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## THESIS ABSTRACT

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# SEISMIC BEHAVIOR AND ANALYSIS OF IRREGULAR RC STRUCTURES

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## Contents

|                                                                                                           |    |
|-----------------------------------------------------------------------------------------------------------|----|
| Preamble.....                                                                                             | 4  |
| Thesis content in brief.....                                                                              | 5  |
| 1 CHAPTER ONE .....                                                                                       | 7  |
| 1.1 Definition of irregular structures according to different codes .....                                 | 7  |
| 1.2 Analysis methods for the seismic performance assessment of irregular structures .....                 | 9  |
| CHAPTER TWO.....                                                                                          | 10 |
| 2 STATE OF THE ART.....                                                                                   | 10 |
| CHAPTER THREE .....                                                                                       | 12 |
| 3 The seismic response of reinforced concrete structures with plan irregularity .....                     | 12 |
| 3.1 General .....                                                                                         | 12 |
| 3.2 Description of the building model .....                                                               | 12 |
| 3.3 Structural regularity .....                                                                           | 14 |
| 3.4 Finite element modelling for nonlinear analysis .....                                                 | 14 |
| 3.5 Determining the performance level and the vulnerability .....                                         | 15 |
| 3.6 Discussion of the analysis results .....                                                              | 16 |
| 3.7 Earthquake incidence angle impact on the seismic performance of the RC plan asymmetric buildings..... | 19 |
| 3.8 Influence of the incidence angle.....                                                                 | 19 |
| 3.9 Discussion of the analysis results .....                                                              | 20 |
| CHAPTER FOUR .....                                                                                        | 22 |
| 4 The seismic response of reinforced concrete structures with vertical irregularity .....                 | 22 |
| 4.1 General .....                                                                                         | 22 |
| 4.2 Description of the studied buildings .....                                                            | 23 |
| 4.3 Vertical geometric irregularity .....                                                                 | 24 |
| 4.4 Quantification of setback irregularity.....                                                           | 25 |
| 4.5 Modeling members nonlinearity.....                                                                    | 26 |
| 4.6 Nonlinear static analysis procedure (NSP) .....                                                       | 26 |
| 4.7 Nonlinear dynamic analysis procedure (NDP).....                                                       | 26 |
| 4.8 Results and discussion of structural behavior .....                                                   | 27 |
| 4.8.1 Nonlinear static analysis procedure (NSP) .....                                                     | 27 |
| 4.8.2 Nonlinear dynamic analysis procedure (NDP).....                                                     | 28 |
| 4.9 Determining the performance level and vulnerability .....                                             | 29 |
| 4.9.1 Development of fragility curves.....                                                                | 32 |
| 4.9.2 Performance of the structural members .....                                                         | 36 |
| CHAPTER FIVE .....                                                                                        | 37 |
| 5 Conclusions.....                                                                                        | 37 |
| 5.1 Conclusions from the first case study .....                                                           | 37 |

|       |                                                                                                                       |    |
|-------|-----------------------------------------------------------------------------------------------------------------------|----|
| 5.1.1 | General conclusions.....                                                                                              | 37 |
| 5.1.2 | Conclusions on earthquake incidence angle impact on the seismic performance of the RC plan-asymmetric buildings ..... | 38 |
| 5.2   | Conclusions from the second case study.....                                                                           | 39 |
| 6     | Contributions to the study of the field.....                                                                          | 40 |

## PREAMBLE

The trend towards building irregular structures has increased in recent decades as a result of architectural and aesthetic requirements, as well as due to the limited availability of areas for construction. Many common types of reinforced concrete construction failures are related to structural irregularity. Depending on the design, the structural irregularities that affect the structural behavior or the resistance to static and dynamic actions differ widely. In design, the reference source in the definition of these irregularities according to the geometric configuration and dynamic behavior are the earthquake codes. The provisions of Eurocode 8 [2] and ASCE/SEI 7-16[2] are largely used. While the new Iraqi seismic code, ISC 2016, is used in Iraq [3], this is based mainly on the International Building Code, IBC 2012[4], and on ASCE/SEI 7-10[5]. Previous research has shown that structures with irregular configurations are more vulnerable to strong ground motions than regular structures. The reliability of the seismic response assessment for buildings with irregularities becomes more complicated due to the shift of the building behavior from the linear to the nonlinear range, and to the continuous changes of stiffness and strength distributions during the earthquake. A superior method for evaluating the nonlinear behavior is the nonlinear static approach (pushover) analysis, meant for the seismic evaluation of structures that primarily have a translational response. In addition to the pushover approach, the nonlinear dynamic analysis is used. This is the most accurate analysis method, of particular utility also in the case of structures with plan irregularity.

The objective of this thesis is to perform an analysis of the various parameters and characteristics (internal forces, period, inter-story drift, vulnerability index, ductility factor, over-strength factor, behavior factor, fragility curves) of reinforced concrete structures with vertical and horizontal irregularity and to assess their influence on the seismic performance. In addition, the thesis aims to study the effects of irregularities in elevation and in plan on the response of structural systems, by comparing the results provided by different methods (response spectrum analysis, static nonlinear analysis and dynamic nonlinear analysis) and to investigate if the current methods and approaches used in codes and regulations are suitable and can reflect accurately the seismic performance of irregular buildings.

Therefore, two categories of analytical case studies were performed. The first one concerned a nine-story reinforced concrete dual system (wall-frame) building located in Bucharest, categorized as plan-asymmetric and designed according to the provisions of Eurocode 8 [1] and Eurocode 2 [6], by applying elastic response spectrum analysis. The verification of the criteria for regularity in plan was discussed according to the codes Eurocode 8-1:2004 [1], ASCE 7-16 [2], and P100-1/2013 [7]. The seismic behaviour of the studied building was investigated by the use of the nonlinear static analysis (pushover) method, performed on the 3-D building model. Next, the results were compared to those obtained by nonlinear time-history analysis (THA). The results concerning the structural (global and local) response were analyzed according to Eurocode 8. Subsequently, this study estimated the critical incidence angle of the studied building, considering the Limitation of Inter-storey Drift (LID), the Maximum Inter-story Drift (MLID) along the height of the building and the Seismic Vulnerability Index (SVI) of the building. The influence of directionality on the response of the building was investigated by non-linear dynamic analyses, by applying seven scaled bi-directional ground motion records oriented about 8-incidence angles with values ranging from  $0^\circ$  to  $315^\circ$ , with a  $45^\circ$  increment. For the assessment of the seismic directionality influences, two demand parameters were combined, i.e. the MLID along the height of the building and the estimated SVI, in order to predict the most critical incidence angle. From the results, it was found that the maximum responses to individual ground motions can occur at any angle of incidence, which is not necessarily at  $0^\circ$  or  $90^\circ$ , and that the responses highly depend on the signal characteristics, not only on the structure's features. The second category of case studies focused on four six-story reinforced concrete buildings, with different setback configurations and designed for locations in Baghdad, Iraq, which were studied in order to assess their

seismic vulnerability. The buildings, with masonry infills at the first two stories, categorized as vertically irregular, were dimensioned according to the provisions of the Iraqi Seismic Code, ASCE 7-16 [2] and ACI 318-19 [8]. Nonlinear static (pushover) analyses were conducted to compute the capacity curves, then sets of fragility curves were developed in order to estimate the seismic damage probability, in terms of spectral displacements. The target displacement was identified on the pushover curve by utilizing the modified coefficient method in FEMA 440 [9], adopted in ASCE/SEI 41-13 [10], and then the idealized force-displacement curve was established, to obtain the yield point of the models. The pushover curve was developed for three lateral load distributions: the equivalent lateral force distribution, the uniform pattern and the first mode pattern, to take into account various possible actions that may occur during the actual seismic response and to identify the worst case, which will be the one governing the subsequent computations. The setback degree of the studied models was assessed to take into account its influence in increasing the local damage hazard. From the results it could be noticed that, as the irregularity setback level increases, the probability of damage hazard rises and the models exhibit poorer seismic performance. The fragility curves determined in this study could be represent a preliminary basis in establishing seismic risk scenarios for Baghdad, for vertically irregular buildings with setbacks. However, further processing of these curves is considered necessary, to account for the potential variation of input parameters selected for the nonlinear analysis, damage state thresholds and the assumptions used for the determination of fragility curves for each of the reference damage states.

## **THESIS CONTENT IN BRIEF**

The first chapter of the thesis presents general aspects on irregular buildings, categories of irregular structures according to different codes and concentrates on the utilized analysis methods and objectives of the current thesis. Some existing irregular structures in different parts of the world are also presented.

The second chapter presents the scientific literature regarding existing irregular buildings, some studies of buildings with plan irregularities (asymmetric one-story buildings and asymmetric multistory buildings) and, respectively, vertical irregularities, also highlighting the seismicity of Iraq. The third chapter encompasses the application of the elastic (response spectrum "RSA") and inelastic (pushover N2 and nonlinear time history "THA") analysis for the first case study - a plan irregular building located in Bucharest, categorized as being plan-asymmetric and designed for the earthquake-prone zone of Bucharest by using the Romanian and European codes (P100-1/2013, EN 1992-1-1:2004 and EN 1998-1:2004). Since the N2 analysis requires the structure to be designed first, this was made using the response spectrum analysis, RSA, according to Eurocode 8 (EN 1998-1:2004) for high ductility class (DCH), in order to compute the seismic force to be used in the load combinations for seismic design. The design and detailing of the model were performed according to Eurocode 8 and Eurocode 2. Then, the results of the response spectrum analysis were compared with those obtained from the application of the N2 method and of THA. Based on results of the linear and nonlinear analyses, various parameters were investigated, regarding the compliance with the criteria specified by the considered codes for plan-irregular buildings, such as the effects of the higher modes on inter-storey drifts and roof displacement, base shear force and seismic vulnerability index (SVI). The third chapter also investigates the impact of the incidence angle of the ground motion by time-history analyses performed on the studied building, which was subjected to different scaled bi-directional ground motion records. The PEER database was used to find the best matching earthquake records, which were scaled for a peak ground acceleration  $PGA = 0.30\text{ g}$ , the elastic response spectrum, specified for Bucharest according to P100-1/2013. The accelerograms were applied about eight incidence angles. Namely, each ground motion record was applied at angles ranging from  $0^\circ$  to  $315^\circ$ , with increments of  $45^\circ$ . Two demand parameters were combined, i.e. the maximum inter-story drift

(MLID) along the height of the building and the estimated Seismic Vulnerability Index (SVI), in order to predict the most critical incidence angle of the seismic action.

The combination of the two demand parameters showed the importance of the evaluation the influence of the Flexible Edge (FE) and of the Stiff Edge (SE) on the structural demand. In addition, the difficulty of correctly predicting the critical angles of incidence when designing an irregular building was highlighted, given that the building response does not depend only on the characteristics of the building, but also on those of the ground motion. This was obvious from the different critical angles at which the highest SVI and MLID values occurred for the different ground motions considered.

The fourth chapter of the thesis encompasses the second case study, focused on vertical irregular buildings. In this chapter, four different setback models (considered as located in Baghdad, Iraq) were modeled. Nonlinear static analysis (NSP) and nonlinear dynamic analyses (NDP) were conducted according to ASCE/SEI 41-13, after designing these models according to ACI 318-19 (the standard code adopted for reinforced concrete design in Iraq), the Iraqi seismic code, ISC 2014, and ASCE 7-16. The aims of the study were to assess the applicability of the two-stage equivalent lateral force analysis for structures that have a flexible upper portion over a rigid lower portion (method adopted in ISC 2014 and ASCE 7-16 codes) and to verify the accuracy of the results (verification of IDRa, the average inter-story drift ratio and the shear ratio of the columns, i.e. the shear demand on the column,  $V$ , to the shear strength  $V_n$ ) obtained from this approach, as compared with those obtained from the nonlinear static and nonlinear dynamic analysis procedures.

In addition, the local vulnerability indices ( $VI_{Fi}$ ) were assessed. The severity or degree of models setbacks also influences the increase in damage, so the setback ratios were studied to take into account their influence on the increase of damage hazard, where the abrupt changes in stiffness or the irregular vertical configurations of the structures were considered local vulnerability locations. The fourth chapter adds also a contribution to the assessment of the seismic vulnerability of the studied buildings, in which the fragility curves developed based on nonlinear static analysis procedure (pushover) were determined in order to estimate the seismic damage probability in terms of spectral displacements. The fragility curves developed in this study could be used as preliminary investigation in seismic risk scenarios for Iraq (Baghdad), for irregular setback buildings.

The fifth chapter presents the conclusions of the thesis.

# 1 CHAPTER ONE

## 1.1 Definition of irregular structures according to different codes

For seismic design, building structures are categorized as regular or irregular, based on their structural configurations. Depending on the design, the structural irregularities that influence the structure's behavior under static and dynamic loads, as well as its resistance to these loads, vary substantially. Earthquake codes provide the reference source for defining these irregularities based on geometric considerations and accompanying dynamic computations. Tables 1-1 and 1-2 synthesize criteria for horizontal and vertical irregularities, according to various codes.

Table 1-1: Irregularity limits prescribed by UBC 97, NBCC 2005 and P100-1/2013

| Type of Irregularity       | UBC 97 [11]                                                                                                             | NBCC 2005 [12]                                                                                         | P100-1/2013 [7]                                                                                  |
|----------------------------|-------------------------------------------------------------------------------------------------------------------------|--------------------------------------------------------------------------------------------------------|--------------------------------------------------------------------------------------------------|
| <b>Horizontal</b>          |                                                                                                                         |                                                                                                        |                                                                                                  |
| a) Torsional Irregularity  | $d_{\max} \leq 1.2 d_{\text{avg}}$                                                                                      | $d_{\max} \leq 1.7 d_{\text{avg}}$                                                                     | $d_{\max} \leq 35\% d_{\text{avg}}$                                                              |
| b) Diaphragm Discontinuity | $O_A > 50\%$<br>$S_{\text{dst}} > 50\%$                                                                                 | -                                                                                                      | -                                                                                                |
| c) Re-entrant Corners      | $R_i \leq 15\%$                                                                                                         | -                                                                                                      | $R_i \leq 10\%$                                                                                  |
| d) Out-of-Plane Offsets    | Discontinuities in the direction of lateral force, such as the vertical elements out-of-plane offsets.                  | Discontinuities in the direction of lateral force, such as the vertical elements out-of-plane offsets. | -                                                                                                |
| <b>Vertical</b>            |                                                                                                                         |                                                                                                        |                                                                                                  |
| a) Mass                    | $M_i > 1.5 M_a$                                                                                                         | $M_i > 1.5 M_a$                                                                                        | Mass, $M_i > 1.5 M_a$                                                                            |
| b) Stiffness               | $S_i < 0.7 S_{i+1}$<br>or<br>$S_i < 0.8(S_{i+1} + S_{i+2} + S_{i+3})$                                                   | $S_i < 0.7 S_{i+1}$<br>or<br>$S_i < 0.8(S_{i+1} + S_{i+2} + S_{i+3})$                                  | Stiffness, $S_i < 0.7 S_{i+1}$                                                                   |
| c) Soft Storey             | $S_i < 0.7 S_{i+1}$<br>or<br>$S_i < 0.8(S_{i+1} + S_{i+2} + S_{i+3})$                                                   | $S_i < S_{i+1}$                                                                                        | -                                                                                                |
| d) Weak Storey             | $S_i < 0.8 S_{i+1}$                                                                                                     | -                                                                                                      | -                                                                                                |
| e) Setback irregularity    | $SB_i < 1.3 SB_a$                                                                                                       | $SB_i < 1.3 SB_a$                                                                                      | $SB_i < 0.2$ of the previous storey plan dimension in the direction of the setback               |
| f) In-plane discontinuity  | The offset of an element of the lateral-load-resisting elements should be not greater than the length of those elements | Should not decrease in lateral stiffness of an element in the below story                              | Structure should not present vertical discontinuities that deviate the force path to foundations |

Table 1- 2: Irregularity limits prescribed by EC8-1:2004, ASCE 7-16, and IBC 2003

| Type of Irregularity       | EC8-1:2004 [1]                                                                                                                                                                                                                                                                                                                                                                                                                     | ASCE 7-16 [2]                                                         | IBC 2003 [4]                                                          |
|----------------------------|------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|-----------------------------------------------------------------------|-----------------------------------------------------------------------|
| <b>Horizontal</b>          |                                                                                                                                                                                                                                                                                                                                                                                                                                    |                                                                       |                                                                       |
| e) Torsional Irregularity  | $e_{ox} \leq 0.3 r_x$<br>$e_{oy} \leq 0.3 r_y$<br>$r_x$ and $r_y \geq l_s$ ,                                                                                                                                                                                                                                                                                                                                                       | $d_{max} \leq 1.2 d_{avg}$<br>$d_{max} \leq 1.4 d_{avg}$              | -                                                                     |
| f) Diaphragm Discontinuity | $r_x > (l_s^2 + e_{ox}^2)^{0.5}$<br>$r_y > (l_s^2 + e_{oy}^2)^{0.5}$                                                                                                                                                                                                                                                                                                                                                               | $O_A > 50\%$<br>$S_{dst} > 50\%$                                      | -                                                                     |
| g) Re-entrant corners      | $R_i \leq 5\%$                                                                                                                                                                                                                                                                                                                                                                                                                     | $R_i \leq 15\%$                                                       | -                                                                     |
| d) Slenderness             | $\lambda = \frac{L_{max}}{L_{min}} \leq 4$                                                                                                                                                                                                                                                                                                                                                                                         | -                                                                     | -                                                                     |
| <b>Vertical</b>            |                                                                                                                                                                                                                                                                                                                                                                                                                                    |                                                                       |                                                                       |
| g) Mass                    | Should not reduce abruptly                                                                                                                                                                                                                                                                                                                                                                                                         | $M_i < 1.5 M_a$                                                       | $M_i < 1.5 M_a$                                                       |
| h) Stiffness               | Should not reduce abruptly                                                                                                                                                                                                                                                                                                                                                                                                         | $S_i < 0.7 S_{i+1}$<br>or<br>$S_i < 0.8(S_{i+1} + S_{i+2} + S_{i+3})$ | $S_i < 0.7 S_{i+1}$<br>or<br>$S_i < 0.8(S_{i+1} + S_{i+2} + S_{i+3})$ |
| i) Soft Storey             | -                                                                                                                                                                                                                                                                                                                                                                                                                                  | $S_i < 0.6 S_{i+1}$<br>or<br>$S_i < 0.7(S_{i+1} + S_{i+2} + S_{i+3})$ | $S_i < 0.7 S_{i+1}$<br>or<br>$S_i < 0.8(S_{i+1} + S_{i+2} + S_{i+3})$ |
| j) Weak Storey             | -                                                                                                                                                                                                                                                                                                                                                                                                                                  | $S_i < 0.8 S_{i+1}$<br>$S_i < 0.65 S_{i+1}$ (extreme)                 | $S_i < S_{i+1}$                                                       |
| k) Setback irregularity    | <ul style="list-style-type: none"> <li><math>SB_i &lt; 0.2</math> of the previous storey plan dimension in the direction of the setback</li> <li><math>SB_i &lt; 0.3</math> plan the dimension of the first storey</li> <li><math>SB_i &lt; 0.5</math> of the previous plan dimension (the lower 15 % of the total height)</li> <li><math>SB_i &lt; 0.1</math> of the previous plan dimension (the individual setbacks)</li> </ul> | $SB_i < 1.3 SB_a$                                                     | $SB_i < 1.3 SB_a$                                                     |

The notations used in the previous tables are the following:

- $e_{ox}, e_{oy}$  - distance between the center of stiffness and the center of mass, measured along the X and Y directions, respectively
- $d_{max}, d_{avg}$  - maximum drift computed at a specific storey level, respectively average of drifts computed at both sides of a structure
- $L_{max}$  and  $L_{min}$  are, respectively, the larger and smaller in plan dimension of the building
- $S_{i+1} + S_{i+2}$  - Stiffness of  $i^{th}$ ,  $i + 1^{th}$  and  $i + 2^{th}$  storey
- $SB_i$  - setback irregularity limits
- $O_A, S_{dst}$  - open area in diaphragm and diaphragm stiffness
- $M_i, M_a$  - mass of  $i^{th}$  storey and the storey adjacent to the  $i^{th}$  storey
- $I_s$  - radius of gyration
- $r_x, r_y$  - torsional radius in X and Y directions
- $R_i$  - re-entrant corner irregularity limit.

## 1.2 Analysis methods for the seismic performance assessment of irregular structures

A large number of researchers have focused their study on plan irregularities. The study of the seismic behavior of torsionally unbalanced structures poses a range of challenging issues from the structural engineering perspective. The seismic torsional response occurs due to a variety of factors, including irregular configuration, but also others, more difficult to predict and quantify. As an example, additional mass eccentricity, causing seismic torsional response, can occur due to asymmetric live load distributions or due to differences between actual and design stiffness and mass distributions. These phenomena are dealt with in codes by specifying an accidental design eccentricity [13]. In a common approach for asymmetric structures, one edge of the structure is called “stiff edge”, because the translation of this edge due to the rotation induced by torsion is less pronounced than that of the other side, the so-called “flexible edge” of the building, which experiences increased displacements due to torsion [14].

Structures having significant physical discontinuities in their vertical configuration or in their lateral force-resisting or bracing systems are classified as vertically irregular. Vertical irregularities are one of the main reasons for failures of building structures during earthquakes, the examples of collapse occurring in soft-story structures being among the most significant. Therefore, the effect of vertical irregularities on the seismic behavior of structures is important and needs additional research, given that the stiffness and mass changes induce different dynamic characteristics as compared to those of regular buildings [15].

Analysis methods are generally classified as linear static, nonlinear static, linear dynamic and nonlinear dynamic methods. The nonlinear static and nonlinear dynamic analysis have improved capabilities to simulate seismic response as compared to the linear approaches. In this case, geometrical nonlinearity and material nonlinearity are considered in the analysis. For irregular buildings located in high seismicity zones, the design and analysis become more difficult, therefore the Modal Response Spectrum Approach, RSA, is considered a more advanced method in comparison with the equivalent lateral force analysis (ELF), since it incorporates the contribution of higher vibration modes and offers a better assessment of the actual force distribution in the elastic range.

Modern seismic design and assessment regulations specify two types of nonlinear analysis methods: (i) the nonlinear static analysis, and (ii) the nonlinear dynamic analysis, which allow engineers to understand structural behavior and damage evolution in structural elements with increasing ground motion intensity.

## CHAPTER TWO

### 2 STATE OF THE ART

The assessments of the performance of buildings during past earthquakes demonstrated that plan irregularity is one of the most common causes of severe damage, given the asymmetric distributions of stiffness, strength, and mass, which results in torsional response.

Using simple one-story models, the torsional effects in asymmetric building systems have been widely studied in the past. These models were considered appropriate to explain the impact of key structural parameters and to evolve design measures applicable to some classes of asymmetric multi-storey buildings as well. Multi-story building models have been used in recent years to research the inelastic seismic response of irregular buildings more realistically. Nevertheless, these models apply to the study of a few cases of real buildings due to their complications. Many researchers use one-story models because they allow them to gather enough general information on the torsional behavior of asymmetric structures.

Previous studies on buildings with plan irregularities focused specifically on the variation of the center of mass (C.M.) location with respect to the center of stiffness (C.S.). The purpose of these studies was to determine, given the eccentricity of C.M. with respect to C.S., the torsional response of buildings. Some researchers varied the C.S. locations to produce eccentricity, keeping the C.M. location constant. The eccentricity produced in this instance was the so-called stiffness eccentricity constant ( $e_s$ ) (Tso and Sadek (1985) [16], Tso and Sadek (1989) [17]).

Other researchers (Ladinovic 2008) [18], changed the locations of C.M. by keeping the locations of C.S. constant. The eccentricity created in this instance is called mass eccentricity ( $e_m$ ). In a different approach, some researchers modified the strengths of resisting elements to vary the place of the center of strength (C.V.). The resulting excentricity is the so-called eccentricity of strength ( $e_v$ ) (Aziminejad and Moghadam (2010) [19], Shakib and Ghasemi (2007) [20]).

One-story models were commonly used in previous analytical researches on plan-asymmetrical buildings due to their simplicity and their capacity to represent the impact of various seismic response parameters. However, several researchers, e.g. Stathi et al. (2015) [21] and Yoon and Smith (1995) [22], proved that one-story models give inaccurate assessments of torsional response. The development of powerful software tools has made it much easier to model and analyze multi-story building models, which provide more realistic torsional response prediction.

Many studies on multi-story buildings were performed by applying pushover analysis to plan-asymmetric buildings. Early research dates to the mid-1990s, with the studies of Kilar and Fajfar (1996) [23] and Tso and Moghadam (1997) [24]. Although studies on plan-asymmetrical structure models started in the 1990s, Fajfar et al. (2002) [25] conducted major research in this field, suggesting a new method that was an extension to 3-D models of the N2 procedure, by the use of a height-wise distribution of lateral forces on the height of the building, applied in the floor centers of mass.

The research works on vertical irregularities started in the early 1970s, e.g., with Chopra (1973) [26] who examined the seismic response of a group of eight-story structures exposed to earthquake ground motions. The author's main objective was to evaluate the influence of yielding of the first story on upper stories. Based on the results of the study, it was found that a low yielding force and an ideal plastic mechanism were needed on the first floor for the protection of the higher floors of the building. As well, e.g. Das and Nau (2003) [27], who assessed the effects of stiffness, strength, and mass

irregularity on the inelastic seismic response of multi-story buildings. D. Van Thuat (2011) [28] who determined the strength demands for a set of irregular structures subjected to severe earthquakes. Sharma and Nasier (2019) [29] who examined the behavior of irregular buildings with a bracing system to the development of new design and construction techniques to assess the performance of the same structure. Aranda (1984) [30] who presented a comparison of the ductility demands between setback and symmetric buildings, using ground motion recorded on soft soil. Moehle and Shahrooz (1990) [31] determined the seismic response of structural systems with vertical setbacks. To refine the methodologies for the design of setback structures, the authors performed both experimental and analytical tests. Naveen E. S. et al. (2019) [32] discussed the seismic response of RC buildings with different irregularities. By integrating irregularities in different types in both elevation and plan, the nine-story regular frame was changed to create 34 configurations with single irregularity.

The first Iraq seismic code was published on 1997 (ISC 1997) [33]. The first edition of the official new Iraqi seismic code, ISC 2016 [5], was issued after the draft version ISC 2013 [34] was published in 2013. The ISC 2016 [3] is based mainly on the IBC 2012 [4] and ASCE/SEI07-10 [2], with local mapping of acceleration parameters  $S_1$  (1.0 sec period) and  $S_s$  (0.2 sec period).

The shape of the response spectrum specified in ISC 2014 is similar to those specified in IBC 2012 and ASCE/SEI7-10, with the exception of the absence of a long-period transition at a large period range, which is required for periods greater than  $T_L$ , as shown in Figure 2-1.

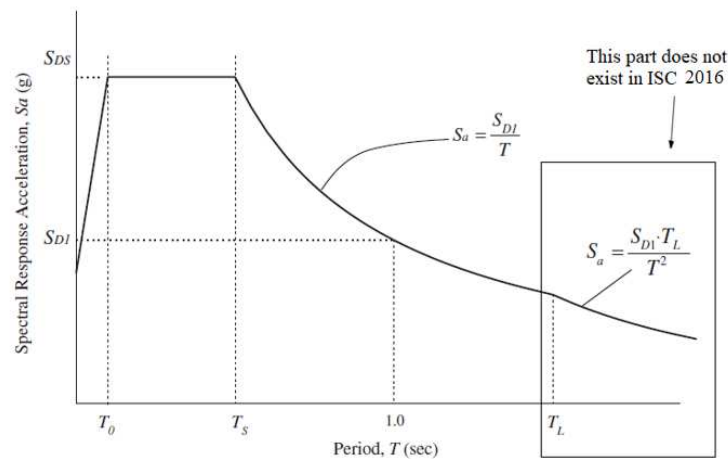


Figure 2-1: Seismic design response spectrum, as defined in ASCE/SEI 7-10 and IBC 2014.

## CHAPTER THREE

### 3 THE SEISMIC RESPONSE OF REINFORCED CONCRETE STRUCTURES WITH PLAN IRREGULARITY

#### 3.1 General

For the presented case study, in-depth linear and nonlinear analyses were conducted on a reinforced concrete (RC) multi-story building, categorized as being a plan-asymmetric and designed for the earthquake-prone zone of Bucharest by using the Romanian and European codes (P100-1/2013, EN 1992-1-1:2004 and EN 1998-1:2004). The nonlinear static method was applied to the building according to Eurocode 8 Part 3 (EN 1998-3:2006) and the results were subsequently compared with those obtained from the nonlinear time-history analysis (THA). Based on results of the linear and nonlinear analyses, various parameters were investigated, regarding the compliance with the criteria specified by the considered codes for plan-irregular buildings, such as the effects of the higher modes on inter-storey drifts and roof displacement, base shear force and seismic vulnerability index (SVI). The present study also investigates the impact of the incidence angle of the ground motion by time history analyses performed on the studied building which subjected to different scaled bi-directional ground motion records (the PEER database was used to find the best matching earthquake records and scaled for a peak ground acceleration  $PGA = 0.30$  g, the elastic response spectrum, specified for Bucharest according to P100-1/2013), oriented along eight incidence angles. Namely, each ground motion record is applied at angles ranging from  $0^\circ$  to  $315^\circ$ , with increments of  $45^\circ$ . Two demand parameters are combined, i.e. the maximum inter-story drift (MLID) along the height of the building and the estimated Seismic Vulnerability Index (SVI), in order to predict the most critical incidence angle of the seismic action.

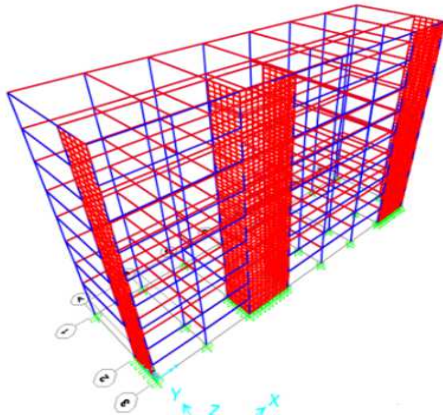
#### 3.2 Description of the building model

The investigated building is a nine-storey reinforced concrete structure located in Bucharest, Romania. The 3D model of the building is shown in Figure 3-1. Two vertical longitudinal and transversal sections through the building and floor plans for the typical floor and ground floor are shown in Figure 3-1. The height of the first and second storeys is 3.0 m, whereas the height of the other storeys is 3.5 m. The total building height is 30.5 m. The building has seven bays in the X-direction and two bays in the Y-direction. The bay widths are 6 m in the X direction and 3.7 m and 8.1 m in the Y-direction. The dimensions of beam cross-sections are 0.85 x 0.35 m for the 8.1 m and 3.7 m spans (transversal direction) and 0.25 x 0.50 m for the 6 m spans (longitudinal direction). The structural system consists of RC shear walls and frames, being categorized as a dual system, according to its conformation. The floor slabs are 0.20 m thick. Concrete class C30/37 was used for walls and columns and C25/30 for beams and slabs. The corresponding moduli of elasticity,  $E_{cm}$ , amount to 33 GPa and 31 GPa, respectively (EN1992-1-1:2004, Table 3.1). Poisson's ratio,  $\nu$ , was taken equal to zero, as prescribed by EN1992-1-1:2004, clause 3.1.3, for cracked concrete. Steel S500 Class C was used. The structure was designed for high ductility class (DCH). The column dimensions are illustrated in Table 3-1. These dimensions were varied in accordance with the building's design. The total wall cross-section area should satisfy the following inequality to reduce the effect of shear force on walls:  $F_b/A_w \leq 2f_{ctd}$ , where:  $A_w$  is the walls total area on the direction of the seismic force and  $f_{ctd}$  is the concrete tensile design strength [35]. It was assumed that all walls have rectangular cross-sections with  $b_w=0.25$  m.

The building is classified as irregular in plan according to Eurocode 8 criteria. The basic behaviour factor,  $q_0$ , is  $4.5 \alpha_u/\alpha_l$  for DCH. For the considered structural type,  $k_w = 1$ , and the factor  $\alpha_u/\alpha_l$  is 1.1, thus the corresponding  $q$  amounts to 4.95. The behaviour factor  $q$  ( $= q_0 k_w \geq 1.5$ ) in both directions is equal to 4.95. The static and dynamic nonlinear analyses were performed with the SAP2000 finite element software [36]. As required by EN1998-1:2005, clause 4.3.2, an accidental mass eccentricity of 5% of the plan dimensions of the building was assumed in both horizontal directions, to account for uncertainty in the position of masses. The investigated building is characterized by an  $\Omega$  ratio (translation to rotation period for the elastic analysis) in the X direction ( $\Omega_x = 1.89$ ) and in the Y direction ( $\Omega_y = 1.59$ ),  $\Omega > 1$ . Consequently, the building was considered torsionally stiff, and the predominant response is the translational mode [37].

Table 3-1. Summary of column sections

| Floor Level  | Section of columns (mm)                |                                        |                                        |
|--------------|----------------------------------------|----------------------------------------|----------------------------------------|
|              | Column Section 1<br>Marginal or Corner | Column Section 2<br>Corner or Internal | Column Section 3<br>Marginal or Corner |
| Ground Floor | 650x900                                | 650x900                                | 450x550                                |
| Level 1      | 550x850                                | 550x850                                | 450x550                                |
| Level 2      | 550x750                                | 550x750                                | 450x450                                |
| Level 3      | 450x650                                | 450x650                                | 450x550                                |
| Level 4      | 450x650                                | 450x650                                | 450x550                                |
| Level 5      | 450x550                                | 450x550                                | 450x550                                |
| Level 6      | 350x450                                | 350x450                                | 350x450                                |
| Level 7      | 350x450                                | 350x450                                | 350x450                                |
| Level 8      | 350x450                                | 350x450                                | 350x450                                |



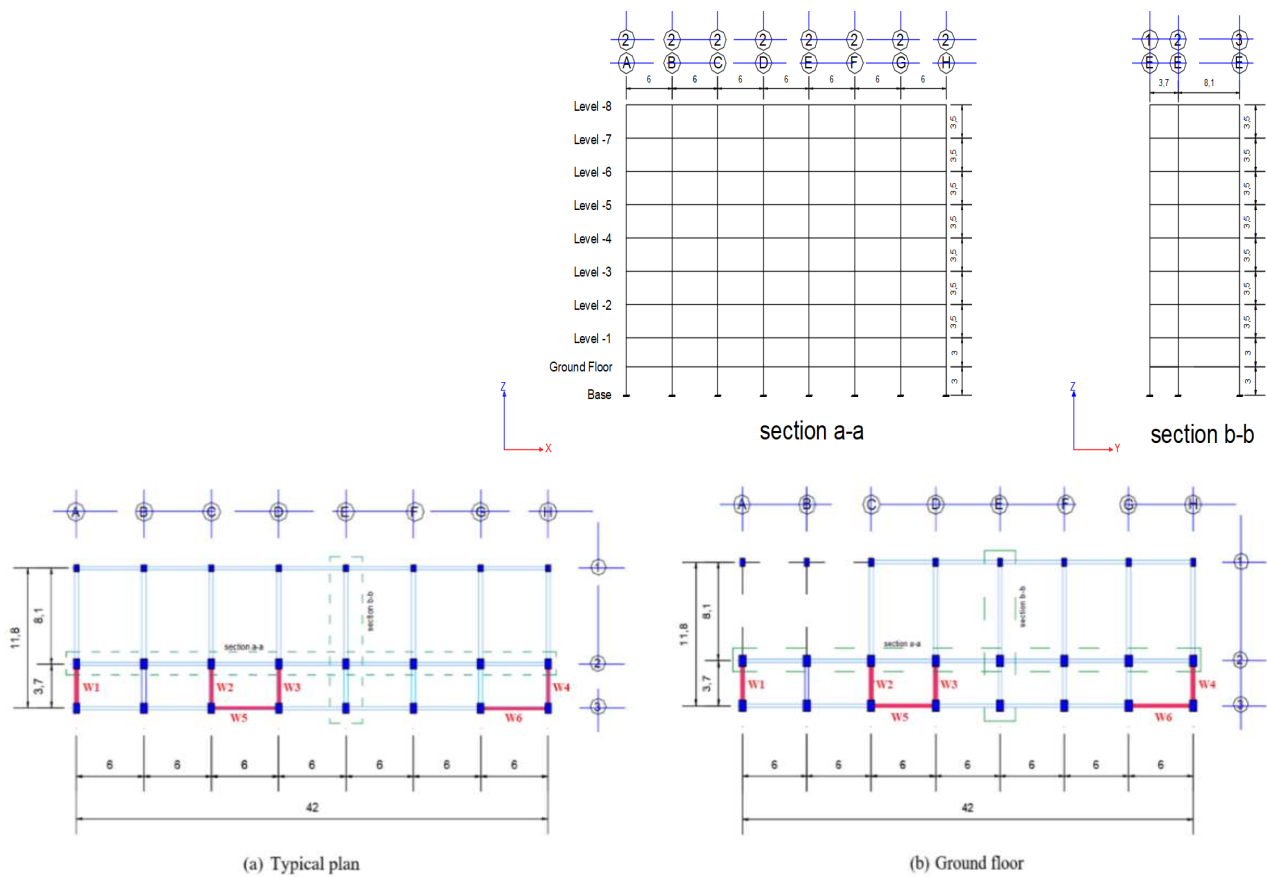


Figure 3-1. 3D Structural model, schematic vertical sections and floor plans of the studied building (dimensions in m)

### 3.3 Structural regularity

The structural regularity was assessed comparatively, according to the provisions of Eurocode 8, ASCE 7-16 and P100-1/2013.

The studied building was categorized as being irregular in plan, in the X and the Y directions.

- According to Eurocode 8, the studied building has three irregularity conditions
- For ASCE 7-16 - two irregularity conditions
- For P100-1/2013 - two irregularity conditions

The building model is torsionally stiff in all cases. The predominant response is given by the translational modes.

### 3.4 Finite element modelling for nonlinear analysis

Shear walls are modeled with mesh sizes of 50x74 cm for 3.7 m-wide walls and 50x60 cm for 6 m-wide walls. The Mander stress-strain relation is used for the nonlinear multi-layer material shell model; for shear walls, two layers have been adopted in the vertical and horizontal directions to calculate reinforcement in the cross-section as shown in Figure. 3-2.\

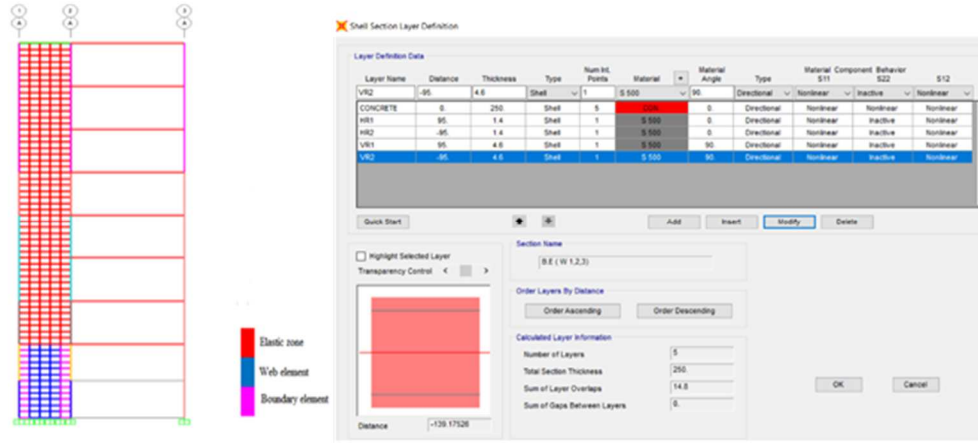


Figure 3-2- (a) Location of boundary elements and web shell elements and elastic zone of wall (1), (b) nonlinear multi-layer shell for boundary elements of walls (1, 2, and 3).

A lumped plasticity model was employed for frame elements (columns and beams). For beams, the moment-rotation relationship was entered in SAP 2000 utilizing user-defined hinge property type M3, while for columns the user-defined hinge property type P-M2-M3 was used, as shown in Figure 3-3.

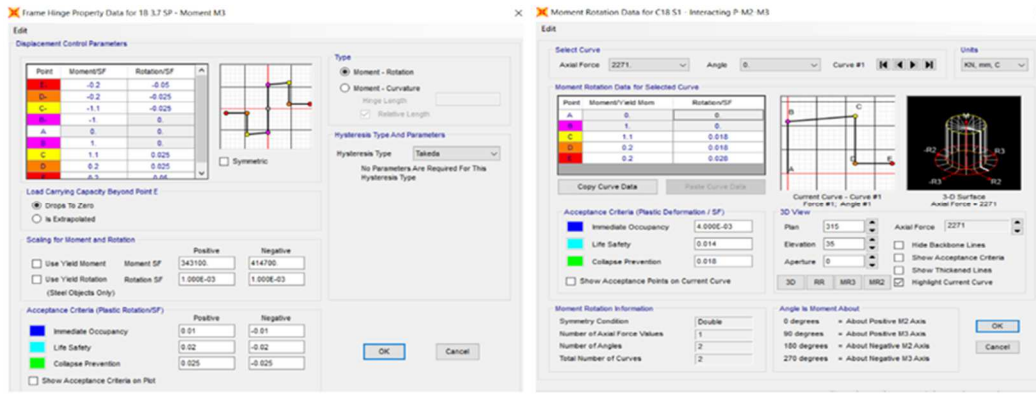


Figure 3-3. (a) M3 plastic hinges at the ends of beam (3.7m, ground floor level, frame B), (b) P-M2-M3 plastic hinges at the ends of column (C18, level 1).

### 3.5 Determining the performance level and the vulnerability

For the assessment of the structural vulnerability, the Structural Vulnerability Index (SVI), proposed in [38], was used. To compute the index, the number of plastic hinges formed in the elements of the frame for each performance level should be determined. A weighting factor ( $X_i$ ) is chosen for each performance level, as a separate analysis, according to Table 2-9. The vulnerability index, defined as a scaled linear combination (weighted average) of performance measures of the hinges in the structural elements, is computed from the performance levels of the components at the end of the nonlinear analysis. An importance factor, equal to 1.5 and 1.0, is assigned to columns and beams, respectively. Then, SVI is estimated by the expression below [38].

$$SVI_{Building} = \frac{1.5 \sum N_i^c X_i + 1.0 \sum N_i^b X_i}{\sum N_i^c + \sum N_i^b} \quad (1)$$

In Eq. (1):

$N_i^c$  is the number of plastic hinges developed in the columns,

$N_i^b$  is the number of plastic hinges developed in the beams,

$i$  is the performance level number; with values from 1 to 6, as shown in Table 3-2 [39].

Table 3-2. Performance level weighting factors [39]

| Performance level | Performance Level ( $i^{th}$ ) | Weighting factor ( $X_i$ ) |
|-------------------|--------------------------------|----------------------------|
| 1                 | < B                            | 0.000                      |
| 2                 | B-IO                           | 0.125                      |
| 3                 | IO-LS                          | 0.375                      |
| 4                 | LS-CP                          | 0.625                      |
| 5                 | CP-C                           | 0.875                      |
| 6                 | C-D, D-E, > E                  | 1.000                      |

For the vulnerability categorization of reinforced concrete buildings based on the SVI, five levels of vulnerability were proposed in [22]: Green (1), Green (2), Orange (3), Orange (4), and Red (5), to assess the seismic performance of the buildings. This classification is shown in Table 3-3 [40]. The vulnerability categories are related to the observed damage, which is defined as “Negligible”, “Minor”, “Moderate”, and “Severe/Partial Collapse”, as shown in Table 3-4 [40].

Table 3-3: RC buildings vulnerability classification according to SVI [40]

| Vulnerability Level | <b>Green</b><br><b>1</b> | <b>Green</b><br><b>2</b> | <b>Orange</b><br><b>3</b> | <b>Orange</b><br><b>4</b> | <b>Red</b><br><b>5</b> |
|---------------------|--------------------------|--------------------------|---------------------------|---------------------------|------------------------|
| SVI                 | 0.10-0.20                | 0.20-0.40                | 0.40-0.55                 | 0.55-0.70                 | 0.70-1.00              |
| SVI, mean           | 0.150                    | 0.300                    | 0.475                     | 0.625                     | 0.850                  |

Table 3-4: Vulnerability categories according to the observed damage [20]

| Damage Categories       | Level           | Description                                                             |
|-------------------------|-----------------|-------------------------------------------------------------------------|
| Negligible              | <b>Green 1</b>  | Negligible to light damage                                              |
| Minor                   | <b>Green 2</b>  | Light for structural elements, and moderate for non-structural elements |
| Moderate                | <b>Green 3</b>  | Moderate for structural elements, and heavy for non-structural elements |
| Severe/Partial Collapse | <b>Orange 4</b> | Heavy for both the structural and non-structural elements               |
| Total Collapse          | <b>Red 5</b>    | Total failure or collapse of the structure                              |

### 3.6 Discussion of the analysis results

A brief discussion of the results of the global response obtained for each type of analysis is presented in the following.

- The studied building was categorized as being irregular in the plan, in the X and the Y directions according to Eurocode 8 because it has three irregularity conditions. For ASCE 7-16, the studied building has two irregularity conditions. For P100-1/2013, the studied building has two irregularity conditions also.
- To have an in-depth understanding of the seismic response of the building, the periods and vibration modes for the elastic analysis (response spectrum analysis, RSA), pushover at target displacement  $d_t$ , and nonlinear time history analysis (THA) of the building were analyzed. The natural periods of the first three modes of vibration and the ratio of translation to rotation periods ( $\Omega$  ratio) of the building model, assuming rigid diaphragms, were shown in Table 2-12. The first mode is predominantly translational in the X direction, the second mode is translational in the Y direction and the third mode is torsional, with the  $\Omega$  ration greater than one. According to Anagnostopoulos et al. [41], it results that building models are torsionally stiff in all cases. The predominant response is given by the translational modes. The edge having the highest ductility demand is the so-called “flexible edge” FE, and the opposite, the “stiff edge”, SE [42]. The

schematic plan of the roof level of the building shows the stiff edge, SE, and the flexible edge, FE (Figure 3-4).

- The effects of higher modes of vibration were also assessed. The inter-story drift ratio was evaluated for the studied building in the X and the Y directions for the CM, the stiff edge (SE) and the flexible edge (FE) [43], as obtained by the nonlinear time history analysis THA, and then compared with the pushover analysis. The inter-story drift ratios over the building height are shown in Figure 3-5. The distribution of the inter-story drift ratio increases gradually over the building height, and reaches larger values in upper stories due to the modification of column sizes. The story drift increases in the flexible edge FE for the X and the Y directions of loading, where the values are larger than those in the stiff edge with 4% to 13% in the X direction and with 6% to 14% in the Y direction, for the results obtained from the THA. Given the differences between the inter-story drift values obtained from the THA and pushover analysis, respectively, it results that the effects of the higher modes are important for the seismic behavior of irregular buildings. In addition, it is clear that torsion induces a significant amplification to inter-story drifts for plan-asymmetric buildings.
- The in-plan distribution of roof displacements was also determined by THA and pushover analyses, to evaluate the torsional effects [44], as shown in Figure 3-6. The values are normalized by the roof displacement at the center of mass, CM. The resulting normalized roof displacements ( $u/u_{CM}$ ) in the X and the Y directions show that the torsional effects are situated between the values of the THA, as compared to those obtained by pushover analyses, especially in the FE. While in the SE the torsional effects are less than in the FE on both directions, this may be due to an increase of the number of elements that reach yielding in the flexible edge, which is larger than that for the stiff edge. The normalized roof displacement in the X direction shows torsional effects that are smaller than for the Y direction with about 22% in the SE and with about 19% in the FE.

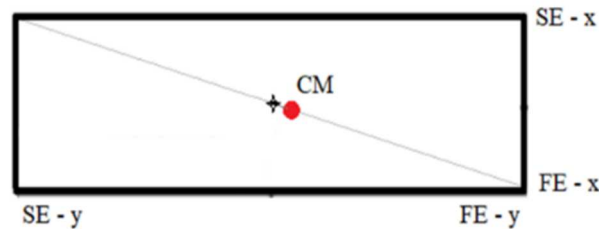


Figure 3-4. Schematic plan of the building (roof level), with stiff edge (SE) and flexible edge (FE)



Figure 3-5. Roof displacements in the horizontal plane for X and Y direction, as resulting from pushover (uniform and modal patterns) and time-history analysis (THA)

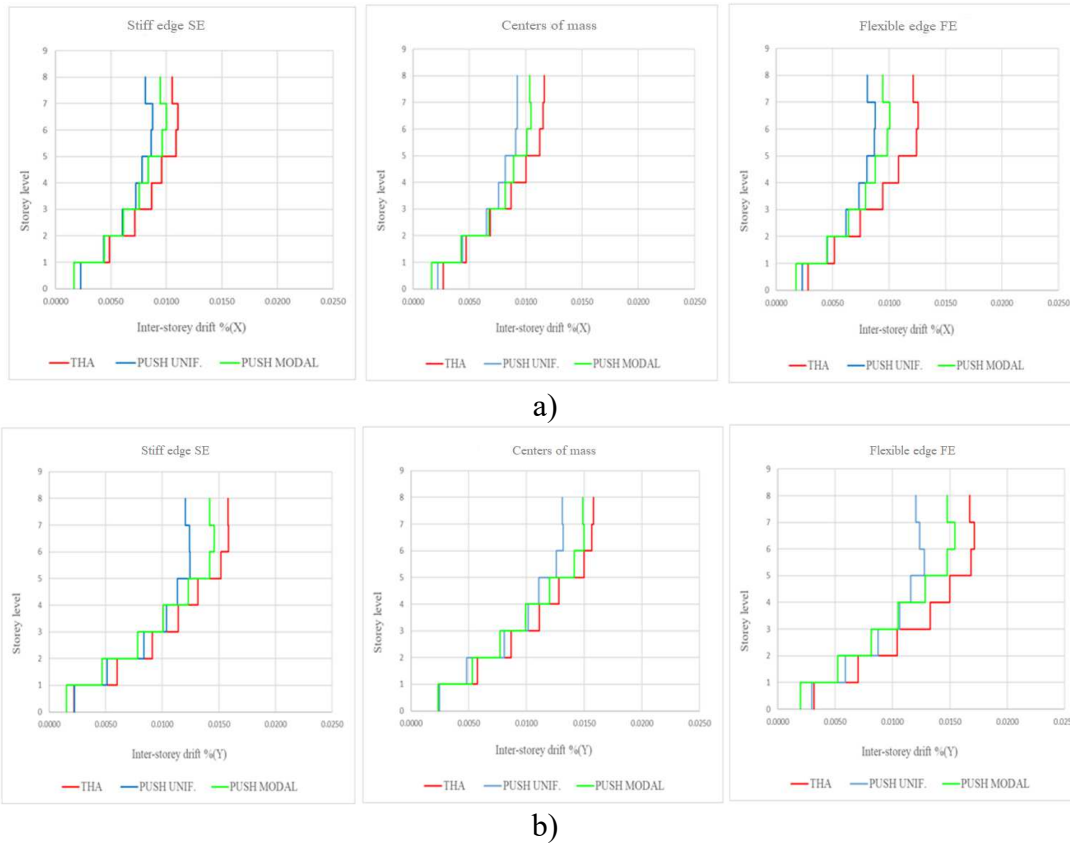


Figure 3-6. Interstory drift (Pushover and THA) at the Stiff Edge (SE), at the center of mass (CM) and at the Flexible Edge (FE), for the: a) X-direction, b) Y-direction

Table 3-5: Natural period and torsional to translational (lateral) frequency ratios ( $\Omega$ ) of the models

| Model                      | Mode X (sec) 1 | Mode Y (sec) 2 | Mode $\Theta$ (sec) 3 | $\Omega_x$ | $\Omega_y$ |
|----------------------------|----------------|----------------|-----------------------|------------|------------|
| Response Spectrum Analysis | 0.821          | 0.692          | 0.434                 | 1.89       | 1.59       |
| Pushover                   | 0.957          | 0.811          | 0.488                 | 1.96       | 1.66       |
| THA                        | 0.784          | 0.666          | 0.406                 | 1.93       | 1.64       |

- The computed values of the structural vulnerability index, SVI, are shown in Table 3-6. It can be noticed that, for the uniform pattern pushover in the X and the Y directions, the values correspond to level “Green 1” (negligible to light damage). For the modal pattern pushover in the X and Y directions, the SVI is between levels “Green 1” (negligible to light damage) and “Green 2” (light for structural elements and moderate for non-structural elements). For THA, level “Green 1” (negligible to light damage) was obtained. Therefore, the THA and the pushover analysis indicate a very similar global performance level, based on the vulnerability index. From the results, it can be noticed however that the SVI values for the pushover analysis are greater as compared to those obtained from the THA. In addition, the SVI values in the Y direction are greater than those in the X direction for the THA, possibly because the torsional effects in the X direction are smaller than those in the Y direction.

Table 3-6: Vulnerability index (SVI) for the THA and the pushover analysis

| Model                          | X+    | X-    | Y+    | Y-    |
|--------------------------------|-------|-------|-------|-------|
| Pushover MODAL                 | 0.226 | 0.231 | 0.185 | 0.239 |
| Pushover UNIFORM               | 0.180 | 0.168 | 0.146 | 0.149 |
| THA ("Irpinia-Italy-01", 1980) | 0.137 |       | 0.141 |       |
| Artificial accelerograms - 1   | 0.130 |       | 0.136 |       |
| Artificial accelerograms - 2   | 0.131 |       | 0.139 |       |
| Artificial accelerograms - 3   | 0.130 |       | 0.135 |       |

### 3.7 Earthquake incidence angle impact on the seismic performance of the RC plan asymmetric buildings

The acceleration response spectrum for Bucharest, according to P100-1/2013, with  $PGA = 0.3 \text{ g}$  (elastic response spectrum) was used as a target spectrum. The PEER database (NGA-West2) [45], was used to find the best matching, for the target spectrum, seven accelerogram records (with magnitudes ranging from 6.6 to 7.2). Each record has a pair of horizontal components, which are scaled in the SAP2000 program for  $PGA = 0.30 \text{ g}$ . These accelerograms were applied to the studied building in a set of eight different directions (the cases  $\beta = 360^\circ$  and  $\beta = 0^\circ$  are the same), with angles of incidence from  $0^\circ$  to  $315^\circ$ , with respect to the relevant axis (assumed in this study to be the X-axis). Increments of  $45^\circ$  were considered. Each pair of ground motion components was decomposed in the  $(X)$  and the  $(Y)$  components, the  $\bar{X}$  and the  $\bar{Y}$  components being rotated with angle  $\beta$  about the X-axis, as shown in Figure 3-7.

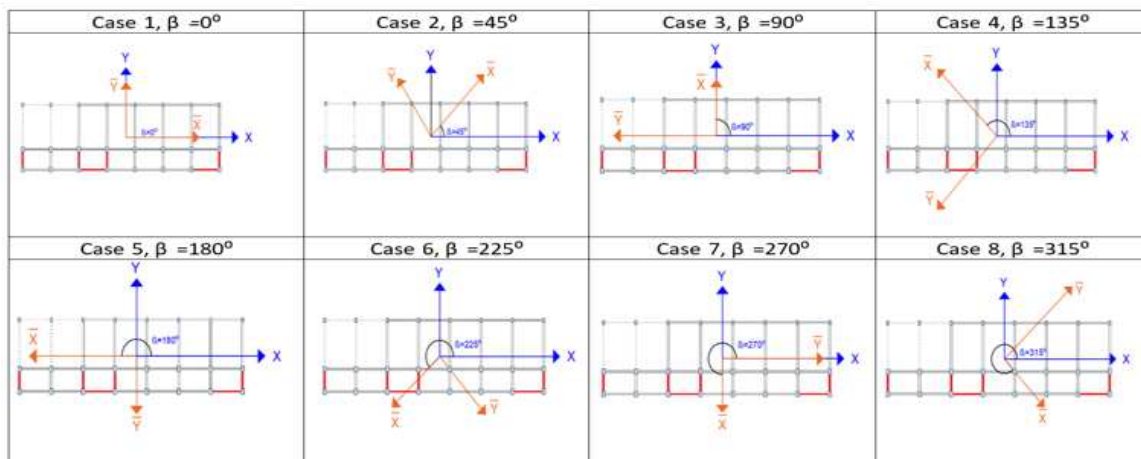


Figure 3-7. Set of values considered for the angles of incidence of the seismic action

### 3.8 Influence of the incidence angle

Two demand parameters were studied in order to investigate the impacts of ground motion orientation variability on the structural demand:

- the maximum inter-story drift (MLID) along the height of the building;
- the Seismic Vulnerability Index (SVI).

The inter-story drift ratio is evaluated for the studied building in the X and the Y directions for the Centre of Mass (CM) of the rigid floor diaphragms and at the Stiff Edge (SE) and the Flexible Edge

(FE) of the building. The edge having the highest ductility demand is the so-called “flexible edge”, FE, and the opposite - the “stiff edge”, SE [46]. Table 3-7 shows the SVI for all cases and for all considered accelerograms. Figures 2-7 and 2-8 show the MLID distribution along the height and the SVI, for all ground motion records, in the FE and SE.

Table 3-7. Seismic vulnerability index (SVI) for all cases and for all accelerograms

| SVI                | Case 1,<br>$\beta=0^\circ$ | Case 2,<br>$\beta=45^\circ$ | Case 3,<br>$\beta=90^\circ$ | Case 4,<br>$\beta=135^\circ$ | Case 5,<br>$\beta=180^\circ$ | Case 6,<br>$\beta=225^\circ$ | Case 7,<br>$\beta=270^\circ$ | Case 8,<br>$\beta=315^\circ$ |
|--------------------|----------------------------|-----------------------------|-----------------------------|------------------------------|------------------------------|------------------------------|------------------------------|------------------------------|
| Accelerogram No. 1 | <b>0.1605</b>              | 0.1388                      | 0.1250                      | 0.1586                       | <b>0.1640</b>                | 0.1315                       | 0.1260                       | 0.1483                       |
| Accelerogram No. 2 | 0.1270                     | 0.1268                      | 0.1250                      | <b>0.1274</b>                | 0.1269                       | 0.1269                       | 0.1250                       | <b>0.1272</b>                |
| Accelerogram No. 3 | 0.1268                     | 0.1250                      | 0.1250                      | <b>0.1286</b>                | 0.1268                       | 0.1254                       | 0.1250                       | <b>0.2209</b>                |
| Accelerogram No. 4 | 0.1268                     | 0.1270                      | 0.1250                      | <b>0.1336</b>                | 0.1312                       | 0.1301                       | 0.1250                       | <b>0.1418</b>                |
| Accelerogram No. 5 | 0.1250                     | 0.1279                      | 0.1275                      | <b>0.1349</b>                | 0.1289                       | 0.1275                       | 0.1250                       | <b>0.1337</b>                |
| Accelerogram No. 6 | <b>0.1597</b>              | 0.1390                      | 0.1405                      | 0.1380                       | 0.1596                       | 0.1538                       | 0.1432                       | <b>0.1628</b>                |
| Accelerogram No. 7 | <b>0.1338</b>              | 0.1313                      | 0.1272                      | 0.1330                       | 0.1329                       | 0.1296                       | 0.1320                       | <b>0.1344</b>                |

### 3.9 Discussion of the analysis results

The main results on the impact of the incidence angle are summarized below. Figures 3-8...3-11 show the variation of the maximum inter-story drift (MLID) values along the height of the building with the angle of incidence for the seven accelerograms considered, in the FE, SE, and CM respectively. Table 3-8 shows the main results.

Table 3-8. Higher values of (SVI) at incidence angle for MLID

| Accelerograms      | MLID        |             |      |      |      |      | Higher values of SVI at $\beta^\circ$ |
|--------------------|-------------|-------------|------|------|------|------|---------------------------------------|
|                    | FE-X        | FE-Y        | SE-X | SE-Y | CM-X | CM-Y |                                       |
| Accelerogram No. 1 | <b>180°</b> | 135°        | 45°  | 225° | 180° | 180° | <b>180°</b>                           |
| Accelerogram No. 2 | 180°, 315°  | <b>135°</b> | 0°   | 135° | 0°   | 90°  | <b>135°</b>                           |
| Accelerogram No. 3 | <b>315°</b> | <b>315°</b> | 90°  | 315° | 315° | 315° | <b>315°</b>                           |
| Accelerogram No. 4 | <b>315°</b> | <b>315°</b> | 270° | 315° | 315° | 270° | <b>315°</b>                           |
| Accelerogram No. 5 | 180°        | <b>135°</b> | 315° | 45°  | 180° | 90°  | <b>135°</b>                           |
| Accelerogram No. 6 | 0°          | <b>315°</b> | 0°   | 45°  | 0°   | 270° | <b>315°</b>                           |
| Accelerogram No. 7 | 0°          | <b>315°</b> | 0°   | 270° | 0°   | 90°  | <b>315°</b>                           |

The positions of the SE and FE changed during the earthquake for all cases corresponding to angles of incidence as illustrated in Table 3-9. This shows that the SE and the FE may switch between one another during the earthquakes, for some values of the incidence angles, due to the random occurrence of yielding, which is dependent on the seismic input.

Table 3-9: Angles of incidence for which SE & FE changed

| Accelerograms                                 | No. 1            | No. 2                | No. 3           | No. 4            | No. 5           | No. 6                   | No. 7                   |
|-----------------------------------------------|------------------|----------------------|-----------------|------------------|-----------------|-------------------------|-------------------------|
| Angles of incidence for which SE & FE changed | 45° 180°<br>225° | 45° 90°<br>225° 270° | 45° 90°<br>225° | 45° 225°<br>270° | 45° 90°<br>225° | 45° 90°<br>225°<br>270° | 45° 90°<br>225°<br>270° |

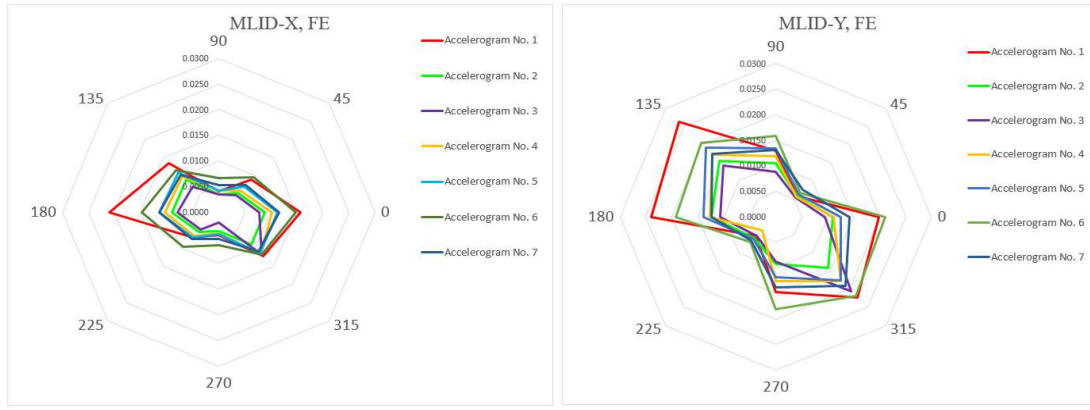


Figure 3-8: Variation of MLID with the angle of incidence for the seven accelerograms, in the FE

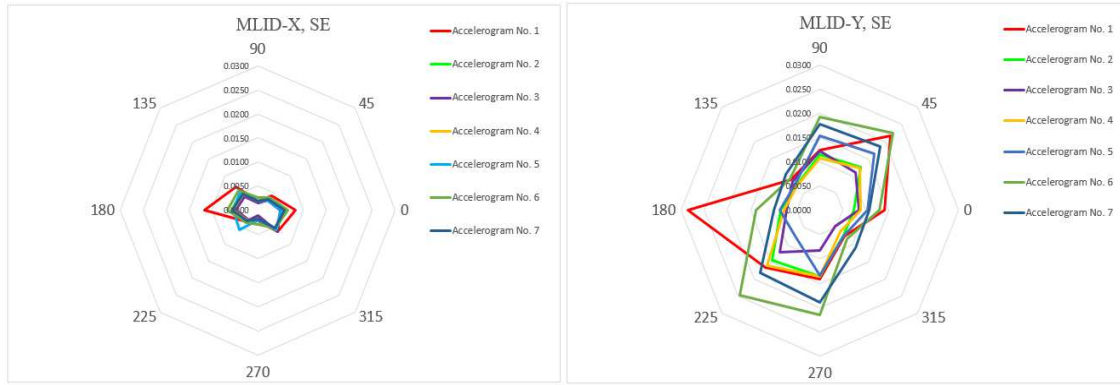


Figure 3-9: Variation of MLID with the angle of incidence for the seven accelerograms, in the SE

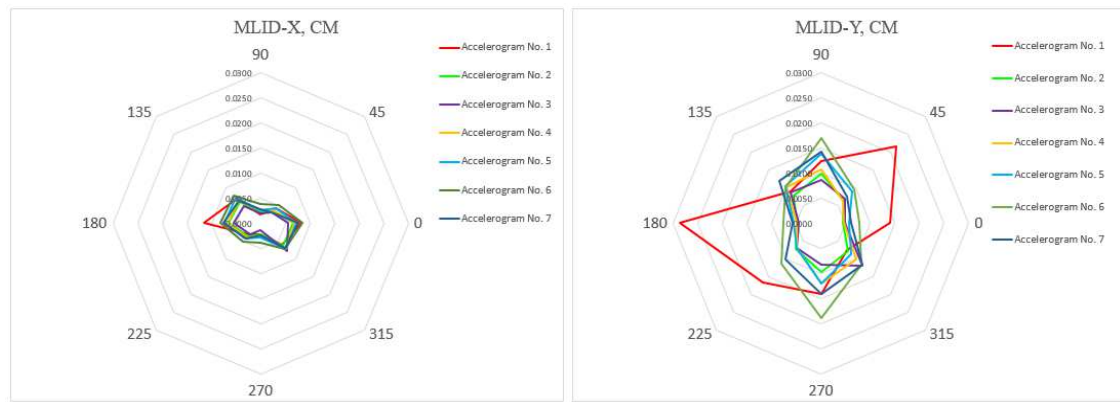


Figure 3-10: Variation of MLID with the angle of incidence for the seven accelerograms, in the CM

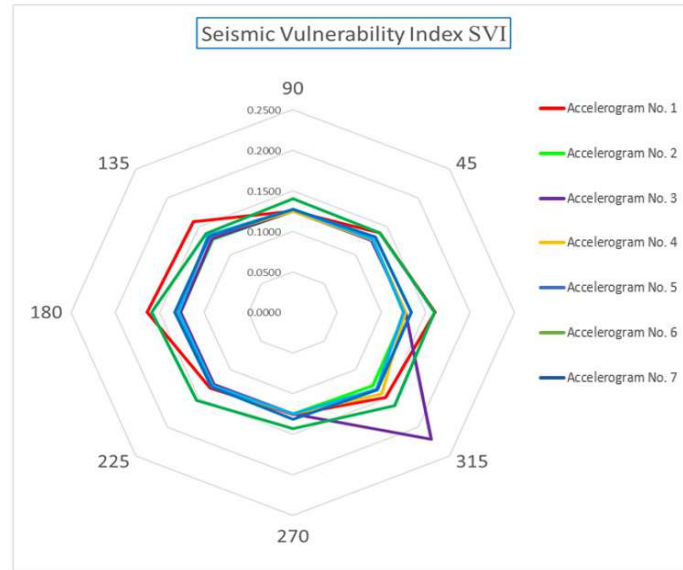


Figure 3-11: Seismic Vulnerability Index SVI for all accelerograms: variation with the angle of incidence

## CHAPTER FOUR

### 4 THE SEISMIC RESPONSE OF REINFORCED CONCRETE STRUCTURES WITH VERTICAL IRREGULARITY

#### 4.1 General

Earlier research on mid-rise structures with setbacks investigated whether dynamic analysis is important to design such buildings, although several design codes (i.e., [1], [2] and [11]) already recommend the dynamic method for the analysis of vertically irregular structures. At present, ASCE 7-16/Section 12.2.3.2, allows the use of a two-stage equivalent lateral force analysis for structures that have a flexible upper portion over a rigid lower portion. To obtain a feasible story stiffness distribution for the upper and lower structures, a simplified seismic design approach, proposed in [47], was applied in the current study. This approach was adopted to avoid the dynamic analysis-based trial-and-error procedure. The aim of the following study is to assess the applicability of the above-mentioned two-stage equivalent lateral force analysis for structures that have a flexible upper portion over a rigid lower

portion. In this study, the seismic behavior of various setback frames, derived based on a type of structural configuration located in Baghdad (Iraq) was studied using the approach proposed in [47]. A verification of IDRa, the average inter-storey drift ratio parameter obtained from this approach, was conducted and a comparison was made with the nonlinear static analysis (NSP) and nonlinear dynamic analysis (NDP) results. In addition, the shear ratio of the columns and the vulnerability index (VI) were assessed. The present study brings also a contribution to the assessment of the seismic vulnerability of the studied buildings, in which the fragility curves developed based on nonlinear static analysis procedure (pushover) were determined in order to estimate the seismic damage probability in terms of spectral displacements. In addition, the setback ratios were studied to take into account their influence on the increase of damage hazard.

## 4.2 Description of the studied buildings

The studied RC frame structures shown in Figure 4-2 are modified based on the archetypal building in Figure 4-1, by introducing setbacks. The archetypal RC frame structures exists in Baghdad. The building has six stories above the ground level (the base). This is an office building, with uniform configuration over the height, constructed in 2015. The story height of the lower and upper structure are 3.0 m, the total building height is 18.0 m (GF+5S). The building has 8 bays in Y direction and 3 bays in X direction. The bay widths are 5.75 m and 6.0 m in X direction and 3.7 m, 3.05 m and 5.45 m in Y direction. The studied frame structures have the same plan layout as the archetypal building at the first two stories. It was assumed that there are masonry infill walls at these stories, as described in Section 4.4 and glass curtain walls for the upper stories, as shown in Figure 4-2.

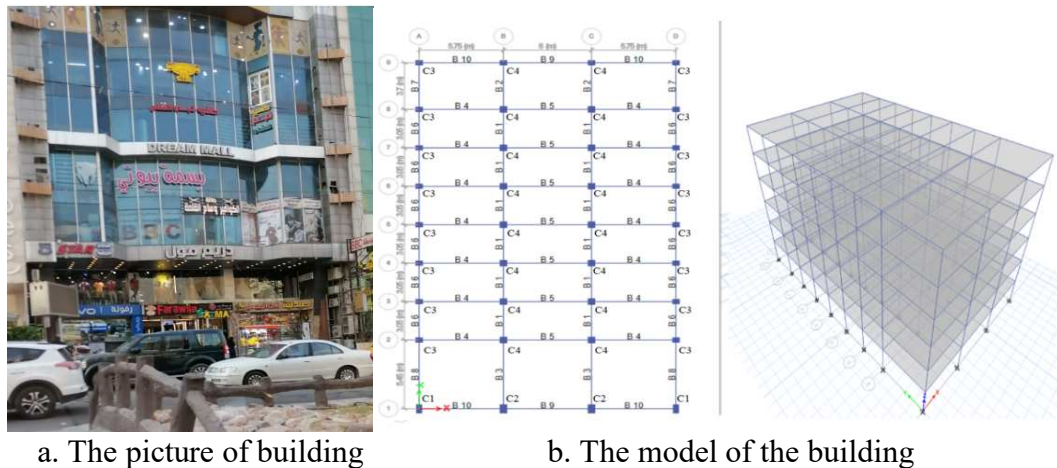


Figure 4-1: Archetypal building

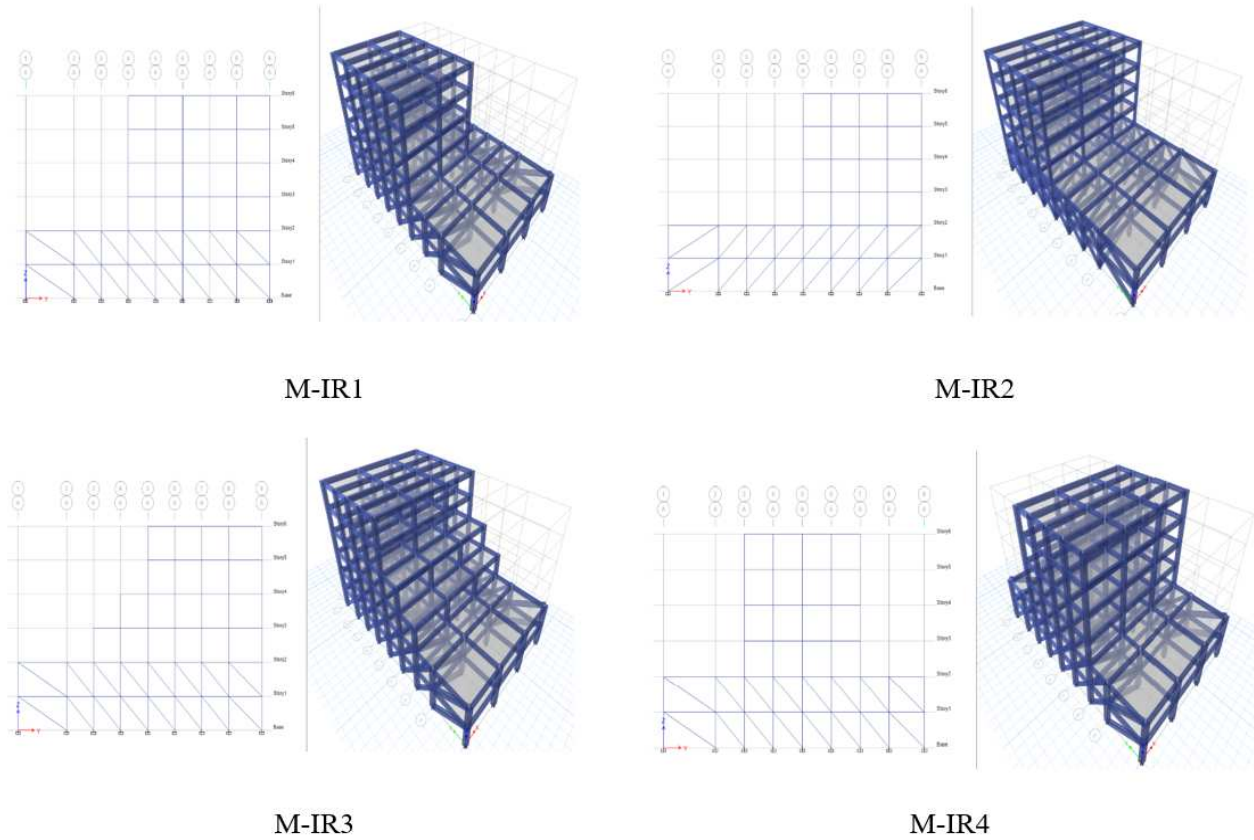


Figure 4-2: Geometries of the setback RC structures considered in this investigation

The material properties are chosen based on the specifications of ACI 318-19 [8], the standard code adopted for design in Iraq. The configuration of the frames is shown in Table 4-1 below.

Table 4-1. Buildings configuration data

| Items                                                         | Building data                      |
|---------------------------------------------------------------|------------------------------------|
| Perimeter columns size (C3)                                   | 500 × 400 mm                       |
| Internal columns size (C4)                                    | 500 × 500 mm                       |
| Axis 1 columns size (C1)                                      | 700 × 400 mm                       |
| Axis 1 columns size (C2)                                      | 700 × 500 mm                       |
| Beam size                                                     | 600mm × 300mm                      |
| Slab thickness                                                | 150 mm                             |
| Compressive strength of the concrete, $f'_c$                  | 25 MPa                             |
| Modulus of elasticity of the concrete, $E_c$                  | 23500 MPa                          |
| Minimum yield strength of steel, $f_y$                        | 414 MPa                            |
| Modulus of elasticity of steel, $E_s$                         | 200000 MPa                         |
| Masonry infill wall thickness (outer and inner, respectively) | 240 mm, 120 mm                     |
| Assuming good condition for masonry strength; $f'_m$          | 6.2 MPa (FEMA 356, Table 7-1)[48]  |
| Modulus of elasticity of masonry $E_m=550 \times f'_m$        | 3410 MPa (FEMA 356, Table 7-1)[48] |

### 4.3 Vertical geometric irregularity

According to the definition in ASCE/SEI 7-16, vertical geometric irregularity occurs when the horizontal dimension of the lateral-resisting system at one level is more than 130% of that at an

adjacent story. Based on this definition, it results that all investigated frame structures have geometric irregularity. These geometric irregularities are described briefly in Table 4-2.

Table 4-2. Vertical Geometric Irregularity

| Model No. | Model Identification | 3 <sup>rd</sup> , 4 <sup>th</sup> , 5 <sup>th</sup> and 6 <sup>th</sup> story                                                                                                         |
|-----------|----------------------|---------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|
| 1         | M-IR1                | 2 bays in the X- direction and 5 bays in the Y- direction                                                                                                                             |
| 2         | M-IR2                | 3 bays in the X- direction and 4 bays in the Y- direction                                                                                                                             |
| 3         | M-IR3                | 3 bays in the X- direction and 6 bays on the 3 <sup>rd</sup> story, 5 bays on the 4 <sup>th</sup> story, 4 bays on the 5 <sup>th</sup> and 6 <sup>th</sup> stories in the Y-direction |
| 4         | M-IR4                | 3 bays in the X- direction and 4 bays in the Y- direction                                                                                                                             |

#### 4.4 Quantification of setback irregularity

To determine the gradual variation of setbacks along the height of the studied frame structures and to quantify the setback irregularity, the irregularity indices  $\phi_b$ , and  $\phi_s$ , proposed in [49], were computed. The expressions of these parameters are according to Equation 2.

$$\phi_b = f(z) = \frac{1}{n_b - 1} \sum_{i=1}^{n_b-1} \frac{H_i}{H_{i+1}}, \quad \phi_s = f(z) = \frac{1}{n_s - 1} \sum_{i=1}^{n_s-1} \frac{L_i}{L_{i+1}} \quad (2)$$

where  $n_s$  is the number of stories,  $n_b$  is the number of bays at the 1<sup>st</sup> storey and  $H_i$  and  $L_i$  are the height and the width of the  $i^{\text{th}}$  storey [49].

A large value of the  $\phi_s$  index corresponds to a large reduction of the floor area. A large value of the  $\phi_b$  index corresponds to a tower-like structure, while for the extreme situation of a regular frame without setbacks, both of the above indices take their minimum value, i.e. unity [49]. The two indices are represented in Table 4-3 and in Figure 4-3, for the studied models.

Table 4-3. Vertical geometric irregularity

| Model No. | Model Identification |          | Y        |              |          | X        |              |
|-----------|----------------------|----------|----------|--------------|----------|----------|--------------|
|           |                      | $\phi_b$ | $\phi_s$ | $\phi_{avg}$ | $\phi_b$ | $\phi_s$ | $\phi_{avg}$ |
| 1         | M-IR1                | 1.286    | 1.139    | 1.212        | 4.500    | 1.116    | 2.808        |
| 2         | M-IR2                | 1.286    | 1.128    | 1.207        | -        | -        | -            |
| 3         | M-IR3                | 1.040    | 1.135    | 1.088        | -        | -        | -            |
| 4         | M-IR4                | 1.286    | 1.125    | 1.206        | -        | -        | -            |

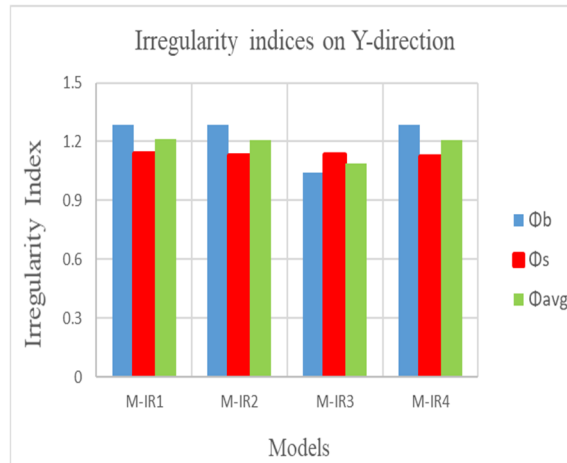


Figure 4-3: Variation of the irregularity index for the studied setback models (M-IR1, M-IR2, M-IR3, M-IR4) – Y direction

#### 4.5 Modeling members nonlinearity

The nonlinear (static and dynamic) analysis is also performed according to ASCE/SEI 7-16 and ASCE/SEI 41-13 codes. The CSI software SAP2000 (2019), is utilized to perform the nonlinear analysis.

The same 3-D models utilized before in the linear analysis are again utilized for the nonlinear analysis. The structures must be first designed, using the response spectrum analysis, RSA, according to ASCE/SEI 7-16. The design and detailing of the models for ductility are performed to achieve the goal of ASCE/SEI 7-16 and also to meet the rules and requirements of ACI 318-19.

According to ACI 318-19 (considered for design in Iraq), for nonlinear analysis the effective stiffness values were input in the analysis by adopting the cracked stiffnesses of the columns and beams. According to ASCE/SEI 41-13, beams and columns were modelled as elastic elements with concentrated plastic hinges at each end, after their effective stiffness was assigned.

#### 4.6 Nonlinear static analysis procedure (NSP)

According to FEMA 356, "the pushover curve is developed for at least two vertical distributions of lateral loads". The first distribution is the Equivalent Lateral Force (ELF) distribution: ( $S_i^* = m_i h_i^k$ ) (with  $i = 1, 2 \dots N$  being the floor number), where  $S_i^*$  is the lateral force at the  $i^{th}$  floor,  $m_i$  is the mass located on or assigned to floor level  $i$ ,  $h_i$  is the height above the base to level  $i$  and the exponent  $k = 1$  for fundamental period  $T_1 \leq 0.5$  second,  $k = 2$  for  $T_1 \geq 2.5$  second; and varies linearly in between. The second distribution is the uniform pattern of lateral force distribution: ( $S_i^* = m_i$ ). Nonlinear static analysis is carried out in both directions ( $\pm X$  and  $\pm Y$ ) of the models. The analyses include P-delta effects and gravity loads.

In addition, and based on ASCE/SEI 41-13 and on the recommendations of FEMA 440, the first mode distribution ( $S_i^* = m_i \phi_{i1}$ ) will also be used as a third distribution. According to the modal analysis of the building models, the fundamental period of vibration does not exceed 1.0 s and the first mode of vibration dominates.

#### 4.7 Nonlinear dynamic analysis procedure (NDP)

According to ASCE/SEI 7-16, the site of the models in this study (Baghdad) is not within 10 km of any known fault, so only far-field ground motions are considered.

According to ASCE/SEI 7-16 requirements on ground acceleration histories needed in analysis, eleven pairs of spectrally matched orthogonal components, obtained from eleven artificial accelerograms pairs, were included in this study (ASCE/SEI 7-16, Section C16.2.2). Because more than seven ground motions were used, the response parameters will be the mean results obtained from all analyses.

For spectral matching, the target response spectrum (design spectrum, specified for Baghdad for Peak Ground Acceleration  $PGA = 0.125$  g) was used as a target spectrum to generate the eleven artificial accelerograms. The target response spectrum, 5%-damped, was developed for single response spectrum. The period range for matching was determined according to ASCE/SEI 7-16, Section 16.2.3.1: "upper [period] bound equal to twice the largest first-mode period in the principal horizontal directions of response. The lower bound period shall not exceed 20% of the smallest first-mode period for the two principal horizontal directions of response".

Two orthogonal seismic actions (in the X and Y directions) were applied independently. The vertical response effects were not included for the studied models, according to ASCE/SEI 7-16, Section 16.1.3. For sources of artificial excitations, the PEER NGA-2022 strong motion database was used to find the best matching (spectral matching with the target response spectrum, design spectrum specified for Baghdad) earthquake records. The two horizontal components were applied for each accelerogram, and then they were scaled (in the time domain) in the SAP2000 program to match the target spectrum.

## 4.8 Results and discussion of structural behavior

### 4.8.1 Nonlinear static analysis procedure (NSP)

#### 4.8.1.1 Drift Check

The inter-story inelastic drift ratios ( $IDR_{x,y}$ ) for both directions are computed at the target displacement ( $\delta_t$ ) [50]. The maximum inter-story inelastic drift ratio ( $IDR_{max}$ ) is the maximum IDR of all stories. The story drift limit is 2% for the risk category II building, according to ASCE 7-16. The inter-story drift ratios should not exceed this limit.

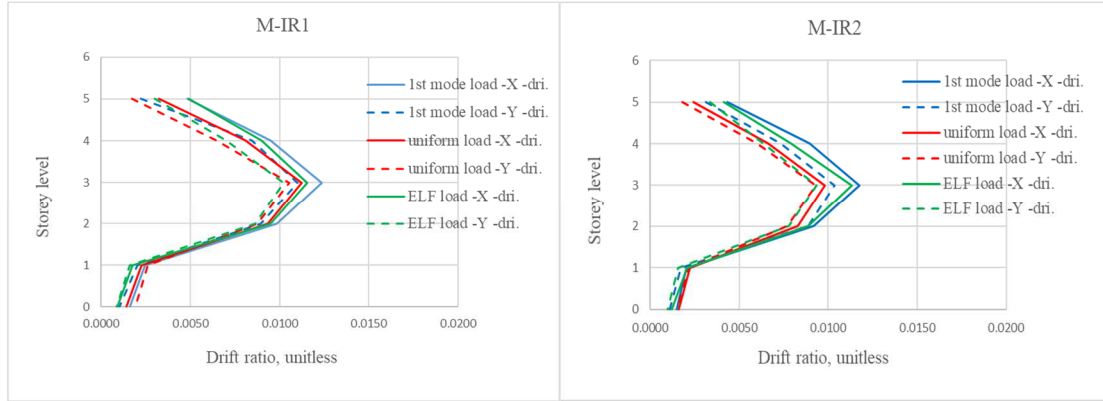


Figure 4-4: Inter-story drift ratios ( $IDR_{x,y}$ ) along the height of M-IR1 and M-IR2 models for NSP

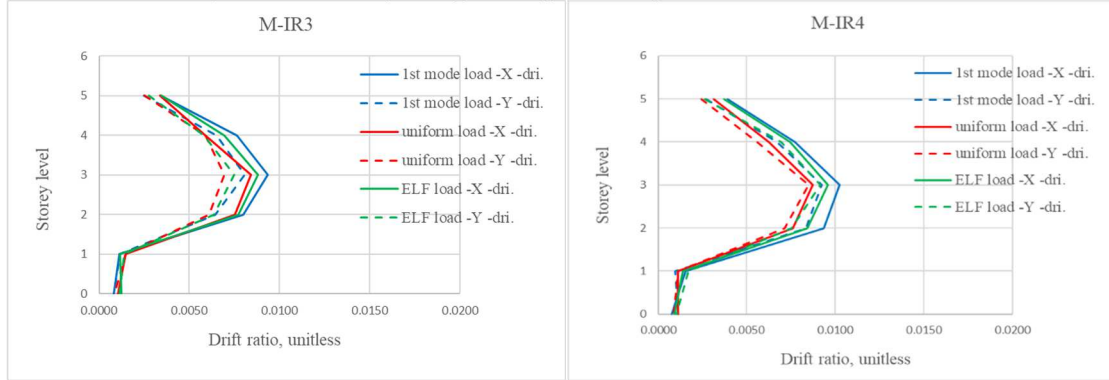


Figure 4-5: Inter-story drift ratios ( $IDR_{x,y}$ ) along the height of M-IR3 and M-IR4 models for NSP

The story drifts are shown in Figures 4-4 and 4-5 for both directions, under the effect of the three load distributions considered in the NSP analysis. It can be noticed that the drift of the 1<sup>st</sup> mode load distribution along the height of the models is larger than the drift of both load distribution (Equivalent lateral force ELF and uniform) for the same model. The first story of the upper structure for all models has the highest story-drift-ratio in the entire building, which is satisfying the assumption of the equation of the minimum story-stiffness ratio  $r_{kU1}$  which is derived in [47].

Nevertheless, the maximum inter-story drift ratios at  $\delta_t$  ( $IDR_{tmax}$ ) do not exceed the limit of 2%, i.e., the performance of the frames is satisfactory in spite of the existence of the setback. In addition, it can be noted that, as the model irregularity indices increase, the inter-story drift increases under the effect of a particular load distribution, as shown in Figure 4-6.

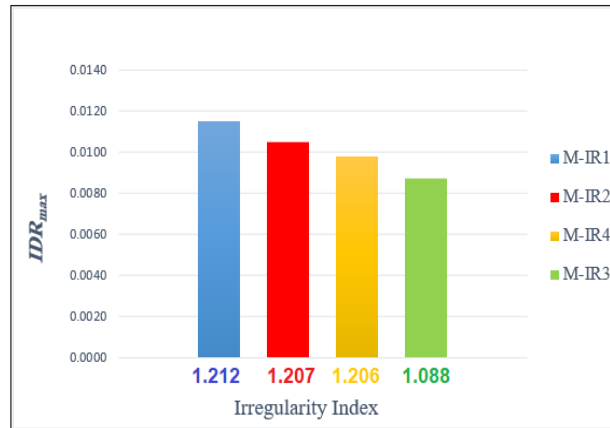


Figure 4-6: Maximum inter-story inelastic drift ratio ( $IDR_{max}$ ) as a function of the irregularity index of all models for NSP

## 4.8.2 Nonlinear dynamic analysis procedure (NDP)

### 4.8.2.1 Drift Check

From the analysis of the NDP results for the setback irregular models and by comparing them with the results of NSP, the first-mode load pattern distribution was chosen, being considered the worst case of the three load pattern distributions considered in NSP. It can be observed, although the inter-story drift ratios for NSP followed the same pattern as inter-story drift ratios for NDP along the height of the models, with very close values in the first story, the differences between the two analyses occurred at the second story. Moreover, the inter-story drift ratios ( $IDR_{x,y}$ ) from the NDP and the NSP are larger than the ( $IDR_{x,y}$ ) values of the ELFP, where the seismic action is represented by the design response spectrum specific for Baghdad. Figure 4-9 shows, for all models, the comparison between ELFP and NDP. The ( $IDR_a$ ) from NDP are larger than those from ELFP. This shows that the equivalent lateral force (ELF) procedure, which is adopted in ISC 2016, is not appropriate for setback structures.

This shows that the equivalent lateral force (ELF) procedure, which is adopted in ISC 2014 is not appropriate for setback structures. Also, this study shows the drawbacks of the new simplified seismic design approach proposed in [47] for structures that have a flexible upper portion over a rigid lower portion (in this study, was adopted a setback frames correspond with this type of structural configuration) to quantify the performances of this type of configurations, also reveals an issue in ISC 2016 which includes only two seismic analysis methods: i) Modal Response Spectrum (MRS); ii) Equivalent Lateral Force (ELFP), which are the methods applied to analyze and design buildings in Iraq. The average resulting inter-story drift ratios, for all models, analyses are shown comparatively in Figure 4-7, for NDP and NSP (1<sup>st</sup>- mode). In all models, there is a sudden increase in the drift values where geometry changes, at the 2<sup>nd</sup> level. The graph shows that, even though the results of the NSP analysis took into account the vertical distribution of lateral forces, this type of analysis is incapable to simulate the impacts of higher modes on the structural response, as these become more important when the irregularity of the structure increases.

Although in the past Iraq was rarely exposed to seismic activity, in recent years, seismic activity has begun to increase in parts of Iraq, including the eastern region bordering Iran, which led to its effects reaching Baghdad. Consequently, there is an urgent need to adopt appropriate and more effective methods for designing and evaluating the performance of reinforced concrete buildings, because they are the most used type in Iraq.

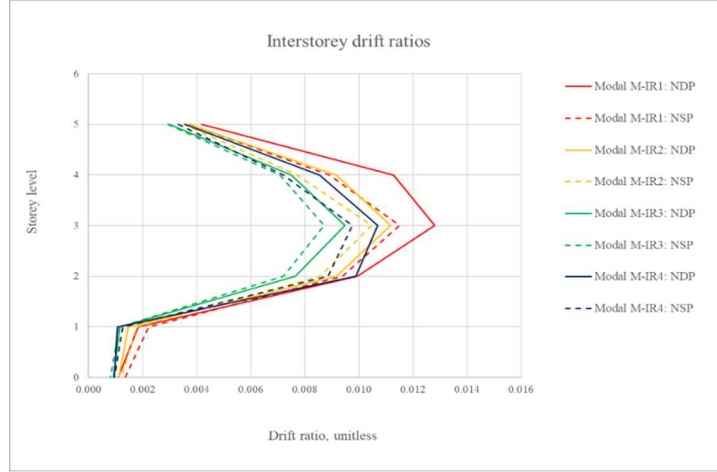


Figure 4-7: Inter-storey drift ratios ( $IDR_{ave}$ ) of the setback irregular models for NSP and NDP

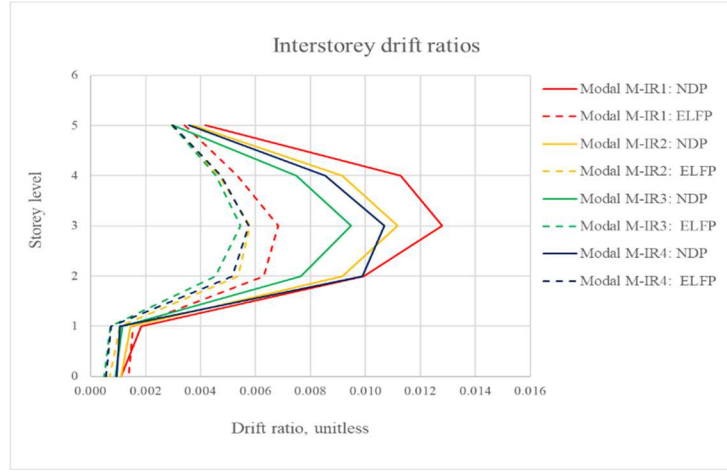


Figure 4-8: Inter-storey drift ratios ( $IDR_{ave}$ ) of setback irregular models for ELFP and NDP

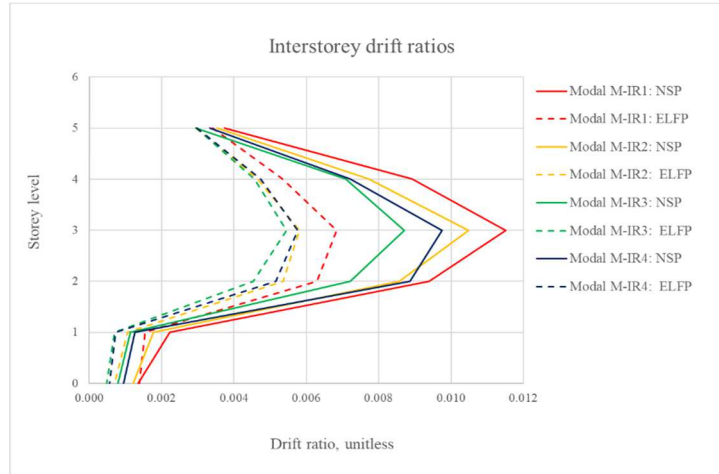


Figure 4-9: Inter-storey drift ratios ( $IDR_{ave}$ ) of setback irregular models for ELFP and NSP

#### 4.9 Determining the performance level and vulnerability

The vulnerability index (VI) can be used to assess the damage caused by seismic actions. It is calculated using the weighting factors of the frame elements and based on the number of plastic hinges formed. Because the essential cause for concern about the risk of irregular buildings, according to several analyses of various types of irregularities, is the increased chance of local failure, a

vulnerability index can be used to determine the increase or distribution of local damage (Dya [51]). Dya proposed a modified approach of the original vulnerability index (Lakshmanan [52]), based on an attempt to derive a local vulnerability index for each story frame. The modified formula is the following:

$$VI_{Loc i} = \frac{[1.5 \sum N_j^c X_j + 1.0 \sum N_j^b X_j] i}{[\sum N_i^c + \sum N_i^b] i} \quad (3)$$

where  $N_j^c$  and  $N_j^b$  are the number of plastic hinges created in columns and beams respectively,  $j^{th}$  is the performance level number ( $j = 1 \dots 6$ ) and weighting factor ( $X_i$ ) as shown in Table 3-2, and  $i$  is the story frame in consideration. The importance factor equals 1.5 and 1.0, for the columns and the beams, respectively. For each irregular model, the score modifier decreases due to the variation in the distribution of the local vulnerability index in comparison with that of the considered regular model. The local vulnerability index for each story frame of the considered buildings is determined using Equation (3) and the distribution of the local vulnerability relative to the entire building for which the vulnerability is determined. The distribution of the local vulnerability is determined using the formula

$$VI_{Di} = \frac{VI_{Loc i}}{Total VI_{Loc}} \times 100 \quad (4)$$

where:

$VI_{Di}$  is the local vulnerability index distribution of the story frame  $i$

$VI_{Loc i}$  is the local vulnerability factor of the frame  $i$ .

The increase in the distribution of the vulnerability index is calculated as,

$$VI_{Fi} = \frac{VI_{Di} \text{ of irregular bulding}}{VI_{Di} \text{ of regular bulding}} \quad (5)$$

where  $VI_{Fi}$  is the local vulnerability index that represents the increase in  $VI_{Di}$  for frame  $i$ .

The NSP was performed in two main directions ( $\pm X$  and  $\pm Y$ ) for three load distributions, for regular and setback irregular models. Conservatively, for each load distribution, the pushover curve with the lowest shear capacity (considered as the worst performance of models) was chosen to be represent the respective model capacity.

Table 4-4: Local vulnerability index for all models (vulnerability increase as compared with the regular model)

| Local Vulnerability factor $VI_{Loc i}$ |            |       |       |       |       |       |       |       |       |       |         |       |       |       |       |
|-----------------------------------------|------------|-------|-------|-------|-------|-------|-------|-------|-------|-------|---------|-------|-------|-------|-------|
| Models                                  | First mode |       |       |       |       | ELF   |       |       |       |       | Uniform |       |       |       |       |
|                                         | Reg.       | M-IR1 | M-IR2 | M-IR3 | M-IR4 | Reg.  | M-IR1 | M-IR2 | M-IR3 | M-IR4 | Reg.    | M-IR1 | M-IR2 | M-IR3 | M-IR4 |
| 1 <sup>st</sup> Frame                   | 0.169      | 0.125 | 0.125 | 0.125 | 0.125 | 0.149 | 0.125 | 0.125 | 0.125 | 0.125 | 0.143   | 0.125 | 0.125 | 0.125 | 0.125 |
| 2 <sup>nd</sup> Frame                   | 0.125      | 0.125 | 0.125 | 0.125 | 0.125 | 0.125 | 0.125 | 0.125 | 0.125 | 0.125 | 0.125   | 0.125 | 0.125 | 0.125 | 0.125 |
| 3 <sup>rd</sup> Frame                   | 0.125      | 0.236 | 0.227 | 0.203 | 0.217 | 0.125 | 0.214 | 0.206 | 0.188 | 0.192 | 0.125   | 0.182 | 0.175 | 0.156 | 0.159 |
| 4 <sup>th</sup> Frame                   | 0.125      | 0.208 | 0.206 | 0.178 | 0.200 | 0.125 | 0.151 | 0.146 | 0.131 | 0.135 | 0.125   | 0.144 | 0.142 | 0.125 | 0.125 |
| 5 <sup>th</sup> Frame                   | 0.125      | 0.125 | 0.125 | 0.125 | 0.125 | 0.125 | 0.125 | 0.125 | 0.125 | 0.125 | 0.125   | 0.125 | 0.125 | 0.125 | 0.125 |
| 6 <sup>th</sup> Frame                   | 0.000      | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000   | 0.000 | 0.000 | 0.000 | 0.000 |
| Total                                   | 0.669      | 0.819 | 0.807 | 0.788 | 0.792 | 0.649 | 0.740 | 0.728 | 0.708 | 0.713 | 0.643   | 0.700 | 0.692 | 0.656 | 0.656 |

Table 4-1: Local vulnerability distribution for all models

| Local Vulnerability distribution $VI_{Di}$ [%] |            |       |       |       |       |       |       |       |       |       |         |       |       |       |       |
|------------------------------------------------|------------|-------|-------|-------|-------|-------|-------|-------|-------|-------|---------|-------|-------|-------|-------|
| Models                                         | First mode |       |       |       |       | ELF   |       |       |       |       | Uniform |       |       |       |       |
|                                                | Reg.       | M-IR1 | M-IR2 | M-IR3 | M-IR4 | Reg.  | M-IR1 | M-IR2 | M-IR3 | M-IR4 | Reg.    | M-IR1 | M-IR2 | M-IR3 | M-IR4 |
| 1 <sup>st</sup> Frame                          | 25.22      | 15.26 | 15.48 | 16.54 | 15.79 | 22.91 | 16.89 | 17.17 | 18.02 | 17.82 | 22.26   | 17.83 | 18.07 | 19.05 | 18.97 |
| 2 <sup>nd</sup> Frame                          | 18.69      | 15.26 | 15.48 | 16.54 | 15.79 | 19.27 | 16.89 | 17.17 | 18.02 | 17.82 | 19.43   | 17.83 | 18.07 | 19.05 | 18.97 |
| 3 <sup>rd</sup> Frame                          | 18.69      | 28.78 | 28.09 | 26.88 | 27.36 | 19.27 | 28.95 | 28.36 | 27.03 | 27.33 | 19.43   | 26.03 | 25.30 | 23.81 | 24.10 |
| 4 <sup>th</sup> Frame                          | 18.69      | 25.44 | 25.47 | 23.28 | 25.27 | 19.27 | 20.38 | 20.12 | 18.90 | 19.20 | 19.43   | 20.47 | 20.47 | 19.05 | 18.97 |
| 5 <sup>th</sup> Frame                          | 18.69      | 15.26 | 15.48 | 16.54 | 15.79 | 19.27 | 16.89 | 17.17 | 18.02 | 17.82 | 19.43   | 17.83 | 18.07 | 19.05 | 18.97 |
| 6 <sup>th</sup> Frame                          | 0.00       | 0.00  | 0.00  | 0.00  | 0.00  | 0.00  | 0.00  | 0.00  | 0.00  | 0.00  | 0.00    | 0.00  | 0.00  | 0.00  | 0.00  |

Table 4-2: Local vulnerability index for all models (vulnerability increase as compared with the regular model)

| Vulnerability index $VI_{Fi}$ |            |       |       |       |       |       |       |       |         |       |       |       |
|-------------------------------|------------|-------|-------|-------|-------|-------|-------|-------|---------|-------|-------|-------|
| Models                        | First mode |       |       |       | ELF   |       |       |       | Uniform |       |       |       |
|                               | M-IR1      | M-IR2 | M-IR3 | M-IR4 | M-IR1 | M-IR2 | M-IR3 | M-IR4 | M-IR1   | M-IR2 | M-IR3 | M-IR4 |
| 1 <sup>st</sup> Frame         | 0.61       | 0.61  | 0.66  | 0.63  | 0.78  | 0.75  | 0.78  | 0.78  | 0.80    | 0.81  | 0.86  | 0.85  |
| 2 <sup>nd</sup> Frame         | 0.82       | 0.83  | 0.88  | 0.84  | 0.92  | 0.89  | 0.93  | 0.92  | 0.92    | 0.93  | 0.98  | 0.98  |
| 3 <sup>rd</sup> Frame         | 1.54       | 1.50  | 1.44  | 1.46  | 1.50  | 1.47  | 1.39  | 1.42  | 1.34    | 1.30  | 1.23  | 1.24  |
| 4 <sup>th</sup> Frame         | 1.36       | 1.36  | 1.26  | 1.35  | 1.06  | 1.04  | 1.00  | 1.02  | 1.05    | 1.05  | 0.98  | 0.98  |
| 5 <sup>th</sup> Frame         | 0.82       | 0.83  | 0.88  | 0.84  | 0.88  | 0.89  | 0.92  | 0.91  | 0.92    | 0.93  | 0.98  | 0.98  |
| 6 <sup>th</sup> Frame         | 0.00       | 0.00  | 0.00  | 0.00  | 0.00  | 0.00  | 0.00  | 0.00  | 0.00    | 0.00  | 0.00  | 0.00  |

A comparison of the results obtained for all setback models is shown in Table 4-6. The results show that almost all plastic hinges have developed at the 3<sup>rd</sup> and 4<sup>th</sup> stories, which are the base of the upper structure, where there are reductions in stiffness and changes of vertical geometry. It can be noticed that the values of the vulnerability index are larger than unity for the 3<sup>rd</sup> and 4<sup>th</sup> levels, while the factor is smaller than one at the rest of the levels.

In addition, the vulnerability index increased at the 3<sup>rd</sup> and 4<sup>th</sup> levels as models irregularity indices increased under a particular load distribution. The values of the vulnerability index for the “First mode” distribution are larger than those obtained for both other two distributions (ELF and Uniform), for the same model, at the 3<sup>rd</sup> and 4<sup>th</sup> levels.

The analysis of the setback structures results shows that the main reason for setback buildings being more subject to seismic activity is earthquake forces localization. Despite the fact that the total demand on the structure is lower due to the lower overall mass, disparate demands on various parts of the structure result in a local risk.

The severity or degree of the structure setback also influences the increase of the risk, so the setback ratios are studied to take into account its severity. The forces are concentrated on the section of the structure where the abrupt stiffness decrease occurs, i.e., at the bottom of the upper structure.

This can be noticed from the development of the plastic hinges and from the story drift at this location. Consequently, the abrupt changes in the stiffness or in the vertical configurations of the structures are considered local vulnerability locations.

The summary of comparison results is also shown in Figure 4-10.

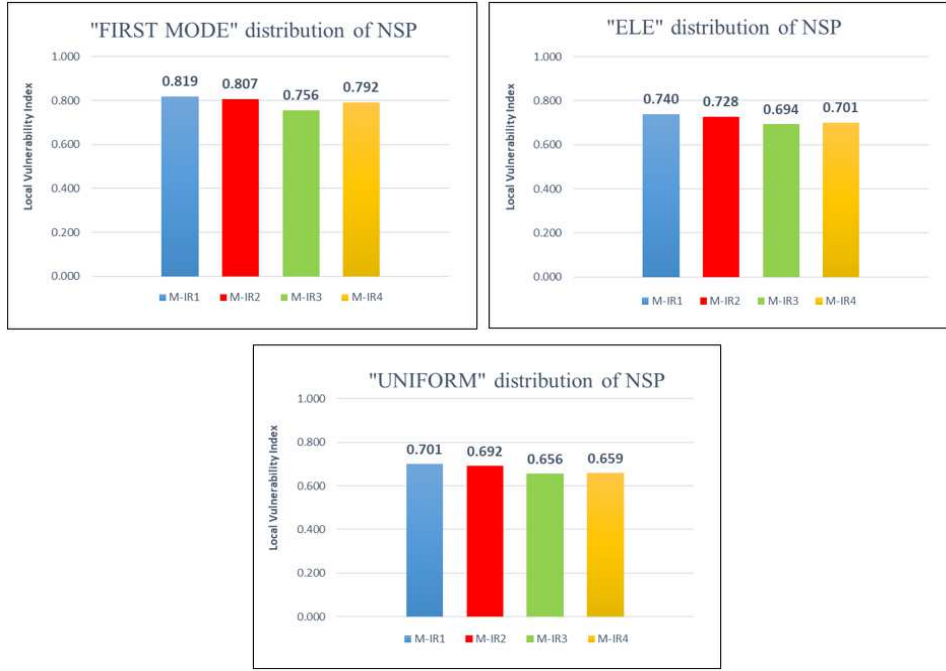


Figure 4-10. Total values (sum for all stories) of each model of the local vulnerability factor  $VI_{Loc i}$

#### 4.9.1 Development of fragility curves

The relations of seismic damage expressed in terms of fragility curves are essential for earthquake risk assessments and simulations of earthquake scenarios. HAZUS-MH-MR1 [53] defines building fragility curves as “lognormal functions that describe the probability of reaching, or exceeding, structural and non-structural damage states, given median estimates of spectral response, for example spectral displacement”. Four damage states are considered, i.e., Slight, Moderate, Extensive and Complete. Seismic motion effects can be expressed in the form of fragility curves, to evaluate the vulnerability of setback irregular structures depending on their probability of damage. The fragility curves of the models under consideration in this study were represented by the probability that the spectral displacement exceeds a specific damage state,  $d_s$ ,  $P(d \geq d_s)$ , where the spectral displacement  $S_d$  is considered as a function aimed to quantify the intensity of the seismic action. The mean displacement,  $\bar{S}_{d_{ds}}$ , and the standard deviation,  $\beta d_s$ , characterize the fragility curves. Thus, for a specific damage state  $d_{si}$ , the fragility curves are described by the lognormal functions shown in Equation 6 [54]:

$$P [d_s / S_d] = \phi \left( \frac{1}{\beta d_s} \ln \left( \frac{S_d}{\bar{S}_{d_{ds}}} \right) \right) \quad (6)$$

where:

$\bar{S}_{d_{ds}}$  - is the median value of the spectral displacement at which the building reaches the threshold of the damage state,  $d_s$ ,

$\beta d_s$ - is the standard deviation of the natural logarithm of spectral displacement of damage state,  $d_s$

$\phi$  – is the standard normal cumulative distribution function.

The thresholds  $S_{d_{ds}}$  represent the yield and ultimate spectral displacement of the models, respectively, obtained from the bilinear representation of the capacity spectra, as illustrated by the following formulas, which were adopted to calculate the damage state thresholds according to [55].

|          |                                    |
|----------|------------------------------------|
| Slight   | $Sd_{ds1} = 0.70 \times D_y$       |
| Moderate | $Sd_{ds2} = D_y$                   |
| Severe   | $Sd_{ds3} = D_y + 0.75(D_u - D_y)$ |
| Complete | $Sd_{ds4} = D_u$                   |

To assess the variability of fragility curves for the damage states, the values of the standard deviation ( $\beta_{dsi}$ ) were established from values provided in HAZUS-MH-MR1 (Tables 6.6) for mid-rise buildings. The following assumptions were made to achieve this aim:

- 1) the systems of models under consideration display moderate capacity curves variability, that is  $\beta_c = 0.3$ ;
- 2) for slight damage, the damage variability is small (0.2),  $\beta_{T,ds} = 0.65$ ;
- 3) for moderate damage, the damage variability is moderate (0.4),  $\beta_{T,ds} = 0.75$ ;
- 4) for severe and complete damage, the damage variability is large (0.6),  $\beta_{T,ds} = 0.9$  (interpolation value between 0.85 and 0.95 for  $\kappa = 0.7$ );
- 5) the degradation factor of post-yield model response ( $\kappa$ ) is determined in accordance to (Table 5.2) [37] as follows: at  $\frac{1}{2}$  yield = 1.0, at yield = 1.0, and post-yield shaking duration (moderate) = 0.7. These values are based on the assumption that the models were designed according to the high-code (HC) seismic design level, and ordinary (O) for construction quality. Consequently, the lognormal standard deviation ( $\beta_{ds}$ ) values were computed from Equation 7, because the response spectrum is known accurately:

$$\beta_{ds} = \sqrt{(\beta_c)^2 + (\beta_{T,ds})^2} \quad (7)$$

In Eq. 7:

$\beta_{ds}$  is the lognormal standard deviation that represents the total variability of damage state,  $ds$ ,  
 $\beta_c$  is the lognormal standard deviation that represents the variability of the capacity curve,  
 $\beta_{T,ds}$  is the lognormal standard deviation that represents the variability of the threshold of damage state,  $ds$ .

The NSP analysis was performed in both main directions (X and Y) for all models. The third load distribution (first mode) was considered because it is the one governing. The pushover curves were converted automatically in SAP2000 into spectral acceleration-displacement curve format. The yield and ultimate spectral accelerations ( $A_y$  and  $A_u$ ) and the spectral displacements ( $D_y$  and  $D_u$ ) of the spectral bilinear capacity are given in Table 4-7, while the thresholds of damage states are illustrated in Table 4-8.

Table 4-7. Characteristic accelerations and displacements

| Model Identification | Yield capacity |           | Ultimate capacity |           |
|----------------------|----------------|-----------|-------------------|-----------|
|                      | $D_y$ (mm)     | $A_y$ (g) | $D_u$ (mm)        | $A_u$ (g) |
| M-IR1, mode-X        | 50.69          | 0.36      | 140.67            | 0.51      |
| M-IR2, mode-X        | 51.38          | 0.36      | 151.05            | 0.59      |
| M-IR3, mode-X        | 61.75          | 0.53      | 194.24            | 0.76      |
| M-IR4, mode-X        | 59.81          | 0.49      | 164.81            | 0.65      |

Table 3-8. Damage state thresholds and beta values

| Model Identification | Damage state thresholds (mm) |            |            |            | Standard deviation |            |            |            |
|----------------------|------------------------------|------------|------------|------------|--------------------|------------|------------|------------|
|                      | $Sd_{ds1}$                   | $Sd_{ds2}$ | $Sd_{ds1}$ | $Sd_{ds2}$ | $Sd_{ds1}$         | $Sd_{ds2}$ | $Sd_{ds1}$ | $Sd_{ds2}$ |
| M-IR1                | 35.48                        | 50.69      | 73.19      | 140.67     | 0.72               | 0.81       | 0.95       | 0.95       |
| M-IR2                | 35.97                        | 51.38      | 76.30      | 151.05     | 0.72               | 0.81       | 0.95       | 0.95       |
| M-IR3                | 43.23                        | 61.75      | 94.88      | 194.25     | 0.72               | 0.81       | 0.95       | 0.95       |
| M-IR4                | 41.87                        | 59.81      | 86.06      | 164.80     | 0.72               | 0.81       | 0.95       | 0.95       |

The fragility curves of the setback irregular models under consideration and of the regular (archetypal) model were developed to investigate of the impact of vertical irregularity location (setback) on the vulnerability of the frame models. The displacement corresponding to the Slight, Moderate and Extensive states for 50% and 90% probabilities and to the Complete damage for 20% and 70% probabilities are shown in Table 4-9 below.

Table 4-9: Displacement (mm) corresponding to damage states for all models

| Building model | Probability of slight damage state |            | Probability of moderate damage state |            | Probability of extensive damage state |            | Probability of complete damage state |          |
|----------------|------------------------------------|------------|--------------------------------------|------------|---------------------------------------|------------|--------------------------------------|----------|
|                | At 50%                             | At 90%     | At 50%                               | At 90%     | At 50%                                | At 90%     | At 20%                               | At 70%   |
| <b>Regular</b> | <b>85</b>                          | <b>115</b> | <b>140</b>                           | <b>168</b> | <b>175</b>                            | <b>244</b> | <b>280</b>                           | <b>-</b> |
| M-IR1          | 20                                 | 36         | 53                                   | 67         | 90                                    | 112        | 135                                  | 170      |
| M-IR2          | 20                                 | 40         | 60                                   | 76         | 103                                   | 126        | 143                                  | 175      |
| M-IR3          | 80                                 | 102        | 123                                  | 146        | 175                                   | 203        | 215                                  | -        |
| M-IR4          | 20                                 | 50         | 69                                   | 89         | 120                                   | 152        | 170                                  | 180      |

From the comparison of the results for the regular model and for the setback models, it is noticeable, based on the definition in [54], that the spectral displacement corresponding to the Slight Damage state for 50% probability is higher by 76.47% as compared with the M-IR1, M-IR2 and M-IR4 models, respectively, and higher by 5.88% as compared with the M-IR3 model (which has two smaller setbacks). The spectral displacement for the regular model, corresponding to the Slight Damage state for 90% probability is higher by 86.70%, 65.22%, 11.30% and 56.52% as compared with the M-IR1, M-IR2, M-IR3 and M-IR4 models, respectively.

The influence of the setback level on the seismic vulnerability is obvious for the Moderate Damage state, where the spectral displacement for 50% probability is higher by 62.14%, 57.14%, 12.14% and 50.71% for the regular model, as compared with the M-IR1, M-IR2, M-IR3 and M-IR4 models, respectively. The spectral displacement corresponding to the Moderate Damage state for 90% probability is higher, in the case of the regular model, by 60.12%, 54.76%, 13.10% and 47.02% as compared with the M-IR1, M-IR2, M-IR3 and M-IR4 models, respectively.

In the same way, for the Extensive Damage state, the spectral displacement corresponding to the regular model, at a probability of 50%, is higher by 48.57 % as compared with the M-IR1 model and by 41.14%, 0.0% (no value) and 31.43% as compared with the M-IR2, M-IR3 and M-IR4 models, respectively. For a probability of 90%, the spectral displacement is higher by 54.10%, 48.36%, 16.80% and 37.70% in the case of the regular model, as compared with the M-IR1, M-IR2, M-IR3 and M-IR4 models, respectively. In the same way, for Complete Damage, the spectral displacement corresponding to this damage state is higher for the regular model by 51.79%, 48.93%, 23.21%, 39.29%, as compared with the M-IR1, M-IR2, M-IR3 and M-IR4 models, respectively, at a probability of 20%.

For the target displacement shown in Figures 4-11 and 4-12, the probabilities corresponding to the Moderate Damage state are about 40%, 55%, 80%, 60% for the M-IR1, M-IR2, M-IR3 and M-IR4 models, respectively, while the probabilities corresponding to the Extensive Damage state are about 60%, 45%, 20%, 40% for the M-IR1, M-IR2, M-IR3 and M-IR4 models, respectively. The probability is 0.0 for the Slight Damage and Complete Damage states. It can be noted from the results that, when the irregularity setback level increases, the damage hazard increases, and the models exhibit poorer seismic performance. In addition, was noticed at target displacement the impact of the setback level on seismic vulnerability are more in the state of a moderate damage and an extensive damage.

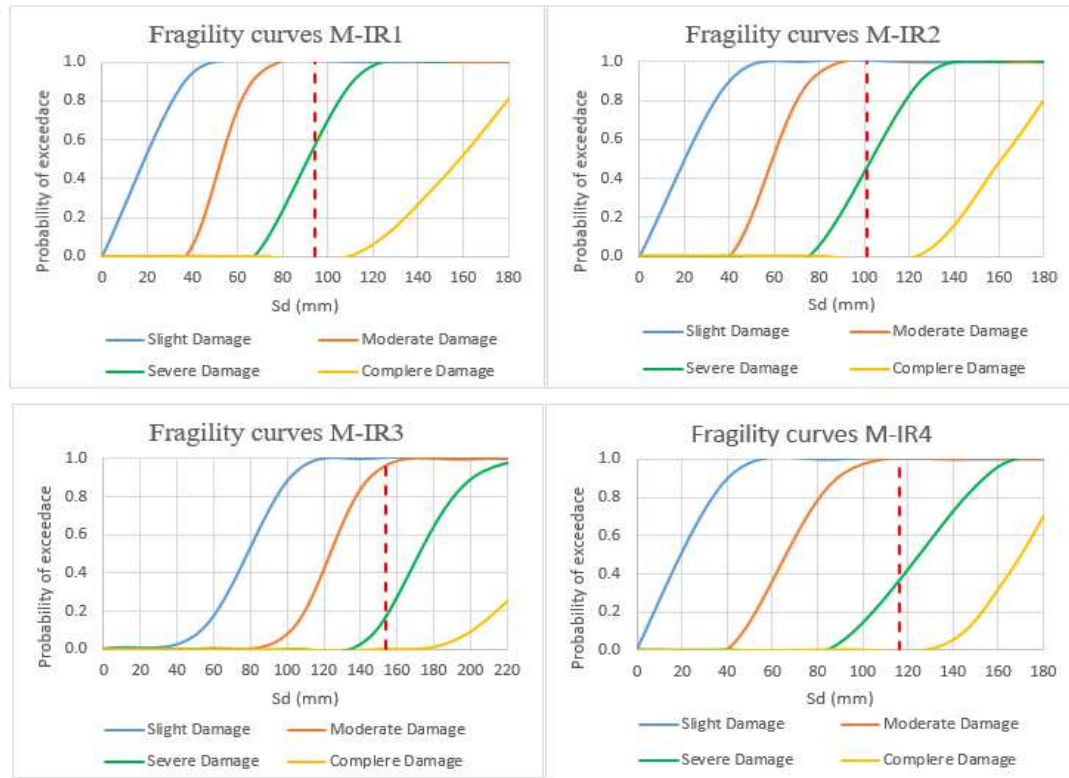


Figure 4- 11. Probabilities at target spectral displacement for M-IR1, M-IR2, M-IR3 and M-IR4

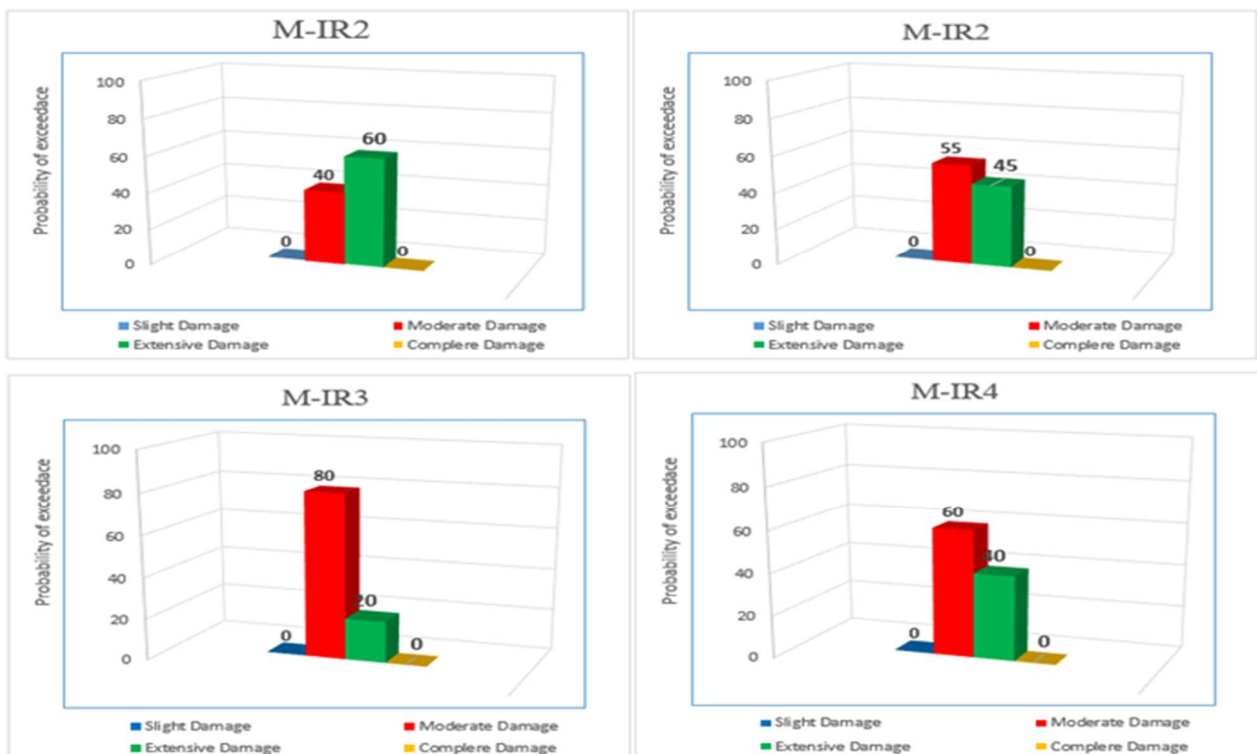


Figure 4-12: Summary of the probabilities of the considered damage states for the target displacements. Models M-IR1, M-IR2, M-IR3 and M-IR4

#### 4.9.2 Performance of the structural members

Figures 4-37 and 4-38 summarize the shear capacity ratio for columns, which represents the ratio of the shear demand on column  $V$ , to the shear strength  $V_n$ . In all cases, the shear demand  $V$  is the maximum shear force occurring in columns of the story levels during the nonlinear static and nonlinear dynamic analysis. The shear strength  $V_n$  is calculated from the following equation, according to ACI 318-19, Section 22.5.1.1, and to equation (22.5.1.1) in this code:

$$V_n = V_c + V_s \quad (8)$$

where:

$V_n$  = nominal shear strength

$V_c$  = nominal shear strength provided by concrete, from table 22.5.5.1,  $= [2\lambda \sqrt{f_c'} + \frac{Nu}{6Ag}] b_w d$

$V_s$  = nominal shear strength provided by the shear reinforcement  $= \frac{A_v f_y d}{s}$

$d$ : is the effective depth of the column ( $d = 0.8h$  was assumed);  $b_w$ : is the width of the column;

$Ag$ : is the gross cross-sectional area of the column;

$Nu$ : is the axial compression force (set to zero for tension force);  $f_y$ : is the yield strength;

$f_c'$ : is the compressive strength of concrete;  $A_v$ : is the area of transverse reinforcement with spacing  $s$ ;

$\lambda$  = modification factor, 1.0 for normal weight aggregate concrete.

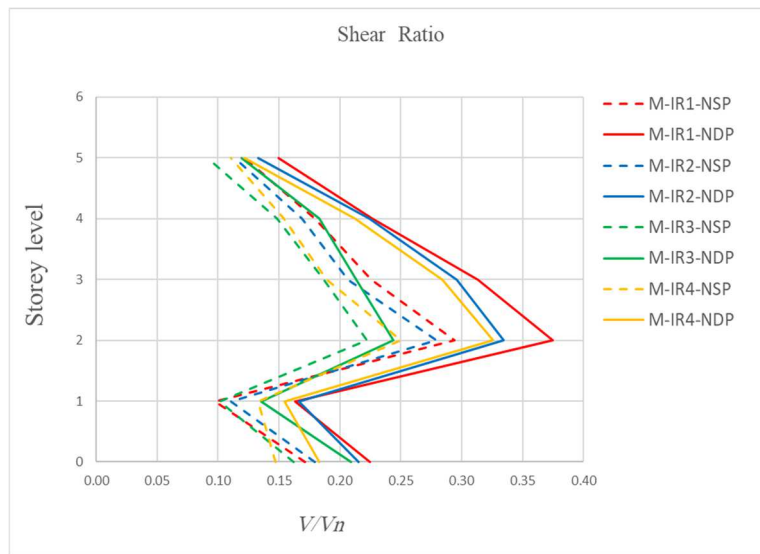


Figure 3-12: Shear demand to shear strength ratios in columns comparison of M-IR1, M-IR2, M-IR3 and M-IR4 models for NDP and NSP

From the resulting values plotted in Figure 4-38, it can be noticed that the shear ratios for the NDP analysis are larger than the corresponding ones for the NSP analysis (1<sup>st</sup> mode load distribution), for all models. For M-IR1, the difference along the height of the model is (19...41)%, whereas for M-IR2 the difference is (17...34)%. For M-IR3 the difference is (9...28) %, and for M-IR4 the difference is (14...33)%. From the results, it can be concluded that the relatively higher values of the shear ratio occurred at the second story for all models, with reduction in ratios at the upper stories. This is because the lower structure is stiffer than the upper one. It is noteworthy that the shear capacity ratios remain lower than 1.0 (actually they do not exceed 0.4), thus the seismic performance of the columns under consideration (choosing the most critical columns in each frame) is quite satisfactory.

## CHAPTER FIVE

### 5 CONCLUSIONS

This thesis encompasses two different studies, focused on various types of structural irregularities. The first one, dedicated to plan irregularities, is focused on the in-depth investigation of the seismic behaviour of a case study consisting of a plan irregular building in Bucharest (Romania), based on various methods and putting in evidence its vulnerability using various criteria. The second study deals with various types of setback models, derived based on an archetypal building in Baghdad (Iraq), evaluating the capacity of various analysis methods, used internationally and in the Iraqi seismic code, to accurately represent the seismic behaviour of this type of buildings. The fragility curves are derived for the studied models and the influence of model irregularities on the displacements specific to each considered damage state is expressed with reference to the same parameters determined for the regular (archetypal) model.

#### 5.1 Conclusions from the first case study

##### 5.1.1 General conclusions

The analysis of the building using the linear response spectrum method (RSA), the nonlinear static (pushover), and the nonlinear dynamic THA analysis was performed. From the results of the analysis, the following observations were made regarding the sufficiency of the EC8 design of the building, as verified by the use of nonlinear static and dynamic approaches:

1. The building has the required ductility and overstrength, sufficient to justify the behavior factor  $q=4.95$ , used in the design.
2. For the building global inelastic response, the torsional effects were evident in the values obtained from THA, compared to those obtained by pushover analyses, especially in the flexible edge, FE, through the assessment of the effects of higher vibration modes and torsion in both elevation (inter-storey drifts) and plan (roof displacement).
3. The building local response from the elastic analysis (RSA) according to EC8 for internal forces (bending moments and shear forces) is lower than the nonlinear response (pushover and THA) analyses.
4. The modal load pattern results seemed more realistic than the uniform load pattern results for the presented building, the explanation residing most probably in the fact that the response is governed by the walls.
5. The comparison of the values of seismic vulnerability index (SVI) for the building showed conservative results; the results of the THA were about (40%) less than for the pushover modal load pattern in the X+, X- and the Y- directions and about (20%) less in the Y+ direction. Also, the results of the THA were about (20%) less than the those obtained using the pushover uniform load pattern in the X+ and the X- directions and with about (3%, 10%) less in the Y+ and Y- directions, respectively.
6. The comparison of the values of the base shear force for the building showed conservative results; the results of the THA were about (6%, 3%) less than those obtained using the pushover modal load pattern, in the X+ and the X- directions, respectively. The results of the THA were larger than those obtained using the pushover modal load pattern with about (2%, 17%) in the Y+ and the Y- directions, respectively.

From the previous, it was concluded that the investigated RC building with plan irregularity, designed according to the Eurocodes (DCH), is eligible to withstand both the design seismic loads (according to P100-1/2013) as well as the Irpinia earthquake with magnitude 6.9, scaled for  $PGA = 0.30\text{ g}$  (specific for Bucharest) and the compatible simulated accelerograms. From the comparison between THA and pushover analyses, it was found that for the local or global response, the pushover analysis applied is not quite efficient, even though it is less time-consuming and less computationally demanding. The obtained results, conservative, are less accurate when compared to the THA results. Based on these conclusions, it was confirmed that the THA is better than pushover analysis for this type of irregular building, with the disadvantage of being more demanding for the point of view of computer processing time. Consequently, further studies were conducted by using THA exclusively, with conclusions as reported in the next section.

### ***5.1.2 Conclusions on earthquake incidence angle impact on the seismic performance of the RC plan-asymmetric buildings***

The studied building was analyzed using nonlinear time history analyses (THA) and several ground motion records were applied to the building at eight incidence angles, ranging from  $0^\circ$  to  $315^\circ$  in  $45^\circ$  increments. Seven pairs of ground motion records, consisting of 14 accelerograms, were used. Each pair was decomposed to the  $(\bar{X})$  and the  $(\bar{Y})$  components. For the assessment of the seismic directionality influences, two demand parameters were combined, the maximum inter-story drift (MLID) along the height of the building and the estimated Seismic Vulnerability Index (SVI), in order to predict the most critical incidence angle. From the results of the analyses, the following conclusions were obtained:

1. For the individual ground motion records, it was noticed that the incidence angle has a considerable impact on the seismic demand in terms of MLID and SVI, showing that the responses depend not only on the structural features but also on the incidence angle of the seismic action.
2. The largest SVI values occurred at the most critical angle of incidence, for each considered ground motion. This angle is the one for which the highest MLID values were obtained in the flexible edge of the building (FE). The above observation, which applies to all considered accelerograms, shows the influence of the FE on the overall structural response, in terms of SVI.
3. In the Y direction, the stiff edge (SE) and flexible (FE) for all seven accelerograms switched between one another at angles of incidence  $\beta=45^\circ$  and  $\beta=225^\circ$ . For 5 out of 7 accelerograms, the change occurred at  $\beta=90^\circ$ , for 4 out of 7 accelerograms - at  $\beta=270^\circ$ , while for 1 out of 7 accelerograms - at  $\beta=180^\circ$ . This shows a large variability of the critical angle and also that the FS and SE may switch due to the random occurrence of yielding, this being dependent also on the ground motion characteristics.

From the above, it can be concluded that determining the structural seismic performance and damage based only on ground motions applied at principal directions may result in inaccurate assessments. The combination of the two demand parameters, the maximum inter-story drift (MLID) along the height of the building and the estimated Seismic Vulnerability Index (SVI), showed the importance of the evaluation the influence of the FE and the SE on the structural demand. In addition, the difficulty of correctly predicting the critical angles of incidence when designing an irregular building was highlighted, given that the building response does not depend only on the characteristics of the building, but also on those of the ground motion. This was obvious from the different critical angles at which the highest SVI and MLID values occurred for the different ground motions considered.

Therefore, the use of a sufficient number of bi-component representative accelerograms for the building site, as well as the detailed study of the seismic response for various angles of incidence is definitely needed for such type of buildings.

## 5.2 Conclusions from the second case study

In this study, the seismic response of multi-story RC frames with different irregular setbacks, designed for Baghdad locations, is studied and analyzed by three different methods: the equivalent lateral force procedure, ELFP; the nonlinear static procedure, NSP (with three load pattern distributions) and the nonlinear dynamic procedure, NDP. Several parameters were investigated (i.e., the inter-story drift, the local vulnerability index (VI) and the seismic performance of the structural members, measured by the shear capacity ratio, i.e. the shear demand on the column,  $V$ , divided by the shear strength  $V_n$ . Furthermore, the vulnerability of the studied buildings was evaluated based on the determination of the fragility curves. The following observations were made.

1. The applicability of a two-stage equivalent lateral force analysis (ELFP) for structures that have a flexible upper portion over a rigid lower portion (proposed in [47] as an improvement of the method in ASCE/SEI 7-16), was investigated by the verification of the inter-story drift (IDRa) parameter obtained from this approach and by its comparison with the results of the nonlinear static analysis, NSP, and of the nonlinear dynamic analysis, NDP. From the results, it was concluded that the ELFP, which is adopted in the Iraqi code ISC 2016 to analyse and design the buildings in Iraq, is not appropriate for the analysis of setback structures. Consequently, there is an urgent need to adopt appropriate and more effective methods for designing and evaluating the performance of reinforced concrete buildings in particular, because they are the most used type in Iraq.
2. The results obtained using the “first mode” load distribution for NSP showed the smallest target shear capacity and the greatest target displacement demand among the three load distribution patterns considered in analysis. Consequently, this was considered the critical pattern.
3. The comparison of the results obtained for all models for NDP and NSP (“first mode” load distribution pattern) of the inter-story drift and shear capacity ratio for columns showed that NSP is unable to simulate the impacts of higher modes on the structural response, which become important when the irregularity of the structure increases. Therefore, NDP is the accurate method for this type of building.
4. It is worth noting that the shear capacity ratio for columns, expressed by the ratio of the shear demand on the column,  $V$ , to the shear strength,  $V_n$ , remained lower than 1.0 (actually did not exceed 0.4), thus the seismic performance of the columns under consideration (choosing the most critical columns in each frame) is quite satisfactory.
5. The fragility curves for the four studied setback models were developed based on NSP, although NDP is more reliable and accurate. The preliminary evaluation of the buildings can, however, allow the use of a simple method as NSP. It can be observed that, when the irregularity (setback) level increases, the damage hazard increases, and the models exhibit poorer seismic performance. The NSP has also been used in many studies to analyze irregular buildings [i.e., 56-62]. However, given the lack of earthquake damage information required to calibrate the levels of damage proposed by vulnerability functions, the reliability of these functions remains for now a critical matter.
6. The fragility curves developed in this study could be used as preliminary investigation in seismic risk scenarios in Iraq (Baghdad) for irregular setback buildings. Further processing of these curves is considered necessary to account for the potential contrast in input parameters, which are selected

for the nonlinear analysis, the damage state thresholds determination, and the hypotheses that have been used for fragility curves for each of the considered damage states.

7. As the model setback level increases, the target shear capacities and the target displacements decrease under the effect of the three load pattern distributions used for NSP. This due to the decrease of the structure capacity.
8. The severity or degree of models setbacks also influences the increase in damage, so setback ratios are studied to take into account their severity. The forces are concentrated on the section of the structure where the abrupt decrease in stiffness occurs, i.e., at the bottom of the upper structure. This can be noticed from the development of the plastic hinges at this location. Consequently, the abrupt change in stiffness or the irregular vertical configurations of the structures are considered to represent local vulnerability locations.

## 6 CONTRIBUTIONS TO THE STUDY OF THE FIELD

Two categories of analytical case studies, aimed for investigating plan and vertical irregularity structures, were performed. The contributions brought by the studies are briefly presented in the following.

- The first category concerned a nine-story reinforced concrete dual system (wall-frame) building located in Bucharest, categorized as being plan-asymmetric. The idea that was put forward in this case was first assessed comparatively, according to the Romanian, European and USA codes. According to the comparison of code criteria, Eurocode 8 resulted as the most restrictive for the evaluation of the plan irregularity conditions.

Considering torsional effects by evaluating the effects of the higher modes in both elevation (inter-story drifts) and plan (roof displacement), the torsional effects, which were evident from the results of the THA, as compared to those obtained from the pushover analysis, were observed especially in the flexible edge of the building.

The seismic vulnerability, expressed by the structural vulnerability index SVI, was investigated through the comparison of the SVI values determined, for the studied building, from the pushover analysis and the THA. Both types of analysis indicated very similar global performance levels, based on SVI values. It was noticed, however, that the SVI values obtained from the pushover analysis were larger as compared to those obtained from the THA.

The combination of the two demand parameters, the maximum inter-story drift (MLID) along the height of the building and the estimated Seismic Vulnerability Index (SVI) was used for the assessment of the seismic directionality influences, in order to predict the most critical incidence angle. The influence of directionality on the response of the building was investigated by non-linear dynamic analyses, by applying seven scaled bi-directional ground motion records oriented on 8-incidence angles with values ranging from  $0^\circ$  to  $315^\circ$ , with a  $45^\circ$  increment. The results showed the importance of the evaluation the influence of the FE and the SE on the structural demand. In addition, the difficulty of correctly predicting the critical angles of incidence when designing an irregular building was highlighted, given that the building response does not depend only on the characteristics of the building, but also on those of the ground motion. Also, the positions of the SE and FE changed during the earthquake for all cases corresponding to angles of incidence, the positions of the SE and FE changed during the earthquake for all cases corresponding to angles of incidence

- The second category of case studies focused on four six-story reinforced concrete buildings, with different setback configurations and designed for locations in Baghdad, Iraq. The idea that was put

forward in this case was the assessment of the applicability of a simplified two-stage equivalent lateral force analysis for structures that have a flexible upper portion over a rigid lower portion (proposed by [Yuan] as an improvement of the method specified by ASCE/SEI 7-16. In this study, setback frames corresponding with this type of structural configuration were investigated first by the verification of the inter-story drift (IDRa) parameter obtained from the mentioned approach and by its comparison with the results of the nonlinear static analysis, NSP, and of the nonlinear dynamic analysis, NDP.

Although, in the past, Iraq was rarely exposed to seismic activity, in recent years, seismic activity has begun to increase in some parts of Iraq, including the eastern region bordering Iran, which led to its effects reaching Baghdad. Consequently, there is an urgent need to adopt appropriate and more effective methods for designing and evaluating the performance of concrete buildings in particular, because they are the most used type in Iraq.

The fragility curves for the four studied setback models were developed based on NSP. From the results, it could be observed when the irregularity (setback) level increases, the damage hazard increases, and the models exhibit poorer seismic performance. The fragility curves developed in this study could be used as preliminary investigation in seismic risk scenarios in Iraq (Baghdad) for irregular setback buildings.

The Structural Local Vulnerability factor ( $SVI_{Fi}$ ) was then considered for the assessment of the structural vulnerability of the setback models. When there is an increased chance of local failure, this index can be used to determine the increase or distribution of local damage in a frame of the structure. For all setback models, the factor of local vulnerability increased for the third and fourth frames, as compared to the regular model, when the irregularity indices,  $\phi_{s,b}$ , increased. This provided a quantitative evidence that the local vulnerability was concentrated at bottom portion of the upper structure of the setback models. The use of this index could be also extended for various types of other irregular structural configurations.

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