric about an axis at right angles to the seismic action component considered, the lateral forces should be applied in both the positive and the negative direction (Fardis, 2009).

According to Eurocode 8-1 (2004), pushover analyses should be applied to buildings using both of the following lateral load patterns:

1. A “modal pattern”, simulating the inertia forces of the 1st mode in the horizontal direction in which the analysis is carried out.
2. A “uniform pattern”, corresponding to uniform unidirectional lateral accelerations. It attempts to simulate the inertia forces in a potential soft-storey mechanism, with the lateral drifts concentrated there and the storeys above moving laterally almost as a rigid body.

2.3 Asymmetric Building

Real structures are almost always irregular as perfect regularity is an idealization that very rarely occurs. There are two types of irregularities, in plan and in elevation. Plan irregularity occurs as a result of the asymmetric distribution of mass, stiffness and strength.

When excited by a lateral ground motion, asymmetric-plan buildings experience irregular coupled translation–torsion motions. Such type of seismic response, producing a non-uniform inelastic demand among the resisting elements of the structure, makes buildings with in-plan non-symmetric strength and stiffness distributions extremely vulnerable to damage under earthquake loads (Lucchini et al., 2009).

Stefano and Pintucchi (2008) stated that, although single-storey models represent the most extreme idealization of asymmetric buildings, single-storey models have been widely used due to their capability of clarifying the influence of the governing parameters and derive effective design criteria. However, multi-storey building models have been used to study more realistic nonlinear earthquake response of asymmetric buildings. Nevertheless, due to their complexity, such models are applicable to the study of few cases of real buildings.

Peruš and Fajfar (2002, 2005) tackled an issue of a general nature, such as the effects of plastic deformations on torsional response in comparison with the corresponding elastic response. Their studies were conducted by means of single-storey models with bi-axial eccentricity without any code-design restrictions.

The major findings can be summarized as follows: from a qualitative point of view, global torsional effects in inelastic structures are similar to the elastic ones, since differences between elastic and inelastic response are more pronounced in the translational part of motion, rather than in the rotational one. Nevertheless, the inelastic torsional response was found to be strongly dependent on the characteristics of the seismic input and affected by greater dispersion than in the elastic range of behaviour.

Generally, Peruš and Fajfar (2002) found a decrease for flexible structures and an amplification for stiff structures (short periods range), according to the shape of response spectrum.

Stefano and Pintucchi (2002) proposed a single-storey model that takes into account the effects of inelastic interaction between axial force and bi-directional horizontal forces in resisting elements. The influence of such effects on torsional response was evaluated for torsionally-stiff systems under two-component earthquake excitations. The authors concluded that previous models of plan asymmetric structures, which make no allowance for interaction phenomena, generally overestimate torsional response; in fact, inelastic interaction phenomena result in a reduction of floor rotation ranging between 20% and 30%, except for short periods.

Marusic and Fajfar (2005) investigated the elastic and inelastic seismic response of plan-asymmetric regular multi-storey steel-frame buildings under bi-directional horizontal ground motions. Symmetric variants of these buildings were designed according to Eurocodes 3 and 8.

Their findings were: The displacement in the mass centre of a plan-asymmetric building is roughly equal to that of the corresponding symmetric building. The amplification of displacements determined by elastic analysis can be used as a rough estimate also in the inelastic range. Any reduction of displacements on the stiff side of torsionally stiff structures compared to the counterpart symmetric building, which may arise from elastic analysis, may disappear in the inelastic range.

According to Peruš and Fajfar (2005), results evidenced a qualitatively similar elastic and inelastic response, with the exception of the stiff edge in the direction undergoing lower plastic deformations in torsionally stiff buildings, and, the same edge, though in the weaker direction, in torsionally flexible ones.

Lucchini et al. (2009) identified the critical parameters that influence the nonlinear seismic response of asymmetric-plan buildings under uni-excitation. They concluded that, with the increase of the earthquake intensity, the maximum displacement demand in the different resisting elements tends to be reached with the same deformed configuration of the system. Also, the resultant of the seismic forces producing such maximum demand is located at the centre of resistances CR, centre of the elements resistances corresponding to the collapse mechanism of the system that provides the maximum lateral strength in the exciting direction of the seismic action.

2.4 Summary

From the literature review, many studies tackled the issue of linear and nonlinear torsional responds under uni- and bi- directional earthquake excitations for single storey systems, however, asymmetric multi-storey structures is a topic that recently has gathered the interest of researchers. Moreover, there is a lack of studies about nonlinear behaviour of asymmetric buildings under repeated earthquake ground motions.

CHAPTER 3

METHODOLOGY

3.1 Introduction

This chapter explains the procedures and approaches used to perform nonlinear analysis for a four storey asymmetric reinforced concrete building in order to achieve a coherent understanding of the nonlinear behaviour under single and repeated earthquake ground motions. The building considered in this study is a four storey RC building with uni-direction eccentricity. The details regarding the analyzed model are provided in the next section.

The sequence of this study is illustrated in Figure (3.1). First of all, literature review is carried out to gain knowledge and information from related studies. Then building modelling steps, these steps consist of developing the basic model by following the information provided in Dolšek (2010) and Dolšek (2008) and running section analysis to determine plastic hinges properties by using CUMBIA program. Then, suitable ground motions are selected from PEER database, scaled and assembled to simulate the action of single and repeated ground motions. Finally, for analysis purposes, two software programs were used: SAP2000 (CSI, 2000) and RUAUMOKO (Carr, 2007) to perform nonlinear analysis. The results from the nonlinear analyses are presented with discussion in Chapter 4. The conclusions of this study are presented in Chapter 5.

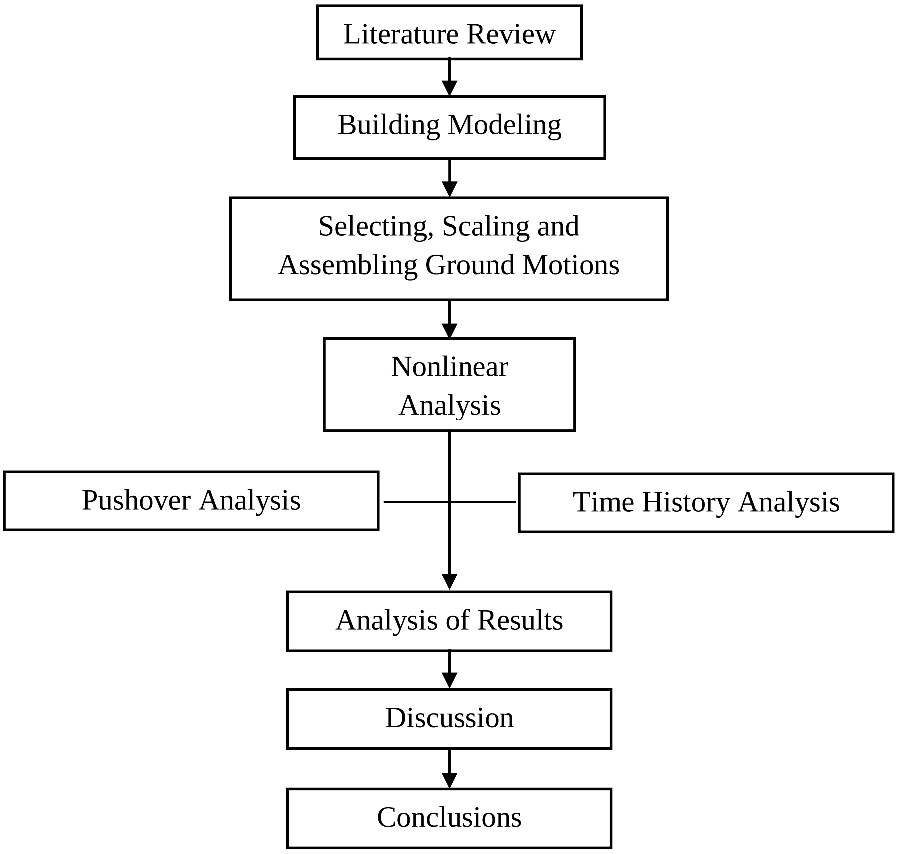

Figure 3.1 Summary of methodology

3.2 Building Modelling

The building model is developed by following the information provided in Dolšek (2010) and Dolšek (2008).

3.2.1 Introduction

A series of pseudo-dynamic tests was conducted on a full scale four-storey reinforced concrete building by European Laboratory for Structural Assessment (ELSA) in Ispra. The first test was performed on a bare frame, then the same input motion was applied to the structure with a uniform configuration of infills (Figure 3.2), and to the structure with a soft-storey infill pattern.

This study considers the bare frame, more information regarding the analyzed structure is introduced in the next section.

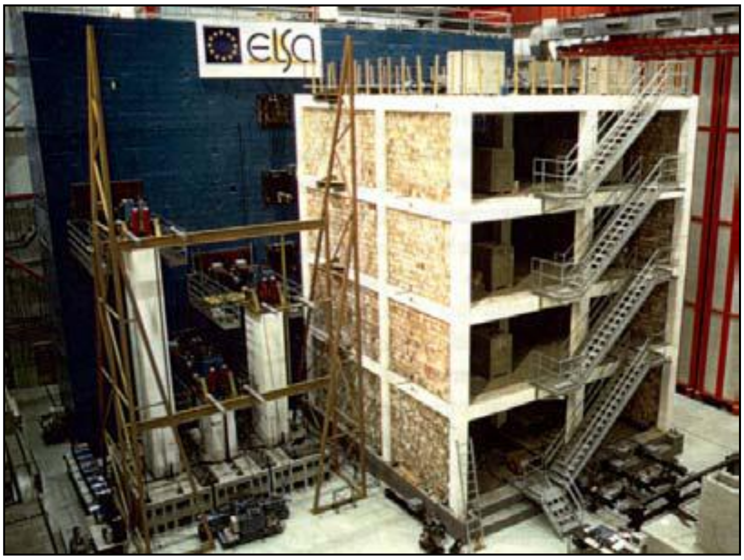

Figure 3.2 The tested frame with masonry infills (Dolsek, 2008)

According to Dolšek (2008), the accelerogram used in the tests is generated from the real accelerogram recorded during the 1976 Friuli Earthquake. The accelerogram and the corresponding spectrum are presented in Figure 3.3, which shows the acceleration spectrum shape approximately corresponds to the EC 8 shape of spectrum and normalized to peak ground acceleration of 0.3 g. The scale factors 0.4 (0.12 g) and 1.5 (0.45 g) for the acceleorgram were used for the low- and high-tests, respectively, and the zero viscous damping was assumed in both tests.

After the low-level test no visible damage were observed. It was assumed that structure become practically in the elastic region. During the high-level test cracks opened and closed in the critical regions of the beams of the first three stories and of most columns. Neither spalling of the concrete cover nor local buckling of reinforcement was observed.

Besides the cracks at the end of beams and columns, which were considered as evidence of yielding in the rebars and of bond-slip in the joints, the specimen remained quite undamaged. However, the fundamental period of the building after the high level test was about 1.22, which is about two times higher than the period measured on the undamaged building (0.56 s).

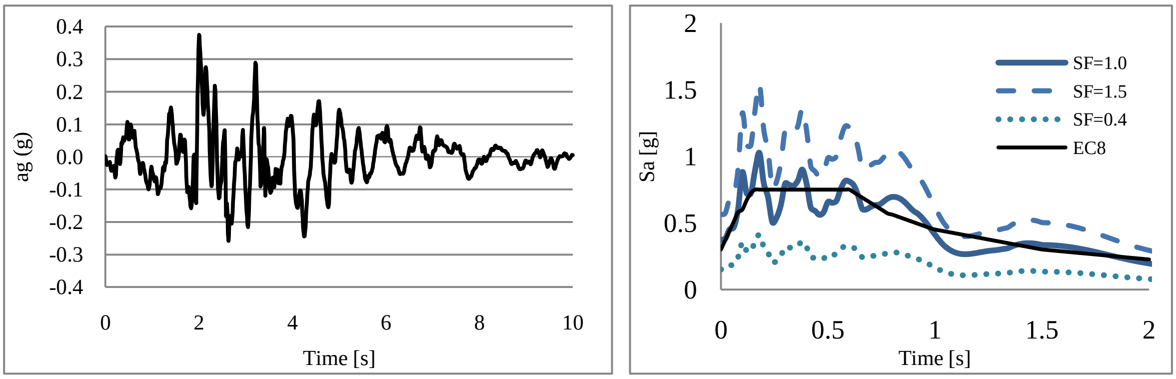

Figure 3.3 The accelerogram used in the pseudo-dynamic test and the corresponding elastic acceleration spectrum compared with EC8 spectrum (Dolsek, 2008)

3.2.2 Building Description

The structure was designed according to previous versions of Eurocodes 2 and Eurocode 8 (Fardis, 1996). In addition to the self weight of the structure 2 KN/m^2 of permanent load was assumed in order to represent floor finishing and partitions, and 2 KN/m^2 of live load was also adopted. The building masses are 87 tons, 86 tons and 83 tons for bottom, second and third, and top storey, respectively. These masses were also taken into account in the pseudo-dynamic tests on the full scale specimen (Negro et al., 1996).

The design base shear versus the weight of the structure corresponded to about 16%, since the design base shear was 529 KN (Fardis, 1996). The design spectrum was defined based on the prescribed peak ground acceleration of 0.3 g, the soil type B, the ductility class high (DCH) and the behavior factor q=5 (Figure 3.4).

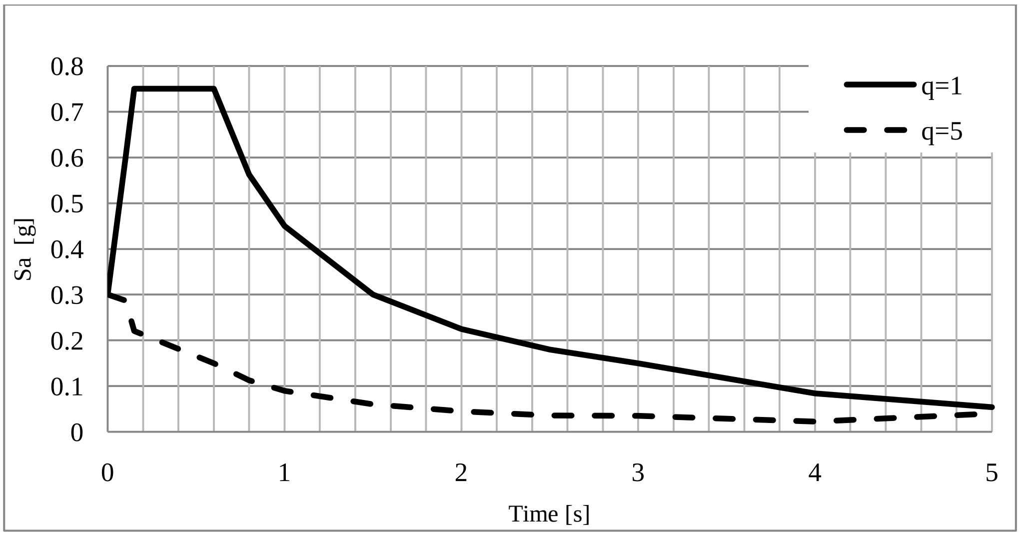

Figure 3.4 EC8: Type 1 elastic response spectra for ground types B (5% damping), ag=0.3g (Eurocode 8-1, 2005).

Figure (3.5) presents the elevation and plan of the four-storey reinforced concrete building, as well as the typical reinforcement of columns and beams. The height of the bottom storey is 3.5 m. In other stories the height is reduced by 0.5 m. The building has two bays in each direction with 5 meters in the X direction and 4 and 6 meters in Y direction, which is the direction of loading in the pseudo-dynamic test. All columns are 40/40 cm except column D which is 45/45 cm and all beams have rectangular cross section with 30 cm width and 45 cm height and the slab has the thickness of 15 cm.

Concrete C25/30 is used for this building beside B500 Tempcore reinforcing steel for which the characteristic yield strength is 500 MPa. However, since the pseudo-dynamic test was performed for the studied building more information regarding material characteristics is available (Tables 3.1 and 3.2).

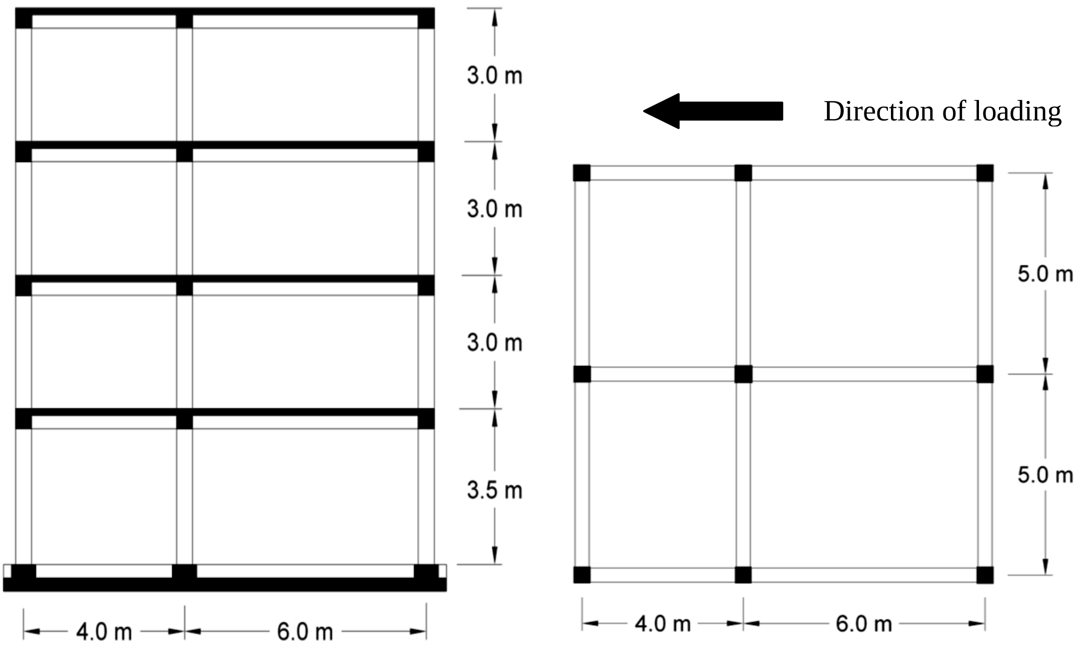

Figure 3.5 The four-storey reinforced concrete frame building (Dolsek, 2008)

Table (3.1) presents the mean concrete strength and modulus of elasticity. The mean concrete strength differs from 32 MPa to 56 MPa. The smallest strength corresponds to columns in third storey and the highest concrete strength corresponds to beams in first storey. Similarly, the modulus of elasticity varies from 28.5 GPa to 35.3 GPa. It should be emphasized that the material characteristics of concrete significantly differs from the nominal material characteristics for C25/30, which are, according to Eurocode 2 (2004), 33 MPa for mean concrete strength and 31 GPa for modulus of elasticity.