An-Najah National University
Faculty of Graduate Studies

Seismic Assessment and Rehabilitation of Existing Buildings Using Nonlinear Static Procedures (NSPs) -Pushover Procedures-

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Dedication

Acknowledgement

First of all, thanks to Allah for everything. I would like to gratefully acknowledge my advisors, Dr. Mahmud Dwaikat and Dr. Abdul-Razzaq Touqan, for their guidance and support throughout my graduate studies and the enormous effort they made to revise this document and make possible my graduation.

Special thanks to my family, for their tremendous and unconditional love, encouragement and support all the time. I would like to thank my friends for their support and friendship.

I owe a massive amount of appreciation to all helped me with my studies.

الإقرار

انا الموقع ادناه مقدم الرسالة التي تحمل العنوان:

Seismic Assessment and Rehabilitation of Existing Buildings Using Nonlinear Static Procedures (NSPs) -Pushover Procedures-

أقر بأن ما اشتملت عليه هذه الرسالة إنما هو نتاج جهدي الخاص، باستثناء ما تمت الإشارة اليه حيثما ورد، وأن هذه الرسالة ككل من أو جزء منها لم يقدم لنيل أية درجة أو بحث علمي أو بحثي لدى أي مؤسسة تعليمية أو بحثية أخرى.

Declaration

The work provided in this thesis, unless otherwise referenced, is the researcher's own work, and has been submitted elsewhere for any other degree or qualification.

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Date:	**التاريخ:**

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Abstract

Design of buildings for seismic loads is becoming mandatory in Palestine. However, what about the existing buildings? Existing buildings, especially old ones, were mostly designed under the influence of static loads. Such buildings may stand vulnerable to earthquakes and thus need to be strengthened; so that they become safe. To achieve the required level of strengthening, advanced analysis and assessment tools must be used.

There is a lack of systematic studies that provide practical "know-how" guidelines for local engineers on the assessment and retrofitting of existing buildings against seismic loads. Generally, the guidelines written in foreign codes (e.g. the ASCE or FEMA) are very broad and general and may pose a challenge to local engineers regarding the consistency of their implementation. This study bridges this gap between local engineers and international codes by putting these guidelines into action through a practical case study.

Generally, four procedures are available for seismic analysis of buildings: two linear procedures, and two nonlinear procedures. The nonlinear procedures include the nonlinear static procedure (NSP) and nonlinear dynamic procedure (NDP). NSP's are deemed to be very practical tools to

assess the nonlinear seismic performance of structures. On the other hand, NDP's require detailed input data, and it is very time-consuming, which is a relevant drawback in design offices, where the deadlines are restrictive. Also, it doesn’t exist in Palestine neither local earthquake records, nor specialized powerful programs for NDP. This makes the NSP best choice for practical assessment of buildings.

The research objective in this thesis is to demonstrate an assessment methodology through studying a local existing building, which was designed under gravity loads only, and then propose retrofitting solutions to remedy the deficiencies in the building.

Based on the above, the case study building is assessed using an NSP that is called capacity spectrum method (CSM) as per ATC-40. The behavior of the structure is generated using nonlinear pushover analyses.

The seismic assessment were conducted based on FEMA 356 performance criteria. According to FEMA 356, there are two approaches for seismic evaluation: global-level and member-level with three performance levels, which are immediate occupancy (IO), life safety (LS) and collapse prevention (CP). In addition, seismic design requirements that are mentioned in ASCE 7-10 were conducted in order to assess the building for irregularities.

Based on the nonlinear pushover analysis and the assessment of the building, it was found that the building suffers from vertical irregularities and concentration of plastic hinges at the ground floor.

In order to improve the performance of the building, two possible retrofitting techniques were applied including the addition of RC column jackets, and moment resisting RC frames.

The capacity curves for the retrofitted structure were compared to those for the un-retrofitted case. FEMA global drift limits were compared with the drift limits of the performance points of each retrofitting techniques based on the FEMA member-level criteria. In addition to this, the ASCE limits were also rechecked and compared to the ratios of the un-retrofitted building. The retrofitting techniques helped improve the performance of the building. This thesis paves the way to further research on seismic assessment of existing buildings with effective tools for judging the efficiency and suitability of retrofitting techniques.

CH1 Introduction

1.1 Forward

Palestine is a seismic zone that it is located along the Dead Sea Transform, which is an extension of ground faults separating the Arabian and African plates (Figure 1.1.1). The seismic history of the region indicates the occurrence of destructive earthquakes. The last devastating earthquake that hit the area was in 1927, which claimed the lives of dozens of residents under the rubble of their homes. [SASPARM Project, 2014]

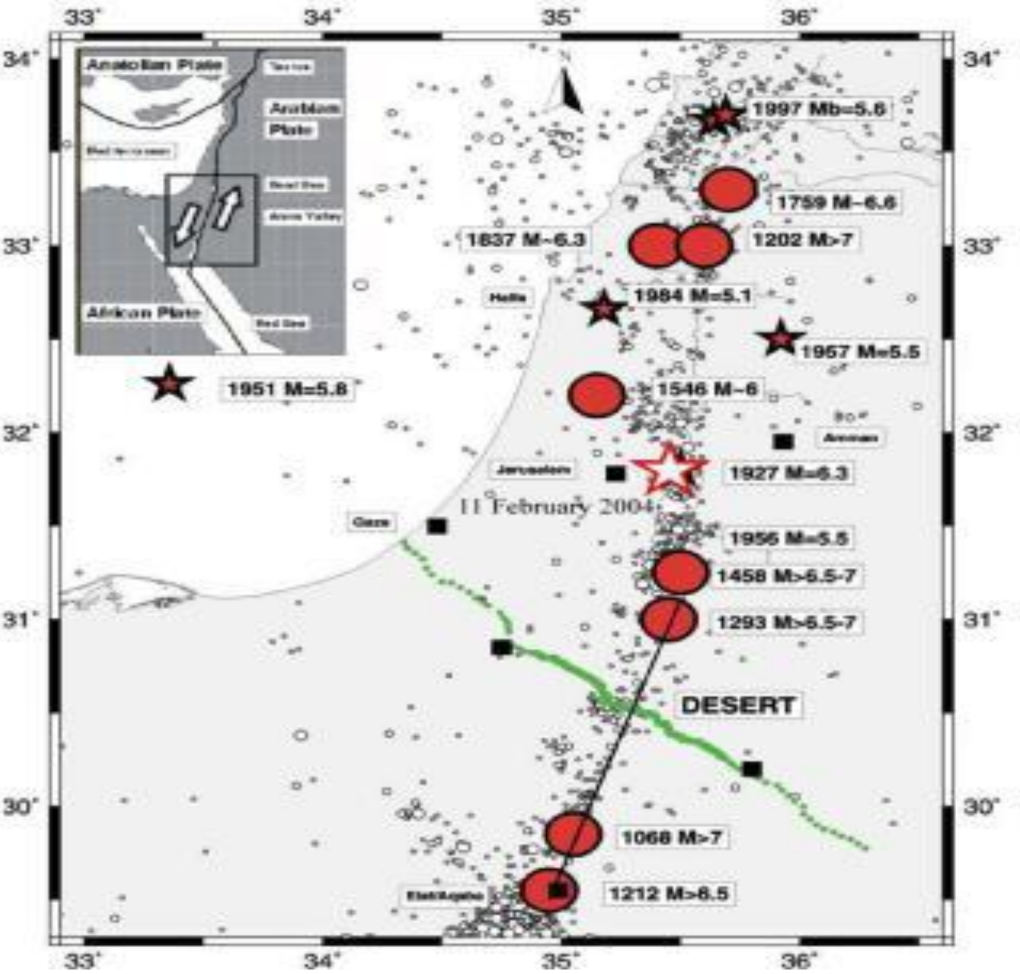

Figure 1.1.1. Seismicity map of the Dead Sea transform region (circles represent seismic events). [SASPARM Project, 2014]

Unfortunately, this bloody history was not enough motivation to work on mitigating the seismic risk or retrofitting of old buildings in this region. There is, however, a glimmer of hope in improving the level of construction by spreading awareness among the society and the designers. The first step was adopting a decision by the Palestinian Engineers Association (PEA), which imposes seismic design as compulsory for new facilities.

The (PEA) decision is a step towards seismic risk mitigation. The (PEA) did not issue a mandatory decision regarding the status of existing buildings. Most of the existing buildings are vulnerable to earthquake events. Ignoring the existing buildings in term of earthquake resistance can cause the following problems:

1- High risk for citizens in event of earthquakes.
2- The risk of closure of major roads or important facilities, which hinders relief efforts.
3- Expensive damage to private and public properties.
4- Legal dilemma: difficulty in specifying responsibility regarding the collapse of buildings that were not designed to resist earthquakes.

One reason behind ignoring the existing buildings is the lack of systematic procedures for evaluating such buildings and for identifying the weaknesses and risks in these buildings, which makes it difficult to adopt retrofitting policies that would improve seismic resistance of such buildings.

In this research, a procedure of using existing method for evaluation of existing buildings, and how to judge their behavior in earthquake events will be displayed. Also, techniques to retrofit and strengthen the existing buildings will be presented. The applicability of the presented methods will be demonstrated through a case study building. This research will provide useful and practical information for engineers who maybe in need for tools to evaluate existing buildings.

1.2 Concept of retrofitting reinforced concrete (RC) buildings

Many of existing buildings in Palestine were not designed to resist earthquake loads, and thus may represent real hazard in event of earthquake. This means that there is a need to retrofit these buildings. This requires the improvement of resistance to earthquake loads by improving and modifying the structural elements that play major role in resisting earthquake loads.

There are variety of structural systems used in buildings, such as, framed systems, shear-wall systems, masonry wall systems, and dual systems. This causes the retrofitting methods to vary from one system to another.

There are three retrofitting domains stiffness, strength, and ductility. The increase in stiffness means increase in lateral resistance to sway drifts in buildings. More strength means the structure can bear and sustain larger loads. More ductility means that the structure can undergo more plastic deformations before failure occurs, when it is compared to other structures.

The structural system and other architectural shapes affect each retrofitting domain. In this thesis, the three domains will be discussed with respect to existing buildings in terms of earthquake resistance.

1.3 Performance based design

Performance based seismic design (PBD) is a new approach to earthquake resistant design. It is more realistic than force based design methods that are based on prescriptive and mostly empirical code formulations. (PBD) is a recent method to design buildings based on predictable and target seismic performance. Therefore, performance objectives such as immediate-occupancy (IO), life-safety (LS), or collapse prevention (CP) are used to define the state of the building when exposed to earthquake loads. In one sense, performance based seismic design is a limit-state design extended to cover the complex range of performance requirements faced by earthquake engineers. There has been much researches on PBD, and many researches tried to come up with the most realistic and accurate procedures for PBD [Chopra, 2012].

One common procedure is the capacity spectrum method (CSM) through pushover analysis. In this study, this method of PBD will be presented and demonstrated through a case study building to provide a tool for local engineers to assess structures against seismic behavior.

1.4 Seismic analysis of (RC) structure

Current seismic design codes in the world are generally carried out by linear static procedures (LSPs), such as equivalent lateral force (ELF) and response spectrum methods (RSA). However, the designed structures can be exposed to large inelastic deformations in strong earthquake events, which is inaccurately accounted for in the current force-based design methods. The drawbacks of (LSPs) will be discussed in chapter two in this thesis.

The most realistic design method must account for the development of plastic deformations in the structure during an earthquake event. In addition, hysteretic behavior of the structure during earthquake event must be considered, in order to predict the capacity of the structure to resist earthquake loads and not to exceed the designed limit level.

The nonlinear time-history analysis method meets the previous consideration. However, it requires high accuracy in the selection of characteristics and assumptions to reach the correct results, and requires very powerful tools for the calculation-intensive nonlinear analysis.

In the last two decades, the need for simple evaluation tools for existing buildings led to new methods related to performance-based approach. These include the nonlinear static analysis (pushover analysis). The main idea in this procedure depends on estimating the capacity curve (pushover curve) and the demand response spectrum curve. The pushover curve represents the behavior of the structure during the elastic and plastic range until collapse, while the demand curve represents the magnitude of predicted earthquake force. The point of intersection between these two curves is called the performance point

The pushover curve (or capacity curve) can be generated by subjecting the structure to one lateral load pattern or more depending on the natural fundamental modal shapes. Then increasing the magnitude of these loads monotonically to generate a nonlinear inelastic force-deformation relationship curve. The load vector is usually chosen to be representative of the load acting on the structure while vibrating in its first mode as a fundamental modes to be compatible with the seismic response of the building.

The seismic demand curve (response spectrum curve) is a representation of the earthquake-induced response to the building, and it is presented in terms of peak acceleration-time relationship. Capacity curve (generated earlier by pushover analysis) must be converted from MDOF into an equivalent SDOF in a format representing peak acceleration and peak displacement. The resulting curve is called capacity spectrum curve. Then response spectrum is also converted into acceleration-displacement response spectra format (ADRS). Both curves are plotted as spectral acceleration with spectral displacement. The response spectrum curve must be reduced such that it accounts for reduction in stiffness and absorbed energy during earthquakes event. The performance point is determined as the intersection of the capacity spectrum and the reduced seismic demand curve.

This method of thinking is gaining popularity among earthquake engineers, and represents a basis for performance based design approach.

1.5 Objectives and scope of the research

The main objective of this work is to present a methodology for evaluating performance of existing buildings under seismic loads. Then improve the performance by means of retrofitting techniques and study the effect of these techniques on the performance of the retrofitted building. Non-linear static procedure will be used in studying the existing building before and after retrofitting until a specific performance target is achieved.

The general objectives in this study are the following:

A- Present a methodology for the seismic assessment of existing buildings.

B- Progress step towards spreading the awareness of seismic performance based analysis and design that gives a clear impression about the realistic behavior of the structure under seismic loads.

C- Present methodology for assessing the effect of different retrofitting techniques on the seismic capacity and demand curves of buildings.

The objectives above can be attained by achieving the following tasks:

1- Selection of a representative existing building as a demonstration vehicle for the methodology.

2- A software for doing the nonlinear pushover analysis will be selected and then verified through comparison to manual calculations for some selected cases.

3- Establishment of a three-dimensional model that simulates the existing building using the program in order to understand its behavior.

4- Performing pushover analysis using both material and geometric non-linearities, in order to draw the capacity curve of the modelled building.

5- Establishing the performance point of the structure based on the intersection of capacity and demand curves.

6- Identifying acceptable performance target for the selected building using relevant codes and standards and logical judgment.
7- Proposing retrofitting techniques and repeating pushover analysis for the retrofitted building until the performance target is achieved.
8- Comparison between different retrofitting techniques and their effect on capacity curves will be done based on their results and performance.

1.6 Thesis outlines

This thesis will be organized according to the following structure:

Chapter 1: - Introduction

The seismic history of area will be presented. Brief talk about retrofitting and performance based design is presented. Also objectives and scope of the work will be discussed briefly.

Chapter 2: - Literature review

A brief review for analytical methods that are used in the design and analysis of structures for seismic loads is presented. In addition, this chapter talks about the criteria used by FEMA 356, which evaluates seismic performance for overall structure and member performance level. Also, a brief review for important studies relevant to the evaluation of the capacity of existing buildings by experimental or analytical methods will be conducted. Then retrofitting strategies will be mentioned and explained.

Chapter 3: - Case study and modeling features

This chapter describes the case study building: site, architectural geometry, structural system, material, and loads. In addition, it talks about assumptions adopted for modeling the building.

Chapter 4: - Analysis of un-retrofitted case study building

In this chapter, modal and static analyses are done to generate modal shapes, and to check static gravity loads, p-delta effects, horizontal and vertical irregularities. Then pushover methodology is illustrated. After that, the capacity spectrum method used by ATC-40 is explained. Then, modeling pushover features that consist of definition of lateral load patterns and cases, and the plastic hinges properties are presented.

Results of pushover analysis are summarized. Then, the performance level of the building is determined based on the results from pushover and the guidelines given by FEMA 356.

Chapter 5: - Analysis of retrofitting techniques for case study building

In this chapter, general retrofitting techniques are displayed. Then specific retrofitting strategies are selected to be applied to the un-retrofitted building in order to be analyzed and studied. Nonlinear analysis is repeated until performance target is achieved.

Chapter 6: - Conclusions

In this chapter, detailed results are displayed for elastic and inelastic analysis and before and after retrofitting. Then, these results are compared and discussed. Finally, recommendations are concluded.

CH 2 Literature review

2.1 Introduction

This chapter gives a brief introduction to the methods used in seismic analysis and design, seismic performance criteria, and general retrofitting techniques for RC buildings. Elastic analysis methods and their major limitations are outlined. After that, PBD methodology is illustrated and the performance levels are explained.

2.2 (FBD) and performance based seismic Design (PBD) methods

Earlier methods of seismic design were based on idealization of earthquake as a lateral force in what called a force-based method. Recently, (PBD) has been widely used by the researchers since the events of 1994 Northridge Earthquake, which was devastating and a very costly earthquake in U.S. history, and 1985 Mexico earthquake. The goal of PBD is to develop design methodologies that produce structures of predictable and intended seismic performance under stated levels of seismic hazards [SEAOC, 1995]. Then the international codes developed guidelines based on PBD to assess and rehabilitate existing buildings, such as ATC-40 (1996) and FEMA 273 (1997).

2.2.1 Force-based design (FBD) methods

Traditional seismic design codes in the world are generally based on elastic analysis methods, where earthquake is presented as static forces. This comes in contrast to reality, where the structures can be exposed to large inelastic deformations in strong earthquake events, and this is not accurately accounted for in current force-based design methods.

Current building codes use static (ELF) procedures for seismic design of regular structures. A brief sequence of the procedure is illustrated in Figure (2.2.1). This procedure is used for buildings with relatively short periods, but for buildings with relatively long periods, (ELF) procedure could be inaccurate, and the structure must be designed using other procedures [Chopra, 2012]. The design lateral forces acting on any structure depend on vibration properties of the structure and the site classification. Based on the estimated fundamental modal behavior of the structure, formulas are specified for calculating base shear, and then lateral forces are distributed over the height of the building accordingly. Static analysis of the building for these forces provides the design forces, including shears and overturning moments for the different stories and structural elements. [Chopra, 2012].

In these methods, the inelastic behavior of the building is incorporated as a reduction factor "R" of the base shear force.

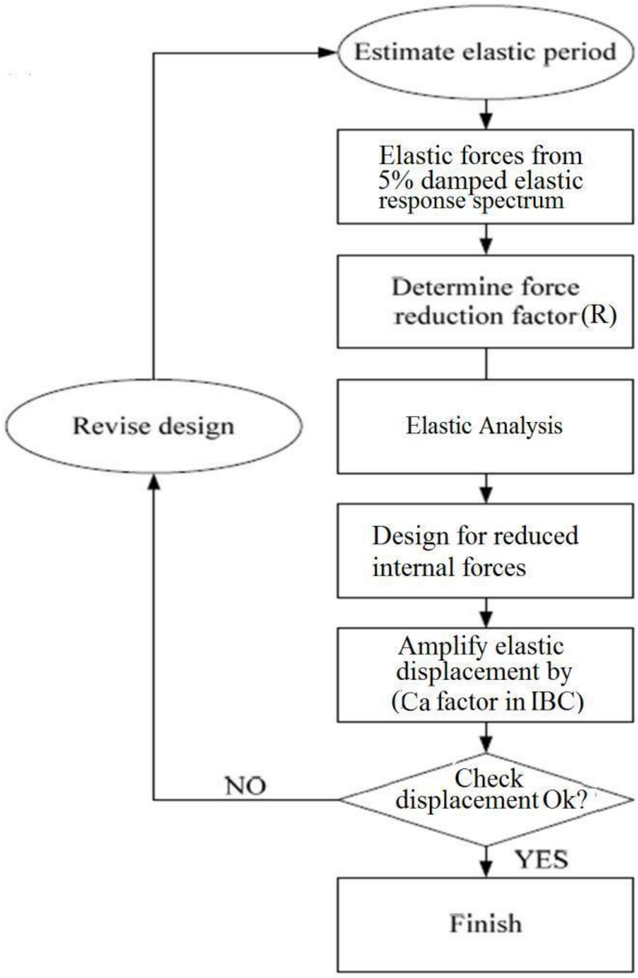

Figure 2.2.1. Force-based design process sequence [Wen-Cheng Liao, 2010]

Figure (2.2.2) shows the process of determining the design base shear as used in ASCE 7-10. The seismic base shear force is generally reduced by a factor (R/I), where (R) represents the force reduction factor depending upon inherent ductility of the structural system, and (I) represents occupancy factor in order to increase the design base shear force for more important buildings according to the category of the building.

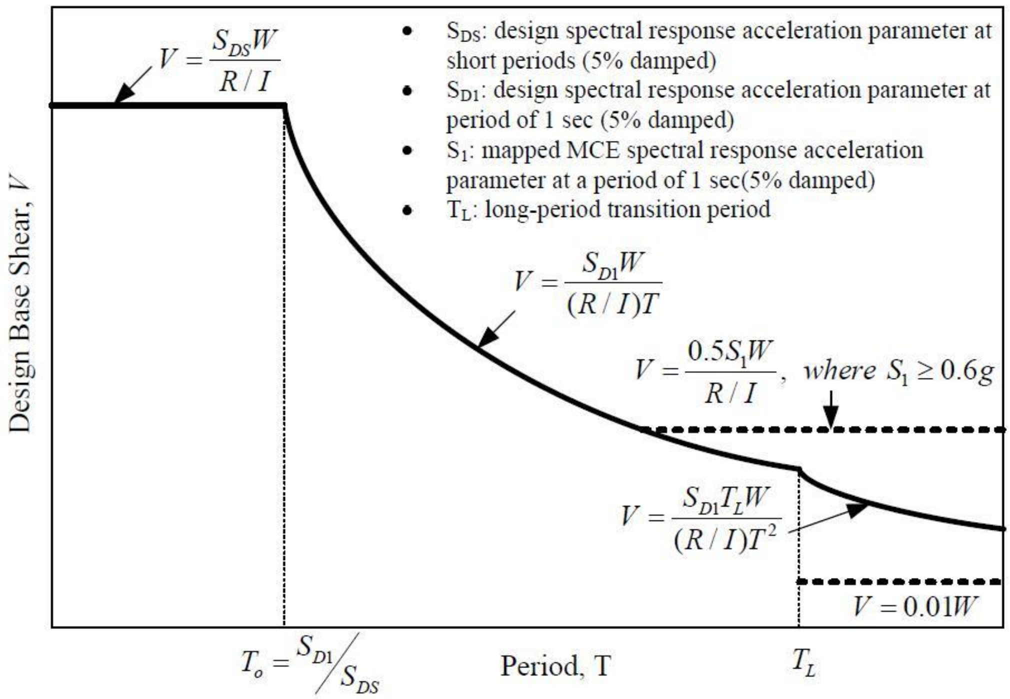

Figure 2.2.2. 5% design response spectrum for seismic design [ASCE 7-10, 2010]

Then lateral design base shear force is distributed along the building height at the floor levels according to the following formulas:

$$Fx = CvxV \quad \ldots\text{Eq. 2.2.1}$$

&

$$Cvx = \frac{w_x h_x^k}{\sum_{i=0}^{n} w_i h_i^k} \quad \ldots\text{Eq. 2.2.2}$$

Where,

F_x = shear force at floor x

C_{vx} = vertical distribution factor

V = total design lateral force or shear at the base of the structure (kN)

w_i & w_x = the portion of the total effective seismic weight of the structure (W) located or assigned to Level i or x

h_i and h_x = the height (ft or m) from the base to Level i or x

k = an exponent related to the effect of modal shape and period as follows:

For structures having a period of 0.5 s or less, k = 1 For structures having a period of 2.5 s or more, k = 2

For structures having a period between 0.5 and 2.5 s, k shall be 2 or shall be determined by linear interpolation between 1 and 2

Elastic analysis is performed to determine the required member strengths. After members design for strength, a deflection amplification factor, Cd according to ASCE 7, is then used to multiply the calculated drift obtained from elastic analysis to check the specified drift limits. The process is repeated in an iterative manner until the strength and drift requirements are satisfied.

Response spectrum depends on computing the statistical peak response of a structure when subjected to a base excitation as shown in Figure (2.2.3). Each of the vibration modes are assumed to respond independently as a SDOF system. Design codes specify response spectra which determine the base acceleration applied to each mode according to its period.

Response Spectrum Analysis (RSA) is used to determine peak displacements and member forces due to support accelerations from each mode of vibration. The "Complete Quadratic Combination" (CQC) method for combining correlated modal responses is generally used to determine the peak response of the structure. This is equivalent to the "Square Root of the Sum of Squares" (SRSS) method if the modes are uncorrelated. RSA is considered as a dynamic procedure. [Chopra, 2012]. The method involves the calculation of only the maximum values of the displacements and member forces in each mode using smooth design spectra that are the average of several earthquake motions.

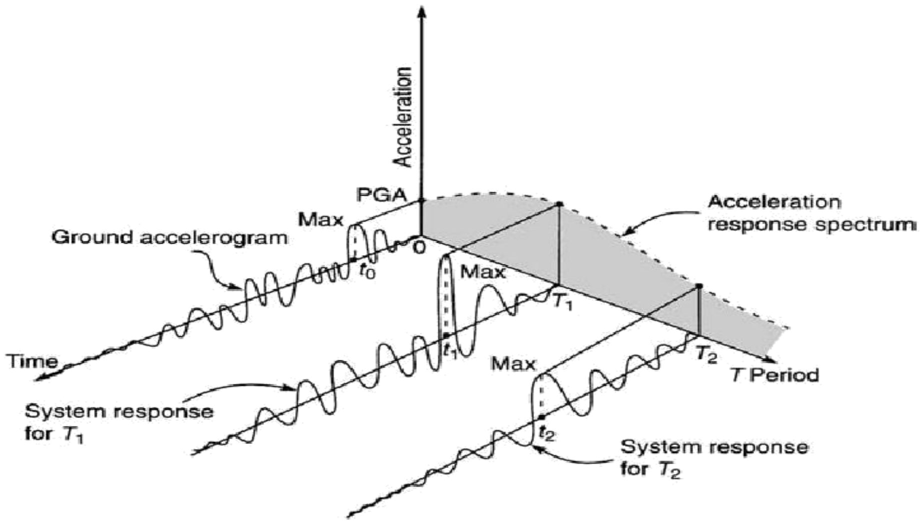

Figure 2.2.3. Statistical maximum response of a SDOF structure subjected to a base excitation

The major limitations and weaknesses of the force based design methods in current codes procedures such as (ELF) and response spectrum analysis (RSA) can be summarized as:

1- In many past earthquakes, it has been observed that in many cases, collapse occurred due to local column damage. This means that safety cannot be guaranteed when the sequence of damage is not clear. [Moehle and Mahin, 1991]. In addition, the distribution of elastic forces depends on stiffness of structural members, which is not accurate, since stiffness of structural members change due to the resulting plastic damage.

2- Nonlinear dynamic analyses research done by Villaverde (1991) showed that using the code distribution of lateral forces, without accounting for the fact that a structure would enter inelastic state during a major earthquake, could be the primary reason leading to numerous upper story collapses during the 1985 Mexico City Earthquake. [Villaverde, 1991]

3- The plastic drift calculated in ELF by using Cd factor or similar factors is not accurate especially for degrading ("pinched") hysteretic behavior and energy dissipation characteristics. [Chao and Goel, 2006]

4- Ductility of higher modes could be different from the ductility of the fundamental modes. Therefore, using the same force reduction factor (R) in all modes may underestimate the higher mode effects in terms of internal forces. [Priestly, 2003]

5- The factor (R) is considered constant for any building with the same structural system.

6- A response spectrum is obtained from an accelerogram by running this record in several single degree of freedom (SDOF) systems with different periods of vibration. The value of the response spectrum corresponding to a certain period is obtained taking the maximum response of the SDOF with that period. As a consequence the duration effects of the dynamic response are ignored, which may not be valid in the case of plastic responses. [Priestly, 2003]

2.2.2 Performance-based seismic design (PBD) methods

As mentioned in chapter (1), performance based seismic design is a limit-state design that is extended to cover the wide range of performance requirements. The performance objectives such as immediate occupancy, life-safety, or collapse prevention (structural stability) are used to define different states of the building when exposed to earthquake loads, see Figure (2.2.4).

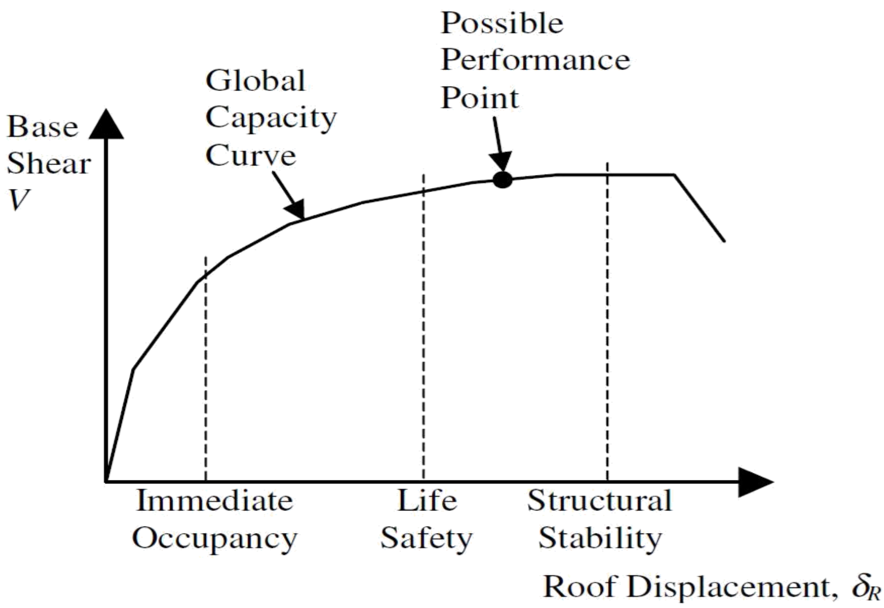

Figure 2.2.4. Building Capacity Curve [ATC-40, 1996]

In performance based seismic design, capacity spectrum is an important description and evaluation for the performance of the structure. There are two basic elements in PBSD method, namely seismic demand and capacity spectrum. The seismic demand represents the earthquake ground motion and it can be observed in terms of spectral accelerations imposed on structures by earthquakes.

The seismic capacity spectrum represents the elastic and inelastic behavior of structure, which is converted from base shear force versus top displacement into spectral acceleration and spectral displacement for equivalent SDOF. The resulting curve is known as the capacity spectrum curve for the building. The process to determine capacity curve relies on the use of nonlinear static analysis (pushover method). The performance point is defined as the intersection point between demand and capacity spectra where the ductility and energy dissipation of structure are matched.

According to FEMA 356, the target performance objectives are divided into two types, Structural Performance Levels (SP-n, where n is a designated number) and Non-structural Performance Levels (NP-n, where n is a designated letter). These may be specified independently, however, the combination of the two determines the overall building performance level. Table 2.2.1 shows possible overall combination. [FEMA 356, 2000]

A description of the structural performance level objectives as per [ATC-40] can be summarized as:

- Immediate Occupancy (SP-1): Limited structural damage with the basic vertical and lateral force resisting system retaining most of their pre-earthquake characteristics and capacities.
- Damage Control (SP-2): A placeholder for a state of damage somewhere between Immediate Occupancy and Life Safety.
- Life Safety (SP-3): Significant damage with some margin against total or partial collapse. Injuries may occur with the risk of life-threatening injury being low. Repair may not be economically feasible.
- Limited Safety (SP-4): A placeholder for a state of damage somewhere between Life Safety and Structural Stability.
- Structural Stability (SP-5): Substantial Structural damage in which the structural system is on the verge of experiencing partial or total collapse. Significant risk of injury exists. Repair may not be technically or economically feasible, which meets collapse prevention in FEMA 356.

- Not Considered (SP-6): Placeholder for situations where only non-structural seismic evaluation or retrofit is performed.

Table 2.2.1. Combinations of Structural and Non-structural Levels to form Building Performance Levels [ATC-40, 1996]

Building Performance Levels						
	Structural Performance Levels					
Non-structural Performance Levels	**SP-1 Immediate Occupancy**	**SP-2 Damage Control (Range)**	**SP-3 Life Safety**	**SP-4 Limited Safety (Range)**	**SP-5 Structural Stability**	**SP-6 Not Considered**
NP-A Operational	1-A Operational	2-A	NR	NR	NR	NR
NP-B Immediate Occupancy	1-B Immediate Occupancy	2-B	3-B	NR	NR	NR
NP-C Life Safety	1-C	2-C	3-C Life Safety	4-C	5-C	6-C
NP-D Reduced Hazards	NR	2-D	3-D	4-D	5-D	6-D
NP-E Not Considered	NR	NR	3-E	4-E	5-E Structural Stability	Not Applicable

Legend

(white)	Commonly referenced Building Performance Levels (SP-NP)
(grey)	Other possible combinations of SP-NP
NR	Not recommended combinations of SP-NP

Non-structural Performance Levels are defined as:

- Operational (NP-A): Non-structural elements are generally in place and functional. Back-up systems for failure of external utilities, communications and transportation have been provided.
- Immediate Occupancy (NP-B): Nonstructural elements are generally in place but may not be functional. No back-up systems for failure of external utilities are provided.
- Life Safety (NP-C): Considerable damage to non-structural components and systems but no collapse of heavy items. Secondary hazards such as

breaks in high-pressure, toxic or fire suppression piping should not be present.

- Reduced Hazards (NP-D): Extensive damage to non-structural components but should not include collapse of large and heavy items that can cause significant injury to groups of people.
- Not Considered (NP-E): Non-structural elements, other than those that have an effect on structural response, are not evaluated.

2.3 Structural analysis types

FEMA 356 divided structural analysis procedures into four procedures: linear static procedure, linear dynamic procedure, nonlinear static procedure (pushover analysis), and nonlinear dynamic procedure (time history). These types will be explained briefly below:

2.3.1 Linear procedures

The linear procedures imply the use of elastic analysis to evaluate the members capacities, then the elastic results are converted to inelastic by multiplying them with empirical inelastic factors. Linear procedures used by FEMA 356 are linear static procedure (LSP) and linear dynamic procedure (LDP). When the linear static procedure is used, the seismic design forces are distributed over the floors, corresponding internal forces and displacement will be determined by linear elastic analysis, and the model will be built using linear elastic stiffness materials, and equivalent viscous damping according to FEMA 356.

2.3.2 Nonlinear procedures

The nonlinear procedures used by FEMA 356 are nonlinear static procedure and nonlinear dynamic procedure. The nonlinear static procedure is done using nonlinear material behavior of members. The lateral load pattern is distributed on each floor of the building in accordance with the dominant mode shapes and floor weights. Then, the load is either statically or dynamically increased until certain deformation target is reached or numerical instability occurs.

Nonlinear procedure is better than linear procedures because it covers inelastic response. On the other hand, nonlinear dynamic procedure NDP simulates reality better than NSP. However, NSP is faster, less data needed, and less calculation intensive than NDP. Because of these advantages of NSP, engineers commonly use NSP in cases of assessment that can be seen in the intensive researches in the subject of performance-based design. [Carlos Augusto, 2011]

The nonlinear dynamic procedure is done by building a model that considers the local nonlinear behavior for individual elements in the model and components, then expose the model to realistic earthquake ground motion records (time history) and transient analysis is conducted in order to find the deflection of the building and internal forces.

Carlos Augusto (2011) investigated the nonlinear static procedures such as Capacity Spectrum Method (CSM), ACSM, N2, N2 extended, and Modal Pushover Analysis (MPA), by applying them on three existing buildings, then he compared the results of the methods and he compared these results with nonlinear dynamic procedure. The results of NSPs were far from NDPs results due to irregularities in the case study buildings. At the end, he proposed a new 3D pushover procedure, in order to overcome the deficiencies of the previous methods in dealing with irregularities. [Carlos Augusto, 2011]

2.4 Seismic performance criteria

2.4.1 FEMA 356 (ASCE 2000)

The Pre-standard and Commentary for the Seismic Rehabilitation of Buildings – FEMA 356 (ASCE 2000) is generally used to evaluate the expected seismic performance of existing structures using qualitative performance levels. The provisions and commentary of this standard are primarily based on FEMA 273 (FEMA 1997a) and FEMA 274 (FEMA 1997b). FEMA 356 covers general information and methodology for seismic rehabilitation of existing building structures. FEMA 356 begins by introducing rehabilitation objectives according to seismic performance level and discusses the general seismic rehabilitation process. In addition, it illustrates general requirements, such as as-built information, and provides an overview of rehabilitation strategies. Finally, it explains the details of the four analysis procedures and the methodology for member-level evaluation according to each structural type. [JONG-WHA BAI, 2004]

In this thesis, the FEMA 356 standards and requirements will be adopted and used for analysis and rehabilitation objectives.

2.4.2 Rehabilitation objectives

The rehabilitation objectives must be selected by the building owner or consultant prior to the evaluation of the existing building and the selection of a retrofitting technique, if needed.

FEMA 356 presents many possible rehabilitation objectives that combine different target building performance levels with associated earthquake hazard levels, as shown in Table (2.4.1). FEMA 356 defines performance levels related to the structural system as follows:

(1) Immediate Occupancy (IO) – Occupants are allowed immediate access into the structure following the earthquake and the pre-earthquake design strength and stiffness are retained.

(2) Life Safety (LS) – Building occupants are protected from loss of life with a significant margin against the onset of partial or total structural collapse.

(3) Collapse Prevention (CP) – Building continues to support gravity loading, but retains no margin against collapse.

Table 2.4.1. FEMA 356 rehabilitation objectives (adapted from ASCE 2000)

		Target building performance levels			
		Operational performance level (1-A)	Immediate occupancy performance level (1-B)	Life safety performance level (1-C)	Collapse prevention performance level (1-D)
Earthquake hazard level	50% / 50 years	a	b	c	d
	20% / 50 years	e	f	g	h
	BSE - 1 10% / 50 years	i	j	k	l
	BSE - 2 2% / 50 years	m	n	o	p

Notes:

1. Each cell in the above matrix represents a discrete Rehabilitation Objective.
2. The Rehabilitation Objectives in the matrix above may be used to represent the three specific Rehabilitation Objectives defined in Section 1.4.1, 1.4.2, and 1.4.3 of FEMA 356, as follows:

   k+p = Basic Safety Objective (BSO)
   k+p+any of a, e, i, b, j, or n = Enhanced Objectives
   o alone or n alone or m alone = Enhanced Objectives
   k alone or p alone = Limited Objective
   c, g, d, h, l = Limited Objective

2.4.3 Global level approach

FEMA 356 defines a wide range of structural performance requirements for specific limit states. Limits are given for many types of structures including concrete frames, steel moment frames, braced steel frames, concrete walls, unreinforced masonry infill walls, unreinforced masonry walls, reinforced masonry walls, wood stud walls, precast concrete connections and foundations. Suggested global-level drift limits for concrete frames and concrete walls are shown in Table (2.4.2) for the main three performance levels.

Table 2.4.2. Structural performance levels and damage (Adapted from ASCE 2000)

Elements	Type	Structural performance levels Collapse prevention S-5	Life safety S-3	Immediate occupancy S-1
Concrete frames	Primary	Extensive cracking and hinge formation in ductile elements. Limited cracking and/or splice failure in some nonductile columns. Severe damage in short columns.	Extensive damage to beams. Spalling of cover and shear cracking (<1/8" width) for ductile columns. Minor spalling in nonductile columns. Joint cracks <1/8" wide.	Minor hairline cracking. Limited yielding possible at a few locations. No crushing (strains below 0.003).
	Secondary	Extensive spalling in columns (limited shortening) and beams. Severe joint damage. Some reinforcing buckled.	Extensive cracking and hinge formation in ductile elements. Limited cracking and/or splice failure in some nonductile columns. Severe damage in short columns.	Minor spalling in a few places in ductile columns and beams. Flexural cracking in beams and columns. Shear cracking in joints <1/16" width.
	Drift	4% transient or permanent	2% transient; 1% permanent	1% transient; negligible permanent
Concrete walls	Primary	Major flexural and shear cracks and voids. Extensive crushing and buckling of reinforcement. Failure around openings. Severe boundary element damage. Coupling beams shattered and virtually disintegrated.	Some boundary element stress, including limited buckling of reinforcement. Some sliding at joints. Damage around openings. Some crushing and flexural cracking. Coupling beams: extensive shear and flexural cracks; some crushing, but concrete generally remains in place.	Minor hairline cracking of walls, <1/16" wide. Coupling beams experience cracking <1/8" width.
	Secondary	Panels shattered and virtually disintegrated.	Major flexural and shear cracks. Sliding at joints. Extensive crushing. Failure around openings. Severe boundary element damage. Coupling beams shattered and virtually disintegrated.	Minor hairline cracking of walls. Some evidence of sliding at construction joints. Coupling beams experience cracks <1/8" width. Minor spalling.
	Drift	2% transient or permanent	1% transient; 0.5% permanent	0.5% transient; negligible permanent

2.4.4 Member level approach

FEMA 356 classifies the structural types by materials, such as steel, concrete, masonry, wood and light metal framing. For each structural type, FEMA 356 describes the procedure for evaluating seismic performance based on member-level limits. For instance, in Chapter 6, the seismic evaluation of concrete structures includes member level limits for concrete moment frames, precast concrete frames, concrete frames with infill walls, concrete shear walls, precast concrete shear walls, concrete-braced frames, cast in-place concrete diaphragms, precast concrete diaphragms and concrete foundation elements.

FEMA 356 addresses several categories of concrete moment frames, including RC beam-column moment frames, prestressed concrete beam-column moment frames, and slab-column moment frames. For concrete moment frames, the plastic rotation of each member is used as a parameter to assess inelastic behavior. Plastic rotation is defined as the amount of rotation beyond the yield rotation of the member.

FEMA 356 provides the maximum permissible plastic rotation corresponding to each performance level as seen in Tables (2.4.3 to 2.4.5). Figure (2.4.1) shows the general capacity curve parameters and numerical acceptance criteria for RC beams, RC columns, and RC beam-column joints.

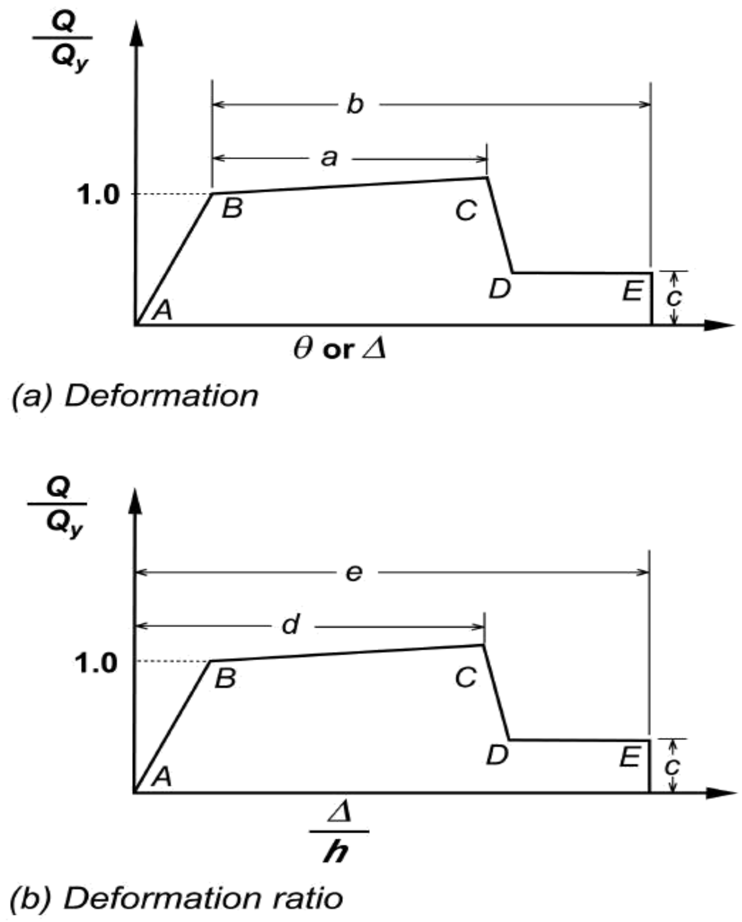

Figure 2.4.1. Generalized Force-Deformation Relations for Concrete Elements or Components [FEMA 356]

Figure (2.4.1) is a generic force-deformation curve for a typical plastic hinge and it shows that point A is the unloaded condition and point B represents yielding of the element. The ordinate at C is the nominal strength and abscissa at C is the deformation at which significant strength degradation begins. The drop from C to D represents the initial failure of the element and resistance to lateral loads beyond point C is usually unreliable. The residual resistance from D to E allows the frame elements to sustain gravity loads. Beyond point E, the maximum deformation capacity, gravity load can no longer be sustained.

Table 2.4.3. FEMA 356 modeling parameters and numerical acceptance criteria for nonlinear procedures - RC beams (adapted from ASCE 2000)

Conditions			Modeling parameters[3]			Acceptance criteria[3]				
			Plastic rotation angle, radians		Residual strength ratio	Plastic rotation angle, radians				
						Performance level				
							Component type			
							Primary		Secondary	
			a	b	c	IO	LS	CP	LS	CP
i. Beams controlled by flexure[1]										
$\frac{\rho-\rho'}{\rho_{bal}}$	Transverse Reinforce-ment[2]	$\frac{V}{b_w d\sqrt{f'_c}}$								
≤ 0.0	C	≤ 3	0.025	0.05	0.2	0.01	0.02	0.025	0.02	0.05
≤ 0.0	C	≥ 6	0.02	0.04	0.2	0.005	0.01	0.02	0.02	0.04
≥ 0.5	C	≤ 3	0.02	0.03	0.2	0.005	0.01	0.02	0.02	0.03
≥ 0.5	C	≥ 6	0.015	0.02	0.2	0.005	0.005	0.015	0.015	0.02
≤ 0.0	NC	≤ 3	0.02	0.03	0.2	0.005	0.01	0.02	0.02	0.03
≤ 0.0	NC	≥ 6	0.01	0.015	0.2	0.0015	0.005	0.01	0.01	0.015
≥ 0.5	NC	≤ 3	0.01	0.015	0.2	0.005	0.01	0.01	0.01	0.015
≥ 0.5	NC	≥ 6	0.005	0.01	0.2	0.0015	0.005	0.005	0.005	0.01
ii. Beams controlled by shear[1]										
Stirrup spacing $\le d/2$			0.003	0.02	0.2	0.0015	0.002	0.003	0.1	0.02
Stirrup spacing $\ge d/2$			0.003	0.01	0.2	0.0015	0.002	0.003	0.005	0.01
iii. Beams controlled by inadequate development or splicing along the span[1]										
Stirrup spacing $\le d/2$			0.003	0.02	0	0.0015	0.002	0.003	0.1	0.02
Stirrup spacing $\ge d/2$			0.003	0.01	0	0.0015	0.002	0.003	0.005	0.01
iv. Beams controlled by inadequate embedment into beam-column joint[1]										
			0.015	0.03	0.2	0.01	0.01	0.015	0.02	0.03

Notes:

1. When more than one of the conditions i, ii, iii, and iv occurs for a given component, use the minimum appropriate numerical value from the table.
2. "C" and "NC" are abbreviations for conforming and nonconforming transverse reinforcement. A component is conforming if, within the flexural plastic hinge region, hoops are spaced at $\le d/3$, and if, for components of moderate and high ductility demand, the strength provided by the hoops (V_s) is at least three-fourths of the design shear. Otherwise, the component is considered nonconforming.
3. Linear interpolation between values listed in the table shall be permitted.

Table 2.4.4. FEMA 356 modeling parameters and numerical acceptance criteria for nonlinear procedures - RC columns (adapted from ASCE 2000)

Conditions			Modeling parameters[4]			Acceptance criteria[4]				
			Plastic rotation angle, radians		Residual strength ratio	Plastic rotation angle, radians				
						Performance level				
							Component type			
							Primary		Secondary	
			a	b	c	IO	LS	CP	LS	CP
i. Columns controlled by flexure[1]										
$\frac{P}{A_g f'_c}$	Transverse Reinforcement[2]	$\frac{V}{b_w d \sqrt{f'_c}}$								
≤ 0.1	C	≤ 3	0.02	0.03	0.2	0.005	0.015	0.02	0.02	0.03
≤ 0.1	C	≥ 6	0.016	0.024	0.2	0.005	0.012	0.016	0.016	0.024
≥ 0.4	C	≤ 3	0.015	0.025	0.2	0.003	0.012	0.015	0.018	0.025
≥ 0.4	C	≥ 6	0.012	0.02	0.2	0.003	0.01	0.012	0.013	0.02
≤ 0.1	NC	≤ 3	0.006	0.015	0.2	0.005	0.005	0.006	0.01	0.015
≤ 0.1	NC	≥ 6	0.005	0.012	0.2	0.005	0.004	0.005	0.008	0.012
≥ 0.4	NC	≤ 3	0.003	0.01	0.2	0.002	0.002	0.003	0.006	0.01
≥ 0.4	NC	≥ 6	0.002	0.008	0.2	0.002	0.002	0.002	0.005	0.008
ii. Columns controlled by shear[1,3]										
All cases[5]			-	-	-	-	-	-	0.003	0.004
iii. Columns controlled by inadequate development or splicing along the clear height[1,3]										
Hoop spacing ≤ d/2			0.01	0.02	0.4	0.005	0.005	0.01	0.01	0.02
Hoop spacing ≥ d/2			0	0.01	0.2	0	0	0	0.005	0.01
iv. Columns with axial loads exceeding $0.70P_o$[1,3]										
Conforming hoops over the entire length			0.015	0.025	0.02	0	0.005	0.01	0.01	0.02
All other cases			0	0	0	0	0	0	0	0

Notes:

1. When more than one of the conditions i, ii, iii, and iv occurs for a given component, use the minimum appropriate numerical value from the table.
2. "C" and "NC" are abbreviations for conforming and nonconforming transverse reinforcement. A component is conforming if, within the flexural plastic hinge region, hoops are spaced at ≤ d/3, and if, for components of moderate and high ductility demand, the strength provided by the hoops (V_s) is at least three-fourths of the design shear. Otherwise, the component is considered nonconforming.
3. To qualify, columns must have transverse reinforcement consisting of hoops. Otherwise, actions shall be treated as force-controlled.
4. Linear interpolation between values listed in the table shall be permitted.
5. For columns controlled by shear, see Section 6.5.2.4.2 for acceptance criteria.

Table 2.4.5. FEMA 356 modeling parameters and numerical acceptance criteria for nonlinear procedures - RC beam-column joints (Adapted from ASCE 2000)

Conditions			Modeling parameters[4]			Acceptance criteria[4]				
			Plastic rotation angle, radians		Residual strength ratio	Plastic rotation angle, radians				
						Performance level				
							Component type			
							Primary		Secondary	
			a	b	c	IO	LS	CP	LS	CP
i. Interior joints[2,3]										
$\frac{P}{A_g f'_c}$	Transverse Reinforcement	$\frac{V}{V_n}$ [3]								
≤ 0.1	C	≤ 1.2	0.015	0.03	0.2	0	0	0	0.02	0.03
≤ 0.1	C	≥ 1.5	0.015	0.03	0.2	0	0	0	0.015	0.02
≥ 0.4	C	≤ 1.2	0.015	0.025	0.2	0	0	0	0.015	0.025
≥ 0.4	C	≥ 1.5	0.015	0.02	0.2	0	0	0	0.015	0.02
≤ 0.1	NC	≤ 1.2	0.005	0.02	0.2	0	0	0	0.015	0.02
≤ 0.1	NC	≥ 1.5	0.005	0.015	0.2	0	0	0	0.01	0.015
≥ 0.4	NC	≤ 1.2	0.005	0.015	0.2	0	0	0	0.01	0.015
≥ 0.4	NC	≥ 1.5	0.005	0.015	0.2	0	0	0	0.01	0.015
ii. Other joints[2,3]										
$\frac{P}{A_g f'_c}$	Transverse Reinforcement[1]	$\frac{V}{V_n}$								
≤ 0.1	C	≤ 1.2	0.01	0.02	0.2	0	0	0	0.015	0.02
≤ 0.1	C	≥ 1.5	0.01	0.015	0.2	0	0	0	0.01	0.015
≥ 0.4	C	≤ 1.2	0.01	0.02	0.2	0	0	0	0.015	0.02
≥ 0.4	C	≥ 1.5	0.01	0.015	0.2	0	0	0	0.01	0.015
≤ 0.1	NC	≤ 1.2	0.005	0.01	0.2	0	0	0	0.0075	0.01
≤ 0.1	NC	≥ 1.5	0.005	0.01	0.2	0	0	0	0.0075	0.01
≥ 0.4	NC	≤ 1.2	0	0	-	0	0	0	0.005	0.0075
≥ 0.4	NC	≥ 1.5	0	0	-	0	0	0	0.005	0.0075

Notes:

1. "C" and "NC" are abbreviations for conforming and nonconforming transverse reinforcement. A joint is conforming if hoops are spaced at $\le h_c/3$ within the joint. Otherwise, the component is considered nonconforming.
2. P is the design axial force on the column above the joint and A_g is the gross cross-sectional area of the joint.
3. V is the design shear force and V_n is the shear strength for the joint. The design shear force and shear strength shall be calculated according to Section 6.5.2.3.
4. Linear interpolation between values listed in the table shall be permitted.

2.5 Seismic retrofitting techniques

2.5.1 Introduction

Seismic retrofitting is concerned with upgrading the existing or damaged buildings into more seismic resistant buildings. The retrofitting can be achieved by:

- Upgrading the lateral strength and/or stiffness of the building
- Increasing the ductility and deformation capacity of the building
- Balancing the previous two points according to a target point

The retrofitting techniques can be divided into two strategies, the first is the global or overall structural level, and the second is local or member level strategy and these are presented in the figures (2.5.1, 2.5.2, and 2.5.3).

There are many factors that govern the selection of retrofitting technique methods, which depend on the type of the structure, structure's importance, and any client special needs. Thermou and Elnashai (2002) summarized these factors as follows [Thermou and Elnashai, 2002]:

- Cost versus importance of the structure
- Available workmanship
- Duration of work/disruption of use
- Fulfillment of the performance goals of the owner
- Functional and aesthetic compatibility and complementarity to the existing building
- Reversibility of the intervention
- Level of quality control

- Political and/or historical significance
- Structural compatibility with the existing structural system
- Irregularity of stiffness, strength and ductility
- Adequacy of local stiffness, strength and ductility
- Controlled damage to non-structural components
- Sufficient capacity of foundation system
- Availability of repair materials and technology

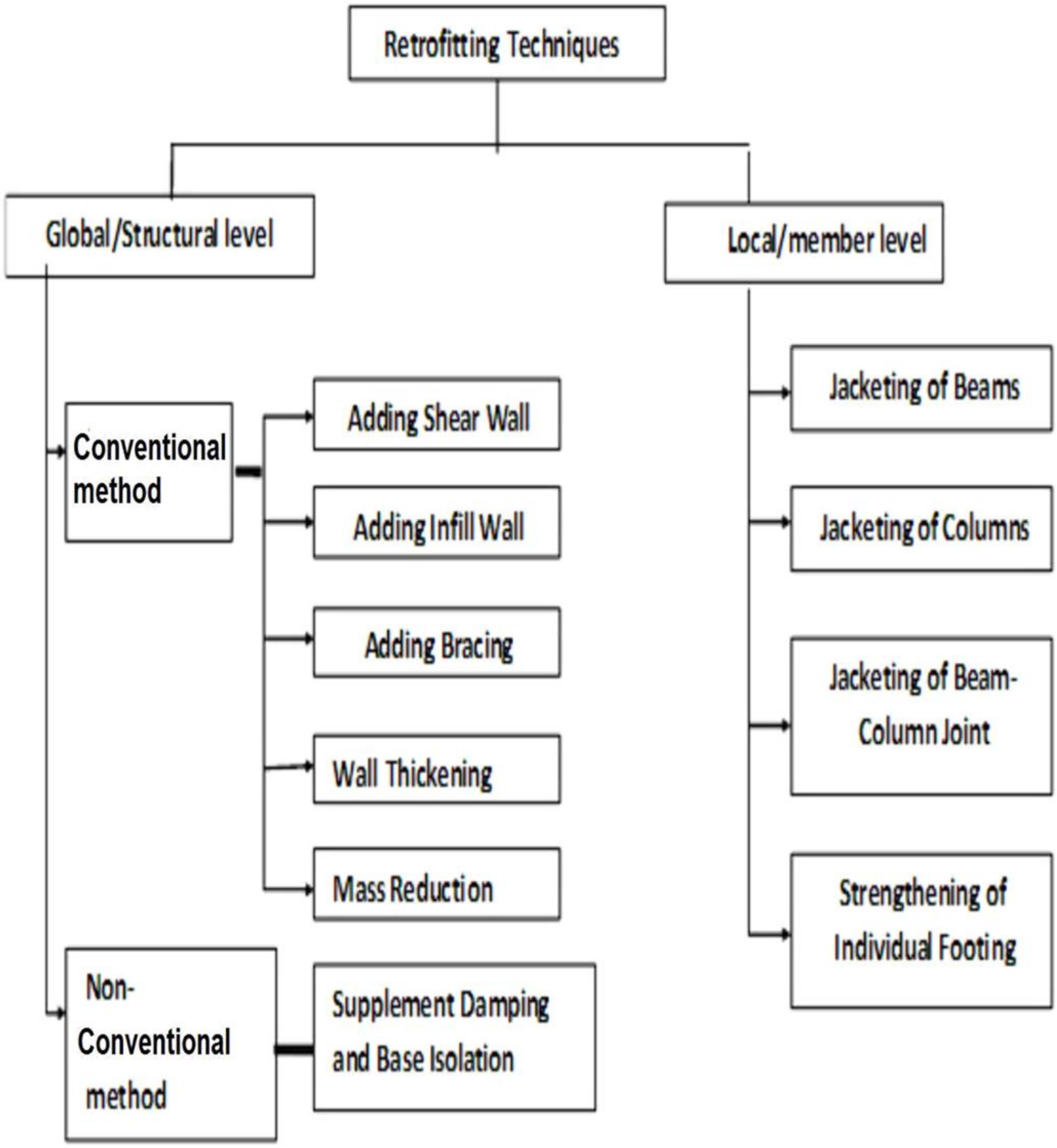

Figure 2.5.1. Strategies of retrofitting techniques and their divisions

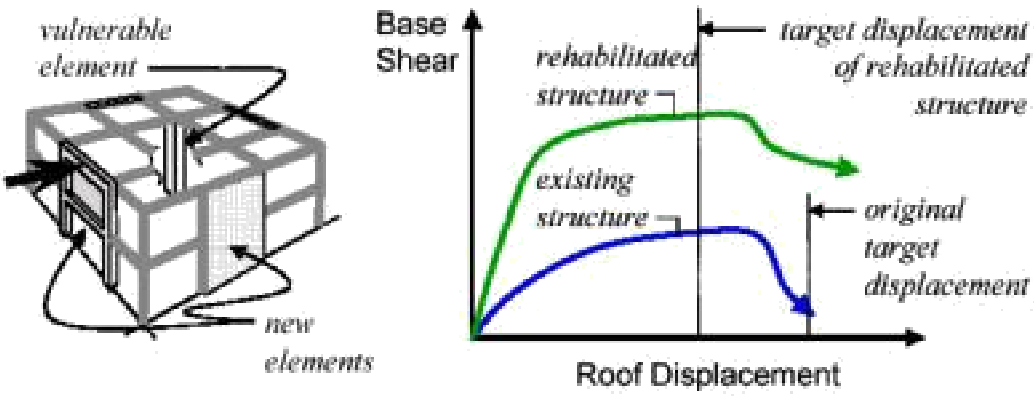

Figure 2.5.2. Global modification of the structural system [Moehle, 2000]

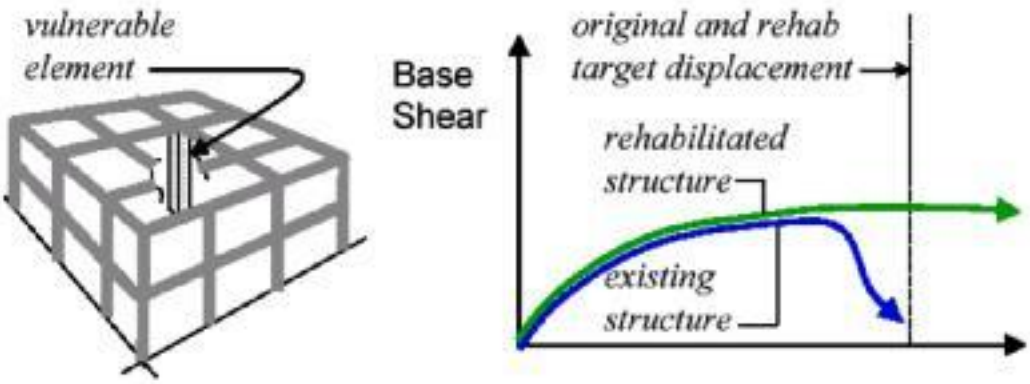

Figure 2.5.3. Local modification of structural components [Moehle 2000]

2.5.2 Global/structural level

Global/structural level strategy can be divided into two types as presented in figure (2.5.1):

1. Conventional techniques: these are based on increasing the seismic resistance of the existing structure by eliminating or reducing the adverse defects of existing design or construction. Below, two examples of such technique are discussed:

- Adding structural walls (shear and infill walls):

This technique is a very powerful and effective technique in increasing the strength and stiffness of existing buildings. It leads to less drift and less damage to the retrofitted building under seismic loads.

There are many studies on the effects of structural walls, such as (Altin et al. 1992, Pincheiraand Jirsa 1995, Lombard et al. 2000). These studies show that adding structural walls increase the stiffness of the overall structure, which leads to an increase in the base shear. The effect of overturning moment and the base shear force are concentrated at stiff ends of the walls. This means that the foundation under these stiff members must be strengthened.

Jirsa and Kreger (1989) made experiments on the construction styles of 1950s. They used four specimens in total to study the effect of walls. The case study was one story building, three-one bay, with non-ductile reinforced concrete frames, and insufficient spacing in columns shear reinforcement and compression splices to develop the required tensile yield strength. The first three walls had different opening locations. Longitudinal reinforcement was added to columns to improve the continuity of steel in the fourth specimen. In their experiments, they exposed the four specimens to equivalent seismic lateral forces. The first three experiment failed due to brittle causes, but the fourth experiment showed high strength and ductility due to sufficient continuity and ductility of rebar. Figure (2.5.4) shows the behavior of the fourth specimens.

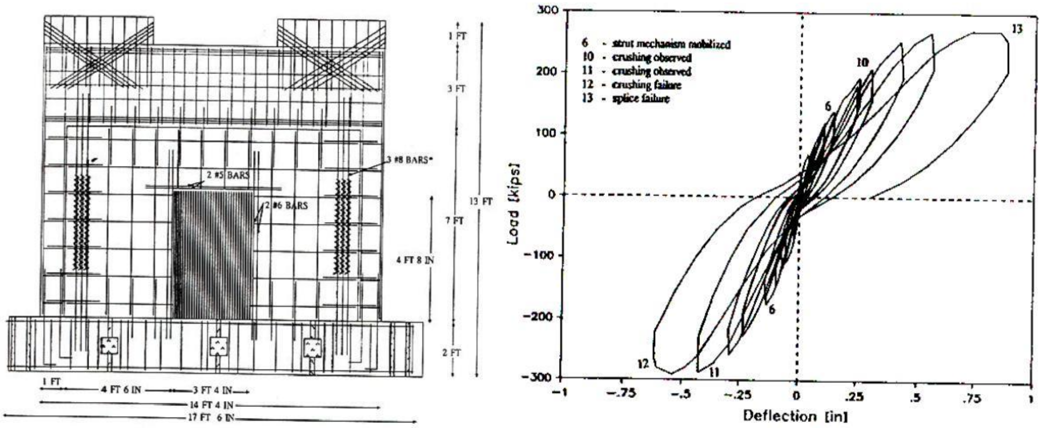

Figure 2.5.4 Infill wall and load-deflection history of the specimen [Jirsa and Kreger 1989]

- Adding steel bracings:

This method is considered a very powerful and effective method for structural-level retrofitting. It increases the stiffness and lateral seismic resistance through concentric or eccentric bracing systems. It must be noted, that the increase of load at bracing location above the foundations must be considered in the design. The other thing is the connection points between the bracing and RC structural elements in order to avoid failure during the earthquake.

Many studies were made about the use of steel bracing, such as Badoux and Jirsa (1990), Bush et al. (1991), Teran-Gilmore et al. (1995), Goel and Masri (1996), and they all showed improved performance due to using bracing.

Pincheira and Jirsa (1995) analytically studied reinforced concrete frame of three-seven-and twelve-story height. They considered many retrofitting techniques, such as, post tensioned bracing, steel bracing, and infill walls as possible retrofitting techniques for low and mid-rise RC buildings. They subjected the models of the RC frames to five earthquake records obtained from stations on hard and soft soils. The infill walls and bracing techniques were added to the perimeter frames only. Figure (2.5.5) shows the base shear coefficient, which is defined as (V/W) and drift of the 12 –story frame in addition to the original behavior and the effect of retrofitting techniques.

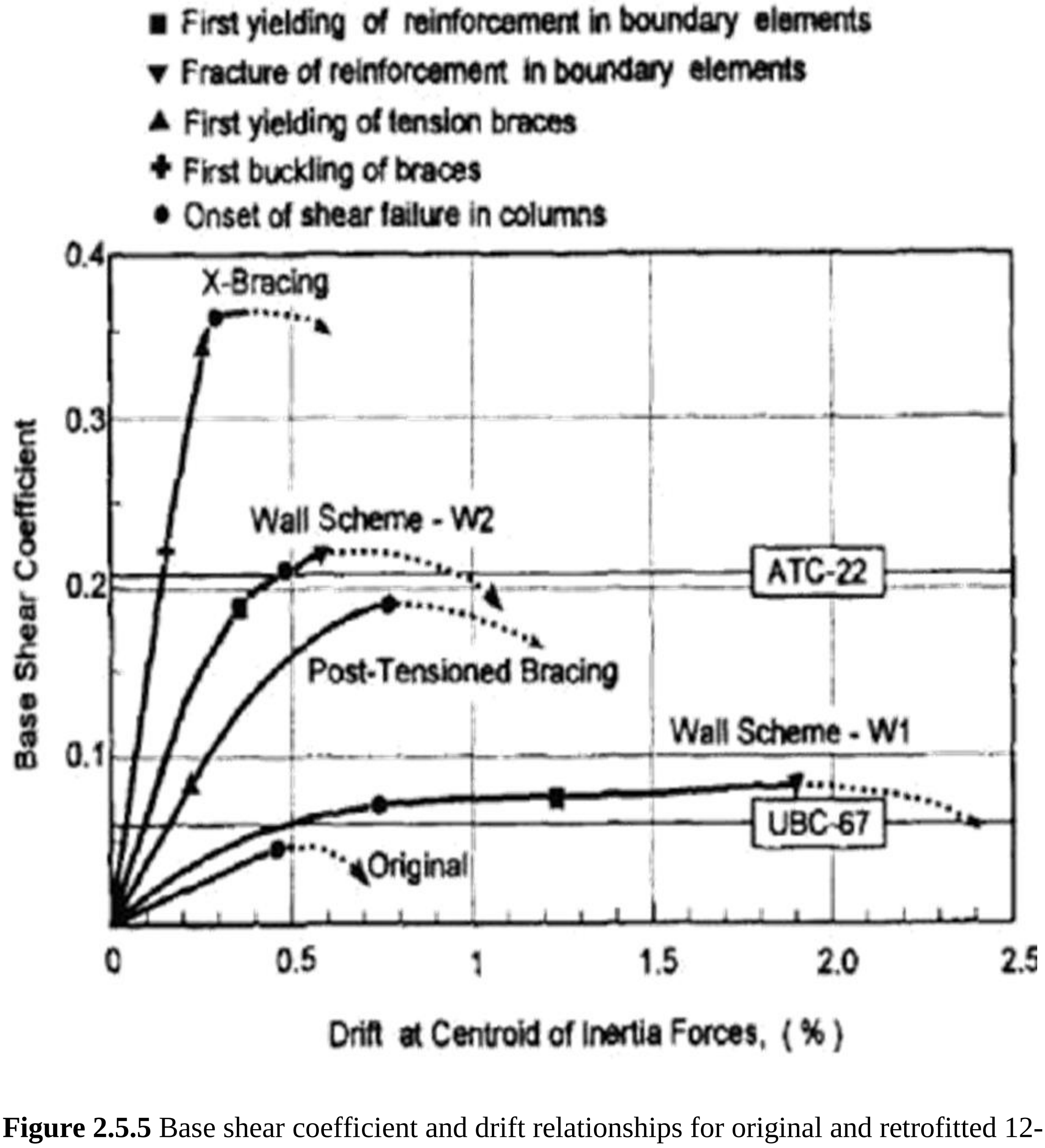

Figure 2.5.5 Base shear coefficient and drift relationships for original and retrofitted 12-story building [Pincheira and Jirsa 1995]

Goel & Masri (1996) worked on weak slab-column building, two-bay, two-story RC slab-column. Using a model with scale one to three of the existing real structure. They worked on two retrofitting techniques, which are two different steel bracing phases on interior and exterior bays. Figures (2.5.6, 2.5.7, and 2.5.8) show one bracing system, and hysteresis loops before and after retrofitting. The results show a noticeable increase in strength, stiffness, and energy dissipation when bracing is used.

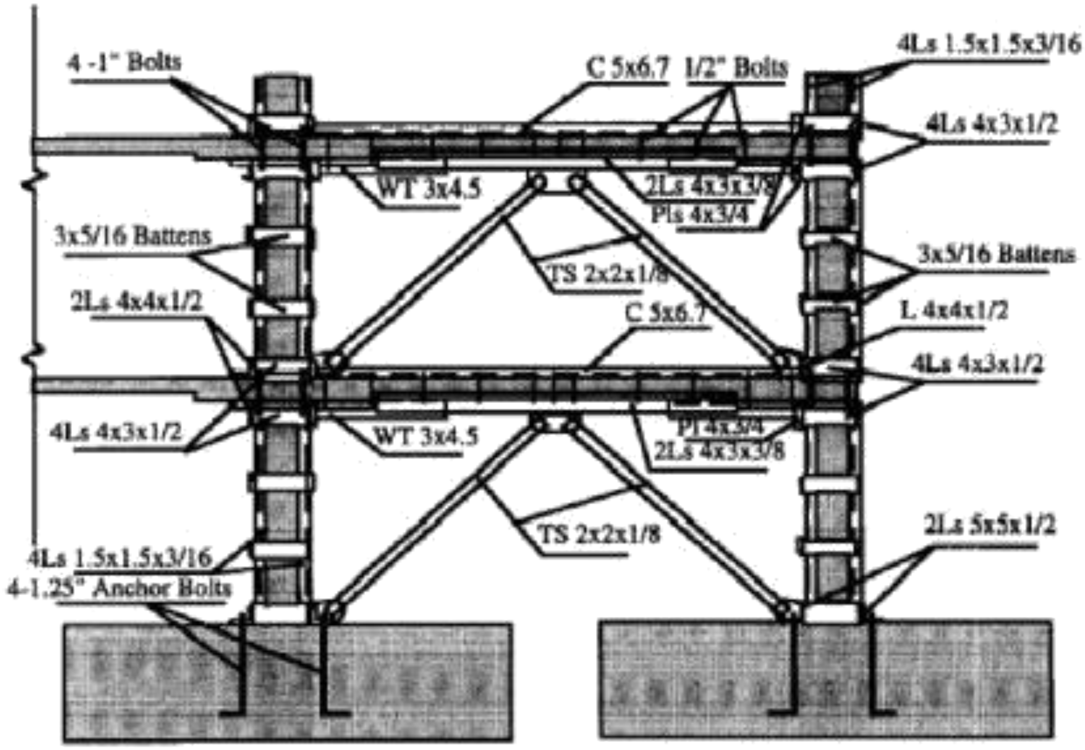

Figure 2.5.6. Layout of the braced frame [Goel and Masri 1996]

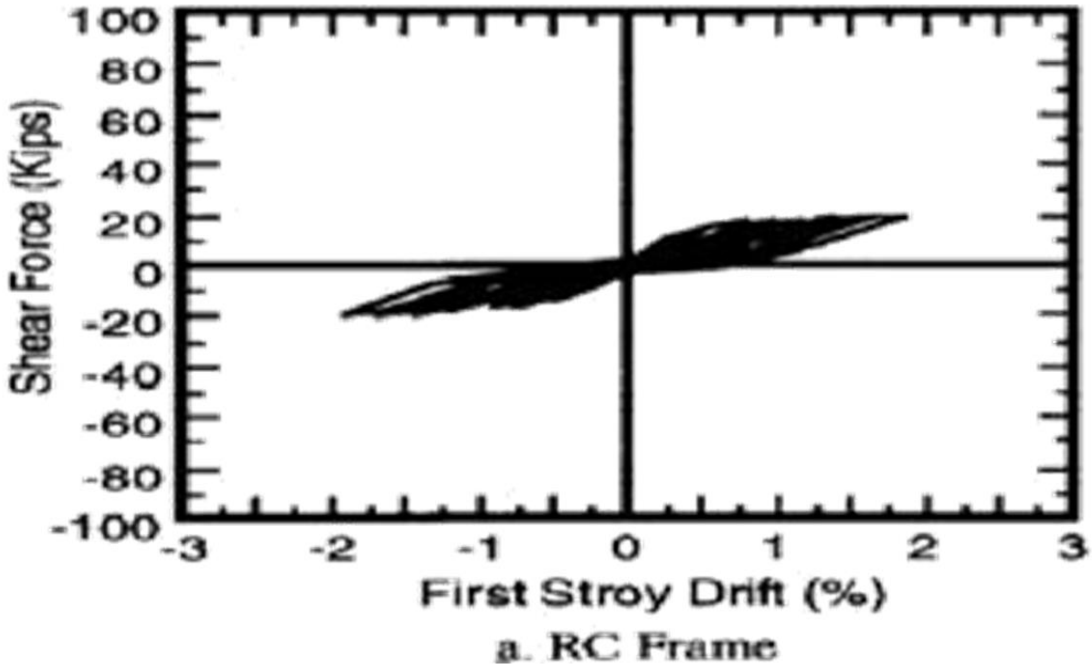

Figure 2.5.7. Hysteretic loops of the RC frames [Goel and Masri 1996]

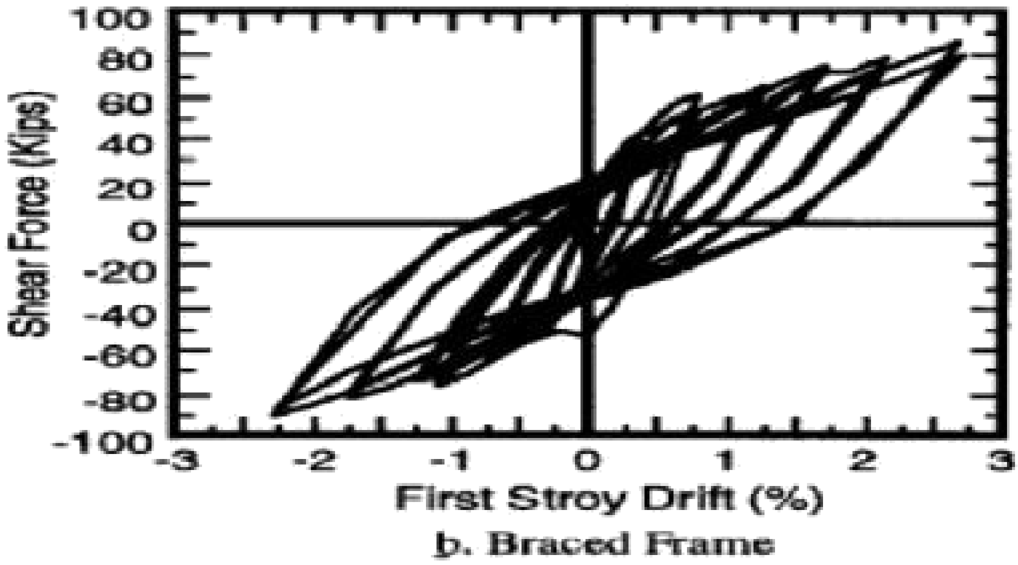

Figure 2.5.8. Hysteretic loops of braced frames [Goel and Masri 1996]

2. Non-conventional techniques based on reducing the seismic demands.

These techniques rely on absorbing the seismic energy through non conventional techniques such as:

- Supplement Dampers (Energy Dissipation)

It is a commonly used method in retrofitting that includes frictional, hysteretic, viscosity, or magneto-logical dampers as components for bracing frames. Many researchers studied energy dissipation techniques, such as (Pekcan et al. 1995, Kunisue et al. 2000, Fu 1996, Munshi 1998, Yang et al. 2002). Generally, reducing the deflection in the structure means increasing of forces in the structural elements, and this needs to be studied carefully (ASCE 2000). Also, the cost of this method in Palestine could be high and very expensive to be practical, because there are neither experts nor trained workers for this technique, in addition to the procurement costs of such advanced technology.

- Seismic Base Isolation

It is an effective method used to isolate the structure from ground motion during earthquakes. Many researchers studied this technique, such as (Gates et al. 1990, Constantinou et al. 1992, Tena-Colunga et al. 1997, Kawamura et al. 2000). This technique is mostly effective when used for relatively stiff, low rise, and heavy structures over stiff or hard ground. [Kawamura et al. 2000]. Again, this technique is technologically expensive to be used in Palestine.

2.5.3 Member level

This kind of techniques is considered more cost effective than structural level retrofitting technique since only the weak members are retrofitted. This strategy includes adding steel, concrete, fiber reinforced polymers jacket for strengthening RC columns, beams, slabs, and joints.

In flat plate slabs, punching failures are more probable. Therefore, these retrofitting techniques are generally used for slab column connections. Many researchers studied this topic, such as (Harries et al. 1998, Luo and Durrani 1994, Farhey et al. 1993, Martinez et al. 1994). Briefly discussion will be presented below on selected types of jacketing and another types will be described with details in chapter (5).

Since columns are critical structural members, they must be strong enough relative to the beams and slabs. The forces that affect the column are axial, shear, and flexure forces. Therefore, jacketing of columns is used to increase the resistance against the previous forces in order to prevent excessive column damage [Bracci et al. 1995].

Researchers show that composite materials such as carbon fiber reinforced polymer composite (FRPC) are most efficient in jacketing columns, because these techniques confine the column, require least intervention in the existing column and does not add an extra weight to it. Column failure due to plastic hinge zone can be delayed using this method because it increases the ductility of column. This effect can be seen in figure (2.5.9).

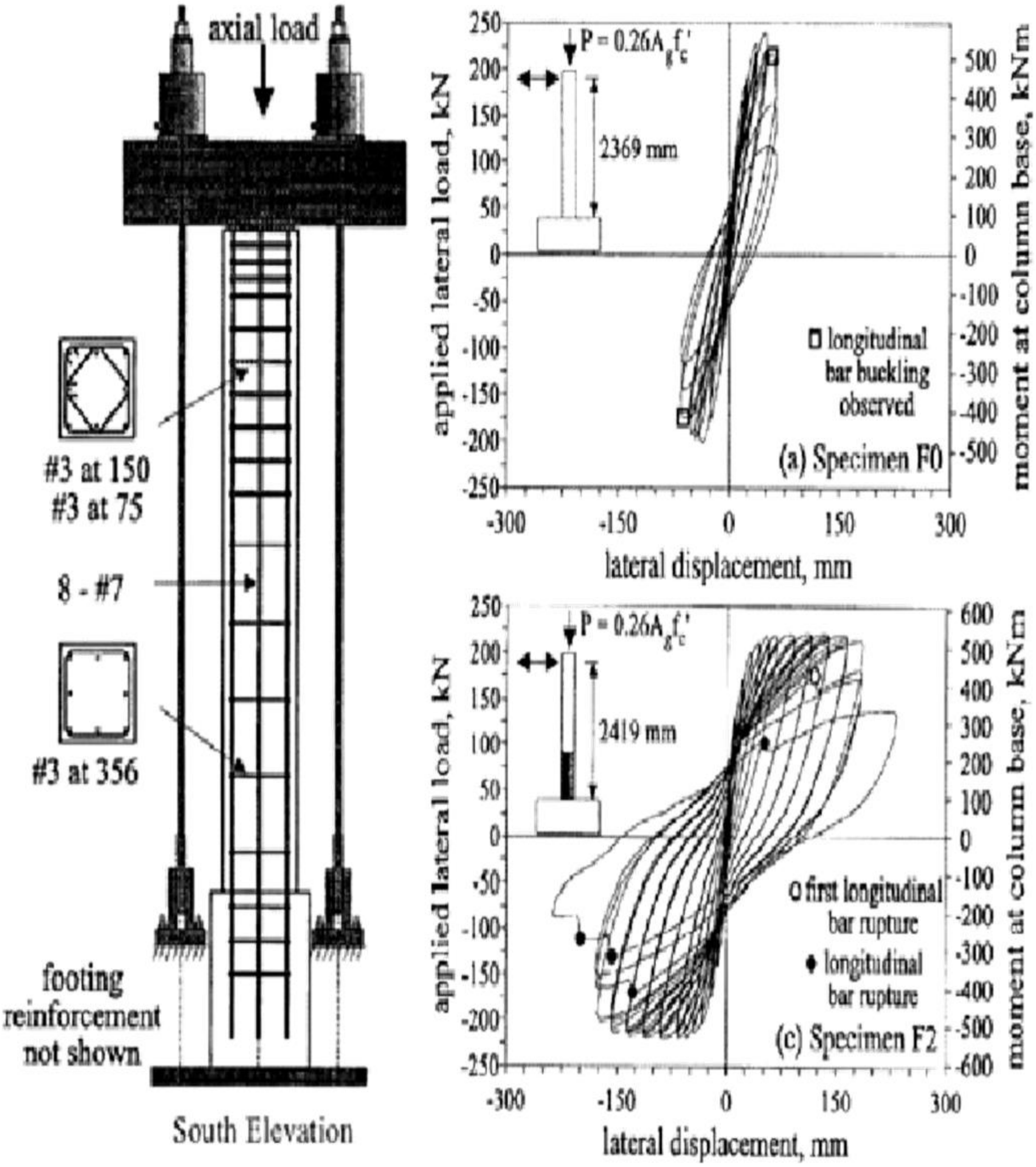

Figure 2.5.9. Column retrofitting by carbon FRPC [Harries et al. 1998]

CH3 Case study description and modelling features

3.1 Introduction

As mentioned in chapter (1), Palestine is located in a relatively active seismic zone. Many of the existing buildings in Palestine were not designed to resist earthquake loads. This means that there is a need to retrofit these buildings in order to reduce any future damage to these buildings in event of earthquake. To satisfy the required level of strengthening for such vulnerable buildings, the actual capacity of these buildings must be examined and their weak elements must be identified.

A common problem that can be observed in many of the residential and commercial buildings is the functional differences between the ground floor and the other floors. The ground floor is usually used as open spaces for parking, markets, or stores, whereas the rest of the floors are typically residential with the external walls are stonewalls, and infill walls are used inside. This means that there is a high possibility of vertical irregularity because the rigidity of ground floor may differ noticeably from the above floors. Figure (3.1.1) shows at least six possible vertical irregularity in buildings in a random location in Nablus city.

Figure 3.1.1. Possible vertical irregularity formations in many buildings [SASPARM project (2), 2014]

Since many building in Palestine are vulnerable to earthquake hazard, there is a need to have an assessment method that can be used to judge the expected behavior of such buildings in the event of earthquake. Such methods exist in foreign codes and maybe useful to our country, but they are not known and not used. Therefore, this study comes to expose these methods and to make them practical and accessible to local engineers.

In order to demonstrate a valid and reliable methodology for seismic retrofitting of existing buildings, a typical residential building is selected as a case study.

The residential building is chosen because it represents most of the common existing residential buildings in the region. The assessment of seismic response and performance of the building under the seismic loads is obtained through nonlinear analysis. After assessment, retrofitting techniques will be proposed, applied, and analyzed. Comparison of results will be made to conclude on the best retrofitting technique for the building.

3.2 Case study description

3.2.1 General

The case study building is located in Nablus city in the western region on Eibal Mountain in an area called "Al-Ma'ajeen", which was built between 2000 and 2010. Al-Ma'ajeen region is one of the green areas in the city of Nablus where olive trees and other trees are spread in there, see figure (3.2.1).

The soil of the site is relatively weak, where it is classified as stiff soil. According to the classification by ASCE 7-10, stiff soil comes as "D" among soil classifications, see table (3.2.1).

The region of the case study building was relatively flat before the building was built, and the area was then leveled, and leveling included the land of the building in addition to a perimeter width of 4-5m around the building land. The building area was excavated below the level of the foundations by 1.2 m, and then the soil was improved with 60 cm sub-base course and 50 cm base course.

Figure 3.2.1. Al-Ma'ajeen area in Nablus city [SASPARM project (2), 2014]

Table 3.2.1. Soil classification [ASCE 7-10, 2010]

Table 20.3-1 Site Classification

Site Class	$\bar{v}_s$	$\bar{N}$ or $\bar{N}_{ch}$	$\bar{s}_u$
A. Hard rock	>5,000 ft/s	NA	NA
B. Rock	2,500 to 5,000 ft/s	NA	NA
C. Very dense soil and soft rock	1,200 to 2,500 ft/s	>50	>2,000 psf
D. Stiff soil	600 to 1,200 ft/s	15 to 50	1,000 to 2,000 psf
E. Soft clay soil	<600 ft/s	<15	<1,000 psf
	Any profile with more than 10 ft of soil having the following characteristics: —Plasticity index $PI > 20$, —Moisture content $w \geq 40\%$, —Undrained shear strength $\bar{s}_u < 500$ psf		
F. Soils requiring site response analysis in accordance with Section 21.1	See Section 20.3.1		

For SI: 1 ft/s = 0.3048 m/s; 1 lb/ft^2 = 0.0479 kN/m^2.

As mentioned earlier many buildings in Palestine adopt the same style, where usually the ground floor is used as either as parking, stores, or markets with floor heights up to 5m with wide gates. The rest of the floors above are residential floors with net height of 3m, and infill walls and external unreinforced concrete-masonry stone walls, with small doors and windows.

Usually, the external stone walls are built without reinforcement. The ceilings are usually built with a thickness of 25 cm ribbed slab in one direction with hidden beams. Columns are usually 25 cm width and the depth ranges from 60 cm to 100 cm.

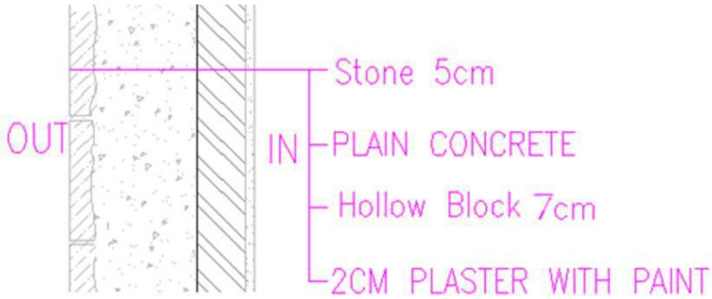

Figure 3.2.2. Typical section in an unreinforced masonry stone wall.

The specifications mentioned above indicate that the ground floor has lateral stiffness less than that of upper floors. The large height of the columns in the ground floor reduces the stiffness, whereas the interior infill walls and exterior stone walls the lateral stiffness of the upper floors.

3.2.2 Architectural description

The building is a six-story RC building, where the ground floor consists of stores with 4.75m height, and the rest floors consist of residential floors with 3.25m height each, and a total height of 21m.

The building has a regular rectangular shape in the vertical projection, with 24m long and 14m wide. The figures (3.2.3, 3.2.4, and 3.2.5) show the top and front views of the building and the distribution of columns.

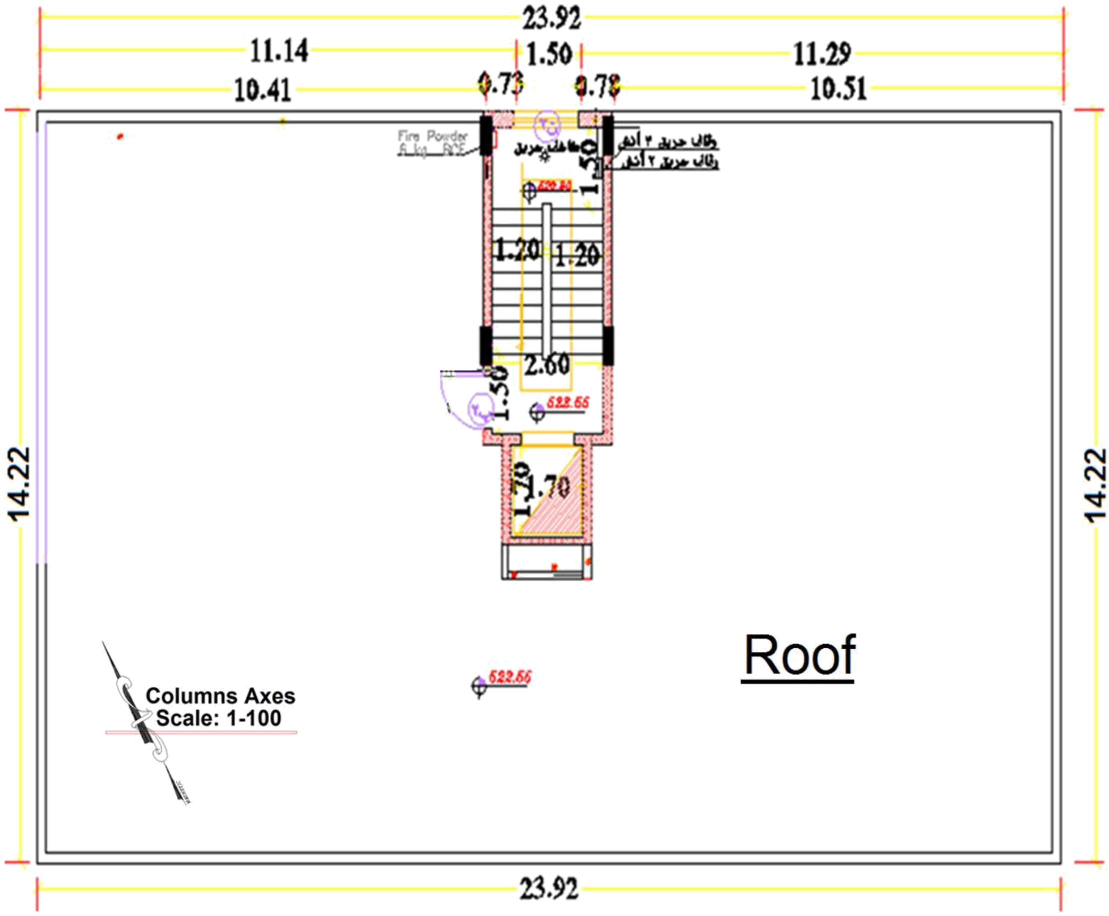

Figure 3.2.3. Top view of case study building

Figure 3.2.4. Ground floor plan view

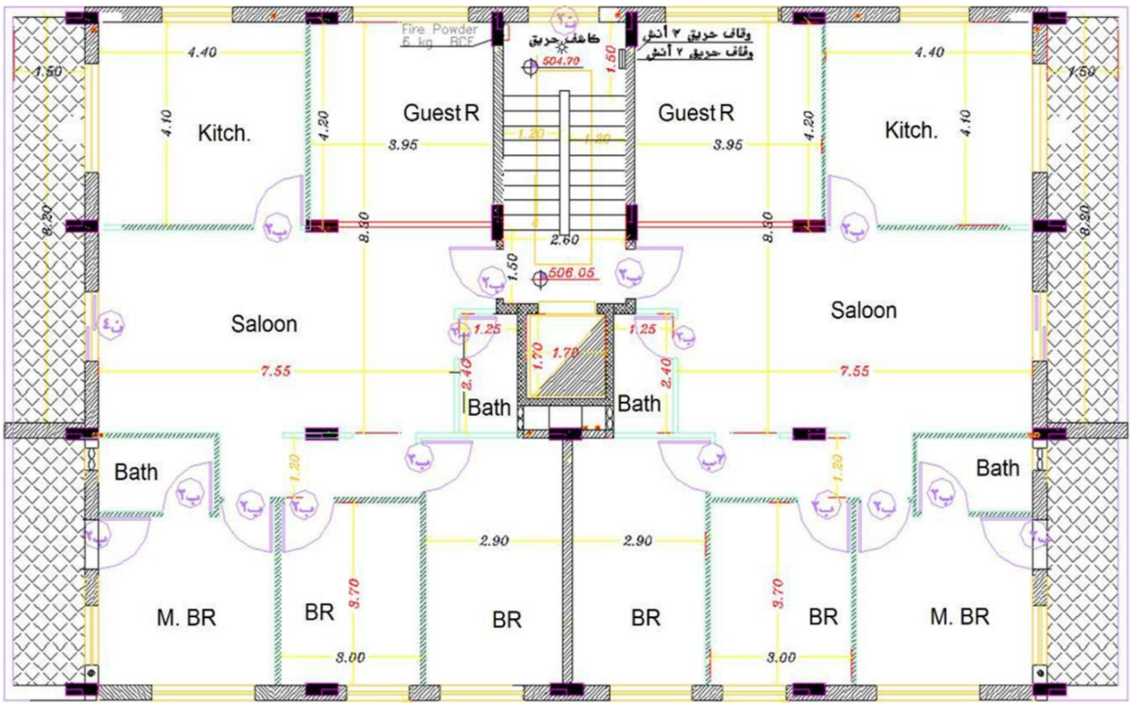

Figure 3.2.5. Repeated floors 1, 2, 3, 4, and 5 plan view

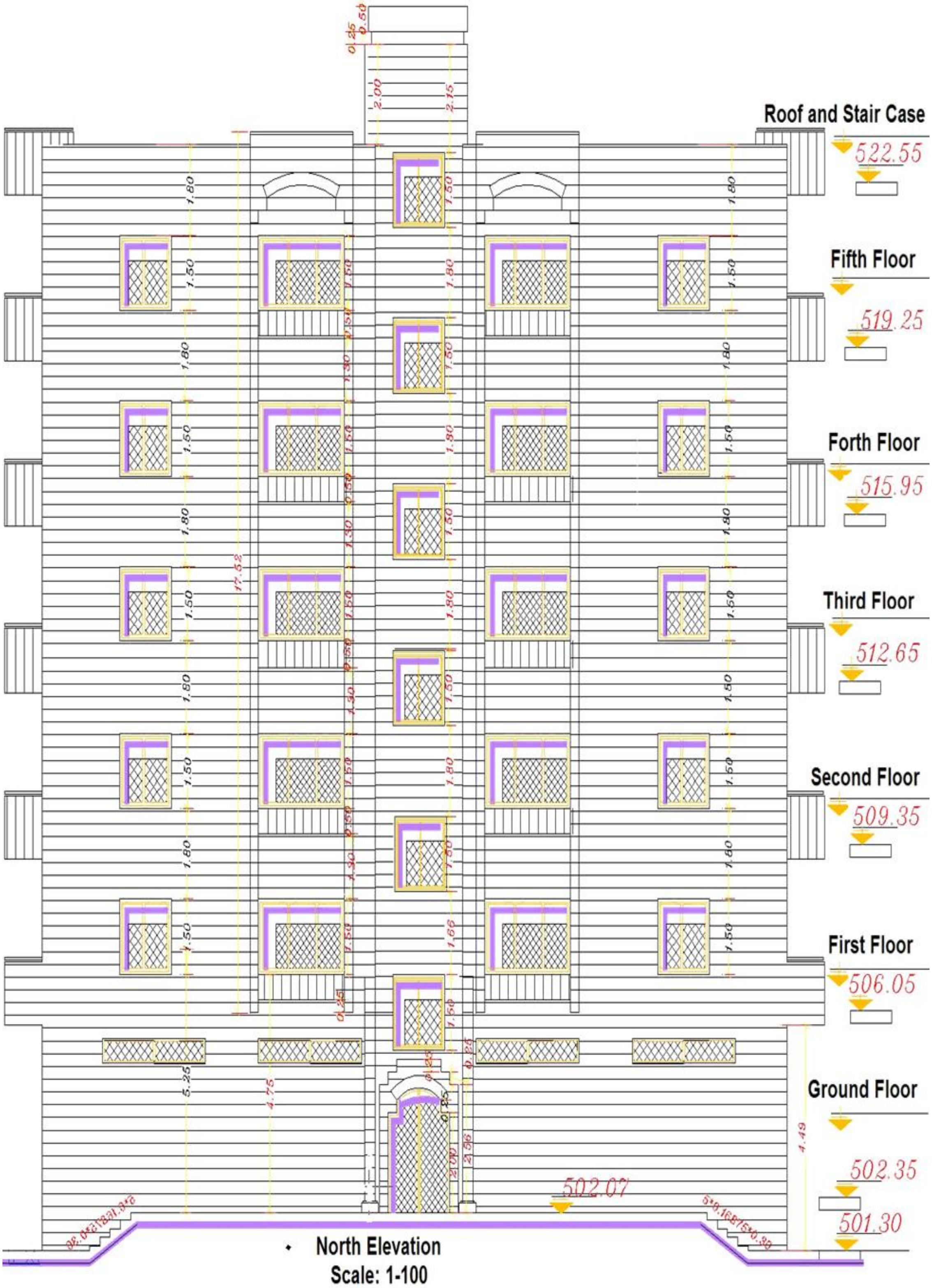

Figure 3.2.6. Elevation view of case study building

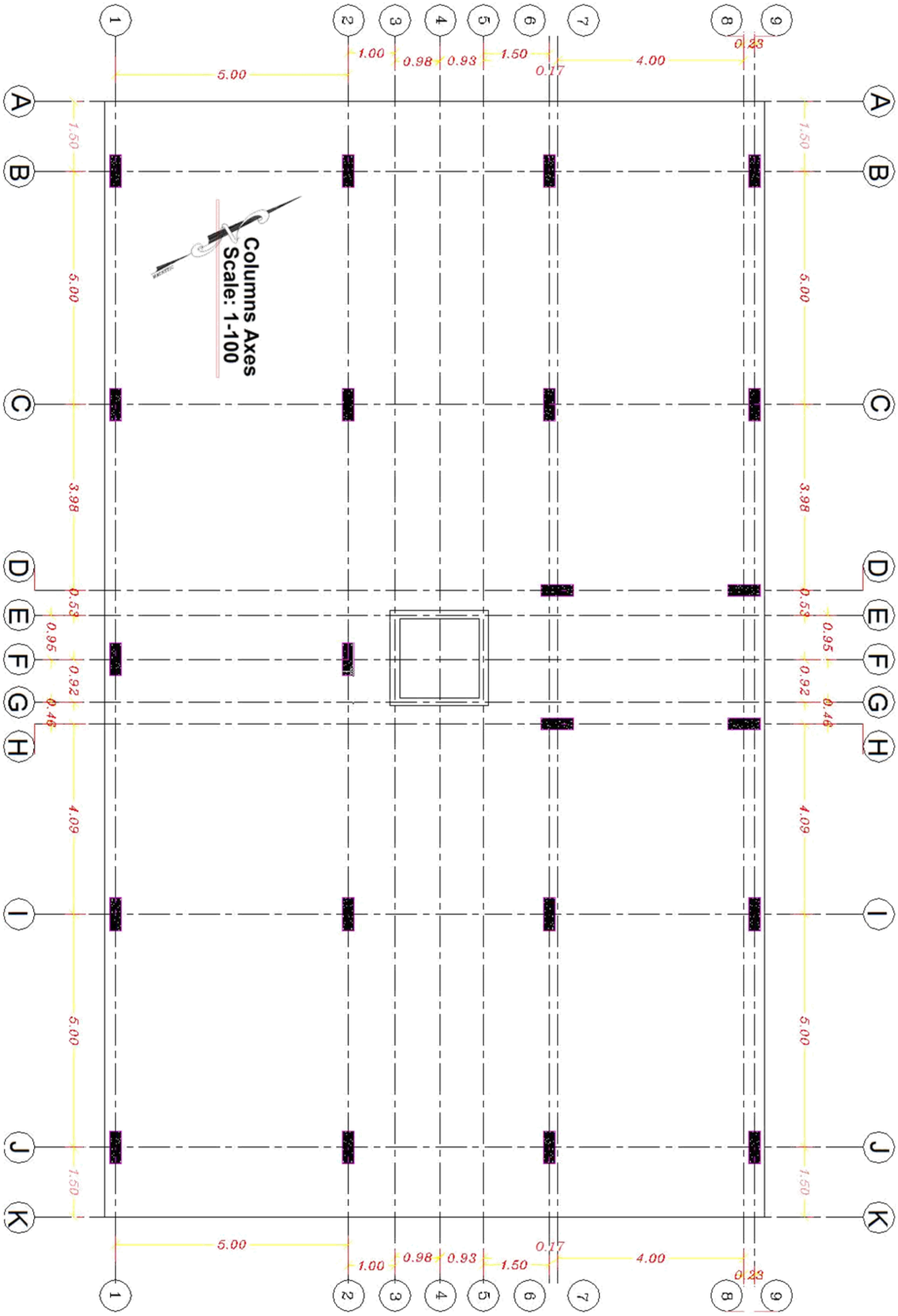

Figure 3.2.7. Columns Grid

3.3 Structural details

3.3.1 General

The gravity loads are mainly transmitted by the main beams, then to columns then to foundations, which spread the loads into ground. The seismic loads are mainly taken by the frame systems consisting of beams and columns. External unreinforced concrete masonry stone walls and internal infill walls, that are considered nonstructural elements have a very large moment of inertia in the horizontal direction. Logically, these masonry and infill walls could take lateral loads, especially the masonry walls, because they are connected to the frames directly. Therefore, the masonry stone walls will be presented in the model in a simplified manner, but the internal infill walls will not be presented in the model in this research.

3.3.2 Structural systems

Both lateral and gravity loads are assumed to be transferred by frames. The selected building consists of one way ribbed slab over beams with the same depth as slab depth, which is 25cm. Beams are distributed in both principal directions. Main beams are designed in north-south direction, and the secondary beams are distributed in the other direction, see figure (3.3.2). All columns have the same dimensions and the same reinforcement, except the columns of the ground floor, which have larger concrete dimensions with similar reinforcement. Stair case has four columns surrounding it, with no shear walls, but there are two block walls. There is one elevator-well walls located at the middle of the building.

The foundation system shown in figure (3.3.1) consists of two strip footings and large single footings connected to each other with tie beams that are 80cm deep with 50cm wide. This leads us to assume that the base joints may act as a fixed supports.

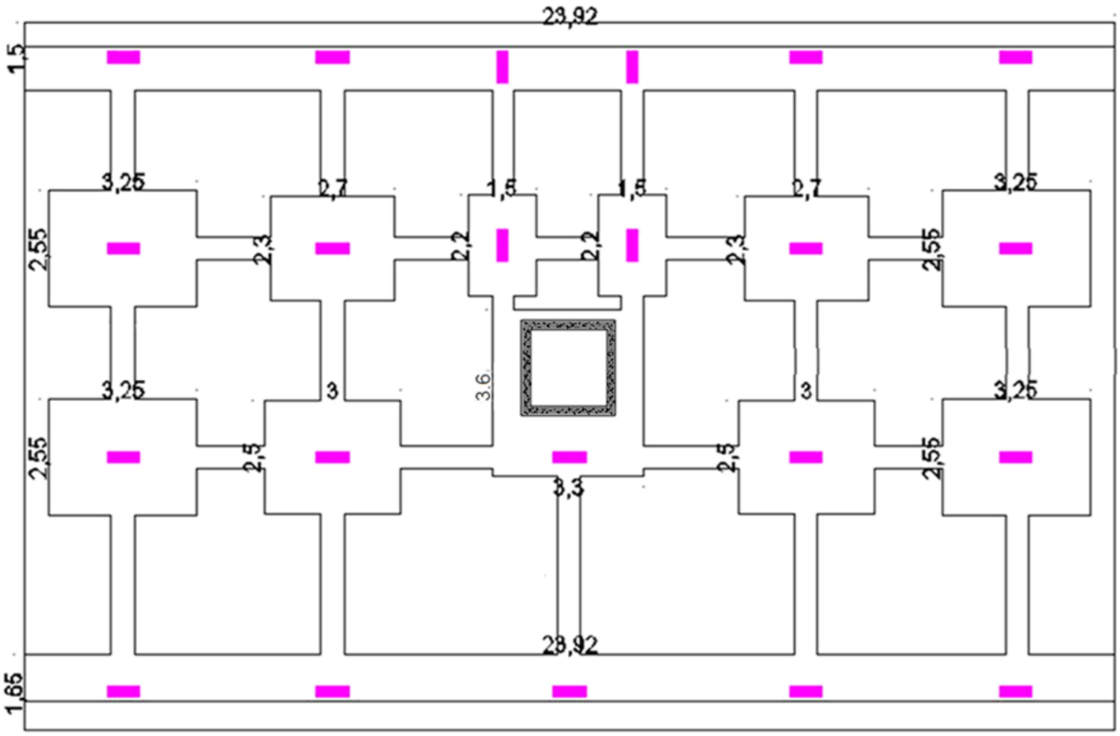

Figure 3.3.1. Foundation system

The characteristics of Structural elements are shown in details in Table (3.3.1).

Table 3.3.1. Characteristics of Structural elements

Name	Width (cm)	Depth (cm)	Longitudinal reinforcement		Ties
GF column	**75**	**30**	**12 ϕ16mm**		**6ϕ8mm/m**
Column	**70**	**25**	**10 ϕ16mm**		**6ϕ8mm/m**
			Top reinf.	**Bott. reinf.**	
Main beam A	**75**	**25**	**9 ϕ16mm**	**6 ϕ16mm**	**6ϕ8mm/m**
Main beam B	**75**	**25**	**6 ϕ16mm**	**5 ϕ16mm**	**6ϕ8mm/m**
Main beam C	**75**	**25**	**5 ϕ16mm**	**5 ϕ14mm**	**6ϕ8mm/m**
Secondary Beam 1	**50**	**25**	**8 ϕ16mm**	**6 ϕ16mm**	**6ϕ8mm/m**
Secondary Beam 2	**50**	**25**	**6 ϕ16mm**	**5 ϕ14mm**	**5ϕ8mm/m**
Secondary Beam 3	**50**	**25**	**6 ϕ14mm**	**5 ϕ14mm**	**5ϕ8mm/m**
Secondary Beam 4	**50**	**25**	**5 ϕ14mm**	**5 ϕ12mm**	**5ϕ8mm/m**
Secondary Beam 5	**25**	**25**	**2 ϕ12mm**	**3 ϕ12mm**	**5ϕ8mm/m**

The cover for beams is considered 2.5cm and for columns and elevator walls is 3cm.

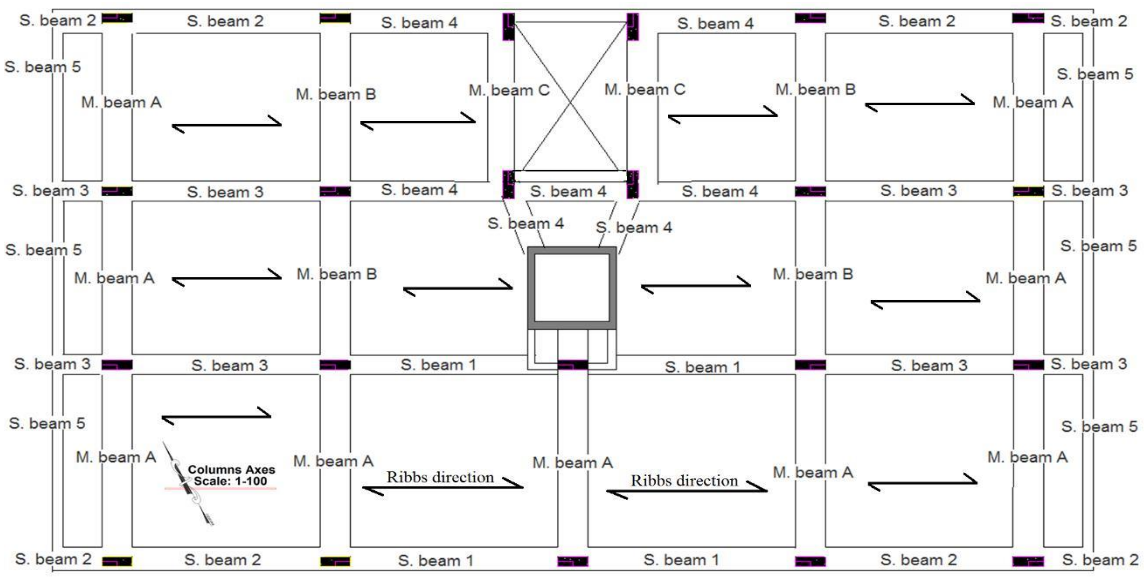

Figure 3.3.2. Beams distribution.

Due to reasons that will be explained in section (3.6.2), the elevator shaft walls will be divided into two basic vertical elements, corner L-section element with 0.5m in both sides and thickness of 0.2m, and straight element with 1m width and 0.2m thick, see figure (3.3.3).

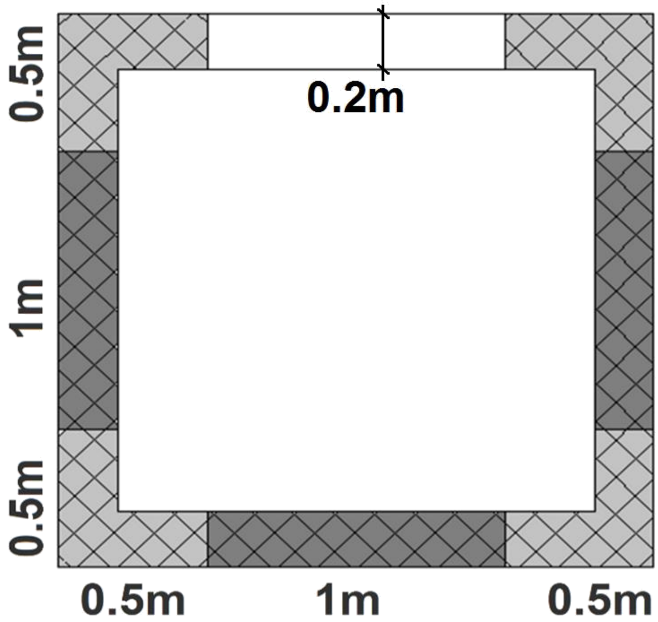

Figure 3.3.3. Assumed divisions of elevator-well shaft

3.4 Materials

The selected building is made of reinforced concrete. The concrete compressive strength f'c differs for slabs, columns, footings and other structural elements. Concrete specific weight is 25 kN/m^3. The steel type used is ASTM A615 Grade 60. More characteristics of materials used is shown in Table (3.4.1).

Table 3.4.1: The characteristic of the used materials

Slab & Beams Concrete		Columns & Footings Conc.	Steel ASTM A615 Gr60	
F'c (cylinder) (MPa)	**21**	**24**	**E (GPa)**	**200**
E (GPa)	**21.54**	**23.03**	**Poisson's ratio (v)**	**0.3**
Poisson's ratio (v)	**0.2**	**0.2**	**Min. yield stress Fy (MPa)**	**414**
			Min. tensile stress Fu (MPa)	**620**
Specific weight (kN/m3)	**25**	**25**	**Specific weight (kN/m^3)**	**77**

The relationship between modulus of elasticity and compressive strength for concrete is taken according to the following empirical formula. [ACI 318M-14, 2015]

$$E = 4700\sqrt{f_c'} \text{ (in MPa) …Eq. 3.4.1 (code eq. 19.2.2.1.b)}$$

Where,

E= Elastic modulus MPa

f_c'= 28 day compressive strength MPa

Dense tie reinforcement in columns can increase the ultimate strain significantly, which means providing more ductility because the deformability is increased due to the tri-axial state of stress. Lateral swelling is prevented by closed ties (hoops) or spirals and the following figures (3.4.1, 3.4.2, and 3.4.3) show the importance of confinement.

The effect of confinement is modeled by changing stress-strain curves of the concrete inside the columns. In this research, the core concrete inside stirrups is considered confined near the joints, where the stirrups are denser. Because the structural details indicate increasing the number of stirrups at both ends of the columns, see (Appendix E) for structural details.

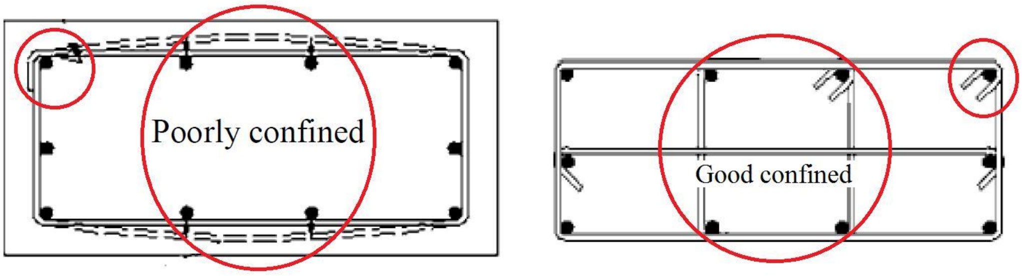

Figure 3.4.1. Typical section for poor and good confined concrete column

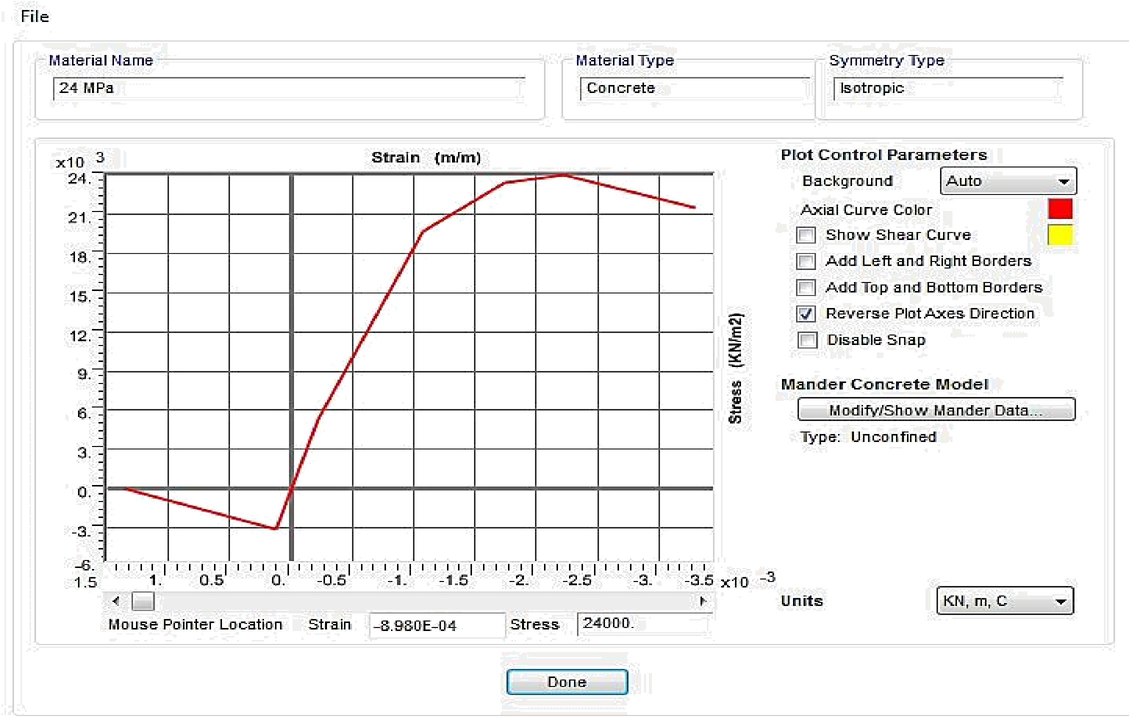

Figure 3.4.2. Unconfined concrete stress-strain curve

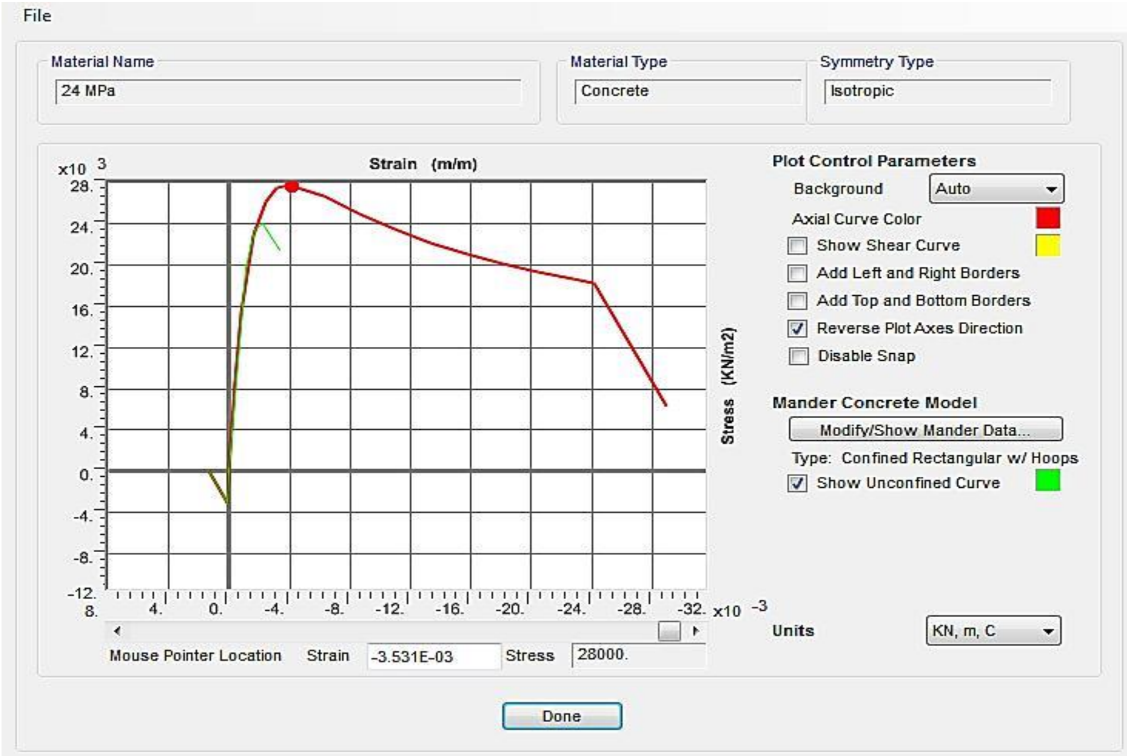

Figure 3.4.3. Confined concrete stress-strain curv

3.5 Vertical loads

The live loads were taken according to "ASCE standard ASCE/SEI 7-10". The super imposed dead load is calculated based on typical finishes in Palestine. The used vertical loads are summarized in Table (3.5.1).

The lateral seismic loads will be discussed in details in the next chapter.

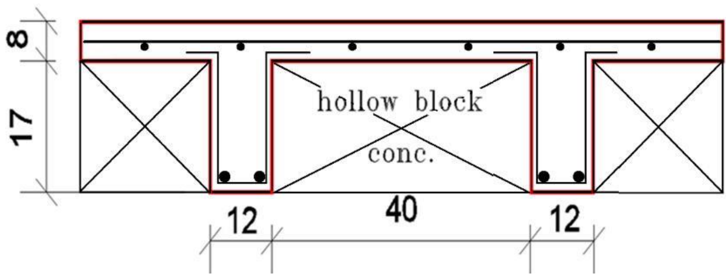

Figure 3.5.1. Section in slab (cm)

Table 3.5.1. Summary of adopted vertical loads

	Design load	Value	Unit
Dead Load	Slab self-weight	4.22	kN/m^2
	Super Imposed load	3.5	kN/m^2
	External stone walls	20.75	kN/m
Live	Private rooms and corridors (residential)	2	kN/m^2
	Balconies	3	kN/m^2
	Roof	3	kN/m^2

3.6 Modelling features

3.6.1 Introduction

The modeling process is the most critical step of any research, where many assumptions must adopted in order to simulate reality. In this section, 3D-model will be built for the selected building. SAP2000 V.18 program is selected to analyze and to evaluate the performance of the selected building before and after retrofitting.

The following sections show the assumptions used for creating the model.

3.6.2 Assumptions & possible scenarios

Generally, in common buildings in Palestine a floor made of concrete can be considered to act as a rigid diaphragm. Rigid diaphragms are constrained by walls or some lateral resisting systems such as masonry walls and moment resisting frames. [Louie L. Yaw, 2015]. Rigid diaphragm distributes the lateral forces to the resisting vertical elements according to the rigidity of each element.

Diaphragm constraints can simplify the analysis and reduce computation time, but they may also affect the accuracy of the results. Therefore, several scenarios will be studied considering diaphragm constraints. The following sections explain these scenarios adopted for comparison:

1- Rigid Diaphragm with Bracing System (D-NA-B model)

   This scenario assumes rigid diaphragm constraints at each floor level. The slabs are disregarded. Only the mass and gravity loads are transferred manually from the slabs to the beams. The effect of masonry walls is modeled as bracing elements.

2- No-diaphragm, with Bracing System (ND-A-B model)

   This scenario assumes the slab to behave as elastic shell elements with stiffness modifications to force the slab to work as one-way slab. Diaphragm constraints are not applied. The bracing elements are also be used to model masonry walls.

3- Frames with Bracing System (ND-NA-B model)

   This scenario assumes no diaphragm constraints and also neglects the slab altogether. The bracing elements are used to model the masonry walls.

   The second group (B) neglect the effect of bracing by masonry walls:

4- Rigid Diaphragm without Bracing System (D-NA-NB model)

   This model is similar to D-NA-B but with removing the bracing elements. The goal of this model is to study the effect of diaphragm working with frames without bracing.

5- Area Sections without Bracing System (ND-A-NB model)

This model is similar to ND-A-B but with removing the bracing elements. The goal of this model is to study the effect of in-plane elastic behavior of slab in conjunction with frames without bracing.

6- Frames without Bracing System (ND-NA-NB model)

This model is similar to ND-NA-B but with removing the bracing elements and with no diaphragm constraints. The goal of this model is to study the efficiency of frames without in-plane stiffness and without bracing.

It is expected that the scenario ND-A-B will be the closest to represent reality, because it contains the least assumptions among them, however such scenario will be computationally demanding and may cause convergence problems in the nonlinear analysis phase.

Therefore, the idea of studying the effect of several simplification scenarios may come handy because if similar results may be obtained using simpler models, this may improve the efficiency of the assessment study.

Other assumptions regarding special elements are also discussed below: The external walls may act as lateral bracing for the structure. It is difficult to represent the unreinforced masonry walls with area sections, because this increases the degrees of freedom significantly, which decreases the efficiency of the F.E. program. Therefore, masonry walls will be modeled in this thesis as bracing elements to study their capacity in resisting seismic loads. These bracing elements are released from resisting moments at their ends and have brittle behavior. A typical masonry wall with window opening was represented by bracing elements and is verified in (Appendix A).

The model for the elevator shaft walls is divided into two basic vertical elements; corner L-section elements with 0.5m in both sides and a thickness of 0.2m, and straight elements with 1m length and 0.2m width, as shown in figure (3.3.3). The reinforcement is the same as the drawings, which is two mesh layers of rebar of dia. 12mm vertical bars per 20cm and dia. 8mm horizontal bars per 20cm.

This assumption of simulating the elevator shaft into divisions is also used to overcome a deficiency in SAP2000, which is the difficulty of modeling nonlinear behavior of area sections. The assumption of separated vertical beam elements was compared with nonlinear shell elements and showed the same results in modal shape analysis. Therefore, these divisions will be represented as line elements as beam-columns.

The staircase area, which is about (4*2.5m) will be modeled as a part of the slab area, for two reasons. The first reason is that stairs end with a 15cm slab and a beam connecting the two columns. The second reason is that the stairs connect the four column, see figure (3.6.2).

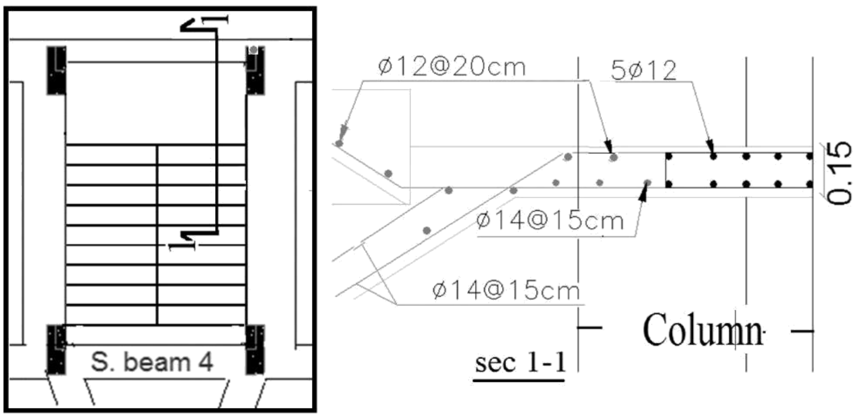

Figure 3.6.1. Flight and landing details in stair

3.6.3 Input data

3.6.3.1 Define materials

There are four types of materials that will be used in modeling. Three of them are mentioned in section (3.4). The forth material is the bracing elements material, which has an "equivalent" value for elastic modulus, see figure (3.6.2), and for more details see Appendix A.

The volume unit weight of bracing elements material is considered zero because the weight of the masonry walls will be added manually as super imposed dead load. Also, the elements weight is considered zero because it will be added among the weight of the slab.

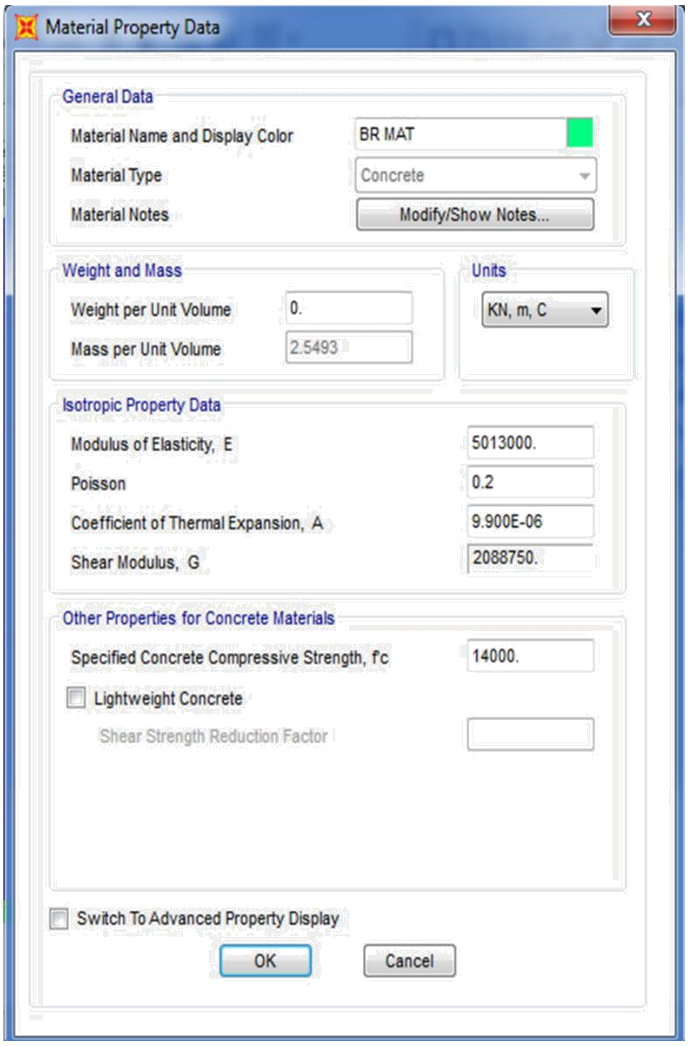

Figure 3.6.2. Bracing elements material

3.6.3.2 Defining sections

Sections need to be defined using the following modifications:

1- Columns:

The columns are two types: The ground floor columns (75*30cm) and the above floors columns (70*25cm) as explained in section (3.3).

According to ACI 318-14, the moment of inertia of the columns must be multiplied by a factor of (0.35) to consider effect of cracking. In addition, the height of the columns are defined as center-to-center length, which reduced the stiffness. Therefore, the moment of inertia will be multiplied by a factor equals to $(L_{c\text{-}c})^3/(L_{net})^3$, which equals to 1.18 for ground floor and 1.27 for the rest floors.

2- Beams of the slab:

The defined beams are the main beams and secondary beams as explained in section (3.3). According to ACI 318-14, the moment of inertia of the beams must be multiplied by a factor of (0.35) to include the effect of cracking.

3- Bracing elements system:

The masonry walls were represented as bracing system with diagonal brittle behavior and have moment releases at the ends of the elements as explained earlier in this section. And it doesn't take tensile strengths.

4- Slab:

The slab system is one-way ribbed slab with 25cm thickness and it is modeled as equivalent solid slab with 18.74cm thickness in one direction. The moment of inertia was factored according to (ACI 318-14) in the direction of the ribs by 0.25 and 0.05 in the perpendicular direction to the ribs.

3.6.3.3 Assign support conditions

As mentioned in section (3.3) the foundation system consist of strip footings and large single footings that are all tied together with large tie beams. For that, the end conditions for columns will be assumed to be fixed supports.

3.6.3.4 Assign diaphragm constraints

This step is done for the D-NA-B and D-NA-NB models. As mentioned earlier in this section (3.6).

3.6.3.5 Define load patterns and assigning loads

This step is to define load patterns, and then assign loads for frames in (D-NA-B) models and for area and frames in (ND-A-B) model.

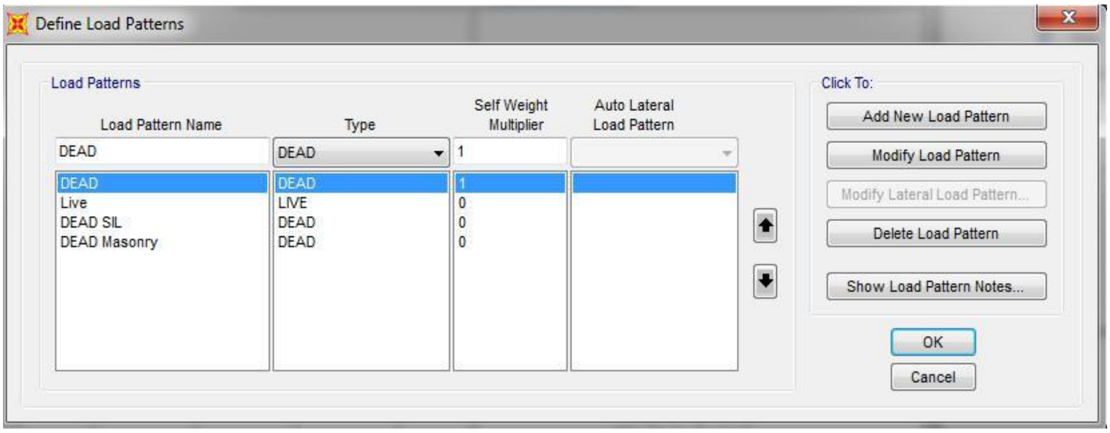

Figure 3.6.3. Loads patterns

3.6.3.6 Assign release/partial fixity

This step is to release bracing elements from resisting moments, because they are representing brittle unreinforced masonry walls.

3.6.3.7 Define mass source

This step is to define the mass source for the elastic modal shape analysis. All dead loads including superimposed dead load are considered to contribute to the vibration mass.

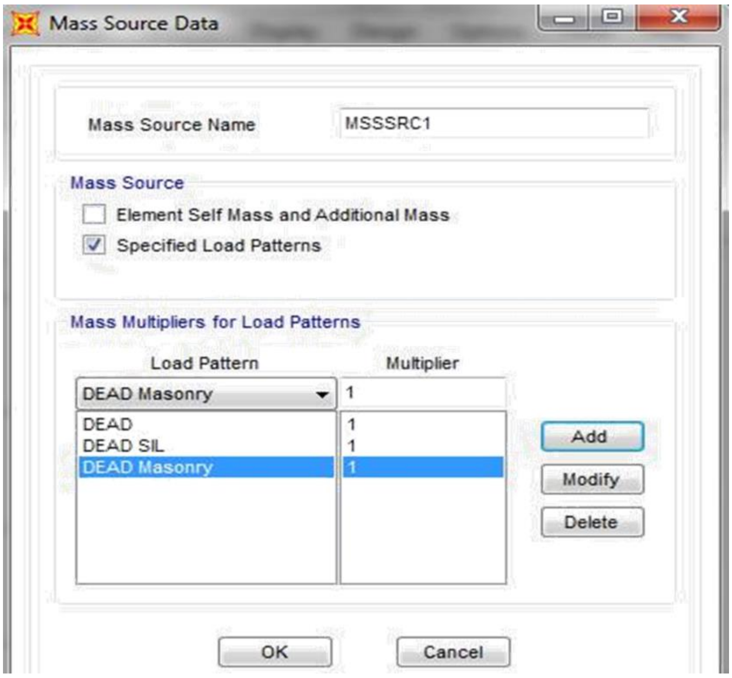

Figure 3.6.4. Mass source

3.6.3.8 Model scenarios

The resultant models (D-NA-B) & (ND-A-B) models represent case study building (reality). The other four models are similar to the first two models, but represent different scenarios as explained earlier.

Group (A): With bracing elements (masonry walls effect)

1- D-NA-B

(With Diaphragm constraints -No Area elements -with Bracing elements)

2- ND-A-B

(No Diaphragm constraints - With Area elements -with Bracing elements)

3- ND-NA-B

(No Diaphragm constraints - No Area elements -with Bracing elements)

Group (B): Without bracing elements, (no masonry walls effect)

4- D-NA-NB

(With Diaphragm constraints -No Area elements -No Bracing elements)

5- ND-A-NB

(No Diaphragm constraints - With Area elements -No Bracing elements)

6- ND-NA-NB

(No Diaphragm constraints - No Area elements -No Bracing elements)

CH4 Inelastic modeling and analysis of case study building

4.1. Introduction

In this thesis, a methodology is being clarified and sequenced for the assessment and retrofitting of existing buildings. This methodology is provided in American building codes for their local use. These codes includes ACI 318-14, ASCE 7-10, FEMA 365, and ATC-40. These methods are developed to predict the behavior of the structure in case of earthquake event and to identify structural elements that are vulnerable to collapse. In addition, retrofitting techniques will be displayed and some of these techniques will be explained in how to model, apply, and assess the retrofitted building.

In order to apply this methodology, the chosen building must be regular shape and of clear structures systems as much as possible. If the building has too many irregularities, then the assumptions that are used in this method may not be valid and thus the assessment may not be realistic.

The models of the case study building that were described in chapter (3) simulate reality with different levels of assumptions. Each model represent a set of assumptions in order to understand the effect and importance of each structural component on the performance of the building. For more details, see section (3.6).

Before moving on to plastic analysis, we must first check behavior of the building under the gravity loads and its conformity to ACI code. Moreover, the state of the building under seismic loads as defined by IBC code must be checked. These checks are designated as elastic checks and are explained in section (4.2).

Section (4.3) talks about pushover analysis methodology, uses, and limitations. Then section (4.4) explains the pushover procedure used by ATC-40, which is called capacity spectrum method (CSM). Modeling pushover properties, loads, and assumptions will be explained in section (4.5). In section (4.6) results of pushover analysis will be displayed, which include pushover curves (base shear vs. top displacement), demand and capacity spectra curves and the point of intersection between the two curves (performance point). The last section (4.7) analyzes the results obtained from pushover analysis, and compares them with the assumed target point for the building.

The line of thinking is summarized in a flow chart of the methodology presented in this thesis in (Figure 4.1.1)

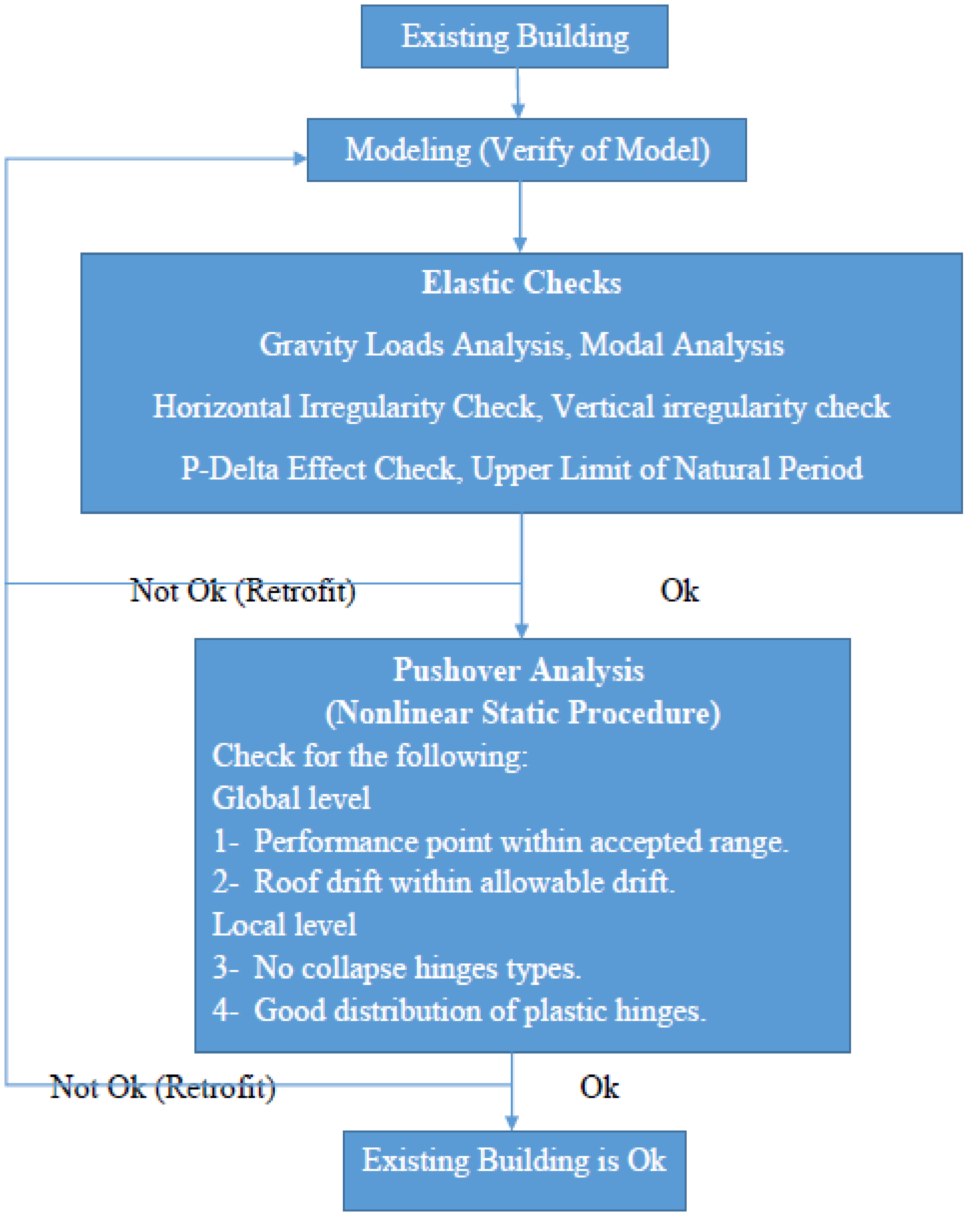

Figure 4.1.1. Methodology presented in this thesis

4.2 Elastic analysis and checks

4.2.1 Gravity loads analysis

Gravity loads studied in this thesis include dead and live loads were assigned to six models that represent six scenarios as mentioned in section (3.6) namely:

1- D-NA-B

(With Diaphragm constraints -No Area elements -with Bracing elements)

2- ND-A-B

(No Diaphragm constraints - with Area elements -with Bracing elements)

3- ND-NA-B

(No Diaphragm constraints - No Area elements -with Bracing elements)

4- D-NA-NB

(With Diaphragm constraints -No Area elements - No Bracing elements)

5- ND-A-NB

(No Diaphragm constraints - with Area elements - No Bracing elements)

6- ND-NA-NB

(No Diaphragm constraints - No Area elements - No Bracing elements)

Equilibrium check was made for the models and can be seen in (Appendix A). The differences between the base reactions in the models do not exceed 3% of the manual calculated values. Therefore, the two these models can be considered equivalent in gravity loads.

The design checks of the structural elements satisfy the gravity loads combination proposed by ASCE-10 (2.3.2) to sustain the ultimate loads predicted in this equation:

1. 1.4D
2. 1.2D + 1.6L …..Eq 4.2.1

In addition, the resulting reinforcement areas are compatible with the real reinforcement as provided in structural detailing, that is shown in the figure (4.2.1).

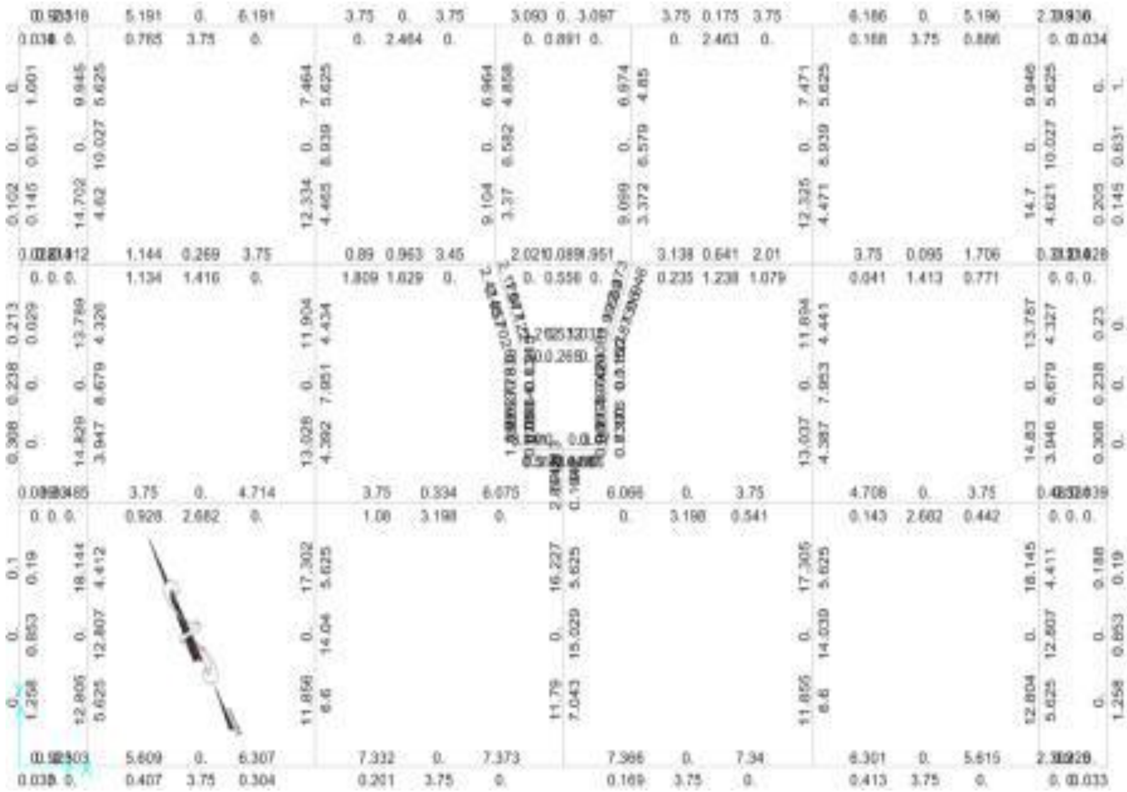

Figure 4.2.1. Resulting reinforcement area in cm^2 of slab beams

4.2.2 Modal analysis

The goal of modal analysis is to determine the natural modal shapes and their frequencies for the structure during free vibration. The modal shapes are computed by SAP2000 using Eigen vectors method, which uses distribution of the masses and stiffness of the structure. The output of the system are Eigenvectors and Eigenvalues that represent frequencies of the modal shapes of the structure. The lowest frequency indicates the fundamental mode.

In order to verify the results of the program, certain checks were done. These checks are compatibility, equilibrium, internal load checks, and elastic period using Rayleigh method (See appendix A). The results showed that the F.E. model simulates the structure as expected according to the adopted assumptions.

The results of modal analysis and modal mass participation ratios for the two models (D-NA-B and ND-A-B) are shown in tables 4.2.1 and 4.2.2.

Table 4.2.1. Modal analysis results for (D-NA-B) (Diaphragm with No Area elements with Bracing elements)

Modal mass participation ratio							
Mode	**Period**	**UX**	**UY**	**RZ**	**SumUX**	**SumUY**	**SumRZ**
	Sec	Unitless	Unitless	Unitless	Unitless	Unitless	Unitless
1	0.893	0.0000	0.9528	0.0040	0.000	0.953	0.004
2	0.888	0.0000	0.0039	0.9883	0.000	0.957	0.992
3	0.726	0.9506	0.0000	0.0000	0.951	0.957	0.992
4	0.265	0.0000	0.0370	0.0000	0.951	0.994	0.992
5	0.217	0.0395	0.0000	0.0004	0.990	0.994	0.993

Table 4.2.2. Modal analysis results for (ND-A-B) (No Diaphragm with Area elements with Bracing elements)

Modal mass participation ratio							
mode	**Period**	**UX**	**UY**	**RZ**	**SumUX**	**SumUY**	**SumRZ**
	Sec	Unitless	Unitless	Unitless	Unitless	Unitless	Unitless
1	0.911	0.0000	0.9585	0.0000	0.0000	0.9585	0.0000
2	0.883	0.0001	0.0000	0.9921	0.0001	0.9585	0.9921
3	0.723	0.9496	0.0000	0.0001	0.9497	0.9585	0.9921
4	0.267	0.0000	0.0348	0.0000	0.9497	0.9933	0.9921
5	0.219	0.0386	0.0000	0.0003	0.9883	0.9933	0.9924

In the previous tables (4.2.1, 4.2.2), the first three modes in the main two models have the largest modal mass participation ratios (MMPR), which means they are the fundamental modes.

As can be seen in the tables (4.2.1, 4.2.2), the differences between the two models (D-NA-B) and (ND-A-B) can be considered negligible in terms of periods, and MMPRs. Therefore, the model (D-NA-B) can be assumed acceptable model to study the building with.

Therefore, the first mode period is T_1 =0.893 sec and has significant MMPR in Y-direction that equals to 95%. This mode is translational mode. The second mode period is T_2 = 0.888 sec, and it has insignificant MMPR in X & Y directions, but has a large MMPR in rotational movement around Z direction that is equal to 98.8%. The second mode is considered a torsional mode, which is undesired mode and needs to be remedied. The third mode period is T_3 = 0.726 sec, and it has MMPR in X-direction equals to 95% with insignificant values in other directions, therefore it is considered translational mode.

Table 4.2.3. Summary of modal analysis results for the dominant modes.

		Group A			group B		
		D-NA-B	ND-A-B	ND-NA-B	D-NA-NB	ND-A-NB	ND-NA-NB
mode1	period	0.893	0.911	1.15	1.87	1.84	1.89
	direction	UY	UY	UY	RZ	RZ	RZ
	MMPR	95%	96%	95%	81%	85%	83%
mode2	period	0.88	0.88	0.96	1.51	1.49	1.53
	direction	RZ	RZ	RZ	UX	UY	UY
	MMPR	99%	99%	98%	85%	85%	86%
mode3	period	0.72	0.72	0.79	1.5	1.39	1.51
	direction	UX	UX	UX	UY	UX	UX
	MMPR	95%	95%	94%	85%	82%	79%

D= with Diaphragm constraints, ND= No Diaphragm constraints

A= with Area elements, NA= No Area elements

B= with Bracing elements, NB= No Bracing elements

UX= translation in X direction, UY= translation in Y direction, RZ= rotation about Z direction

The results in table (4.2.3) shows that torsional mode is one of the three fundamental modes. Clearly, using bracing system has a direct effect on changing the first mode from torsional mode to translational mode. Also, the bracing increased the lateral stiffness and thus decreased the period.

4.2.3 Check for seismic design requirements according to ASCE 7-10 and IBC-2012

According to (ASCE 7-10) code, structures are not allowed to cross certain limits in order to guarantee safe seismic behavior. These limits cover horizontal irregularity and vertical irregularity. Other checks must be done, such as p-delta effect in order to determine if P-delta effect can be neglected or not.

Applying these checks to existing buildings can give an idea about the state of the building during seismic assessment. In addition, these tests can tell what retrofitting strategy may be adopted.

In order to do all of these checks, the model should be exposed to seismic loads according to the site and the building characteristics, which are specified in (ASCE) 7-10 and IBC-2012 design codes. According to design category of the building "D", the analysis method that can be used to assess the building are (ELF) method and response spectrum analysis (RSA) method. The following table (4.2.4) shows the input parameters needed to perform equivalent lateral seismic loads.

The building may resist lateral seismic forces with two different systems, and that are ordinary moment resisting frames on the ground floor only and unreinforced masonry walls on the above floors. These resisting systems are classified in ASCE 7-10 code as case (C-7) which gives the factors R=3, Ω=3, and Cd=2.5 for ground floor and case (B-19) with R=2, Ω=2.5, and Cd=2. The values of R and Ω for our building should be between these values. R represent the load reduction factor due to inherent ductility of the structure while Ω represent the over strength factor of the building. Therefore, the average values is taken as the following in table (4.2.4).

Table 4.2.4. Site and building characteristics and the base shear calculation of ELF method according to ASCE 7-10. [ASCE 7-10, 2010]

Seismic zone factor Z	0.2	Importance factor I	1
Ss	0.5	Seismic Force-Resisting Systems as in ASCE 7-10 table 12.2-1	
S1	0.25		
Sds	0.467	At ground floor	C-7
Sd1	0.317	At upper floors	B-19
Ct	0.0488	Average of R	2.5
X	0.75	Average of Ω	2.75
Height (m)	21	Average of Cd	2.25
Period Ta (sec)	0.48	Structure weight (kN)	27760
Sa	0.187	Seismic base shear V (kN)	5191
k	1	Seismic Design Category	D

The following table (4.2.3) shows manual calculation and the distribution of equivalent lateral seismic loads on each floor of the building.

Table 4.2.5. Distribution of base shear on each floor. [ASCE 7-10, 2010]

Story	Wx (kN)	hx (m)	hx^k	wx*hx^k	$\frac{wx * hx}{\sum wx * hx}$	Fx (kN)	Vx (kN)
6	3726.667	21	21	78260	0.23	1168.55	1168.55
5	4776.667	17.75	17.75	84785.83	0.24	1265.99	2434.54
4	4776.667	14.5	14.5	69261.67	0.20	1034.19	3468.72
3	4776.667	11.25	11.25	53737.5	0.15	802.39	4271.11
2	4776.667	8	8	38213.33	0.11	570.59	4841.70
1	4926.667	4.75	4.75	23401.67	0.07	349.42	5191.12
0	0	0	0	0	0.00	0.00	
SUM	27760			347660	1	5191.12	

The following figure (4.2.2) shows the definition of the design response spectrum according to ASCE 7-10 that is used for RSA method. Figure (4.2.3) shows the definition of the load case and ($\frac{g*I}{R}$) factor for analysis.

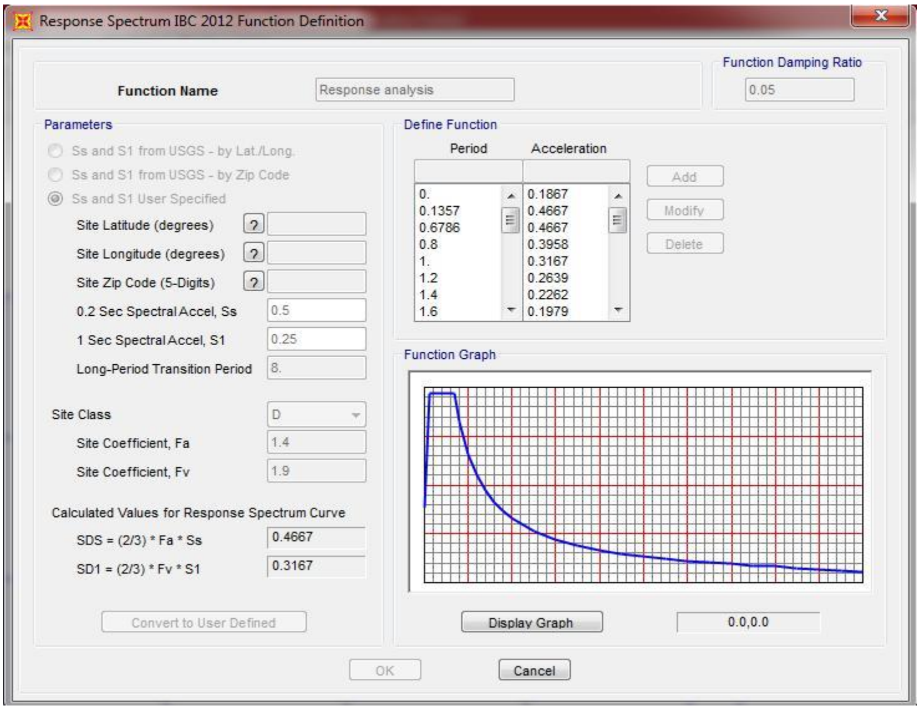

Figure 4.2.2. Design response spectrum according to ASCE 7-10

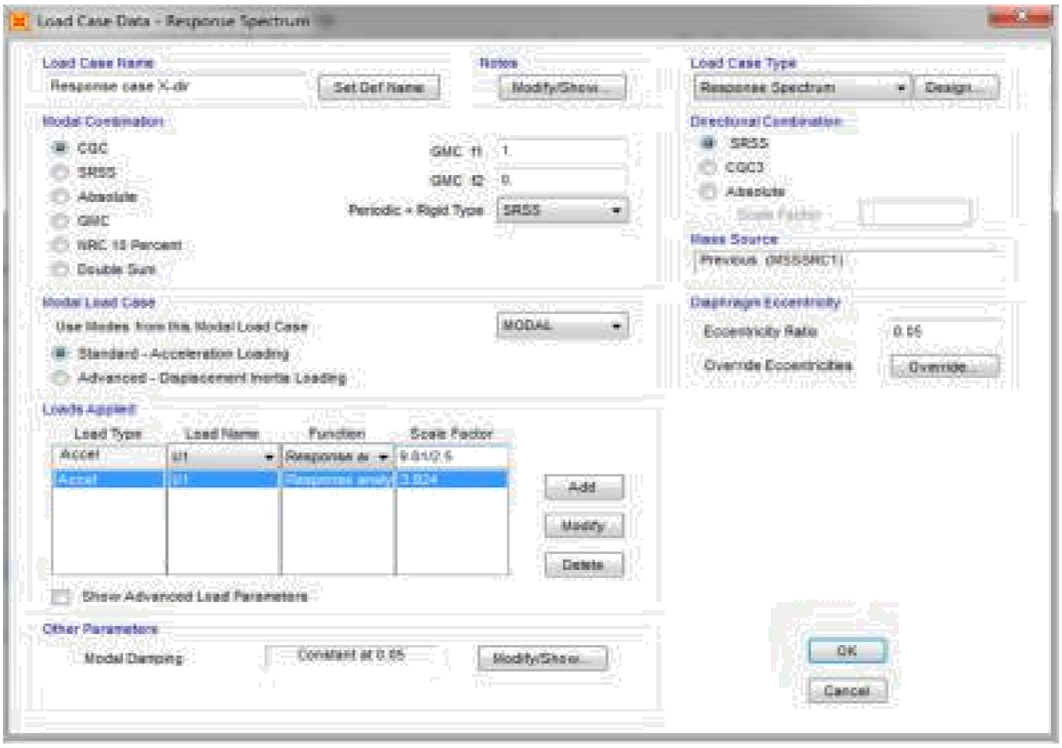

Figure 4.2.3. Definition of the load case and ($\frac{g*I}{R}$) factor for analysis

The results obtained from SAP2000 are shown in figure (4.2.4). It can be seen that the difference between manual and program results of ELF does not exceed ($\frac{5113.34-5191.12}{5113.34}$ = 1.5%). Figure also shows that RSA method gives base shear of 4622.7 kN in X-direction and 3796.3 kN in Y-direction compared to 5113.3 kN for the ELF. The cause of this difference is due to the natural period obtained from the model being larger than the natural period obtained from the code method.

According ASCE7-10 requirements the vertical component must counted for, which equals to (0.2*Sds*DL). Also, 30% of perpendicular force must be added in case of category (D).

Base Reactions

File View Format-Filter-Sort Select Options

Units: As Noted

Filter:

Base Reactions

OutputCase Text	CaseType Text	StepType Text	GlobalFX KN	GlobalFY KN	GlobalFZ KN	GlobalMX KN-m	GlobalMY KN-m	GlobalMZ KN-m
EQ-X-IBC12	LinStatic		-5113.339	-3.855E-13	-9.436E-12	-1.346E-10	-76570.1634	39349.0496
EQ-Y-IBC12	LinStatic		1.197E-11	-5113.339	-1.728E-11	76570.1634	5.13E-10	-67524.7996
Response case X-dir	LinRespSpec	Max	4622.741	0.186	0.374	4.6927	64159.1698	36015.5225
response case Y-dir	LinRespSpec	Max	0.186	3796.285	14.871	52033.2722	178.7146	50339.5957
0.9D+Ex+0.3Ey-Ev	LinStatic		-5113.339	-1534.002	22391.552	179850.4746	-345491.18	19091.6097
0.9D+Ey+0.3Ex-Ev	LinStatic		-1534.002	-5113.339	22391.552	233449.589	-291892.069	-55720.0847
1.2D+Ex+0.3Ey+Ev+L	LinStatic		-5113.339	-1534.002	40551.018	307726.9513	-563538.99	19091.6097
1.2D+Ey+0.3Ex+Ev+L	LinStatic		-1534.002	-5113.339	40551.018	361326.0657	-509939.88	-55720.0847
1.2D+1.6L	Combination		1.662E-13	4.775E-13	40745.612	286507.9711	-489278.21	2.564E-12
service DL	Combination		1.028E-13	4.547E-13	27760.417	194494.7007	-333400.72	3.403E-12
1.4D	Combination		1.439E-13	6.366E-13	38864.583	272292.581	-466761.01	4.765E-12

Figure 4.2.4. Results obtained from SAP2000

1- Horizontal irregularity checks

(ASCE) 7-10 has classified horizontal irregularity into several types as the following:

Table 4.2.4: Horizontal Structural Irregularities. [ASCE 7-10, 2010]

Type	Description
1a.	**Torsional Irregularity:** Torsional irregularity is defined to exist where the maximum story drift, computed including accidental torsion with $A_x = 1.0$, at one end of the structure transverse to an axis is more than 1.2 times the average of the story drifts at the two ends of the structure. Torsional irregularity requirements in the reference sections apply only to structures in which the diaphragms are rigid or semirigid.
1b.	**Extreme Torsional Irregularity:** Extreme torsional irregularity is defined to exist where the maximum story drift, computed including accidental torsion with $A_x = 1.0$, at one end of the structure transverse to an axis is more than 1.4 times the average of the story drifts at the two ends of the structure. Extreme torsional irregularity requirements in the reference sections apply only to structures in which the diaphragms are rigid or semirigid.
2.	**Reentrant Corner Irregularity:** Reentrant corner irregularity is defined to exist where both plan projections of the structure beyond a reentrant corner are greater than 15% of the plan dimension of the structure in the given direction.
3.	**Diaphragm Discontinuity Irregularity:** Diaphragm discontinuity irregularity is defined to exist where there is a diaphragm with an abrupt discontinuity or variation in stiffness, including one having a cutout or open area greater than 50% of the gross enclosed diaphragm area, or a change in effective diaphragm stiffness of more than 50% from one story to the next.
4.	**Out-of-Plane Offset Irregularity:** Out-of-plane offset irregularity is defined to exist where there is a discontinuity in a lateral force-resistance path, such as an out-of-plane offset of at least one of the vertical elements.
5.	**Nonparallel System Irregularity:** Nonparallel system irregularity is defined to exist where vertical lateral force-resisting elements are not parallel to the major orthogonal axes of the seismic force-resisting system.

According to Figure (3.2.7) columns distribution, the case study building has no horizontal irregularities of type 2, 3, 4, and 5 according to table (4.2.4). To check for the torsional irregularity, the building model (D-NA-B) was exposed to ELF in X and Y-directions with eccentricity ratio of 5% according to the ASCE 7-10. The resultant deformations at the roof showed that the maximum story draft do not exceeds 20% more than average drift either in the direction of X-axis or in the direction of Y-axis as shown in the following calculation:

Average drift in X-direction = $\frac{0.0298 + 0.0276}{2}$ = 0.0287

% of corner drift difference = $\frac{|0.0298 - 0.0287|}{0.0287}$ = 3.8% less than 20%

Average drift in Y-direction = $\frac{0.05234 + 0.03969}{2}$ = 0.046

% of corner drift difference = $\frac{|0.05234 - 0.046|}{0.046}$ = 13.9% less than 20%

2- Vertical irregularity

(ASCE) 7-10 has classified vertical irregularity into several types as the following:

Table 4.2.5. Vertical Structural Irregularities. [ASCE 7-10, 2010]

Type	Description
1a.	Stiffness-Soft Story Irregularity: Stiffness-soft story irregularity is defined to exist where there is a story in which the lateral stiffness is less than 70% of that in the story above or less than 80% of the average stiffness of the three stories above.
1b.	Stiffness-Extreme Soft Story Irregularity: Stiffness-extreme soft story irregularity is defined to exist where there is a story in which the lateral stiffness is less than 60% of that in the story above or less than 70% of the average stiffness of the three stories above.
2.	Weight (Mass) Irregularity: Weight (mass) irregularity is defined to exist where the effective mass of any story is more than 150% of the effective mass of an adjacent story. A roof that is lighter than the floor below need not be considered.
3.	Vertical Geometric Irregularity: Vertical geometric irregularity is defined to exist where the horizontal dimension of the seismic force-resisting system in any story is more than 130% of that in an adjacent story.
4.	In-Plane Discontinuity in Vertical Lateral Force-Resisting Element Irregularity:

	In-plane discontinuity in vertical lateral force-resisting elements irregularity is defined to exist where there is an in-plane offset of a vertical seismic force-resisting element resulting in overturning demands on a supporting beam, column, truss, or slab.
5a.	Discontinuity in Lateral Strength–Weak Story Irregularity: Discontinuity in lateral strength–weak story irregularity is defined to exist where the story lateral strength is less than 80% of that in the story above. The story lateral strength is the total lateral strength of all seismic-resisting elements sharing the story shear for the direction under consideration.
5b.	Discontinuity in Lateral Strength–Extreme Weak Story Irregularity: Discontinuity in lateral strength–extreme weak story irregularity is defined to exist where the story lateral strength is less than 65% of that in the story above. The story strength is the total strength of all seismic-resisting elements sharing the story shear for the direction under consideration.

To check for soft-story irregularity, lateral stiffness of the ground floor in both directions X&Y were calculated directly using SAP2000 according to the following procedure:

1- Fixed restraints where assigned to the first floor joints

2- Add lateral load in X-direction at the face column-beam joint of the ground floor

3- Calculate the drift in the ground floor at the center of mass

4- Lateral stiffness at this floor level = $\frac{Lateral\ load\ (kN)}{Lateral\ drift\ (m)}$ …..Eq 4.2.3

5- Repeat the previous steps for Y-direction.

6- To calculate the lateral stiffness of the first floor, restrain the joints of the ground and second floors only. Then follow the previous steps.

The results of lateral stiffness and their percentages to 1st floor are shown in table 4.2.6.

Table 4.2.6: Vertical Structural Irregularities. [ASCE 7, 2010]

Floor	Direction	Lateral force (kN)	Drift (m)	Lateral stiffness (kN/m)	% Lateral stiffness to 1st F
GF	X	200	1.145e-4	1.750e6	68%
1st F	X	200	7.712e-4	2.590e6	
GF	Y	200	2.078e-4	0.962e6	64%
1st F	Y	200	1.334e-4	1.500e6	

The resulting percentages of stiffness ratios indicate the existence of vertical irregularity of type 1 in both directions. These results must be considered when adopting retrofitting technique.

3- P-delta effect:

P-delta effects may increase the story shears and moments, and the story drifts induced by these effects. According to ASCE requirements P-delta effect is not required to be considered if the stability coefficient (θ) as determined by the following equation is equal to or less than 0.10:

$$\boldsymbol{\theta} = \frac{P_x \Delta I_e}{V_x h_{sx} C_d} \text{.....Eq 4.2.4}$$

Where,

P_x = the total vertical design load at and above Level x (kN); where computing P_x, no individual load factor need exceed 1.0

Δ = the design story drift that is the difference of the deflections at the centers of mass at the top and bottom of the story under consideration, and occurring simultaneously with V_x (mm).

I_e = the importance factor

V_x = the seismic shear force acting between Levels x and x – 1 (kN)

h_{sx} = the story height below Level x (mm)

C_d = the deflection amplification factor

The stability coefficient (θ) values are calculated in table (4.2.7) for the ground floor that shows that (θ) values in both directions are smaller than the upper limit, which means that P-delta effects can be neglected. However, it will be included later with retrofitting techniques.

Table 4.2.7. Stability coefficient for X and Y directions. [ASCE 7, 2010]

X-direction						
θ	Px	Δ	Ie	Vx	hx	Cd
0.01	31610	0.0166	1	5113.4	4.5	2.25
Y-direction						
θ	Px	Δ	Ie	Vx	hx	Cd
0.01	31610	0.02	1	5113.4	4.5	2.25

4- The upper limit value for natural period

According to table (12.8-1) in ASCE 7-10 requirements, the upper value for natural period of the building should not exceed the value of (CuTa) with Cu factor shown in table (4.2.8). For the case study building, this gives 0.5*1.5 = 0.7sec. This means that the natural period of the building exceeds the upper limit of the approximate period of the code.

This difference adds to the reasons for retrofitting of the building. It relates to the same problem of vertical irregularity. Retrofitting techniques may reduce or cancel these problems, and this will be discussed in chapter five.

Table 4.2.8. Coefficient for upper limit on calculated period. [ASCE 7-10]

Design Spectral Response Acceleration Parameter at 1 s, S_{D1}	Coefficient C_u
≥ 0.4	1.4
0.3	1.4
0.2	1.5
0.15	1.6
≤ 0.1	1.7

4.3 Pushover analysis

4.3.1 Introduction

The pushover analysis is a method used to predict the nonlinear behavior of structures, by exposing the structure to increasing lateral loads, until failure occurs. The F.E. software SAP2000 generates the nonlinear behavior curve through incremental analysis assuming elastic behavior between each increment where plastic hinges form at each increment. The resulting curve is called capacity curve whose shape and values depend on the stiffness, strength, sequence of plastic hinges formation, and ductility of the component of the structure.

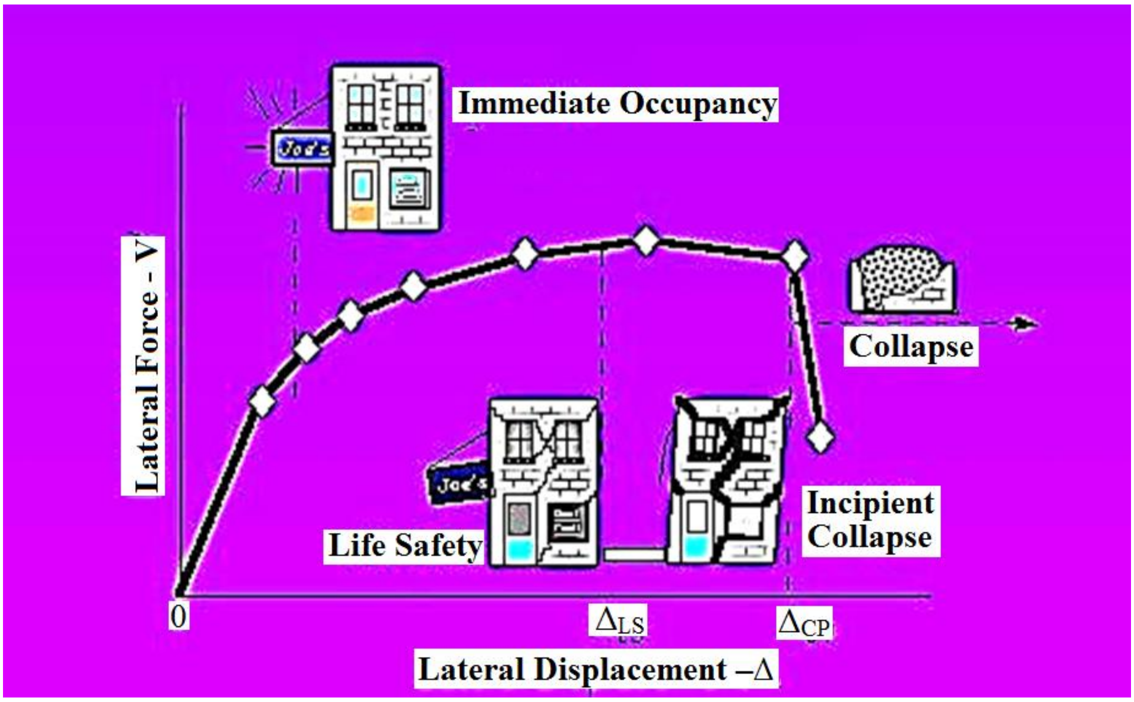

Figure 4.3.1. Typical pushover curve and performance levels

To be useful for earthquake design, the distribution of the pushover lateral loads must be related to the expected loads from the earthquake acceleration. Such forces must depend on the representative modal shape from modal analysis of the structure. Usually the first modal shape, which is considered a fundamental mode, is used as a pattern for the distribution of the lateral pushover loads.

The pushover curve generally relates to the top displacement of the structure against base shear force. Top displacement is taken at the center of the roof mass in order to be related to the mass of the floor as a single degree of freedom equivalence.

The pushover analysis can be done using either force-controlled or displacement controlled procedure. The force-controlled is best used for certain cases where the capacity curve stays monotonic, such as, for gravity loads. Full load is then applied and behavior is expected to remain elastic. The displacement controlled is better used for pushover analysis with lateral loads until failure, where the curve may decrease after certain ultimate value. The advantage of using displacement control with lateral loads can be noticed in the figure (4.3.2). Because the nature of the overall structural behavior, displacement is always increasing, but the load starts to decrease at ultimate point. Therefore, in the displacement-control procedure, the structure is exposed to equal displacement increments and these increments can show the curve turning down. On the other hand, the load increments cannot show turning point of the curve.

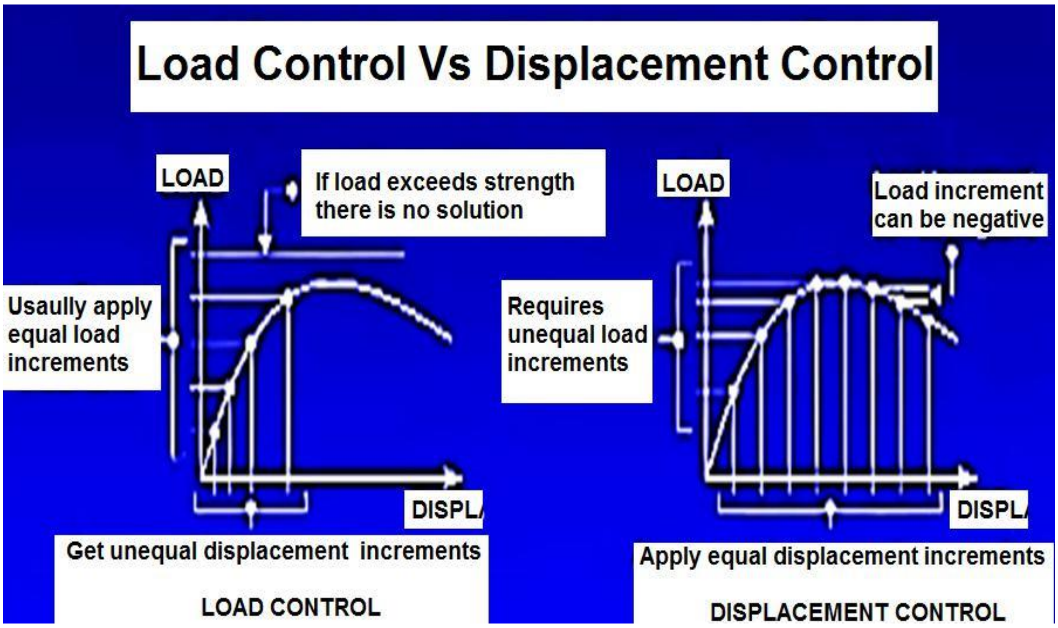

Figure 4.3.2. Load control vs. displacement control

4.3.2 Usage of pushover analysis

Pushover method is a very good method for evaluating the realistic behavior of the existing structures under exposure of lateral loads. It is simple for calculation, straightforward, and clear in concept. Because of these advantages, it is preferable than nonlinear dynamic procedure (NDP). NDP is considered time consuming and complex to perform. Pushover analysis is preferred more than linear static procedures (LSPs) and linear dynamic procedures (LDPs), since these linear methods do not show the real behavior of the structure and the effect of weak members in the structure.

Pushover analysis can show the following information about the analyzed structure [Christiana, 2013]:

- The overall structure drift at each floor level.
- The demand forces on each member of the structure specially columns and beams.
- Location of weak elements of the structure.
- Collapse sequence through formation of plastic hinges.
- The effect of the failing members on the overall structure behavior.
- The irregularity of the strength and stiffness of the building in horizontal plane or vertically.
- The load path adequacy

4.3.3 Limitations of pushover analysis

Pushover analysis is an effective and accurate procedure when compared with the elastic analysis, but the limitations of this method must be known and identified. These limitations are generally related to the selection of representative horizontal load patterns, and target displacement at the center of roof mass. Limitations of pushover analysis can be summarized in the following points:

- The top roof displacement is generally selected to represent (as SDOF) target displacement for a multi-DOF the structure. However, if the structure is dominated by more than one mode shape, then the top roof displacement may not be a good indicator for the behavior of the overall structure. Therefore, this method is meaningful if the structure vibrates in a single dominant mode.
- Distribution of inertia forces are represented by the lateral loads, which are imposed on the structure during an earthquake. The distribution of inertial forces vary with the severity of earthquake and with time during earthquake due to varying acceleration and load reversal. However, usually in pushover analysis, an invariant lateral load pattern is used and the distribution of inertial forces is assumed to be constant during earthquake.

Therefore, selection of load pattern distribution is as important as selection of target displacement.

The previous limitations made many researchers try to adopt adaptive load patterns in order to consider changes in load pattern with the level of inelasticity. The improvements of these adaptive methods include the redistribution of the lateral load shape as a function of the current inelastic deformations. Also, other researchers tried to apply displacement loads instead of static forced, in what is called (displacement based pushover analysis). These new methods are not yet well developed for codicils use.

However, even if these load patterns are invariant or adaptive, they are still static loads and cannot represent inelastic dynamic response with high accuracy. The above discussion about target displacement and lateral load pattern shows that pushover analysis supposes that the response of a building can be related to that of an equivalent SDOF system. This means that the building must be controlled by major fundamental mode even if with adaptive methods.

4.4 Capacity spectrum method CSM

4.4.1 Introduction

CSM is a pushover method presented in 1975 by Freeman and his collaborators, and then adopted by ATC-40, and then FEMA 440 adopted this method and developed it in order to increase its accuracy.

CSM is a nonlinear static procedure (pushover procedure) used for evaluation and retrofitting of the existing structures by showing the performance of the structure. Figure (4.4.1) shows the fundamental components of the method.

CSM is based on assuming idealistic hysteretic models for building model, and then modifying the demand curve on different equivalent stiffness and damping ratios. The capacity curve (top displacement vs. base shear) and the demand spectrum are plotted as spectral acceleration vs. spectral displacement response format.

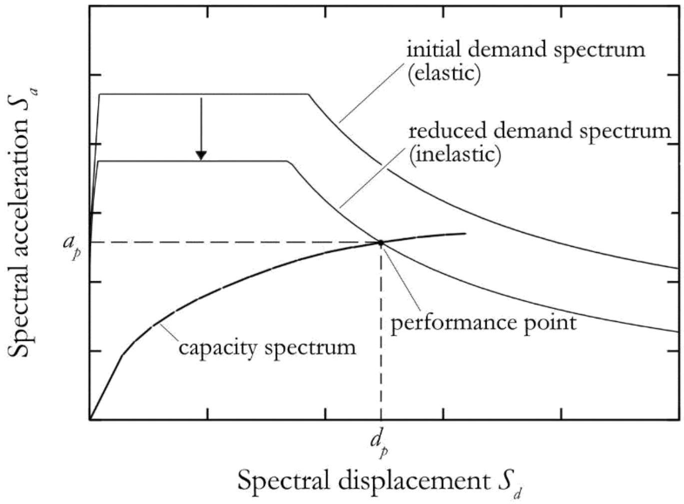

Figure 4.4.1. CSM procedure components and determination of performance point

The performance point is the point of intersection of the capacity and demand spectra, which represent the nonlinear load demand that meets the top displacement of the structure.

The reduction factors of the demand spectrum are based on approximate effective damping, which is calculated based on the capacity curve, estimated displacement demand, and expected hysteresis loop.

4.4.2 CSM procedure as per ATC-40

4.4.2.1 Conversion of MDOF system into an equivalent SDOF system

The CSM is based on the conversion of a MDOF system into an equivalent SDOF system. In the method, it is assumed that a single modal shape "the fundamental mode" that is derived from a MDOF can represent the behavior of an equivalent SDOF. The SDOF is equivalent to MDOF in terms of equivalent absorbed energy and equivalent lateral stiffness. The method iterates between the two systems (MDOF with the resulting capacity curve and the demand load calculated from the equivalent SDOF) until the performance point is reached, which represents the point where the absorbed energy and equivalent stiffness for the two systems are identical.

One of the fundamental parameters is the distribution of the horizontal pushover load along the structure floors. This is usually related to each floor mass according to modal shape of the fundamental mode as follow:

$$Ft = V * \frac{mi*\phi i}{\sum mj*\phi j}, \qquad \text{….Eq 4.4.1}$$

$$V=\sum Ft \text{ the total base shear,} \qquad \text{….Eq 4.4.2}$$

mi: mass of floor,

ϕi: represents the first modal shape. Normalized to the top floor displacement.

The transformations between MDOF into SDOF are made using the following equations:

$$PF_1 = \left[\frac{\sum_{i=1}^{N} (w_i \phi_{i1}) / g}{\sum_{i=1}^{N} (w_i \phi_{i1}^2) / g} \right] \qquad \text{….Eq 4.4.3}$$

$$\alpha_1 = \frac{\left[\sum_{i=1}^{N} (w_i \phi_{i1}) / g \right]^2}{\left[\sum_{i=1}^{N} w_i / g \right] \left[\sum_{i=1}^{N} (w_i \phi_{i1}^2) / g \right]} \qquad \text{….Eq 4.4.4}$$

$$Sa = \frac{V/W}{\alpha_1} \qquad \text{....Eq 4.4.5}$$

$$Sd = \frac{\Delta_{roof}}{PF1 \emptyset_{roof,1}} \qquad \text{....Eq 4.4.6}$$

Where:

PF_1 : modal participation factor for the first natural mode

α_1: modal mass coefficient for the first natural mode

w_i / g : mass assigned to level i

$\emptyset_{i1}$: amplitude of mode 1 at level i

N : Level N, the level which is the uppermost in the main portion of the structure

V : Total base shear

W: building dead weight plus likely live loads

Δ_{roof} : Top roof displacement

Sa : spectral acceleration

Sd : spectral displacement

Figure below shows the participation factor and the modal mass coefficient vary according to the relative story displacement over the height of the building.

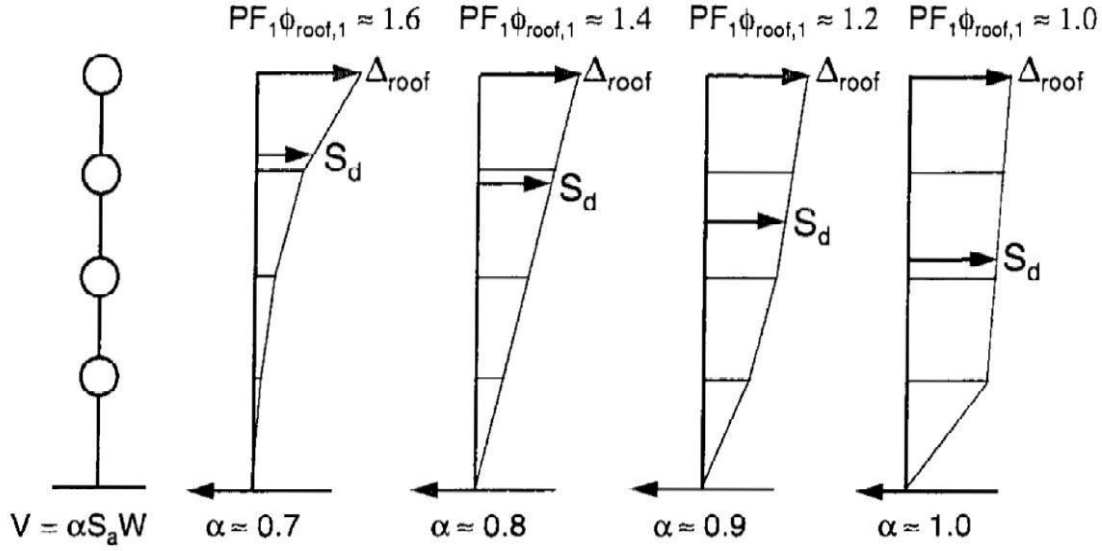

Figure 4.4.2. Example modal participation factors and modal mass coefficients

4.4.2.2 Convert the elastic response (demand) spectrum (5%) from (Sa-T) format into (Sa-Sd) format

The common format used to represent the seismic demand response spectrum is acceleration vs. period, but in this method it is redefined as acceleration vs. displacement response spectrum ADRS. Displacement for SDOF can be calculated using the following equation:

$$Sd = \frac{T^2}{4\pi^2} Sa \qquad \text{....Eq 4.4.7}$$

Where:

Sa: Spectral acceleration (m/s^2)

Sd: Spectral Displacement (m)

T: Period (s)

This process is illustrated in figure (4.4.3)

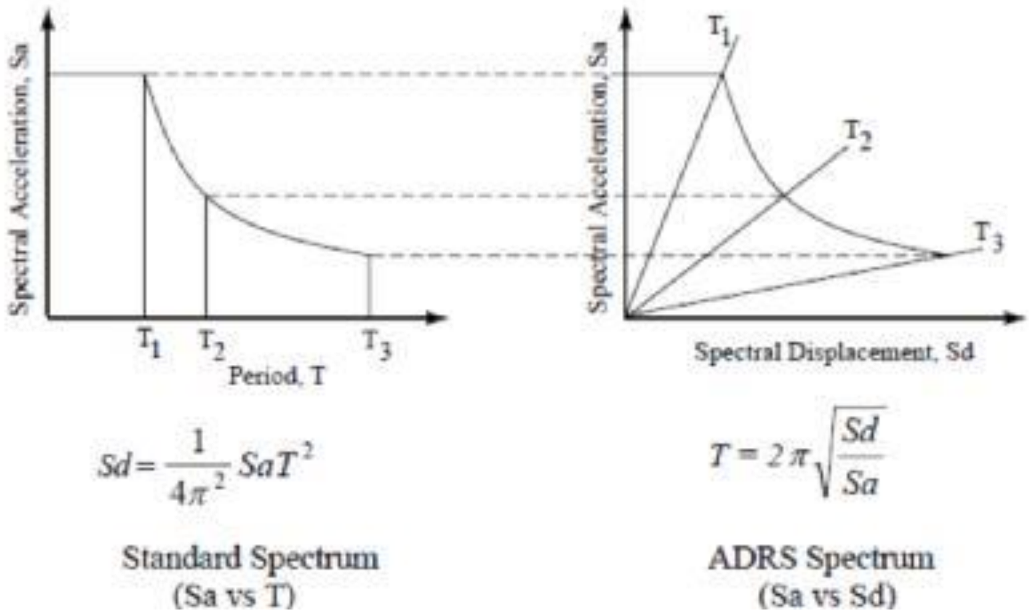

Figure 4.4.3. Convert Sa vs. T for 5% damping into ADRS format

4.4.2.3 Idealizing the capacity curve into bilinear representation

To construct the bilinear representation one line is drawn up from the origin at the initial stiffness of the building. Then a second line is made back from the trial performance point a_{pi}, d_{pi} as shown in figure (4.4.4). After that the second line is sloped and it intersects the first line. At point a_y, d_y, the area designated A1 in the figure is approximately equal to the area designated A2. The importance of the setting area A1 is the same to the area A2 is to have equivalent area under the capacity spectrum and its bilinear representation. That is to have equal energy associated with each curve, see figure (4.4.4). Now, this idealized curve is used in the iterative procedure to find the performance point.

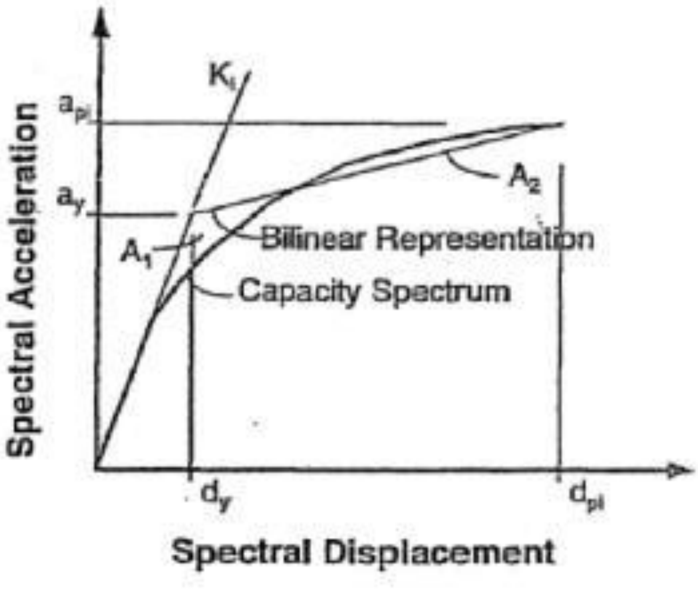

Figure 4.4.4. Bilinear representation of capacity spectrum

4.4.2.4 Reduction of 5% damped response spectrum

Damping of a structure is a combination of a viscous damping and a hysteretic damping. Hysteretic damping is related to the area inside the loops that are formed when the earthquake force (base shear) is plotted against the structure displacement. This method defines an equivalent viscous damping to represent this combination and it can be calculated using (Eq. 4.4.8).

Since existing buildings are not usually ductile, ATC40 introduces the concept of effective viscous damping that can be obtained by multiplying the equivalent damping by a modification factor K using (Eq. 4.4.9).

$$\beta eq = \beta 1 + 5 \quad \text{.......Eq. 4.4.8}$$

$$\beta eff = K\beta 1 + 5 \quad \text{.......Eq. 4.4.9}$$

Where:

βeq : equivalent viscous damping

βeff : effective viscous damping

K : damping modification factor

β1 : hysteretic damping represented as equivalent viscous damping

5 : 5% viscous damping inherent in the structure (assumed constant)

Hysteretic damping β1 can be calculated according to Eq. 4.4.10, [Chopra 2012]

$$\beta 1 = \frac{1}{4\pi} . \frac{E_D}{E_{So}} \qquad \text{.......Eq. 4.4.10}$$

Where:

E_D - energy dissipated by damping

E_{So} - maximum strain energy

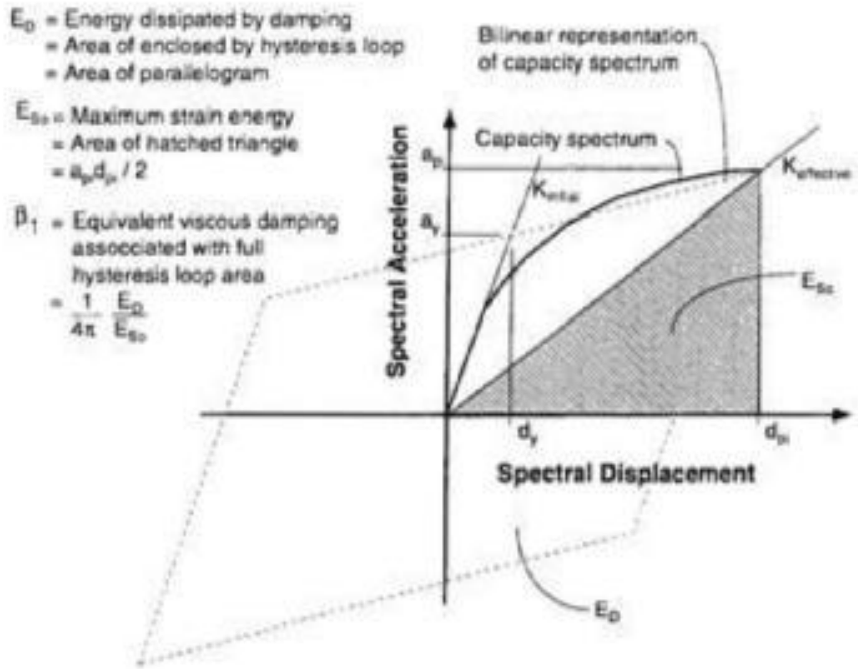

Figure 4.4.5. Hysteresis parallelogram

The physical meaning of both E_D and E_{So} is represented in (Figure 4.4.6). E_D is the energy dissipated by the structure in a single cycle of motion, that is, the area enclosed by a single hysteresis loop. E_{So} is the maximum strain energy associated with that cycle of motion that is the area of the hatched triangle.

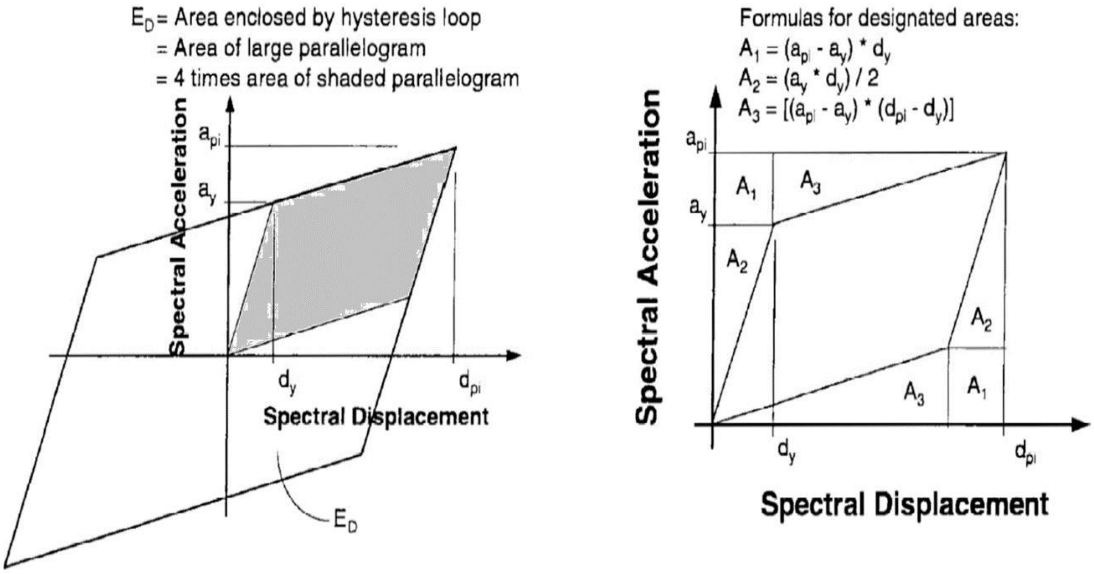

Figure 4.4.6. Derivation of energy dissipated by damping

β1 can be written as following:

$$\beta 1 = \frac{63.7(a_y d_{pi} - a_{pi} d_y)}{a_{pi} d_{pi}} \qquad \text{……..Eq 4.4.11}$$

Where:

a_{pi} , d_{pi} : trial performance point on the bilinear capacity.

a_y , d_y : yield point of the bilinear curve

Therefore,

$$\beta_{eff} = \left(\frac{63.7(a_y d_{pi} - a_{pi} d_y)}{a_{pi} d_{pi}} \right) K + 5 \qquad \text{……..Eq 4.4.12}$$

Factor K measures the extent to which the actual building hysteresis is similar to that illustrated in the figure above, either initially, or after degradation. K-factor depends on the structural behavior of the building, which is related to the quality of seismic resisting system and the duration of ground shaking. ATC40 defines three categories of structural behavior. Type A represents stable, reasonably full hysteresis loops like in figure above; Type B represents a moderate reduction of area; Type C represents poor hysteretic behavior with a significant reduction of loop area (severely pinched). In this research, Type B characterizes the structural behavior of the case study buildings and it represents a moderate reduction in the area of the parallelogram.

Table 4.4.1. Structural behavior types for the quality of seismic resisting system

Shaking Duration	Essentially New Building	Average Existing Building	Poor Existing Building
Short	Type A	Type B	Type C
Long	Type B	Type C	Type C

Values of K are given in following table:

Table 4.4.2. Values for damping modification factor K

Structural Behavior Type	ζ_o (percent)	κ
Type A	≤ 16.25	1.0
	> 16.25	$1.13 - \frac{0.51(Sa_y Sd_i - Sd_y Sa_i)}{(Sa_i Sd_i)}$
Type B	≤ 25	0.67
	> 25	$0.845 - \frac{0.446(Sa_y Sd_i - Sd_y Sa_i)}{(Sa_i Sd_i)}$
Type C	Any value	0.33

4.4.2.5 Factors of spectral reductions

Spectral reduction factors are calculated in the following equations:

$$SR_A = \frac{3.21 - 0.68\ln(\beta eff)}{2.12} = \frac{3.21 - 0.68\ln\left[\frac{63.7k(a_y d_{pi} - d_y a_{pi})}{a_{pi} d_{pi}} + 5\right]}{2.12} \geq 0.44$$

…..Eq 4.4.13

$$SR_V = \frac{2.31 - 0.41\ln(\beta eff)}{1.65} = \frac{2.31 - 0.41\ln\left[\frac{63.7k(a_y d_{pi} - d_y a_{pi})}{a_{pi} d_{pi}} + 5\right]}{1.65} \geq 0.56$$ ..Eq 4.4.14

These values are used on the response spectrum curve (RSC) in order to reduce it. SRV is used for region of constant velocity in the (RSC), and SRA is used for region of constant acceleration in (RSC).

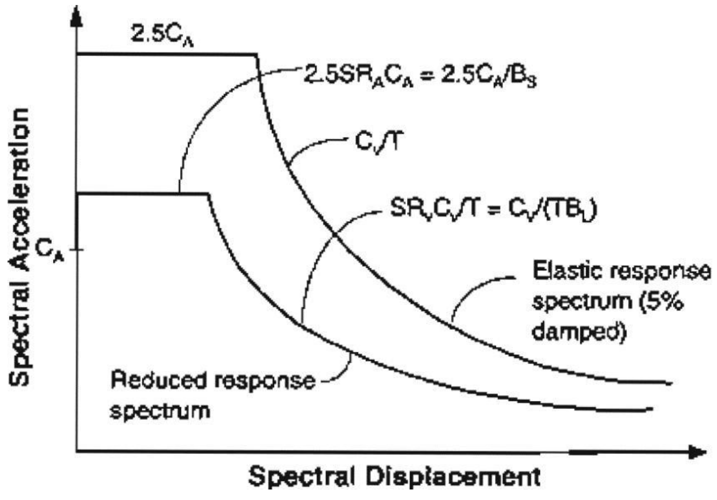

Figure 4.4.7. Reduced response spectrum

4.4.2.6 Point of intersection between demand and capacity curves

In this step, capacity curve is plotted with reduced demand curve. Also a bilinearized capacity trial curve produced in section (4.4.2.3) is plotted with reduced demand curve. The displacement of the intersection points are di from capacity curve and dpi from bilinearized capacity curve.

If the displacement at the intersection between demand and capacity spectra di, is within ±5% of the displacement of the trial performance point, the, a_{pi}, d_{pi} becomes the performance point. If it is not within the acceptable tolerance, then a new point is selected and the process is repeated. Figure (4.4.8) illustrates the procedure. The performance point represents the maximum structural displacement expected for the demand earthquake ground motion.

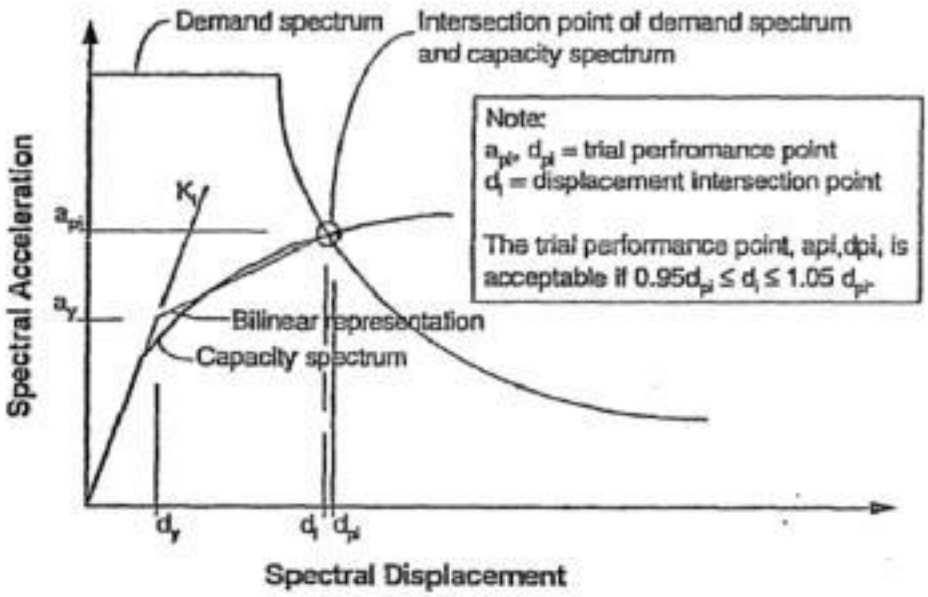

Figure 4.4.8. Performance point (intersection point of demand and capacity spectra)

4.4.2.7 Structure performance point

After the convergence of the values, the intersecting point is converted to global (roof) displacement by multiplying the estimated spectral displacement demand of the equivalent SDOF system with the first modal participation factor at the roof level.

4.5 Modeling pushover analysis

4.5.1 Introduction

In order to do the pushover analysis, the inelastic behavior of the elements must be defined first. The software "SAP2000" captures the plastic behavior through concentrated plasticity approach. In this approach, a single point called plastic hinge wherein all plastic deformation is concentrated represents the inelastic zone in the element. Therefore, the behavior of such plastic hinge must be identified.

4.5.2 Definition of plastic hinges

There are two methods to define the properties of plastic hinges by SAP2000. The manual method and the automatic method. The automatic characteristics are defined according to FEMA 356 tables and Caltrans standards. The automatic method used in this thesis is based on FEMA 356 tables for un-retrofitted members, which are explained in chapter (2), and verified in (Appendix B).

Plastic hinges appear usually at the ends of the beams and columns, because of exceeding the yielding point and the beginning of plastic range where the occurrence of permanent deformation is started. The main cause of the appearance of plastic hinges in the beams is the bending moment. As for the columns, the main cause that leads to plastic hinge is the interaction of the axial force (P) with the moments (M).

- Beam elements:

As mentioned in the previous paragraph, flexure is the main cause of plastic hinge appearance; therefore, automatic flexural hinges will be assigned at the end of the beams.

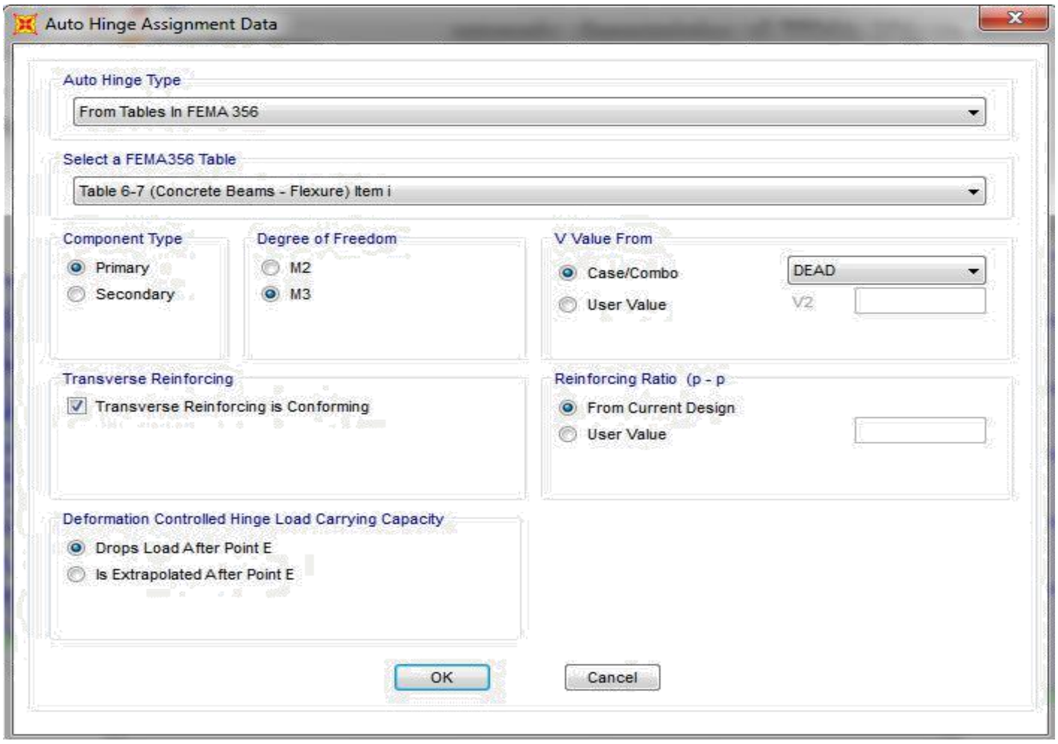

Figure 4.5.1 Assign plastic hinges for beams.

- Columns:

Columns are exposed to axial loads and moments, and these loads are the cause of plastic hinges occurrence. Therefore, automatic (P-M) hinges will be assigned at the columns ends.

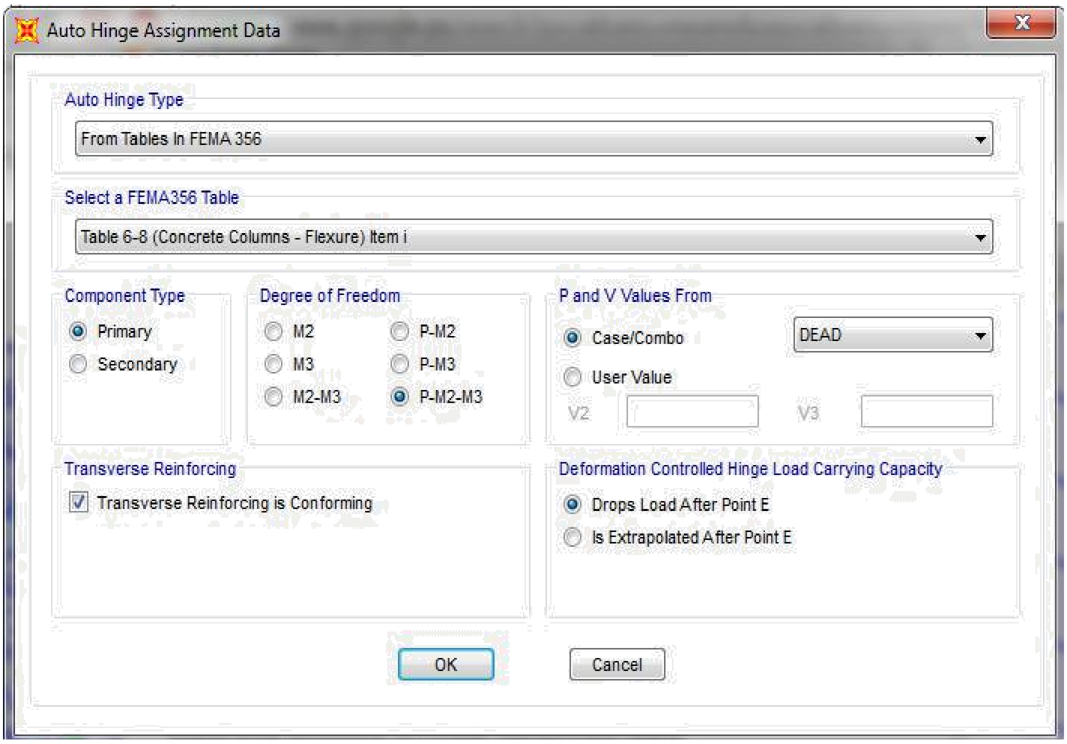

Figure 4.5.2. Assign plastic hinges for columns.

- Bracing:

Bracing members are used to represent the effect of masonry concrete walls, which are considered brittle, because the material used to fill the space between the stone face and the hollow blocks is plain concrete with no reinforcement, except the two small beams under and above the window opining. Therefore, hinges were defined manually as axial brittle hinge. Then assign the plastic hinges for the bracing elements system at the end of bracing members.

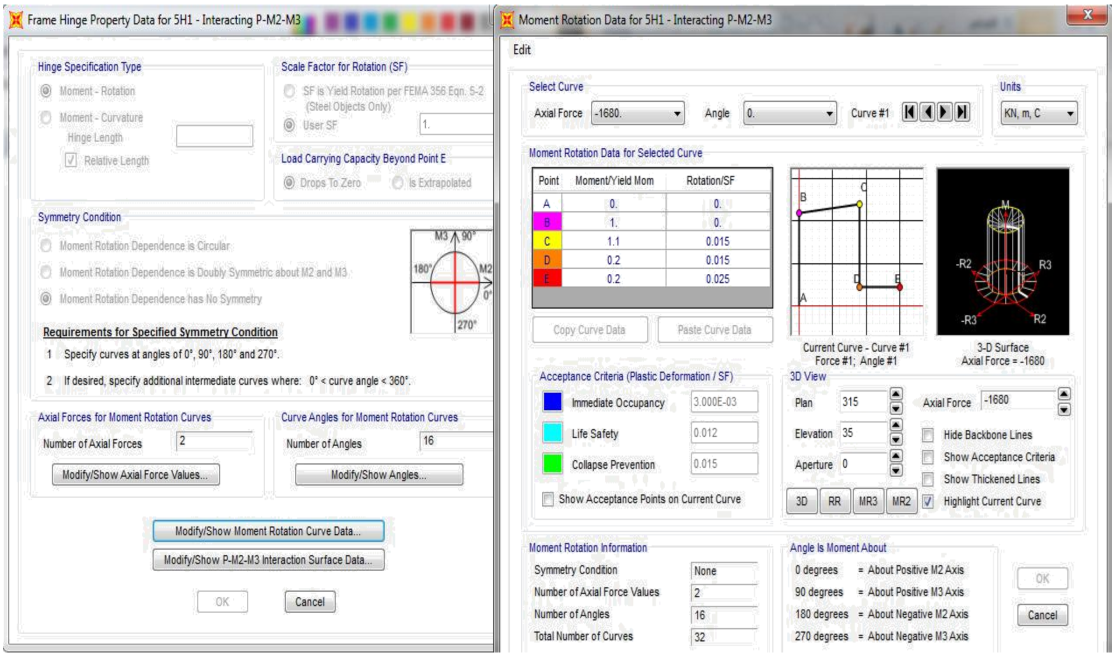

Figure 4.5.3. Generated properties by FEMA356 criteria of column sec.

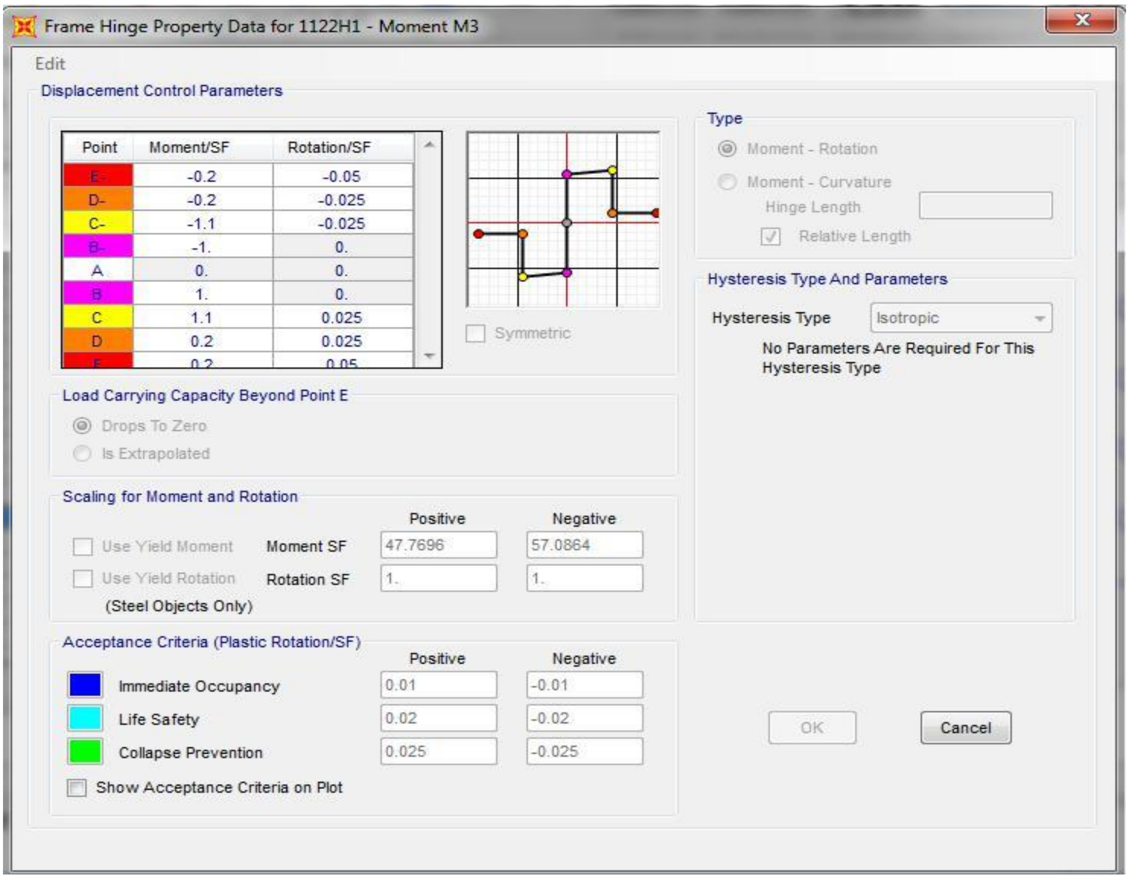

Figure 4.5.4. Generated properties by FEMA356 criteria of beam sec.

4.5.3 Loads

4.5.3.1 Introduction

The loads affecting the structure are divided into gravity loads and lateral loads, and these loads are transmitted through structural elements to the ground. Gravity loads are mainly transmitted by the slabs, to beams and shear walls, then through columns and shear walls to footings and finally to ground. Beams are exposed to flexure, shear, and torsion while transmitting loads to vertical components (columns and shear walls).

Each type of loading needs to be designed for, in order to create safe load paths to ground.

4.5.3.2 Defining initial load conditions for pushover analysis

The gravity loads must be accounted for as initial loading condition before the seismic loads start to take effect on the structure. The gravity loads include the dead loads (structure weight and super imposed loads) and live loads. [IBC 2012]

The gravity loads are defined as nonlinear load case and the full gravity loads are applied during this stage, except live load, where only 25% participation ratio is assumed.

4.5.3.3 Lateral load patterns

ATC-40 method proposes that pushover analyses is performed by using lateral load patterns related to fundamental mode shapes, in order to be as close as possible to the expected earthquake load distribution. The tables (4.5.1 and 4.5.2) below display the modal shape vectors for the two translational fundamental modes per each model scenario, and normalized by the top floor drift.

Table 4.5.1. Normalization of fundamental mode shape vectors for group (A) models

Group (A)												
	D-NA-B model				ND-A-B model				ND-NA-B model			
	Modal shape X-dir.		Modal shape Y-dir.		Modal shape X-dir.		Modal shape Y-dir.		Modal shape X-dir.		Modal shape Y-dir.	
floor	Drift	Norm.	Drift	Norm.	Drift	Norm.	Drift	Norm.	Drift	Norm.	Drift	Norm.
6	0.023	1.00	0.0229	1.00	0.0234	1.00	0.0232	1.00	0.0247	1.00	0.0241	1.00
5	0.0219	0.95	0.0218	0.95	0.0224	0.96	0.0221	0.95	0.0234	0.95	0.0226	0.94
4	0.0202	0.88	0.0201	0.88	0.0207	0.88	0.0206	0.89	0.0216	0.87	0.0203	0.84
3	0.0179	0.78	0.0179	0.78	0.0185	0.79	0.0185	0.80	0.0187	0.76	0.017	0.71
2	0.0152	0.66	0.0153	0.67	0.0157	0.67	0.0159	0.69	0.014	0.57	0.0124	0.51
1	0.0109	0.47	0.0115	0.50	0.0111	0.47	0.0117	0.50	0.0075	0.30	0.0067	0.28
0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00

Table 4.5.2. Normalization of fundamental mode shape vectors for group (B) models

	D-NA-NB model				Group (B) ND-A-NB model				ND-NA-NB model			
	Modal shape		Modal shape		Modal shape		Modal shape		Modal shape		Modal shape	
	X-dir.		Y-dir.		X-dir.		Y-dir.		X-dir.		Y-dir.	
floor	Drift	Norm	Drift	Norm.	Drift	Norm	Drift	Norm	Drift	Norm.	Drift	Norm
6	0.0272	1.00	0.0264	1.00	0.0276	1.00	0.0269	1.00	0.0272	1.00	0.0252	1.00
5	0.0242	0.89	0.024	0.91	0.0249	0.90	0.0246	0.91	0.024	0.88	0.0228	0.90
4	0.0201	0.74	0.0206	0.78	0.0211	0.76	0.0213	0.79	0.0199	0.73	0.0194	0.77
3	0.0151	0.56	0.0161	0.61	0.0162	0.59	0.0168	0.62	0.015	0.55	0.0152	0.60
2	0.0096	0.35	0.0109	0.41	0.0105	0.38	0.0115	0.43	0.0086	0.32	0.0102	0.40
1	0.0043	0.16	0.0054	0.20	0.0049	0.18	0.0059	0.22	0.004	0.15	0.0051	0.20
0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00

The lateral load patterns that will be used for pushover analysis are defined in SAP2000 as shown in Figure (4.5.5).

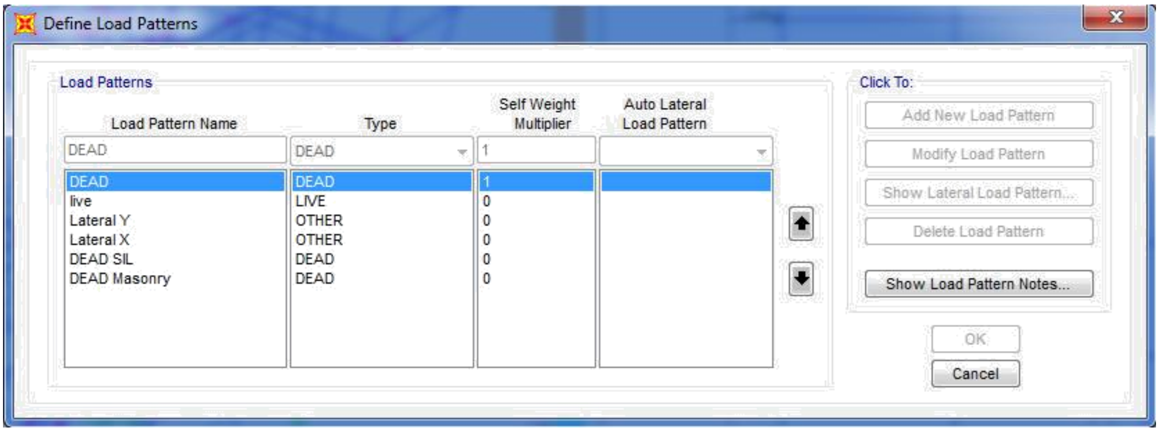

Figure 4.5.5. Load pattern for lateral loads.

The next step is to assign lateral loads on the models. The loads are assigned on each floor according to the modal shapes at the joints connecting beams with columns. Such that the center of the loads match with center of mass for each floor.

4.5.4 Define load cases for pushover

The lateral loads assigned in the previous step are to be defined in a pushover load case, and this pushover load case should continue after initial load case, which is (DL nonlinear).

The analysis in pushover is displacement controlled with monitoring the roof floor movement. The maximum allowable displacement of the master node is assigned 1m, where the analysis stops.

4.6 Results of pushover analysis

4.6.1 Introduction

In this section, the results of the pushover analysis will be presented for the proposed models of the case study building. The results will be in terms of base shear force versus the top displacement of the building. These results represent the behavior of the building under the influence of lateral forces within the assumptions that were explained earlier. The results can be used to show weaknesses and collapse sequence in the building through the shape of the curve and distribution of resulting plastic hinges and the amount of displacement. This information is used to assess the performance of the structure according to performance levels as suggested in the ASCE and FEMA regulations.

4.6.2 Base shear vs. top displacement

Base shear force and top displacement (V-D) was taken as a result of the pushover analysis using SAP2000 program for the six models. The resulting curves are presented in two groups: group (A) and group (B), in both X and Y directions.

The reduction in slope of the curve indicates the degradation of stiffness due to the appearance of new plastic hinges, and this continues until the slope becomes zero and then comes negative slope, which indicates the beginning of collapse.

Unfortunately, when the stiffness of the model approaches zero, it may cause divergence problem in the program algorithm. Generally, this makes the program fail in drawing the descending curve after collapse for some cases.

The following figures (4.6.1, 4.6.2, 4.6.3, 4.6.4) show the (V-D) in both directions X & Y for each group of models. The performance points for each model are also shown in the figures, and these points are automatically calculated using SAP2000 according to ATC-40 iterative procedure.

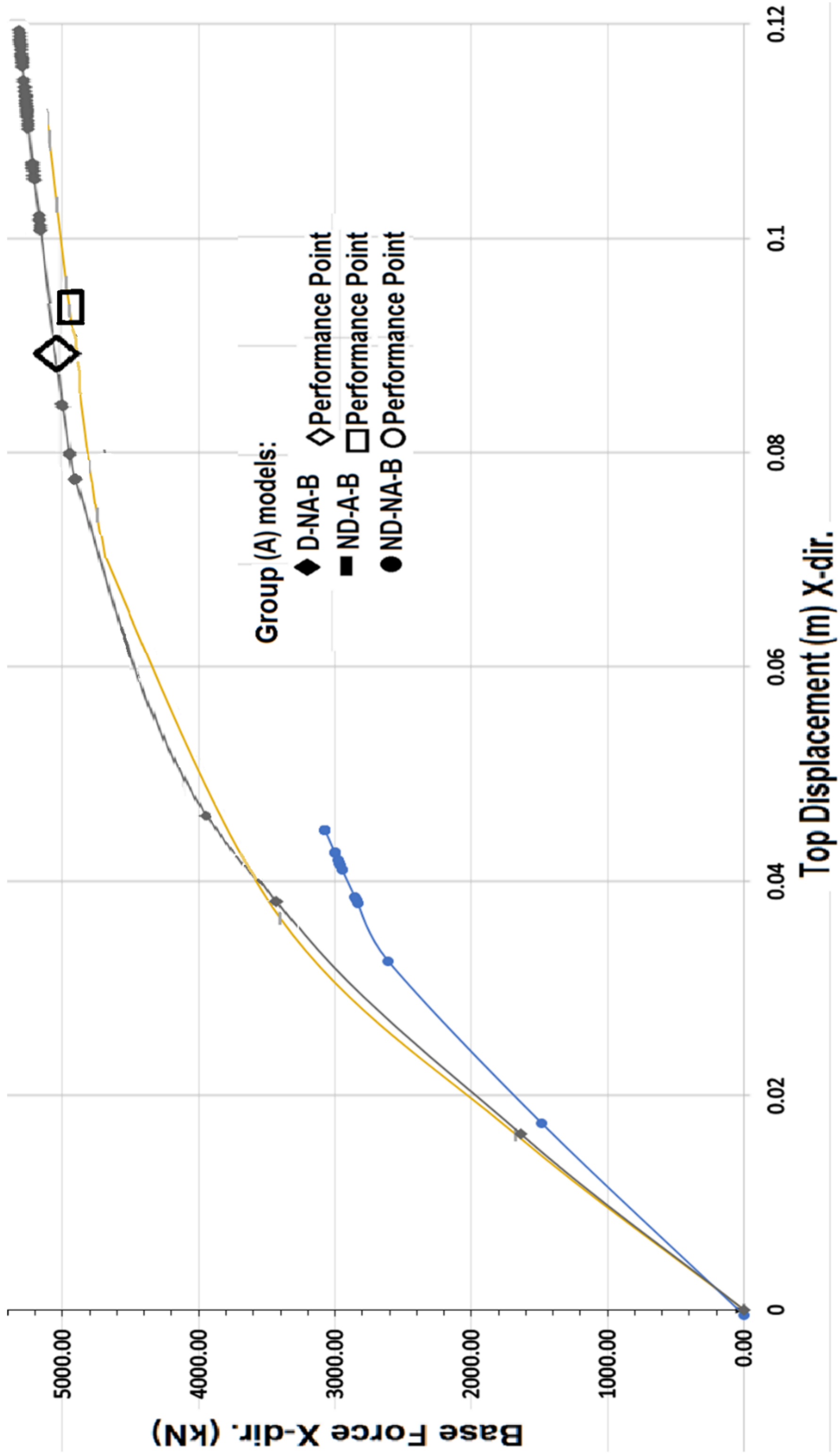

Figure 4.6.1. Group (A) models pushover curves in terms (V-D), X-dir.

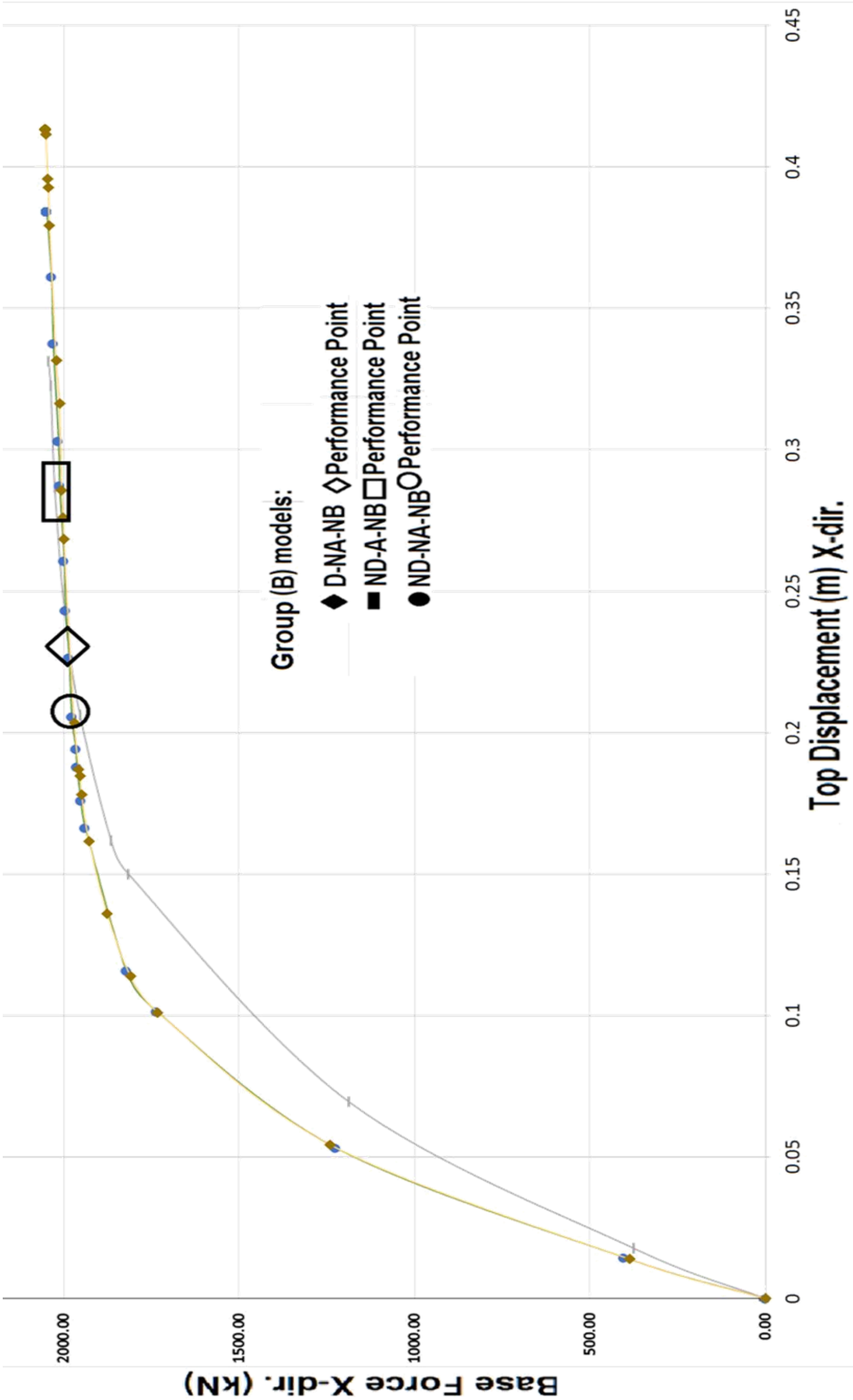

Figure 4.6.2. Group (B) models pushover curves in terms (V-D), X-dir.

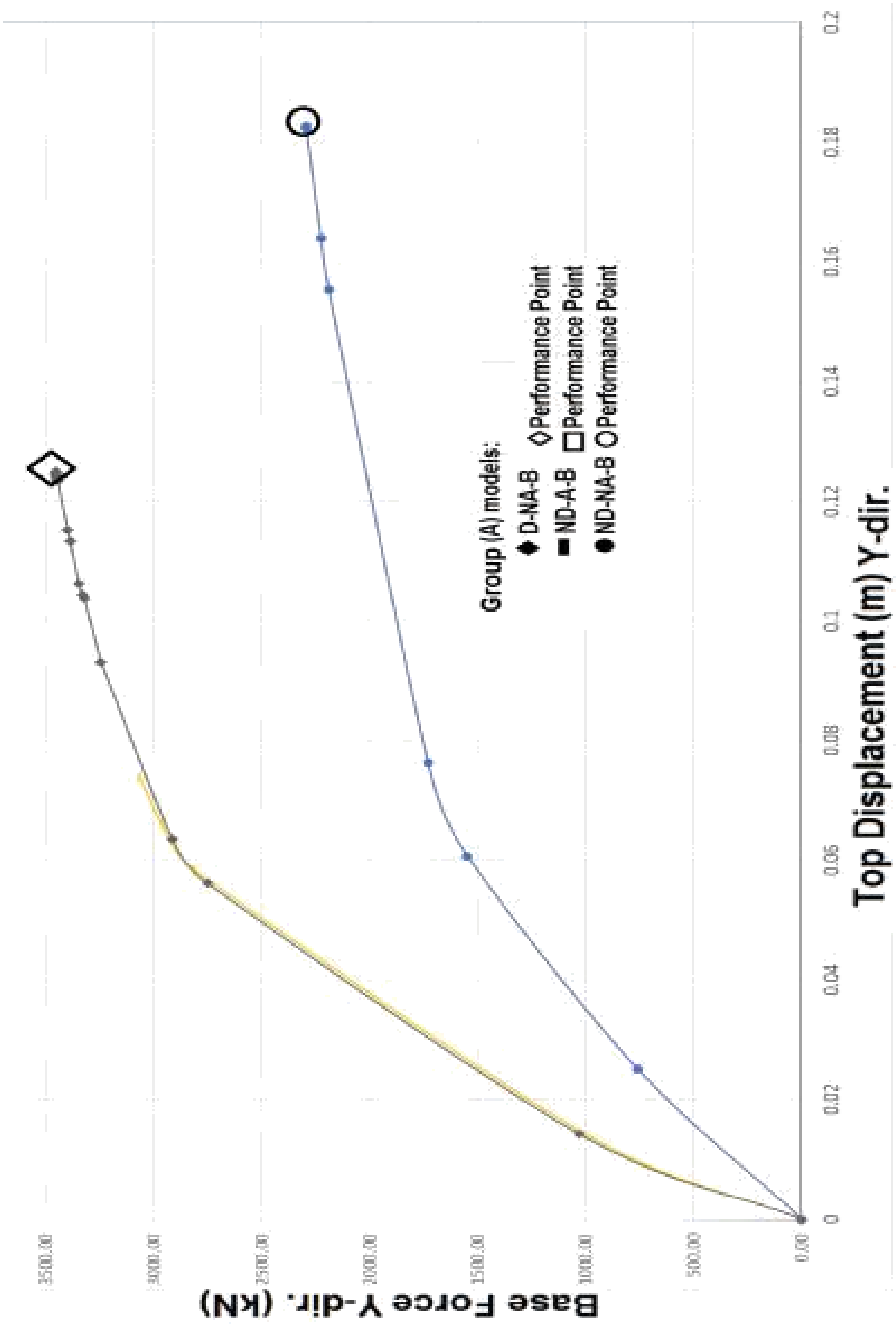

Figure 4.6.3. Group (A) models pushover curves in terms (V-D), Y-dir.

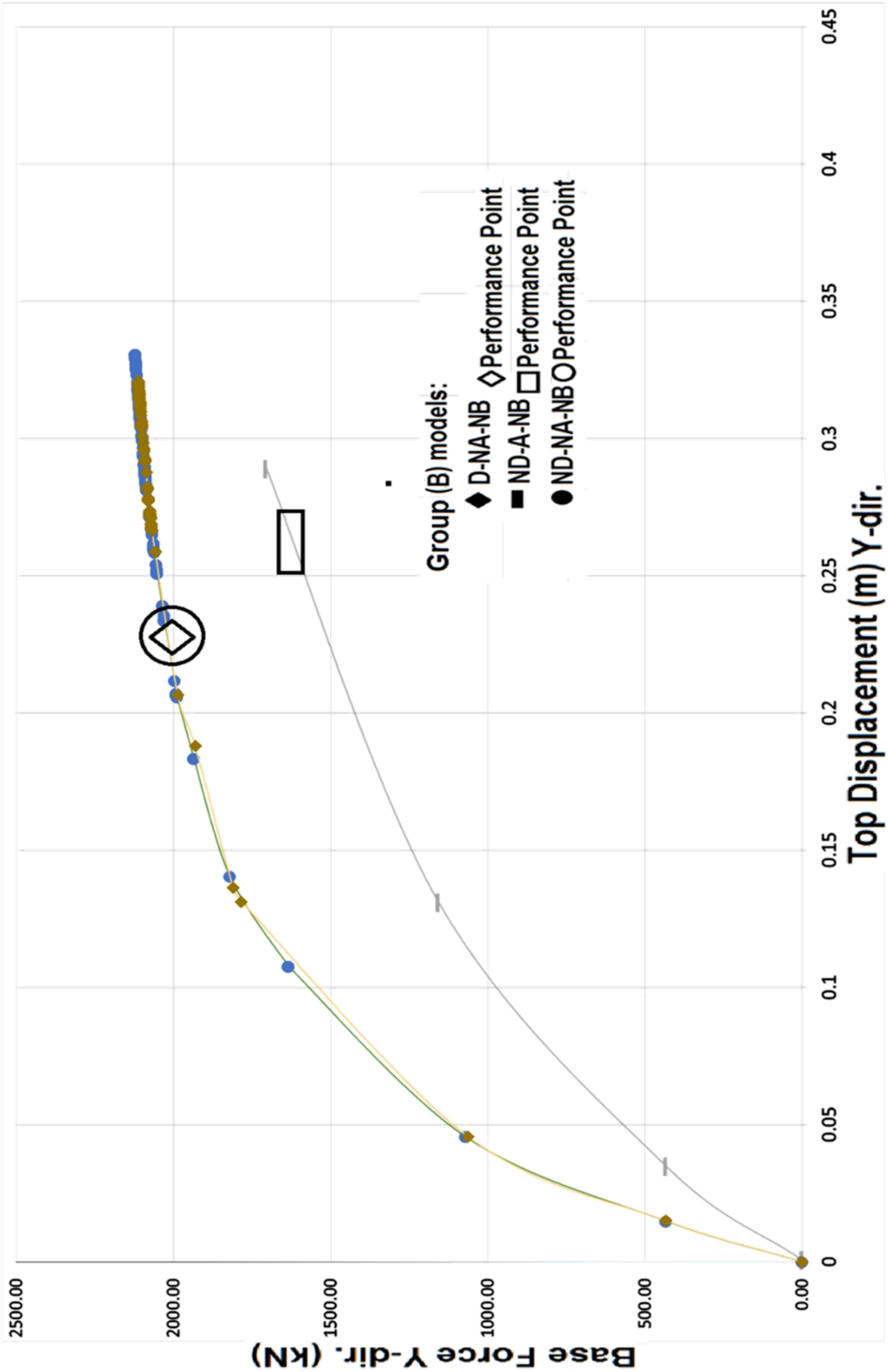

Figure 4.6.4. Group (B) models pushover curves in terms (V-D), Y-dir.

Based on the capacity curves shown in figures (4.6.1, 4.6.2, 4.6.3, and 4.6.4), the following points can be noticed:

1- As expected, using bracing in group (A) leads to increasing the stiffness and strength of the building.
2- When using frames without bracing system, the ductility increases. On the other hand, the stiffness and strength decrease severely.
3- When using bracing, the models D-NA-B and ND-A-B seem to produce the same inelastic behavior. That is, whether the area slabs are modeled explicitly, or removed, but the diaphragm constraints are applied, the behavior does not get affected by such assumptions.
4- It can be noticed that the difference between the curve based on slab area sections and the curve based on diaphragm assumption is not more than 10%. The two curves reach the same drift when collapse hinges appear.
5- In this case study or similar cases, the Diaphragm assumption is a an effective way to replace the area sections in ceilings in order to reduce the degrees of freedom while maintaining the same effect of area sections on stiffness and strength of floors.
6- For the previous reasons, the diaphragm assumption will be effectively used for studying the structure and for representing the effect of slab area sections. Therefore, D-NA-B model will be used to represent the case study building.
7- Reducing the degrees of freedom makes pushover more effective in terms of time and convergence with sufficient accuracy of analysis.

8- Plastic hinges start to spread in the columns of the ground floor, which indicates soft story problem in all models.

9- In group (A) curves, collapse hinges were located on the columns in the ground floor at the performance point. This means that the structure is expected to fail in earthquakes by forming these plastic hinges.

10- In group (B) curves, IO hinges were located all over the columns and beams in all floors at the performance point. This means that structure may sustain earthquakes with the same magnitude.

For the previous reasons, the diaphragm assumption will be effectively used for studying the structure and for representing the effect of slab area sections. Therefore, D-NA-B model will be used to represent the case study building.

4.6.3 Performance point of (D-NA-B) model

In this section, the behavior and performance of the structure at the instance of performance point will be discussed.

X-direction

The analysis stopped after 110 steps due to convergence difficulty. The performance point is located at the last step 110, with spectral acceleration (Sa) equals to 0.171g and spectral displacement (Sd) equals to 0.081m.

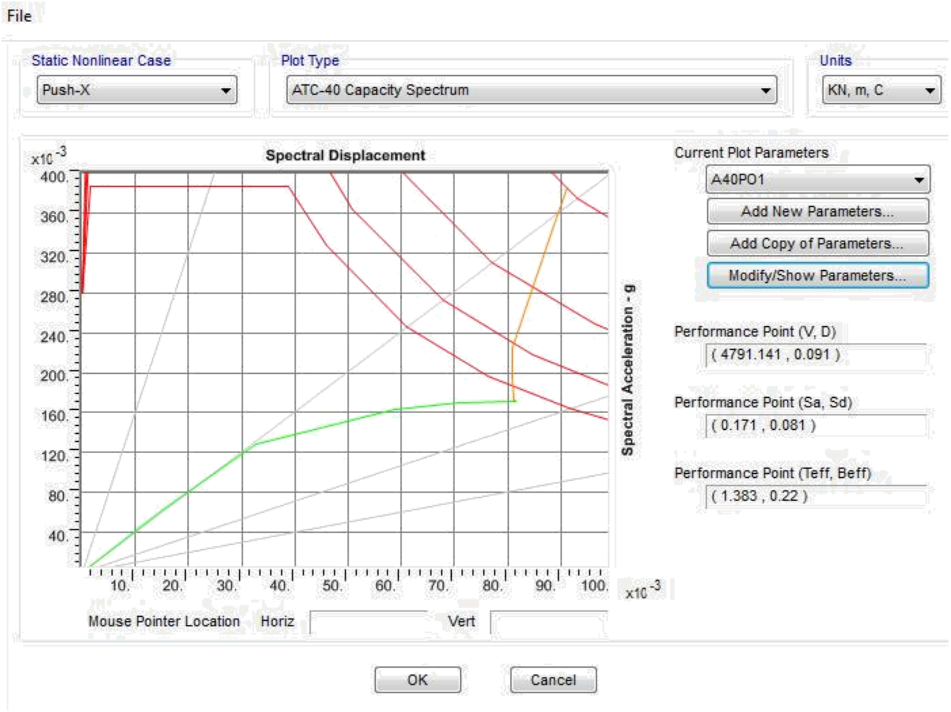

Figure 4.6.5. Pushover curve in terms (Sa-Sd), X-dir.

The following two tables display detailed data for the step of the performance point.

Table 4.6.1. Capacity and demand curves data at the performance point for [X-direction]

X-direction								
Capacity curve			Capacity and demand curve					
step	**diplacement (m)**	**Base force KN**	**Effective period Teff**	**Effective damping ratio Beff**	**Sd Capacity (m)**	Sa Capacity	**Sd Demand (m)**	**Sa Demand**
109	0.091	4791.14	1.3816	0.2197	0.0816	0.1712	0.0814	0.1709
110			1.3817	0.2198	0.0816	0.1712	0.0814	0.1709

Table 4.6.2. Base shear and top displacement of studied building and number of each P.H. type at the performance point for [X-direction]

Step	Disp. (m)	Base Force kN	B to IO	IO to LS	LS to CP	CP to C	C to D	D to E	Beyond E
109	0.091	4791.14	42	54	0	0	0	0	0
110			2	54	0	0	0	0	0

The figure (4.6.6) displays the distribution of P.H. at the performance point of the structure.

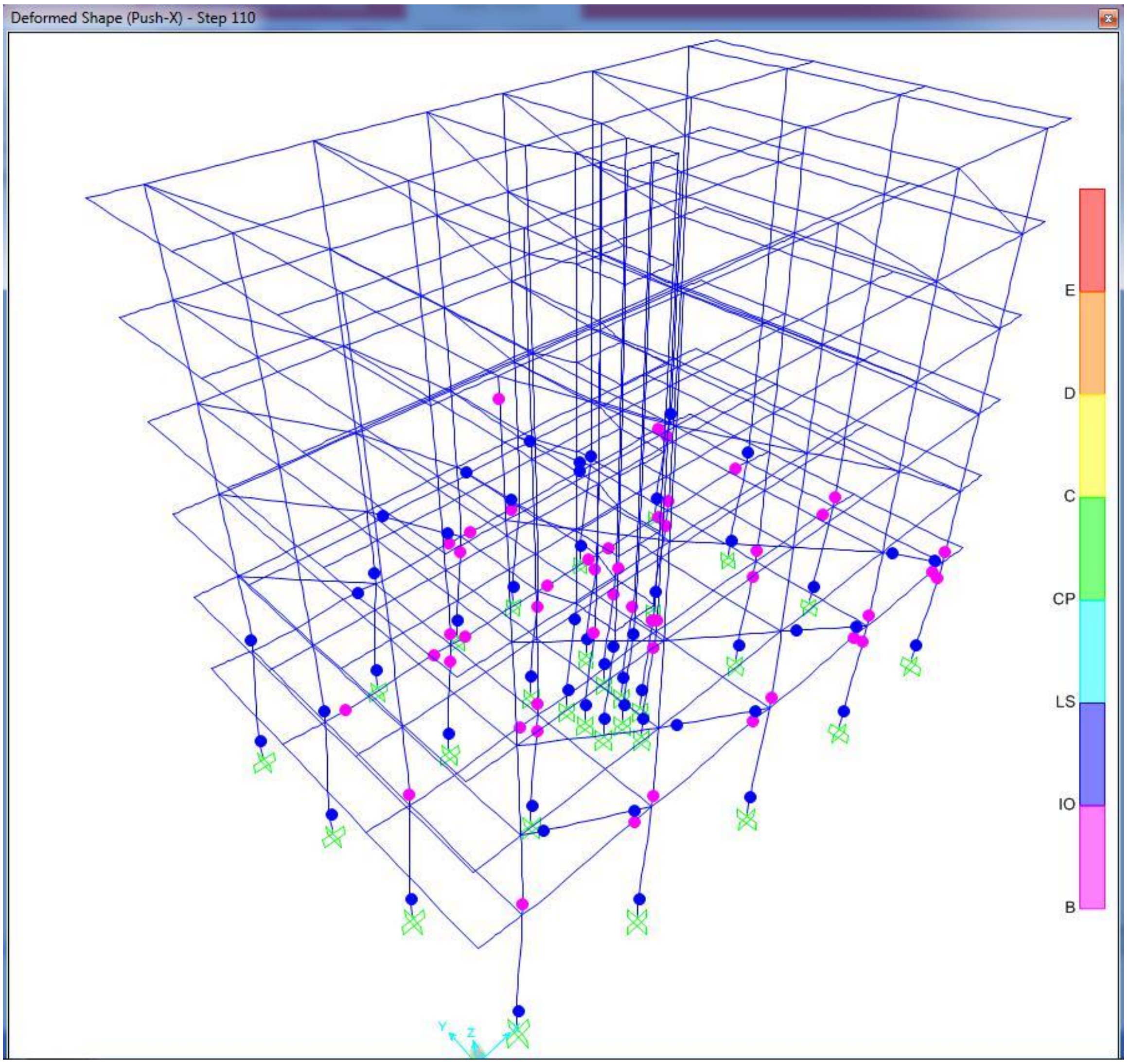

Figure 4.6.6. Distribution of P.H. at the performance point, [X-dir].

Y-direction

The analysis stopped after 428 steps due to convergence. The performance point is located at the last step 428, with spectral acceleration (Sa) equals to 0.123g and spectral displacement (Sd) equals to 0.115m..

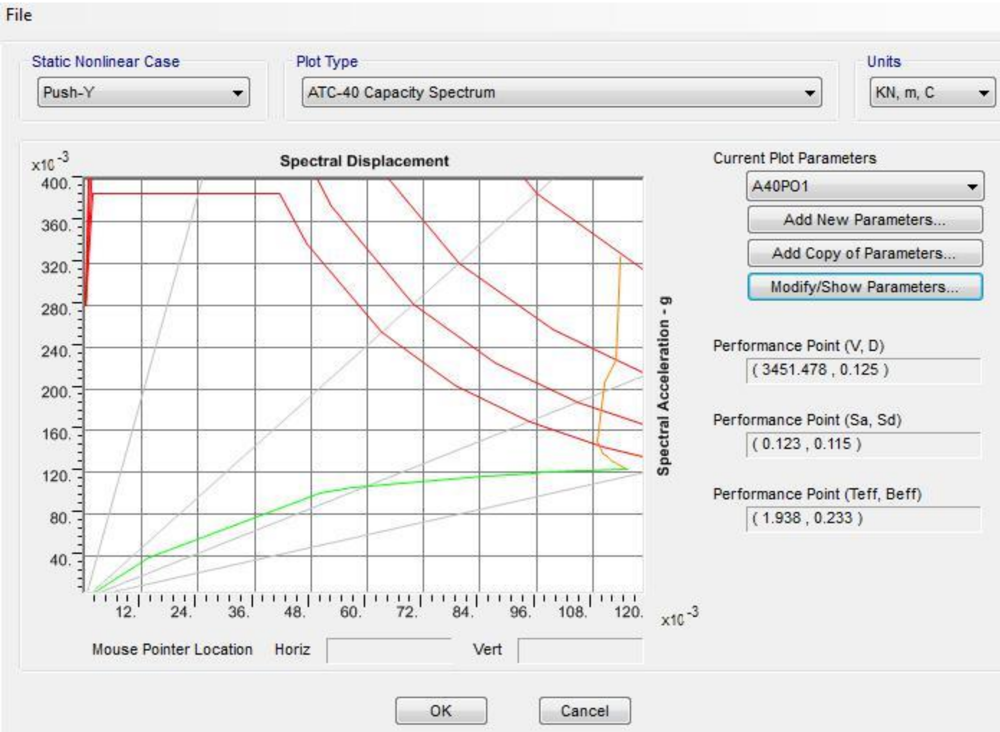

Figure 4.6.7. Pushover curve in terms (Sa-Sd), Y-dir.

Following tables display detailed data for the last two steps.

Table 4.6.3. Capacity and demand curves data at the performance point for [Y-direction]

Y-direction								
Capacity curve			Capacity and demand curve					
step	displacement (m)	Base force KN	**Effective period Teff**	**Effective damping ratio Beff**	Sd Capacity (m)	Sa Capacity	Sd Demand (m)	Sa Demand
427	0.125	3451.5	1.940361	0.233674	0.115326	0.123311	0.115079	0.123047
428			1.940361	0.233674	0.115326	0.123311	0.115079	0.123047

Table 4.6.4. Base shear and top displacement of studied building and number of each P.H. type at the performance point for Y-direction

Step	Disp. (m)	Base Force kN	B to IO	IO to LS	LS to CP	CP to C	C to D	D to E	Beyond E
428	0.125	3451.5	20	50	21	0	0	0	0

The figure (4.6.8) displays the distribution of P.H. at the performance point of the structure.

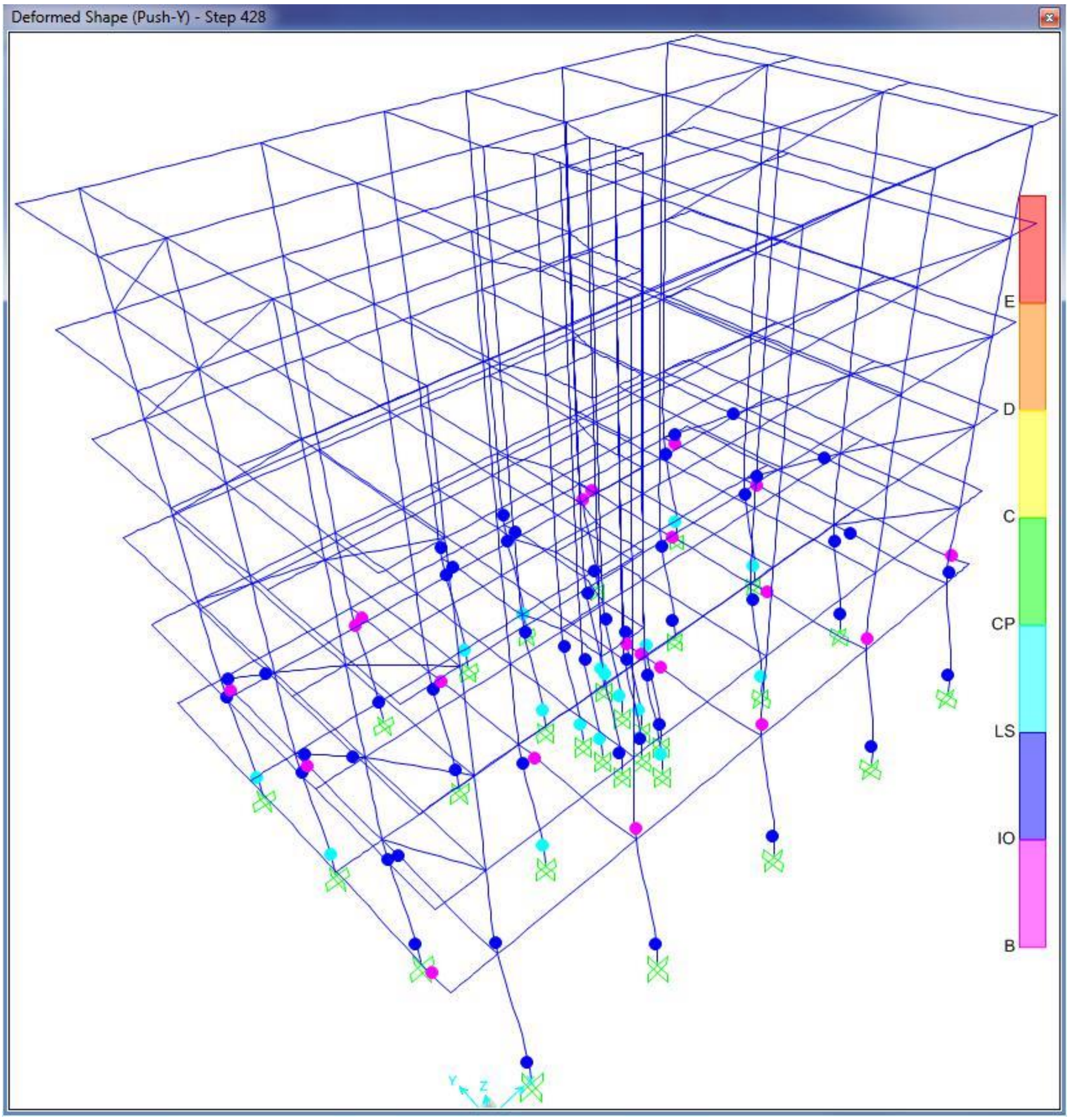

Figure 4.6.8. State of the last step of the structure, Y-dir.

4.7 Assessment of the Un-retrofitted Case Study Building

The structure has the following weaknesses that can be summarized as follows:

1- Vertical irregularity in both directions X & Y:

   According to the checks of ASCE 7-10 that is done in (4.2), there is vertical irregularity in the ground floor, where the columns are 4.5m height, and the columns in rest floors are 3m height. This difference reduces the stiffness of the ground floor with respect to the rest floors. In addition, the bracing system in the upper floors increases the stiffness significantly.

2- Torsional mode effect:

   According to ASCE 7-10 there is no significant torsional effect, but the modal analysis shows that the torsional mode is one of the three fundamental modes. This is not a desired mode because the torsion produces very large shear forces and flexural moments especially on columns far from the center of rigidity, and such interaction is difficult to include in pushover analysis.

The performance level desired for the case study building is life safety (LS) level according to the definition of FEMA 356 standards that mentioned in (2.4). This decision specify the global target performance of structure.

The transient drift limit recommended by FEMA 356 for frame system at life safety limit is 2% of height and for concrete walls 1%. For dual system it is approximated to 1.5%, therefore allowable drift can be considered

$$1.5\% * 21m = 0.32m$$

The un-retrofitted building satisfies the limit in terms of drift, but with the formation of collapse hinges. Therefore, the performance of the building is not satisfactory.

For the resultant pushover curve in the X-direction, performance point was located at the curve, and all the hinges in the structure were in the range of IO-LS. This means that the global performance level of the structure can be considered life safety level in X-dir.

For the resultant pushover curve in the Y-direction, performance point was located at the end of the curve, and appearance of collapse hinges at the bottom of columns on the ground floor columns. The rest of hinges ranging from (IO) to (LS) plastic hinges. These hinges do not mean collapse.

The performance of the structure in Y-direction must be improved from collapse state to life safety state.

In the next chapter, retrofitting techniques will be displayed, in terms of level of using, availability, and the way of using them. Then applying some of these techniques to the case study model in order to improve the performance of the building in its weakest direction.

CH 5 Retrofitting of the case study building

5.1 Introduction

Existing buildings in seismic zones, which do not meet the seismic design requirements, are more vulnerable to destruction or exposure to serious damage. In order to limit their vulnerability, existing buildings must be evaluated and rehabilitated or strengthened to increase their resistive capacity to the seismic forces. To satisfy the required level of strengthening for such vulnerable buildings, the actual capacity of these buildings must be examined and its weak elements must be identified.

The objectives of strengthening of existing buildings are to mitigate seismic risks, which range from preventing damage to structural elements and non-structural elements to avoiding complete collapse of the building. For certain types of structures the goal rises up to keep the building at full readiness for use after being subjected to the seismic forces and have negligible damage in some non-structural elements. This enables residents to use the building after the earthquake ends naturally. FEMA 356 illustrates these ranges of performance levels in the Table (5.1.1).

Table 5.1.1. Damage Control and Building Performance Levels [FEMA 356, 2000]

Table C1-2 Damage Control and Building Performance Levels

	Target Building Performance Levels			
	Collapse Prevention Level (5-E)	**Life Safety Level (3-C)**	**Immediate Occupancy Level (1-B)**	**Operational Level (1-A)**
Overall Damage	Severe	Moderate	Light	Very Light
General	Little residual stiffness and strength, but load-bearing columns and walls function. Large permanent drifts. Some exits blocked. Infills and unbraced parapets failed or at incipient failure. Building is near collapse.	Some residual strength and stiffness left in all stories. Gravity-load-bearing elements function. No out-of-plane failure of walls or tipping of parapets. Some permanent drift. Damage to partitions. Building may be beyond economical repair.	No permanent drift. Structure substantially retains original strength and stiffness. Minor cracking of facades, partitions, and ceilings as well as structural elements. Elevators can be restarted. Fire protection operable.	No permanent drift. Structure substantially retains original strength and stiffness. Minor cracking of facades, partitions, and ceilings as well as structural elements. All systems important to normal operation are functional.
Nonstructural components	Extensive damage.	Falling hazards mitigated but many architectural, mechanical, and electrical systems are damaged.	Equipment and contents are generally secure, but may not operate due to mechanical failure or lack of utilities.	Negligible damage occurs. Power and other utilities are available, possibly from standby sources.
Comparison with performance intended for buildings designed under the *NEHRP Provisions,* for the Design Earthquake	Significantly more damage and greater risk.	Somewhat more damage and slightly higher risk.	Less damage and lower risk.	Much less damage and lower risk.

One of the most common problems in many of the existing buildings in Palestine is the vertical irregularity, which tends to affect the ground floor. The reason behind this problem is the need for parking, stores, markets, or open halls in the ground floor. It may also appear in one of the upper floors of buildings. The following figures (5.1.1 and 5.1.2) are examples of possible vertical irregularity:

Figure 5.1.1. Possible soft floor formations [SASPARM project (2), 2014]

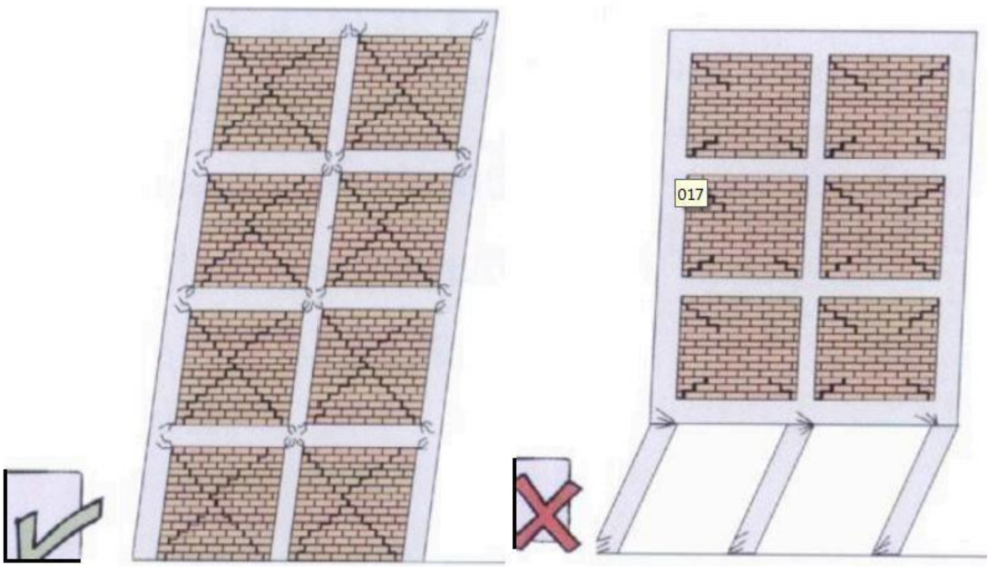

Figure 5.1.2. Typical vertical regularity vs. vertical irregularity [SASPARM project (2), 2014]

Figures (5.1.1) shows a building with short column in the ground floor that has a possible vertical irregularity caused by external masonry walls on the above floors. The second figure (5.1.2) also shows a view for two buildings with vertical regularity vs. vertical irregularity. In addition, figure (3.1.1) which is a photo taken in Nablus city shows at least six building that have possible vertical irregularity.

5.2 Retrofitting concept

The main goal for retrofitting is to improve the performance of existing buildings to resist expected seismic loads. The concept to achieve this goal is to make the structural elements work altogether in a safe and smooth sequence, and all of them contribute in transmitting loads to ground.

In the case of soft story, the soft story is considered a weak link of the chain that may lead to collapse, especially on the ground floor, where maximum forces are affecting it.

Therefore, any proposed retrofitting technique must respect the uniform distribution of stiffness in the structural elements. In addition, retrofitting may change the properties of the building, which may change its behavior, not only in the wanted direction but also in other directions. Therefore, retrofitting is an integrated process that must be studied in all aspects that affect the building, even if it is not related directly to the wanted direction.

When proposing any retrofitting technique, some points should be considered:

1- The way of implementing the retrofitting technique in order to give full efficiency.

2- Vertical and horizontal irregularities, and p-delta effect must be checked with the presence of proposed retrofitting technique.

3- Stiffer retrofitted floor does not mean safer building. Therefore, any changes in stiffness must be studied carefully.
4- Proposed retrofitting technique may create new fundamental modes that was not counted before retrofitting, such as torsional modes and vertical modes in case of long distances between columns relatively with slab and beam depth.
5- Choose the suitable location that can help fix deficits at the same time, in order to keep interventions at minimum.

5.3 Retrofitting techniques

5.3.1 General

As mentioned in section (2.5), the goal of retrofitting methods is to improve the performance of the existing structures against earthquake motions. Existing buildings may need strengthening in the following conditions: [Nikita et al, 2013]

1- Buildings that have not been designed and detailed to resist seismic forces.
2- Buildings that might have been designed for seismic forces, but as per old seismic codes.
3- The lateral strength of the building does not satisfy the seismic forces as per the revised seismic zones or designed base shear.
4- Construction is apparently of poor quality.
5- There have been additions to the building, which increased its vulnerability.

5.3.2 Available retrofitting techniques

5.3.2.1 Introduction

Availability of appropriate retrofitting techniques and its materials in the country should be considered when choosing a technique. It is important issue to make sure that there are specialist technicians and experts that know the details for installing each technique. Therefore, when proposing any retrofitting technique, these conditions must be taken into consideration.

Here are some retrofitting techniques, which can be feasible in Palestine. Crew of technicians can be easily trained to implement these techniques under the presence of the engineering supervision. Sometimes, necessary tests must be done immediately when and where needed to ensure proper installation of the retrofitting technique.

In the following sections, retrofitting techniques are presented briefly, and then the modeling features are discussed. Finally, the outcomes of selected retrofitting techniques are displayed and discussed in terms of their effect on performance.

5.3.2.2 Global level retrofitting techniques (Concrete Frame, Steel Bracing, and Shear wall)

These techniques include adding concrete frames, steel bracings, and shear walls to the structure. The addition of these global techniques is a common seismic retrofitting technique for reinforced concrete frame structures. These techniques increase both the stiffness and the strength of the structure.

Because lateral stiffness has the major effect, the proposed techniques will be checked by stiffness ratios according to ASCE 7-10. These global techniques can be used to solve lateral resistance or bearing gravity loads deficits. These techniques can affect directly more than one structural element at the same time because they connect these elements in a stronger and stiffer bond, which affects the overall behavior.

5.3.2.3 Local level

The following techniques can be applied either to single or to group of structural elements

5.3.2.3.1 Concrete jacketing

Jacketing technique means that the structural element is encased by reinforced concrete materials or other materials, in order to increase the strength in the first place, and as a result the stiffness and ductility maybe increased. It is commonly used for strengthening of reinforced concrete columns. There are many jacketing types other than concrete, these types include steel jacket and fiber reinforced polymer FRP composite jacket.

Although this method is considered destructive method, but it is viable and effective. There are many experiments and studies done about limits and conditions for using this method, such as Indian code. [Nikita et al, 2013]

A research done by Nikita et al (2013) talks about the procedure of providing concrete jacketing to the column. In their research, the overall performance of the column has been significantly improved after jacketing. [Nikita et al, 2013]

For the installation method, the following figure (5.3.1) shows how to connect the new reinforcement with the old one in a typical concrete column section.

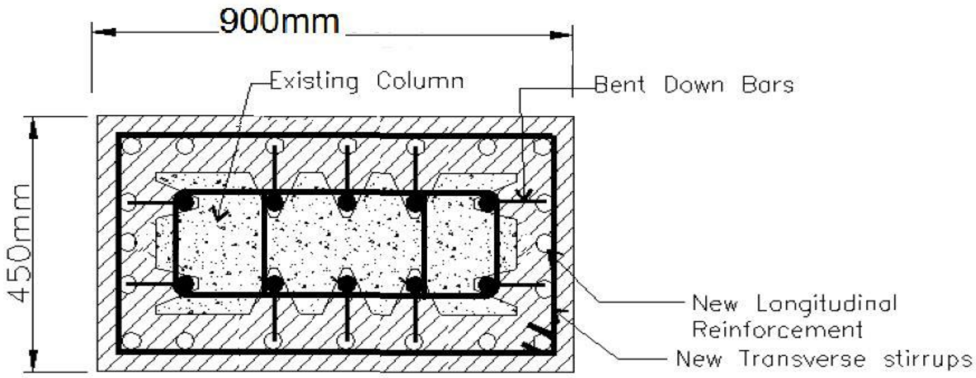

Figure 5.3.1. Typical retrofitted column section showing Jacketing method.

It is necessary that the retrofitted section becomes integrated as one part. Therefore, installation must be studied carefully. The table below (5.3.1) shows a detailed limits for concrete jacketing introduced in a report done by Shri. (2011).

Table 5.3.1. Details for Reinforced Concrete Jacketing.[Shri., 2011]

Properties of jackets	• Match with the concrete of the existing structure. • Compressive strength greater than that of the existing structures by 5 N/mm2 or at least equal to that of the existing structure.
Minimum width of jacket	• 10 cm for concrete cast-in-place and 4 cm for shotcrete. • If possible, four-sided jacket should be used. • A monolithic behavior of the composite column should be assured. • Narrow gap should be provided to prevent any possible increase in flexural capacity.
Minimum area of longitudinal reinforcement	• 3Afy, where, A is the area of contact in cm2 and fy is in kg/cm2 • Spacing should not exceed six times of the width of the new elements (the jacket in the case) up to the limit of 60 cm. • Percentage of steel in the jacket with respect to the jacket area should be limited between 0.015and 0.04. • At least, 12 mm bar should be used at every corner for a four sided jacket.
Minimum area of transverse reinforcement	• Designed and spaced as per earthquake design practice. • Minimum bar diameter used for ties is not less than 10 mm or 1/3 of the diameter of the biggest longitudinal bar. • The ties should have 135-degree hooks with 10bar diameter anchorage. • Due to the difficulty of manufacturing 135-degree hooks on the field, ties made up of multiple pieces, can be used.
Shear stress in the interface	• Provide adequate shear transfer mechanism to assured monolithic behavior. • A relative movement between both concrete interfaces (between the jacket and the existing element) should be prevented. • Chipping the concrete cover of the original member and roughening its surface may improve the bond between the old and the new concrete. • For four-sided jacket, the ties should be used to confine and for shear reinforcement to the composite element.
Connectors	• Connectors should be anchored in both the concrete such that it may develop at least 80% of their yielding stress. • Distributed uniformly around the interface, avoiding concentration in specific locations. • It is better to use reinforced bars (rebar) anchored with epoxy resins of grouts.

5.3.2.3.2 Steel jacketing

Confining reinforced concrete column with steel jacket is one of the effective methods in order to improve the earthquake resistant capacity. When it was compared with conventional hoops or spirals, steel jacket has two advantages: [Kenji and Yuping, 1999]

1- Easy to provide a large amount of transverse steel, hence strong confinement to the compressed concrete.

2- Prevent spalling of the concrete.

Kenji and Yuping describe in their report: [Kenji and Yuping, 1999]

(1) Stress-strain curve model for concrete confined by the steel jacket.

(2) Methods to evaluate ultimate bending strength and shear strength of the retrofitted RC columns.

(3) Design formula to predict deformation capacity of the retrofitted columns.

Kenji and Yuping made analytical research about steel jacket retrofitting for square RC columns and they proposed methods and formula, which were verified by many experimental results of the retrofitted RC column specimens tested by the Japanese researchers. They proposed design formulae that can predict experimental results [Kenji and Yuping, 1999]

Table 5.3.2. List of available test data concerning retrofitted RC columns [Kenji and Yuping, 1999]

Ref.	Number of column	D (mm)	a/D	B/t	p_g (%)	f_c' (MPa)
Sasaki 1972	2	390	1.38, 2.28	172	3.0	21.8
Tomii. 1987	6	160	1.07	29	3.82, 7.65	32.2～42.3
Minami. 1988	3	196	1.53	50	2.64	19.9～22.9
Sun *et al*., 1989	4	165	1.06	77	3.71	40.2～47.8
Sasaki. 1989	7	195	1.02～1.53	48～135	2.65	21.9～31.9
Yoshioka , 1989	6	300	2.0	55～72	4.47～5.36	46.1
Yamamoto . 1990	1	294	1.53	98	2.77	28.4
Asakawa. 1994	9	180	1.25	42～152	7.06	37.4～42.0
Yoshikawa. 1995	5	300	0.9～1.8	52～69	1.69～2.26	48.4～55.2
Jinno., 1996	2	316-356	1.11, 2.11	101,103	1.60, 2.03	36.7
Masuo, 1996	2	460	1.30	146	0.75, 2.16	22.5
Sun *et al*., 1997	7	163	2.18	29	3.82～7.65	39.6～56.6
Sakino. 1997	8	250	2.0	28～117	2.44	32.0
Aklan. 1997	5	250	2.0	28～117	2.44	47.9～51.1
Sakino. 1998	6	250	1.0～1.5	43～82	2.44	29.4～34.9
Sun *et al*. 1998	7	250	1.0	28～117	2.44	27.8～37.2

p_g = ratio of total area of the longitudinal bars to gross sectional area of the concrete

J H Wang et al (2005) studied another type of steel jacket retrofitting which is circular steel jacket, where the authors carried out an experimental study on retrofitting the columns with rectangular and circular steel jackets. The experimental results indicated that the circular jacket specimens are much more effective than the rectangular jacket specimens, especially in the columns under high axial-stress. [J H Wang et al, 2005]

Shri., 2011 also mentioned in their report detailed limits about steel jacketing. The following table (5.3.3) gives a summary of these limits: [Shri., 2011]

Table 5.3.3. Typical details of Steel Jacketing. [Shri., 2011]

Steel plate thickness	• At least 6 mm.
Height of jacket Shape of jackets Free ends of jackets bottom clearance.	• 1.2 to 1.5 times splice length in case of flexural columns. • Full height of column in case of shear columns. • Rectangular jacketing, prefabricated two L-shaped panels The use of rectangular jackets has proved to be successful in case of small size columns up to 36 inch width that have been successfully retrofitted with thick steel jackets combined with adhesive anchor bolt, but has been less successful on larger rectangular columns. On larger columns, rectangular jackets appear to be incapable to provide adequate confinement. • Welded throughout the height of jacket, size of weld1" • 38 mm (1.5 inch), steel jacket may be terminated above the top of footing to avoid any possible bearing of the steel jacket against the footing, to avoid local damage to the jacket and/or an undesirable or unintended increase in flexural capacity.
Gap between steel jacket and concrete column Size of anchor Number of anchor bolts	• 25 mm fill with cementations grout. • 25 mm in diameter and 300 mm long embedded in 200 mm into concrete column. • Bolts were installed through pre-drilled holes on the steel jacket using an epoxy adhesive. • Two anchor bolts are intended to stiffen the steel jacket and improve confinement of the splice.

5.3.2.3.3 Fiber-reinforced polymer (FRP)

Fiber reinforced polymer is a composite material made of a polymer matrix, and reinforced with fibers. FRP can be used as bars or mounted sheets to strengthen and retrofit the structural elements such as beams, columns, and slabs of buildings and bridges.

For the strengthening of beams, there are two techniques adopted, which are flexural strengthening and shear strengthening. In many cases, it may be necessary to provide both strength enhancements. Columns are typically wrapped with FRP around their perimeter, as with closed or complete wrapping. This not only results in higher shear resistance, but more crucial for column design, it results in increased compressive strength under axial loading. The FRP wrap works by restraining the lateral expansion of the column, which can enhance confinement in a similar manner as spiral reinforcement does for the column core.

A research done by A. Mortezaei and H.R. Ronagh (2012) presents an analytical investigation about the behavior of FRP strengthened RC columns. They concluded that FRP strengthened RC columns develop longer plastic hinges than those without FRP sheets. In addition, the plastic hinge length in FRP strengthened RC columns subjected to near-fault earthquakes is lower than that plastic hinge length in RC columns subjected to far-fault earthquakes.

They developed the following two equations that allow a better estimation of the plastic hinge length of FRP strengthened RC members under various far-fault and near-fault ground motions: [A. Mortezaei, H.R. Ronagh, 2012]

$$\frac{l_p}{h} = \left[0.4\left(\frac{P}{Po}\right) + 3\left(\frac{As}{Ag}\right) - 0.1\right]\left(\frac{H}{h}\right) + 0.65$$

≥0.65 (For far-fault earthquakes)

$$\frac{l_p}{h} = \left[0.4\left(\frac{P}{Po}\right) + 3\left(\frac{As}{Ag}\right) - 0.1\right]\left(\frac{H}{h}\right) + 0.55$$

≥0.55 (For near-fault earthquakes)

Where:

lp: plastic hinge length

H: distance from critical section to the point of contra flexure

h: overall depth of column

Po: nominal axial load capacity

P: applied axial load

As: area of tension reinforcement

Ag: gross area of concrete section

These equations will be very useful for defining the plastic hinges in case of retrofitting using FRP.

5.4 Elastic analysis and assessment

5.4.1 Introduction

In the previous chapter, it was shown that the building had a potential for weak and sift story irregularities. Therefore, the retrofitting techniques must restore the uniform distribution of stiffness in the building. In this section, the parameters of retrofitting will be estimated by achieving the required stiffness and shear strength ratios that are compatible with the ASCE and IBC requirements.

After including the effect of the retrofitting techniques in the model, the building will be checked for vertical and horizontal irregularities and for P-delta effect according to ASCE 7-10 in order to give a final elastic assessment. After these elastic checks, nonlinear static procedure will be used again for final judgment on the effectiveness of the proposed retrofitting techniques.

5.4.2 State of un-retrofitted case study

As mentioned in section (4.2), the state of the un-retrofitted case study building can be summarized as follows:

1- Has no problems with carrying gravity loads.

2- Satisfies serviceability conditions (deflection).

3- The torsional mode comes as a second mode and before the transitional mode in X-direction.

4- The building showed that it has no torsional problems according to ASCE 7-10 standards when it is exposed to equivalent lateral loads with accidental 5% eccentricity according to IBC 2012.

5- The building has vertical irregularity between type (1-a) to (1-b) that is: soft story to extreme soft story.

6- Potential weak-story problem

For calculating specific parameters for proposed retrofitting techniques, stiffness deficits of the case study is converted to numbers that can be seen in (Appendix A). The resultant stiffness ratios between ground to first floor are:

Table 5.4.1. Stiffness ratios between ground floor (GF) and first floor (F1)

Lateral stiffness ratio	X-dir.	Y-dir.
(GF+F1) / (F1+F2)	0.66	0.67

There is another check among the checks proposed by ASCE 7-10 within the vertical irregularity check in table (4.2.5), which is the strength– weak story irregularity check. Strength–weak story irregularity is defined to exist where the story lateral strength is less than 80% of that in the story above, and called extreme if the percentage is less than 65%.

The structural elements that participates in shear strength resistance are columns, elevator shaft walls, and part of the unreinforced masonry walls. This part is the masonry wall length between columns minus windows or door openings.

The following tables (5.4.2, 5.4.3, and 5.4.4) shows the calculations of each story lateral strength in both horizontal directions and the ratio between stories.

Table 5.4.2. Shear strength calculations for floors (X-direction)

Ground Floor								
X-Direction								
Column	b_w	h	d	b_w*d	f'c	Vc	Item No.	∑Vc
GF 1	0.3	0.7	0.67	0.201	24000	164.12	17	2789.9688
GF 2	0.7	0.3	0.27	0.189	24000	154.32	4	617.27142
Elev. Corn.	0.2	0.5	0.47	0.235	24000	76.75	4	307
Elev. Straight 1	0.2	1	0.97	0.194	24000	158.40	1	158.40034
Elev. Straight 2	1	0.2	0.17	0.17	24000	138.80	2	277.60884
							SUM	3886.21
First to Fifth Floor								
X-Direction								
Column	b_w	h	d	b_w*d	f'c	Vc	Item No.	∑Vc
Column 1	0.25	0.7	0.67	0.1675	24000	136.76	17	2324.974
Column 2	0.7	0.25	0.22	0.154	24000	125.74	4	502.96189
Elev. Corn.	0.2	0.5	0.47	0.235	24000	76.75	4	307
Elev. Straight 1	0.2	1	0.97	0.194	24000	158.40	1	158.40034
Elev. Straight 2	1	0.2	0.17	0.17	24000	138.80	2	277.60884
Bracing element	0.2	2	1.95	0.094	14000	243.2	8	1945.6
							SUM	5831.8

Where:

b: structural element cross section width

h: structural element cross section length

d: effective depth

f'c: compressive strength of concrete

Vc: capacity shear force

Table 5.4.3. Shear strength calculations for floors (Y-direction)

Ground Floor								
Y-Direction								
Column	b_w	h	d	b_w*d	f'c	Vc	Item No.	∑Vc
GF 1	0.7	0.3	0.27	0.189	24000	154.32	17	2623.4035
GF 2	0.3	0.7	0.67	0.201	24000	164.12	4	656.46325
Elev. Corn.	0.2	0.5	0.47	0.235	24000	76.75	4	307
Elev. Straight 1	1	0.2	0.17	0.17	24000	138.80	1	138.80442
Elev. Straight 2	0.2	1	0.97	0.194	24000	158.40	2	316.80067
							SUM	3778.43
First to Fifth Floor								
Y-Direction								
Column	b_w	h	d	b_w*d	f'c	Vc	Item No.	∑Vc
GF 3	0.7	0.3	0.27	0.189	24000	154.32	17	2623.4035
GF 4	0.3	0.7	0.67	0.201	24000	164.12	4	656.46325
Elev. Corn.	0.2	0.5	0.47	0.235	24000	76.75	4	307
Elev. Straight 1	1	0.2	0.17	0.17	24000	138.80	1	138.80442
Elev. Straight 2	0.2	1	0.97	0.194	24000	158.40	2	316.80067
Bracing element	0.2	2	1.95	0.094	14000	243.2	6	1459.2
							SUM	5237.7

The resulted shear strength ratios are:

Table 5.4.4. Shear strength ratios between ground floor (GF) & first floor (F1)

Shear strength ratios	X-dir.	Y-dir.
(GF) / (F1)	0.66	0.72

According to ASCE 7-10, the structure has irregularity of (type 1) and (type 5) in both (X) and (Y) directions, where the ratios lie between 65-80% for (type 5), which are classified as weak to extreme weak story.

5.4.3 Selecting proposed retrofitting techniques and their parameters

The estimation of the parameters is done through four logical steps:

1- The state of the building before retrofitting and the applied forces at P.P. In chapter (4), the analysis showed two major problems, which are the soft story and the torsional mode effect. The defects of the soft story can be seen through the ASCE 7-10 code requirements, where the ratios of vertical irregularity are out of limited ranges. In addition, the push over analysis results showed that the building is in danger of collapse because of appearance of collapse hinges at the ground floor columns in the y-direction.

The location of the two problems is mainly in the ground floor columns, where the retrofitting procedure should be applied.

The current state design capacities and ratios for the structure before retrofitting are:

1- Shear strength capacity for the ground floor
2- Shear strength capacity ratio (GF/F1)

3- Flexural stiffness ratio (GF+F1/F1+F2)
4- Axial-moment interaction diagram value for maximum loaded column at performance point.

The previous current values need to satisfy and meet the following values in table (5.4.5)

Table 5.4.5. The status of the building at the GF level

<table>
<tr><td></td><td colspan="2">Current value Before retrofitting</td><td colspan="2">Allowable capacity value</td><td colspan="2">required value-current value</td></tr>
<tr><td></td><td>X</td><td>Y</td><td>X</td><td>Y</td><td>X</td><td>Y</td></tr>
<tr><td>Shear strength ratio</td><td>68%</td><td>74%</td><td>>80%</td><td>>80%</td><td>12-32%</td><td>6-26%</td></tr>
<tr><td>Flexural stiffness ratio</td><td>67%</td><td>66%</td><td>>80%</td><td>>80%</td><td>13-33%</td><td>14-34%</td></tr>
<tr><td>Max. drift at P.P.</td><td>0.097m</td><td>0.13m</td><td>0.31m</td><td>0.31m</td><td>Ok.</td><td>Ok.</td></tr>
<tr><td colspan="7"></td></tr>
<tr><td rowspan="3">P-M interaction ratio see figure (5.4.1)</td><td colspan="2">Mn (kN.m)</td><td colspan="2">Pn (kN)</td><td colspan="2">State (Safe/Fail)</td></tr>
<tr><td>Col. D-8</td><td>586</td><td>Col. D-8</td><td>1051</td><td>Col. D-8</td><td>Safe</td></tr>
<tr><td>Col. B-9</td><td>257</td><td>Col. B-9</td><td>2682</td><td>Col. B-9</td><td>Fail</td></tr>
</table>

2- Specify the locations where retrofitting is needed

In the step of choosing retrofitting technique, many things must be considered as mentioned in (5.2) such as the location. Therefore, the columns on the axes B-(1, 2, 6, and 9) and J-(1, 2, 6, and 9) will be retrofitted. The reasons of choosing (B & J) axes are the following:

a) These columns are the farthest from the center of mass, which gives them the largest arm to resist torque.

b) The pushover results indicate that the weak direction is the y-direction, so framing technique would be useful in on (B & J) axes.

c) Easy to modify because they are on perimeter of the building

d) Less destruction than the other two sides, because there are no masonry walls between the columns on (B & J) axes, but only gates are existing.

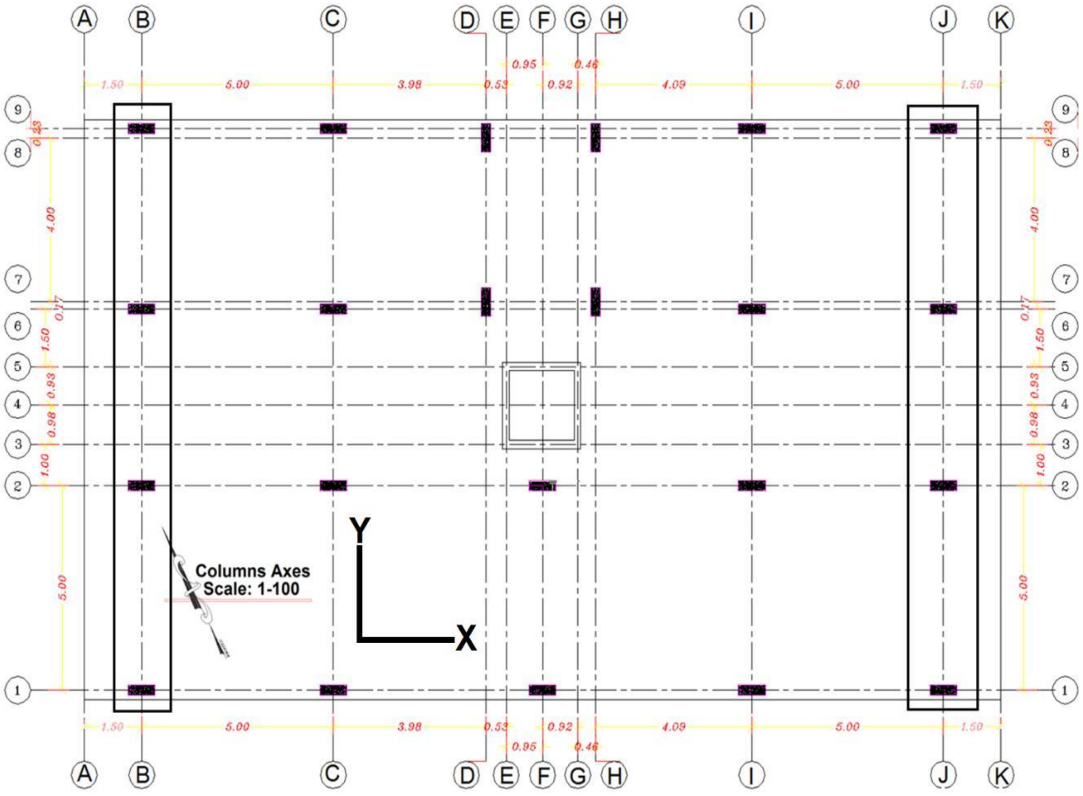

Figure 5.4.1. Columns axes showing the proposed columns on the ground floor to be retrofitted.

3- Proposing suitable retrofitting techniques

Six available retrofitting techniques can be used:

A- Global level retrofitting

a) Adding concrete frames

b) Adding steel bracing

c) Adding shear walls

B- Local level retrofitting

a) Concrete jacketing

b) Steel jacketing

c) FRP retrofitting

However, there are practical limitations that prevent the application of certain retrofitting techniques at the selected locations, namely:

A. As shown in Figure (3.2.4), the presence of gates will prevent adding bracing elements or adding shear walls,

B. As mentioned in section (4.7), the Lack of rigidity that causes vertical irregularity requires increasing the stiffness of the sections of existing vertical structural elements considerably. This is not practical to achieve using steel jackets or (FRP) techniques.

The remaining techniques are "concrete framing" and "concrete jacketing".

4- Determination of techniques parameters

The parameters will be determined according to the values in the previous table (5.4.5). Therefore, the goal is to raise the stiffness and strength ratios to be equal to one in both directions.

The following figure (5.4.2) shows the proposed change in dimensions that meet the required ratios in order to prevent vertical irregularity.

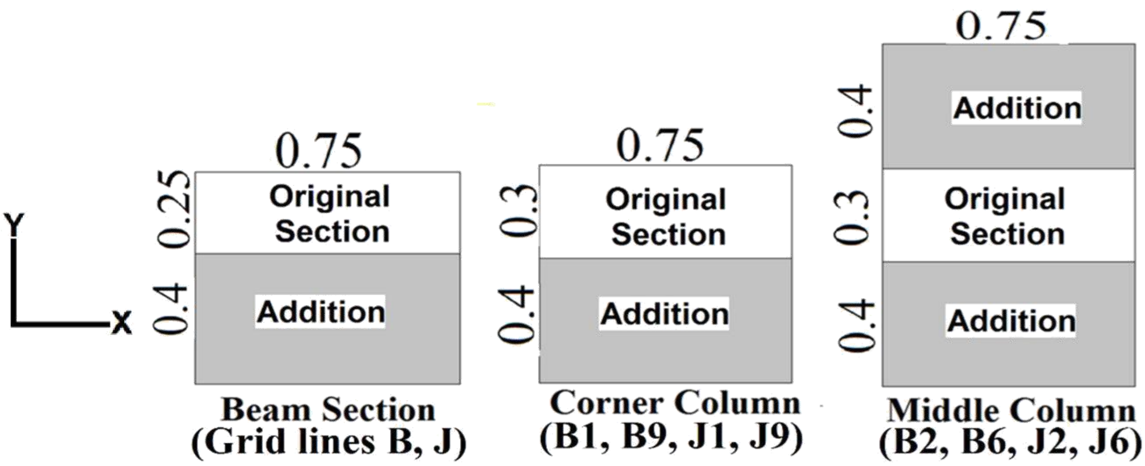

Figure 5.4.2. Concrete framing Technique with the proposed column sections showing framing parts (units in meter).

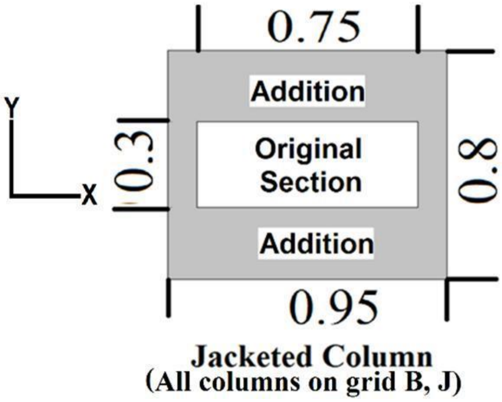

Figure 5.4.3. Concrete Jacketing Technique with the proposed column sections showing jacket concrete thickness in each side (units in meter).

The new stiffness ratios after changing dimensions for both retrofitting techniques are:

Table 5.4.6. Stiffness & shear strength ratios after concrete frame technique for ground floor

Concrete framing technique for GF		
Stiffness Ratio	X-Dir.	Y-Dir.
(GF+F1) / (F1+F2)	0.84	1.26
shear strength Ratio	X-Dir.	Y-Dir.
(GF) / (F1)	1.15	1.44

Table 5.4.7. Stiffness & shear strength ratios with concrete jacketing technique for ground floor

Concrete jacketing technique for GF		
Stiffness Ratio	X-Dir.	Y-Dir.
(GF+F1) / (F1+F2)	1.00	1.12
shear strength Ratio	X-Dir.	Y-Dir.
(GF) / (F1)	1.52	1.9

The value of stiffness ratio in X-direction is still smaller than one. However, this is not detrimental, because:

1- The value is still greater than the code limit value, which is 80%.

2- The structure does not collapse in the X direction.

3- Increasing the stiffness at the edges may lead to collapse of the bracing elements (masonry walls) at the first floor in X-direction, because the bracing elements are brittle and cannot take large deformations

The reinforcement of the added sections of the retrofitting techniques will be estimated based on either the maximum loads on the structural elements at the step of P.P. or minimum requirements according to ACI 318-14. As mentioned in table (5.4.5) the maximum loads lies on the column (B-9) on the ground floor (Mn=257kN.m, Pn=2681.9kN) and these loads will be used to design the sections of the retrofitting techniques. The reinforcement will be as follows:

Table 5.4.8. Reinforcement details for concrete frame and jacket sections for ground floors

Name	Long. Reinforcement.	Stirrups
Corner column	24Φ16mm	2Φ8/15cm
Middle column	38Φ16mm	2Φ8/15cm
Jack. column	36Φ16mm	2Φ8/15cm

The loads on column (B-9) at the performance point lies inside the P-M interaction diagrams of the new composite sections.

The interaction diagrams that were calculated by SAP2000 program are verified and can be seen in (Appendix A).

The same retrofitting techniques will be used to retrofit the previous columns on (B & J) axes, but on the ground and first floors. The following steps can achieve this:

1- Remove the masonry walls between the columns on (B & J) axes at the first floor for the following reasons:

a- Appling the retrofitting techniques require removal of the masonry walls on the sides of the intended columns.

b- This will cause the bracing effect of the masonry walls to decrease as the proportion of openings increases to the percentage of wall area.

2- Apply concrete jacketing to the columns on (B & J) axes at ground floor and first floor.

3- The other retrofitting technique is to apply concrete framing sections to the columns on (B & J) axes at ground floor and first floors.

The reinforcement of the added sections of the retrofit techniques for the ground floor will be the same as mentioned in the table (5.4.8). Ratios and reinforcement will be as follows:

1- Concrete jacketing for the columns for first floor is considered to be as the same as in ground floor in order to keep the center of retrofitted columns on the top of each other.

Table 5.4.9. Stiffness & shear strength Ratios with concrete jacket technique for ground and first floors

Concrete jacketing technique for GF + F1		
Stiffness Ratio	X-Dir.	Y-Dir.
(GF+F1) / (F1+F2)	1	1.08
shear strength Ratio	X-Dir.	Y-Dir.
(GF) / (F1)	1.14	1.23

2- Concrete frame technique for the first floor is:

Table 5.4.10. Dimensions and reinforcement of frame sections of first floor

Dimensions and reinforcement of frame sections of first floor			
Name	Dimension	Long. Reinf.	Stirrups
Corner column	70*70 (40cm added)	24Φ16mm	2Φ8/15cm
Middle column	70*70 (23cm added)	30Φ16mm	2Φ8/15cm
Frame beam	75cm width* 55cm depth	6Φ16mmTop	2Φ8/15cm
	(30cm added in depth)	7Φ16mmBott.	2Φ8/15cm

Table 5.4.11. Stiffness & shear strength Ratios with concrete frame technique for ground and first floors

Concrete frame technique for GF + 1st F		
Stiffness Ratio	X-Dir.	Y-Dir.
(GF+F1) / (F1+F2)	0.88	1.15
shear strength Ratio	X-Dir.	Y-Dir.
(GF) / (F1)	1.24	1.25

5.4.4 Modeling retrofitting techniques

In this section, the proposed retrofitted sections will be modeled and assigned to the model of the case study building. Each technique will be studied separately.

It should be mentioned that during the application of the jacketing technique, the masonry wall will have to be reconstructed. Therefore, its bracing effect will be reduced. To be on the conservative side, the bracing effect of the masonry wall will neglected. This means that special provisions must be ensured during the construction of masonry wall such that the bracing effect is not developed.

5.4.4.1 Definition of concrete jacketing technique for ground floor

Here in this section, column sections in the ground floor are replaced by the jacketed column section that will be assigned to columns on the axes (B & J). Figure (5.4.3) shows the new dimensions with original dimensions and figure (5.4.4) shows the modeled section with the added reinforcement.

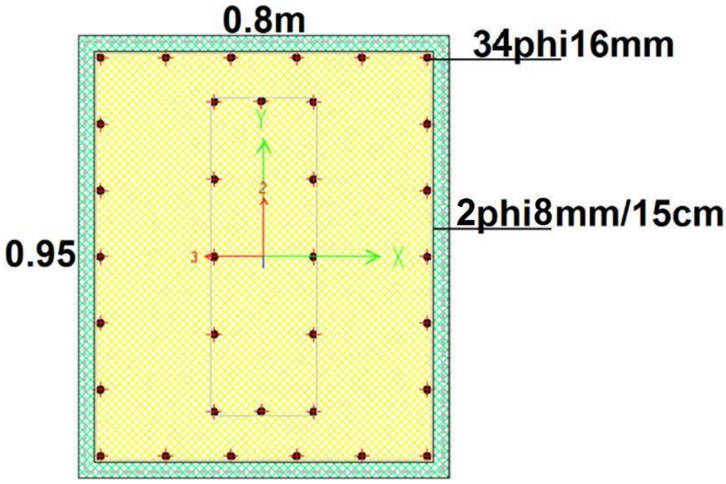

Figure 5.4.4. Defined cross section in jacketed column (in meter)

The retrofitted section will be modeled directly as composite section in section designer in SAP2000. The software will take the properties directly from the detailed section using fiber model and Caltrans bilinear approximation for the plastic hinge moment-curvature behavior for the section.

5.4.4.2 Define concrete frame technique for ground floor

This technique has three sections, which are:

1- The addition of frame column to the original column located at the corner as shown in figure (5.4.5):

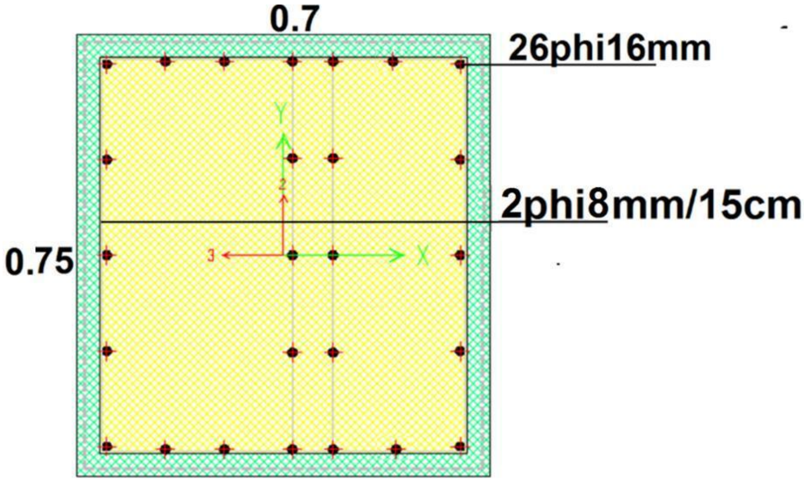

Figure 5.4.5. Defined cross section in corner frame column (in meter)

2- Two additions of frame columns to the original column located at the middle as shown in figure (5.4.6):

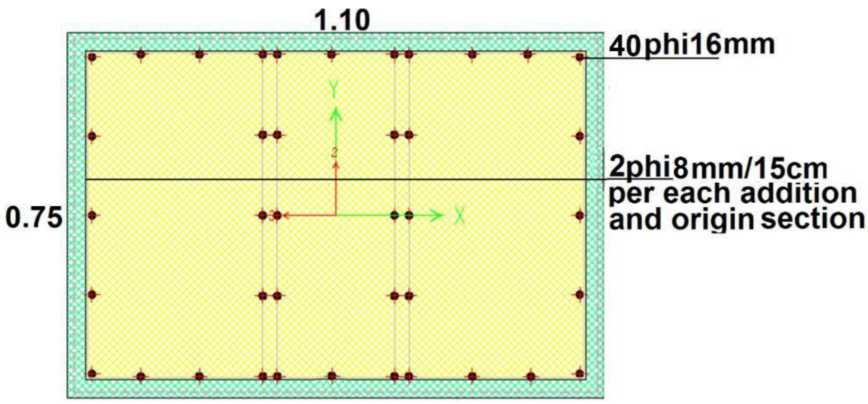

Figure 5.4.6. Defined cross section in middle frame column (in meter)

3- The addition of frame beam to the original beam as shown in figure(5.4.7):

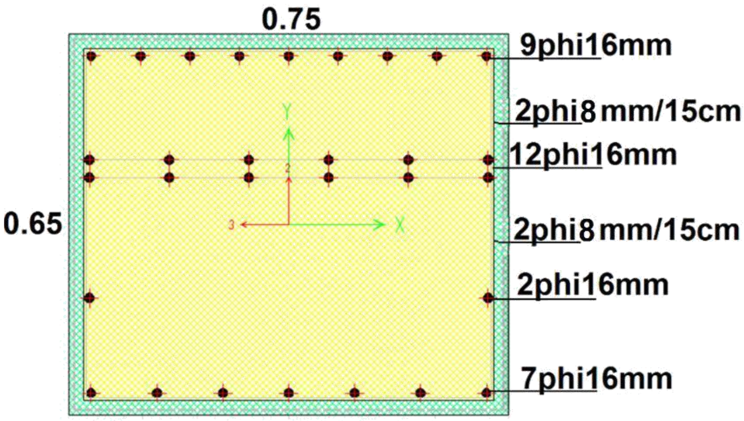

Figure 5.4.7. Defined cross section in frame beam (in meter)

5.4.4.3 Define concrete jacket technique for ground and first floors

This technique has one section only as (5.4.4.1), which is the jacketed column that will be assigned instead of columns on the axes (B & J) in the ground floor and first floor.

5.4.4.4 Define concrete frame technique for ground and first floors

This technique, has three sections for each floor. The sections for the ground floor are the same as in (5.4.4.2). and the sections for the first floor are the following:

1- The addition of frame column to the original column located at the corner is the same as in ground floor
2- Addition of two frame columns to the original column located at the middle as shown in figure (5.4.8):

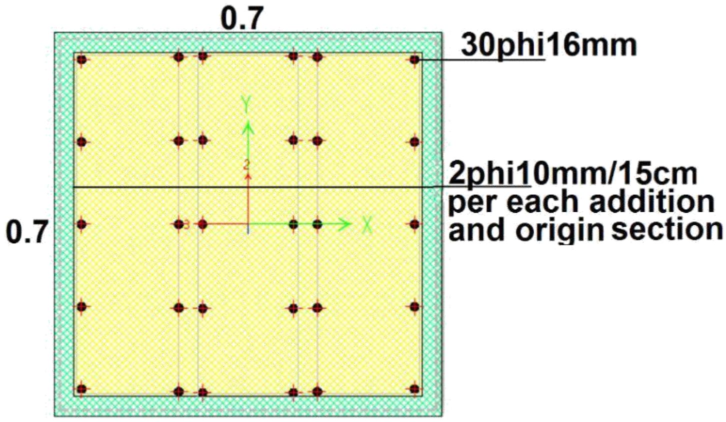

Figure 5.4.8. Defined cross section in middle frame column for first floor (in meter)

3- The addition of frame beam to the original beam located as shown in figure (5.4.9):

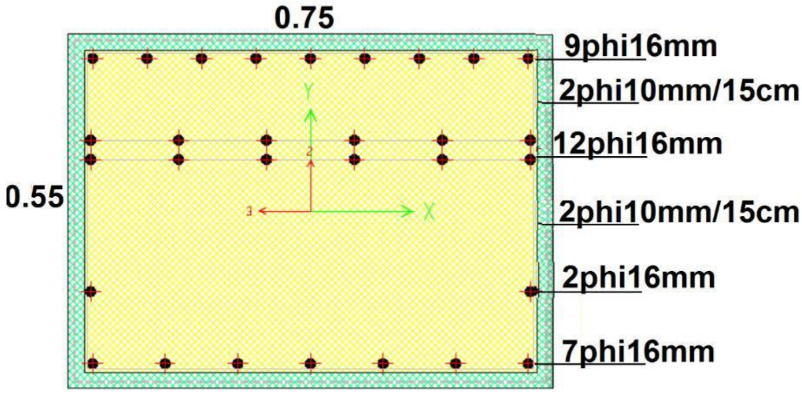

Figure 5.4.9. Defined cross section in frame beam for first floor (in meter)

5.4.5 Elastic analysis results

In this section, the elastic analysis will be applied to the proposed technique and then the design is rechecked to make sure that the new capacities are acceptable:

1- Bearing capacity of ultimate gravity loads according to ACI 318-14.

The retrofitted columns are checked for bending, axial, and shear, and found to be ok. This check is satisfied in the four proposed retrofitting models.

2- Serviceability.

This check is already satisfied in the four proposed retrofitting models, because the un-retrofitted model had already achieved the code serviceability requirements.

3- Horizontal irregularity test

For applying the checks of horizontal irregularity and P-delta effect, the building was exposed to equivalent static earthquake loads once in X direction and the second Y direction with eccentricity ratio of 5% according to the IBC 2012, as mentioned in section (4.2).

The resultant deformations at the roof showed that the horizontal irregularity does not exceed 20% of the average drift in both directions.

Table 5.4.12: Summary of horizontal irregularity ratios of top displacement.

concrete frame technique for GF							
EQ-X-direction				EQ-Y-direction			
Left	Right	Avg	% disp.	Left	Right	Avg	% disp.
0.0508	0.0494	0.0501	1.4	0.0457	0.0612	0.0535	14.5
concrete Jacket technique for GF							
EQ-X-direction				EQ-Y-direction			
Left	Right	Avg	% disp.	Left	Right	Avg	% disp.
0.0456	0.0445	0.0451	1.2	0.0527	0.0699	0.0613	14.0
concrete frame technique for GF+1st F							
EQ-X-direction				EQ-Y-direction			
Left	Right	Avg	% disp.	Left	Right	Avg	% disp.
0.0496	0.0488	0.0492	0.8	0.0465	0.0621	0.0543	14.4
concrete jacket technique for GF+1^{st} F							
EQ-X-direction				EQ-Y-direction			
Left	Right	Avg	% disp.	Left	Right	Avg	% disp.
0.0453	0.044	0.0447	1.5	0.0678	0.0902	0.079	14.2

4- Vertical irregularity

This check is done in (5.4.3) which satisfies ASCE 7-10 limits.

5- P-delta effect

The table below shows stability coefficient (θ) for estimating P-delta effect for the ground floor, because it is the critical floor in the building. The results of (θ) are smaller than the limit (0.1), which means that P-delta effect is not considerable. In addition, the check was also done for the above floor and showed that P-delta effect can be ignored.

The maximum value was (0.019) for the first floor in Y-direction with the concrete jacket technique for ground and first floor.

Table 5.4.13. Summary of stability coefficient (θ) for estimating P-delta effect for the ground floor.

concrete frame technique for GF													
θ	Px	Δ	Ie	Vx	hx	Cd	θ	Px	Δ	Ie	Vx	hx	Cd
X-direction							Y-direction						
0.008	37260.5	0.0131	1	5705	4.5	2.5	0.005	37260.5	0.0056	1	3508	4.5	2.5
concrete Jacket technique for GF													
θ	Px	Δ	Ie	Vx	hx	Cd	θ	Px	Δ	Ie	Vx	hx	Cd
X-direction							Y-direction						
0.006	36950	0.011	1	5651	4.5	2.5	0.009	36950	0.012	1	4423	4.5	2.5
concrete frame technique for GF+1st F													
θ	Px	Δ	Ie	Vx	hx	Cd	θ	Px	Δ	Ie	Vx	hx	Cd
X-direction							Y-direction						
0.008	40705.3	0.0128	1	5735.7	4.5	2.5	0.007	40705.3	0.006	1	3161.7	4.5	2.5
concrete jacket technique for GF+1st F													
θ	Px	Δ	Ie	Vx	hx	Cd	θ	Px	Δ	Ie	Vx	hx	Cd
X-direction							Y-direction						
0.006	40582.7	0.0094	1	5710	4.5	2.5	0.013	40582.7	0.021	1	5710	4.5	2.5

6- Modal shapes check

The results of modal shape analysis for the retrofitted models shows that first three retrofit techniques caused the torsional mode to be delayed behind the flexural modes, except the forth retrofit technique, which is Concrete Jacketing technique for (GF+1st F), which produced torsional effect in the second and third modes.

Table (5.4.13) shows a summary of the first three fundamental modes for the retrofitted models.

Table 5.4.14. Summary of modal analysis results for the dominant modes.

<table>
<tr><th colspan="2"></th><th>Un-retrofitted</th><th>Conc. Fr. GF</th><th>Conc. Fr. GF+F1</th><th>Conc. Jack. GF</th><th colspan="2">Conc. Jack. GF+F1</th></tr>
<tr><td rowspan="3">mode1</td><td>period</td><td>0.92</td><td>0.66</td><td>0.68</td><td>0.71</td><td colspan="2">0.88</td></tr>
<tr><td>direction</td><td>UY</td><td>UX</td><td>UY</td><td>UY</td><td colspan="2">UY</td></tr>
<tr><td>MMPR</td><td>95%</td><td>85%</td><td>87%</td><td>91%</td><td colspan="2">91%</td></tr>
<tr><td rowspan="3">mode2</td><td>period</td><td>0.92</td><td>0.64</td><td>0.64</td><td>0.61</td><td colspan="2">0.62</td></tr>
<tr><td>direction</td><td>RZ</td><td>UY</td><td>UX</td><td>UX</td><td>UX</td><td>RZ</td></tr>
<tr><td>MMPR</td><td>99%</td><td>99%</td><td>92%</td><td>89%</td><td>34%</td><td>58%</td></tr>
<tr><td rowspan="3">mode3</td><td>period</td><td>0.75</td><td>0.48</td><td>0.5</td><td>0.54</td><td colspan="2">0.59</td></tr>
<tr><td>direction</td><td>UX</td><td>RZ</td><td>RZ</td><td>RZ</td><td>UX</td><td>RZ</td></tr>
<tr><td>MMPR</td><td>95%</td><td>91%</td><td>91%</td><td>93%</td><td>90%</td><td>35%</td></tr>
</table>

5.5 Pushover analysis of the retrofitted building

In this section, most steps are similar to what was done in chapter (4); only the following steps need to be updated here:

1- Assign new auto hinges to the retrofitted sections, first for retrofitted column sections, and second, for the retrofitted beam sections in frame techniques.

2- Assign equivalent lateral seismic loads according to the new flexural modes of each model in X & Y directions.

5.6 Results of pushover analysis

In this section, the results of the updated pushover analyses are presented for the models of the retrofitted building for each technique, see figure (5.6.1 and 5.6.2). The results will be in terms of base shear force versus the top displacement of the building. These results represent the behavior of the building under the influence of seismic forces within the assumptions adopted earlier. The information obtained is used to assess the performance of the structure in order to accept or reject the proposed retrofit techniques.

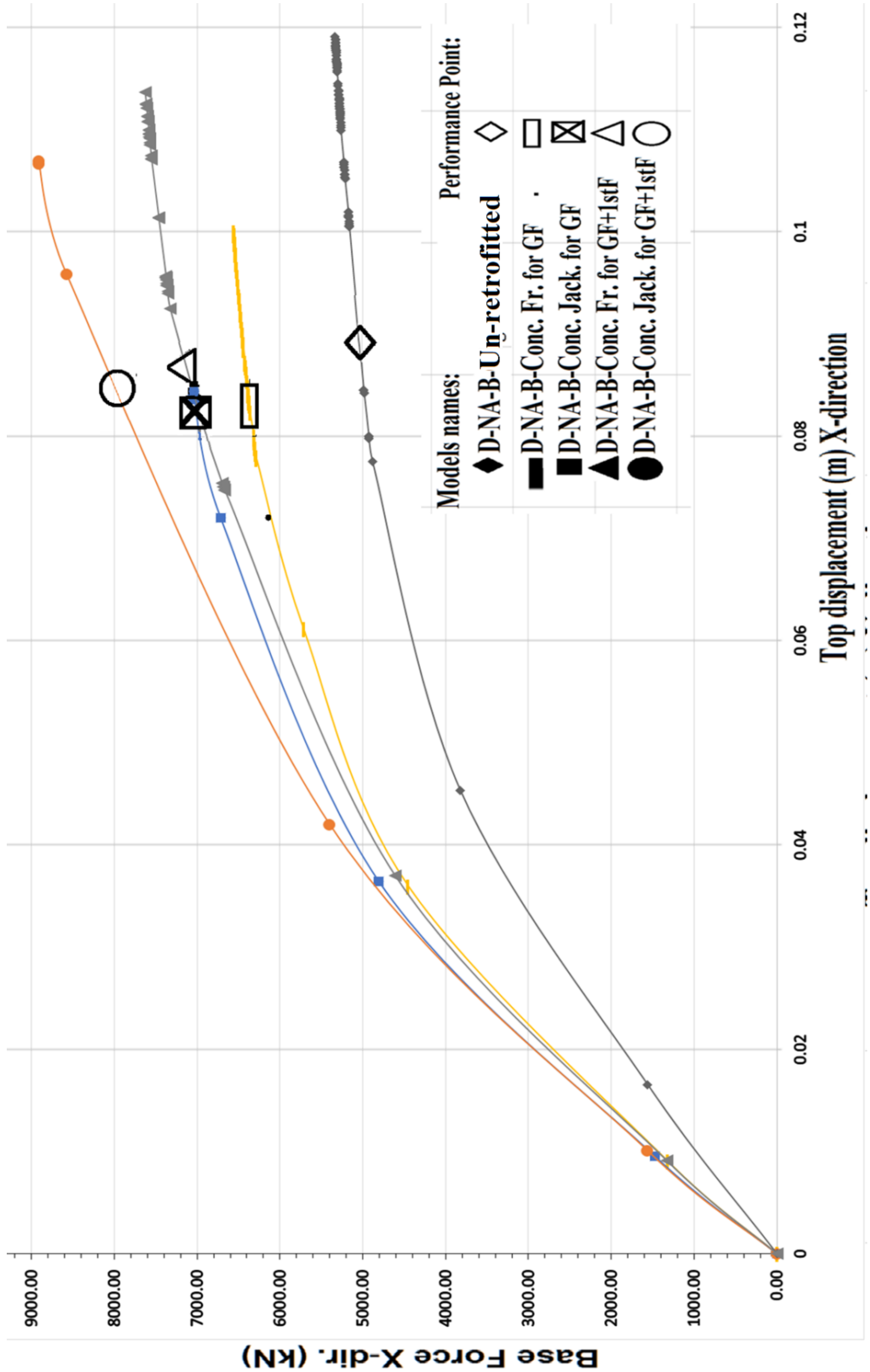

Figure 5.6.1. Models pushover curves in terms of (V-D), X-dir.

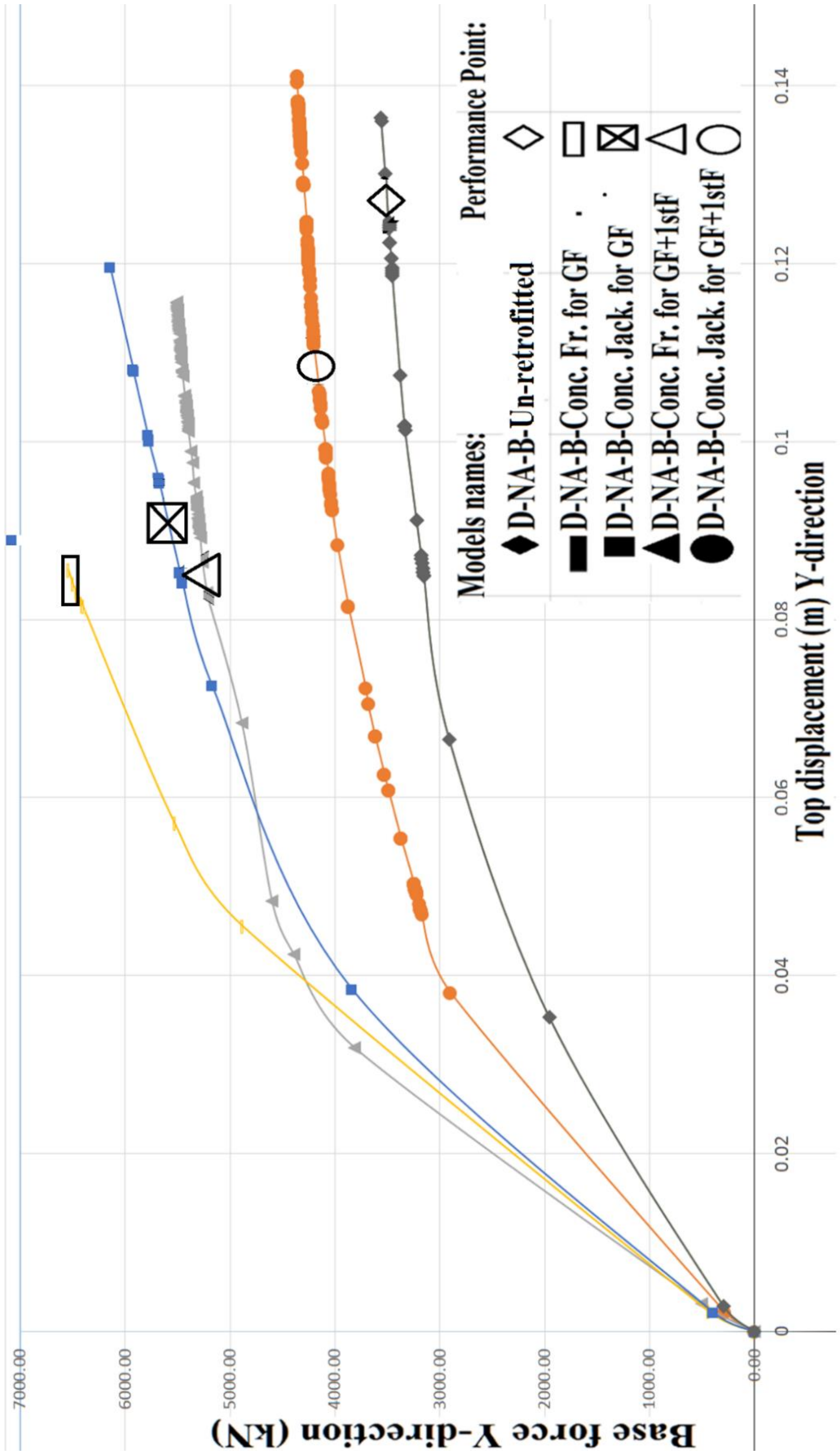

Figure 5.6.2. Models pushover curves in terms of (V-D), Y-dir.

1- Concrete frame technique for ground floor:

X-direction

The performance point is located between the two steps of (21-22) with Sa equal to 0.24g and Sd equal to 0.07m.

Y-direction

The performance point is located at step (9) with Sa equal to 0.26g and Sd equal to 0.068m.

The following two tables display detailed data for the step of the performance point including types of plastic hinges that had formed.

Table 5.6.1. Capacity and demand curves data at the performance point for X & Y-directions for concrete frame technique.

Concrete frame GF								
Capacity curve			Capacity and demand curve					
Step	Disp (m)	V (kN)	Teff	Beff	Sd Cap. (m)	Sa Cap.	Sd Dem. (m)	Sa Dem.
X-direction								
21	0.083	6365	1.0659	0.196	0.067522	0.2392	0.069846	0.2474
22			1.079	0.202	0.06969	0.24062	0.070032	0.24180
Y-direction								
9	0.085	6533	1.026	0.18	0.068	0.261	0.0681	0.2604

Table 5.6.2 Numbers and types of plastic hinges at the performance point for X & Y-directions for concrete frame technique

Concrete frame GF									
X-direction									
Step	Disp. (m)	V (kN)	B to IO	IO to LS	LS to CP	CP to C	C to D	D to E	Beyond E
22	0.083	6365	91	59	0	0	0	0	0
Y-direction									
Step	Disp. (m)	V (kN)	B to IO	IO to LS	LS to CP	CP to C	C to D	D to E	Beyond E
9	0.085	6533	149	35	8	0	0	0	0

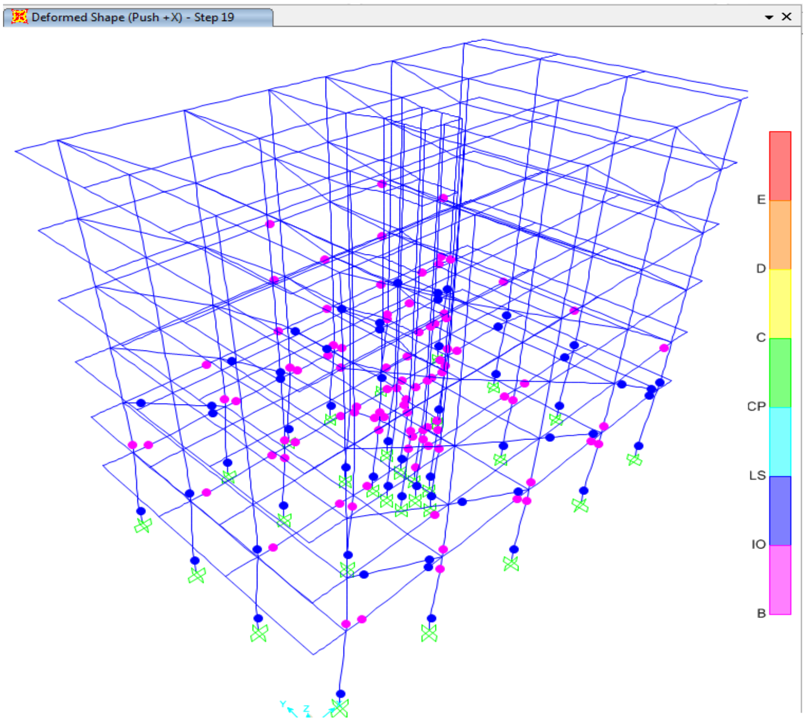

Figure 5.6.3. State of the model at the performance point with concrete frame retrofitting tech. for GF, [X-dir.].

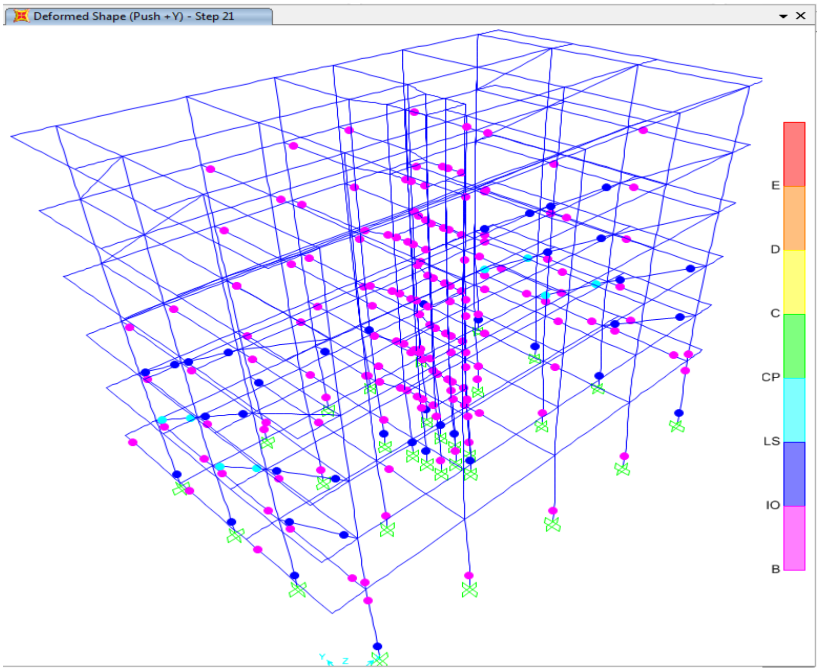

Figure 5.6.4. State of the model at the performance point with concrete frame retrofitting tech. for GF, [Y-dir.].

2- Concrete jacket technique for ground floor:

X-direction

The performance point is located between the two steps of (3-4) with Sa equal to 0.26g and Sd equal to 0.066m.

Y-direction

The performance point is located between the two steps of (13-14) with Sa equal to 0.215g and Sd equal to 0.077m.

The following two tables display detailed data for the step of the performance point.

Table 5.6.3. Capacity and demand curves data at the performance point for X & Y-directions for concrete jacketing technique.

Concrete jacket GF									
Capacity curve			Capacity and demand curve						
Step	Disp (m)	V (kN)	Teff	Beff	Sd Cap. (m)	Sa Cap.	Sd Dem. (m)	Sa Dem.
X-direction									
3	0.081	6956	0.958	0.167	0.059	0.259	0.066	0.292
4			1.022772	0.197624	0.069366	0.266947	0.066928	0.257565
Y-direction									
13	0.093	5623	1.168	0.193	0.071	0.209	0.077	0.227
14			1.224	0.211	0.080	0.215	0.078	0.209

Table 5.6.4 Numbers and types of plastic hinges at the performance point for X & Y-directions for concrete jacketing technique

Concrete jacket GF									
X-direction									
Step	Disp. (m)	V (kN)	B to IO	IO to LS	LS to CP	CP to C	C to D	D to E	Beyond E
4	0.081	6956	106	54	0	0	0	0	0
Y-direction									
Step	Disp. (m)	V (kN)	B to IO	IO to LS	LS to CP	CP to C	C to D	D to E	Beyond E
14	0.093	5623	152	55	8	0	0	0	0

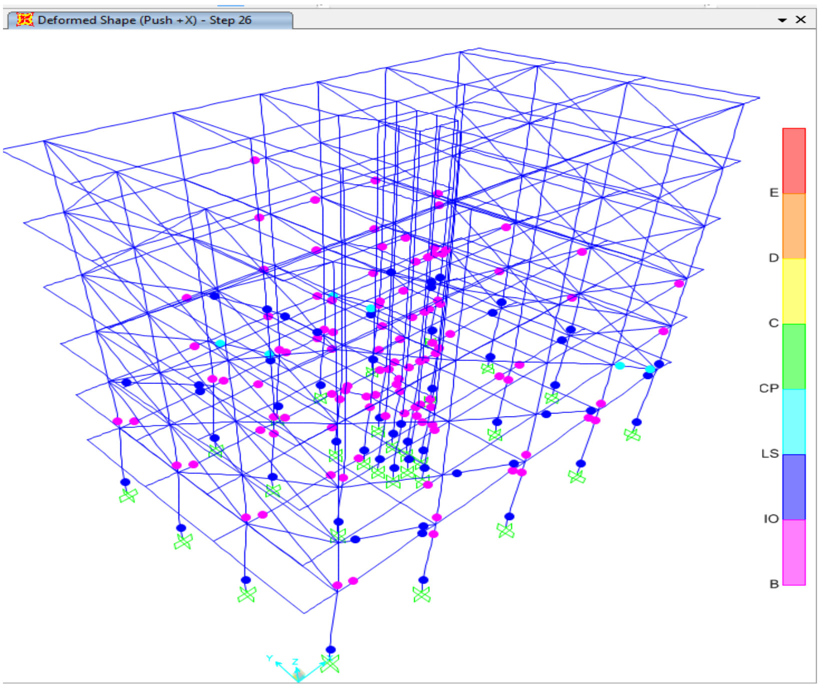

Figure 5.6.5. State of the model at the performance point with concrete jacketing tech. for GF, [X-dir.].

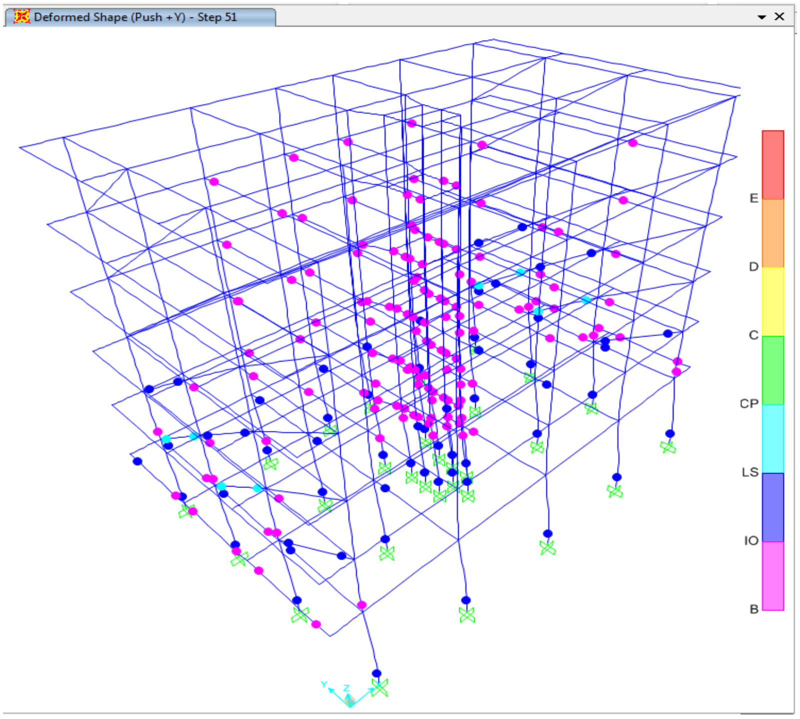

Figure 5.6.6. State of the model at the performance point with concrete jacketing tech. for GF, [Y-dir.].

3- Concrete frame technique for ground and first floors:

X-direction

The performance point is located between the two steps of (11-12) with Sa equal to 0.26g and Sd equal to 0.071m.

Y-direction

The performance point is located between the two steps of (9-10) with Sa equal to 0.20g and Sd equal to 0.071m.

The following two tables display detailed data for the step of the performance point.

Table 5.6.5. Capacity and demand curves data at the performance point for X & Y-directions for concrete frame technique for GF+1stF.

Concrete frame GF+1st								
Capacity curve			Capacity and demand curve					
Step	Disp (m)	V (kN)	Teff	Beff	Sd Cap. (m)	Sa Cap.	Sd Dem. (m)	Sa Dem.
X-direction								
11	0.085	7070	1.010	0.163	0.067	0.265	0.070	0.279
12			1.061	0.181	0.0777	0.277	0.071	0.255
Y-direction								
9	0.086	5251	1.188	0.247	0.071	0.204	0.071	0.203
10			1.207	0.250	0.074	0.205	0.07	0.198

Table 5.6.6. Numbers and types of plastic hinges at the performance point for X & Y-directions for concrete frame technique for GF+1stF.

Concrete frame GF+1st									
X-direction									
Step	Disp. (m)	V (kN)	B to IO	IO to LS	LS to CP	CP to C	C to D	D to E	Beyond E
12	0.085	7070	110	44	16	0	0	0	0
Y-direction									
Step	Disp. (m)	V (kN)	B to IO	IO to LS	LS to CP	CP to C	C to D	D to E	Beyond E
10	0.086	5251	133	41	0	0	0	0	0

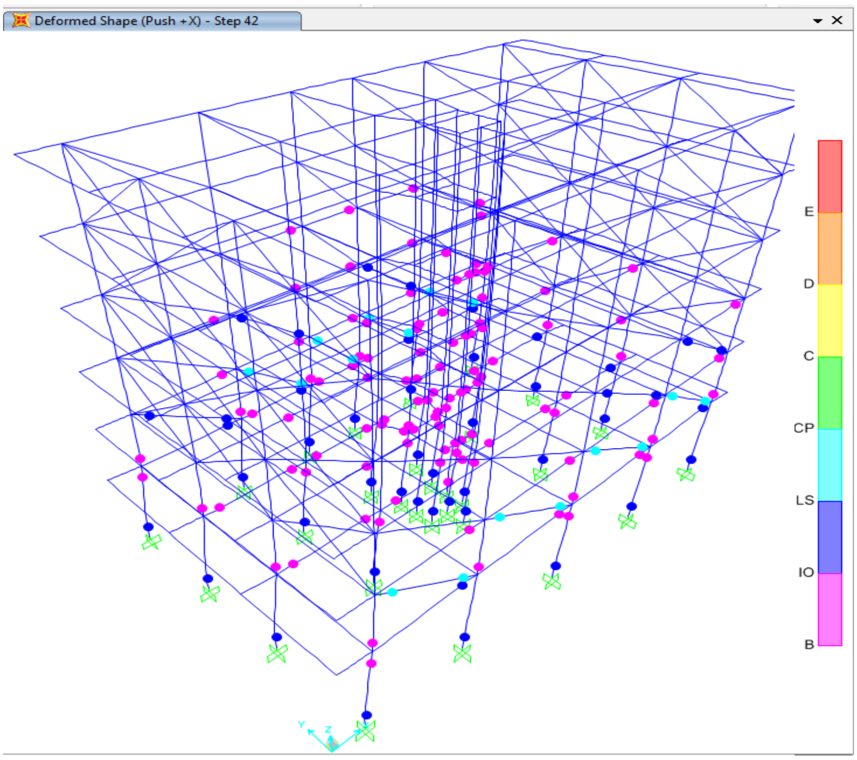

Figure 5.6.7. State of model at the performance point with the conc. Fr. tech. for GF & 1stF, [X-dir.].

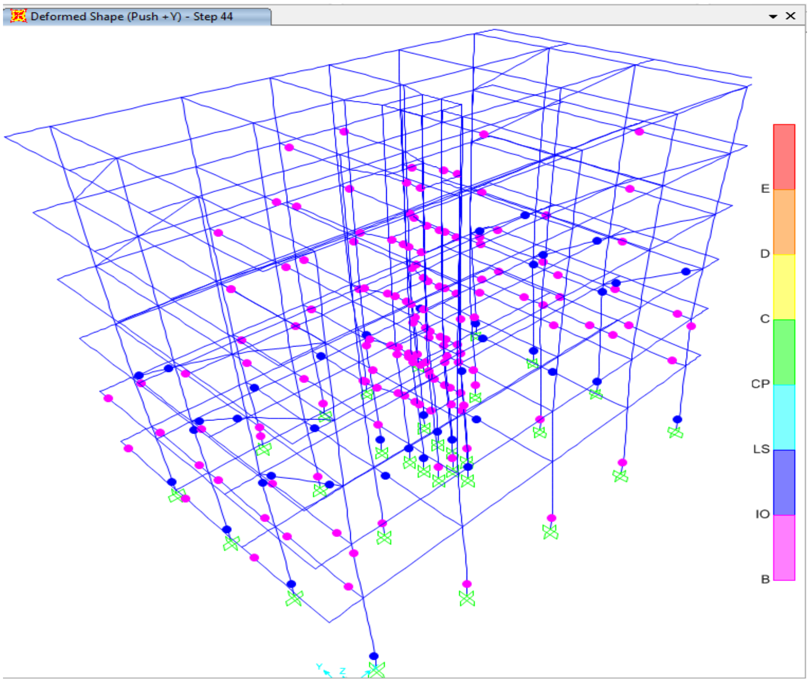

Figure 5.6.8. State of model at the performance point with the conc. Fr. tech. for GF & 1stF, [Y-dir.].

4- Concrete jacketing technique for ground and first floors:

X-direction

The performance point is located between the two steps of (2-3) with Sa equal to 0.33g and Sd equal to 0.07m.

Y-direction

The performance point is located between the two steps of (86-87) with Sa equal to 0.158g and Sd equal to 0.096m.

The following two tables display detailed data for the step of the performance point.

Table 5.6.7. Capacity and demand curves data at the performance point for X & Y-directions for concrete jacketing technique for GF+1stF.

Concrete jacket GF+1st								
Capacity curve			Capacity and demand curve					
Step	Disp (m)	V (kN)	Teff	Beff	Sd Cap. (m)	Sa Cap.	Sd Dem. (m)	Sa Dem.
X-direction								
2	0.086	8004	0.784	0.065	0.033	0.218	0.072	0.475
3			0.976	0.154	0.078	0.331	0.069	0.294
Y-direction								
86	0.11	4186	1.539	0.230	0.092	0.156	0.094	0.161
87			1.569	0.233	0.097	0.158	0.096	0.157

Table 5.6.8. Numbers and types of plastic hinges at the performance point for X & Y-directions for concrete jacketing technique for GF+1stF.

Concrete jacket GF+1st									
X-direction									
Step	Disp. (m)	V (kN)	B to IO	IO to LS	LS to CP	CP to C	C to D	D to E	Beyond E
3	0.086	8004	107	41	18	0	0	0	0
Y-direction									
Step	Disp. (m)	V (kN)	B to IO	IO to LS	LS to CP	CP to C	C to D	D to E	Beyond E
87	0.11	4186	79	98	6	0	0	0	0

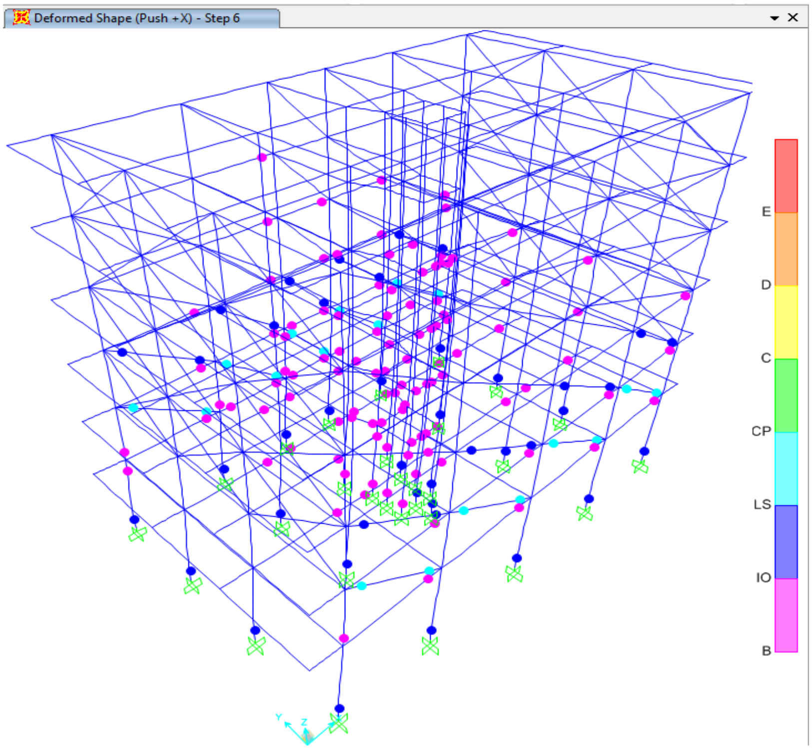

Figure 5.6.9. State of the model at the performance point with conc. jacket. Tech. for GF & 1stF, [X-dir.].

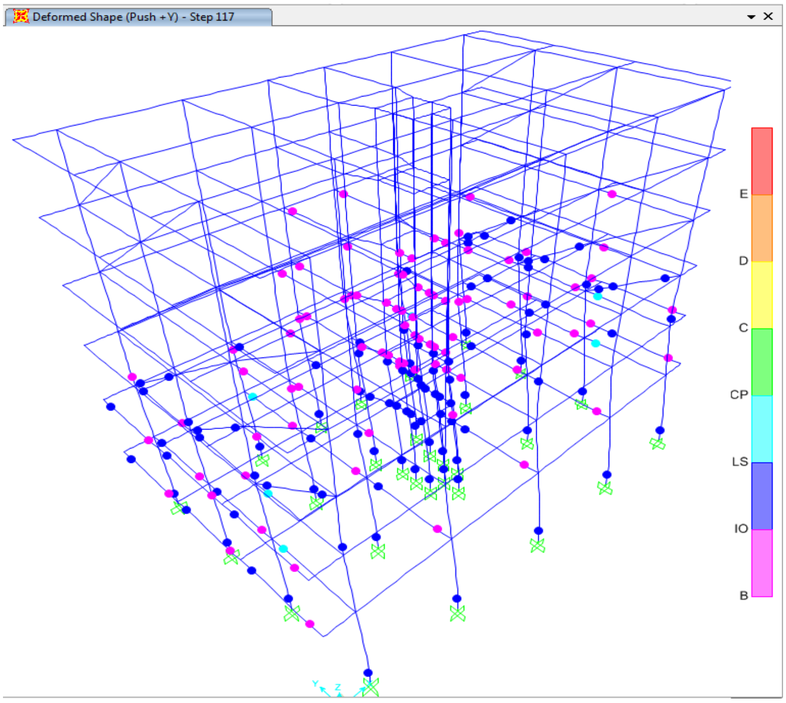

Figure 5.6.10. State of the model at the performance point with conc. jacket. Tech. for GF & 1stF, [Y-dir.].

5.7 Assessment of retrofitted case study

The retrofitted structure should be assessed through two procedures:

A- Linear assessment:

Linear assessment was done in section (5.4.5) and included:

1- Capacity to carry ultimate gravity loads as per design codes.

2- Serviceability (allowable deflection under service gravity loads)

3- Horizontal irregularity

4- Vertical irregularity

5- P-delta effect

6- Modal shapes

The assessments in (5.4.5) indicates that the retrofit techniques are within allowable limits except the fourth technique that is concrete jacket for ground and first floor. This technique did not eliminate the torsional mode. The cause of this deficit is the removal of bracing system (masonry walls) in the Y-direction in order to apply concrete jackets. These bracings were holding the structure from torsional effect. Still, this technique was assessed through the nonlinear pushover analysis in order to study its effect on ductility and strength of the structure with retrofitted columns.

B- Nonlinear assessment

The following criteria are used for the assessment:

1- Allowable drift resulted from pushover analysis

2- Number, location, and type of plastic hinges

The results of the nonlinear analysis shows that the drifts in all techniques are within allowable limit of life safety according to FEMA 356 which is 31cm. The maximum drift was in Y-direction from the analysis of the jacket technique for ground floor and jacket technique for ground and first floors, which were (11.5-14.4cm) respectively, which are less than the limit in FEMA 356.

The plastic hinges should be assessed according to different considerations, which are:

1- The type of plastic hinge and its location:

No collapse hinges (C-D) are allowed to appear at performance point, and if there are collapse hinges it should not be on critical structural elements such as columns, girders, or main beams.

No collapse hinges (C-D) appeared at the performance point at each model of the four techniques. All of the techniques had (LS-CP) hinges, except the concrete framing technique for GF, which had only IO-LS hinges, which makes it more desired.

2- Distribution of plastic hinges:

They should be distributed uniformly in the horizontal plan. The figures that show the plastic hinges at performance point indicates that the hinges are indeed distributed uniformly on each horizontal plane.

The figures also show that the hinges had spread to the upper floors, which means that the structure has better stiffness and strength distribution after retrofitting than before.

3- The continuity of pushover curve beyond performance point:

This is an important assessment point, which indicates improvement in ductility behavior of the structure. Some structural elements such as unreinforced masonry walls are brittle and have sudden failure. Therefore, the curve should extend after the performance point to guarantee that there is no sudden failure.

Therefore, the only retrofit technique that meets the criteria in the two directions is the "concrete framing technique for ground and first floors".

Therefore, the technique that can give the best behavior of the proposed techniques is the "concrete framing technique for ground and first floor", which passed the eight tests.

CH 6 Summary and Conclusions

6.1 Summary

In this thesis, a line of thinking concerning seismic assessment and retrofitting of existing buildings has been presented and demonstrated through a case study building. The assessment technique makes use of nonlinear static pushover procedure. The selected case study building is a typical six story RC building, with 14m width, 24m length, and 21m height. The building was built in the mid-2000s. The building was designed to carry ultimate gravity loads and satisfies ACI 318-14 for gravity loads. The external walls are unreinforced masonry walls. The ground floor consists of stores with large gates on the east-west sides and very wide windows with small heights. These conditions made the masonry wall of little effect on the ground floor in lateral loading capacity. This produced vertical irregularity in the building. The selected building represents many buildings common in shape and function to Nablus district.

Because of this irregularity, the building was assessed using nonlinear static procedure using SAP2000 program in order to predict the seismic behavior of the building under lateral seismic loads. Many scenarios were adopted to arrive at a practical and numerically efficient model for the building. The scenarios include assuming rigid diaphragm instead of using shell slab area sections and using bracing elements to model the masonry walls. The results showed that the assumptions of using rigid diaphragm and neglecting area elements of slab gives reasonably similar behavior to that from considering area elements as flexible elements. Since this assumption made calculations more efficient, it was adopted for nonlinear modeling of the building. The results also confirmed that the building has deficit in vertical irregularity and needs retrofitting.

The seismic evaluations were conducted based on FEMA 356 performance criteria as mentioned in chapter two. According to FEMA 356, the basic safety objective must meet the life safety (LS) performance in basic safety earthquake 1(BSE-1) hazard level, and must meet the collapse prevention (CP) in basic safety earthquake 2 (BSE-2) hazard level. BSE-1 is defined as the smaller of an event corresponding to 10% probability of exceedance in 50 years (10% in 50 years) and 2/3 of BSE-2, which is the 2% probability of exceedance in 50 years (2% in 50 years) event.

Since the building needs retrofitting due to soft story and torsional mode effects, two available retrofitting techniques were adopted and applied, which are concrete framing and concrete jacketing techniques. These techniques were added to the columns located on (B) and (J) axes. First, the ground floor was considered alone, and the second case, the ground floor and first floor were considered.

The nonlinear results show that the concrete framing technique for ground and first floors has produced the best performance and satisfied linear and nonlinear analysis criteria requirements. Moreover, the formed hinges types do not exceed (IO-LS) limit at the performance point of the building.

6.2 Conclusions

Based on the study and results obtained in the thesis, the following conclusions can be drawn:

1- The pushover is an essential tool for studying the realistic and nonlinear behavior of buildings. It gives a live picture on the sequence of plastic hinges formation and development of local failure in buildings. It however, must be used carefully and its limitations must be understood before conducting the analysis.

2- There is a lack of systematic studies that provide practical "know-how" guidelines for local engineers on the assessment and retrofitting of existing buildings against seismic loads. Generally, the guidelines written in foreign codes (e.g. the ASCE or FEMA) are very broad and general and may pose a challenge to local engineers regarding the consistency of their implementation. This study bridges this gap between local engineers and international codes by putting these guidelines into action through a practical case study.

3- The existence of masonry walls increases the rigidity of the building significantly. On the other hand, their presence may be a cause for soft story irregularity.

4- In this case study building and similar reinforced concrete structures, the assumption of rigid diaphragm and the exclusion of the shell area elements for the floors seemed to provide reasonably accurate behavior in the inelastic range when compared to a model that directly considers the area elements in floor. This is very important, because the rigid diaphragm assumption can significantly improve convergence of the 3D model in the plastic range.

5- When using frames without bracing system, the ductility increases, while the stiffness and strength decrease.

6- The formation of plastic hinges are a good indication of the behavior of the building in earthquakes in different terms such as type, location, and distribution of plastic hinges. The plastic hinges indicate the weak elements and integrity of the building through the distribution.

7- Concrete Jacketing technique increases the strength and stiffness but does not alleviate torsional effect. The concrete framing technique provides better alternative for reducing torsional effects.

8- Even if the performance point satisfies the wanted level of performance, the continuity of the pushover curve after performance point may indicates better ductility performance in the building.

6.3 Recommendations and possible future researches

This study gives a practical point of view on how to use nonlinear static pushover for assessment and retrofitting of building. The prime recommendation from this study is to be careful when using the pushover analysis, and to always check for limitations and validity of the assumptions made for the analysis. For instance, if there is torsional behavior in the building, it must be either accounted for in the definition of the plastic hinges and load pattern or be avoided altogether. Possible future researches related to this study include:

1- The study can be extended to cover other types of retrofitting techniques such as polymer composites.

2- Soil structure interaction was ignored in this study.

3- Comparison to nonlinear dynamic time history analysis of the building

4- Effect of elevator shaft and stair case walls and their importance and their possible effect in causing fundamental torsional mode.

5- This thesis can be considered a step in a full methodology for assessing and retrofitting existing vulnerable buildings.

The methodology may include the following general points:

a- Rapid visual screening RVS to identify vulnerable buildings

b- The processes of analysis, assessment, and retrofitting must be iterational in order to improve the quality of existing buildings and to increase the capacity of seismic loads resistance.

c- Good implementation of the technique is necessary to guarantee efficient retrofit.

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Appendices

Appendix A. Verification of 3D model

1- 3D model verification

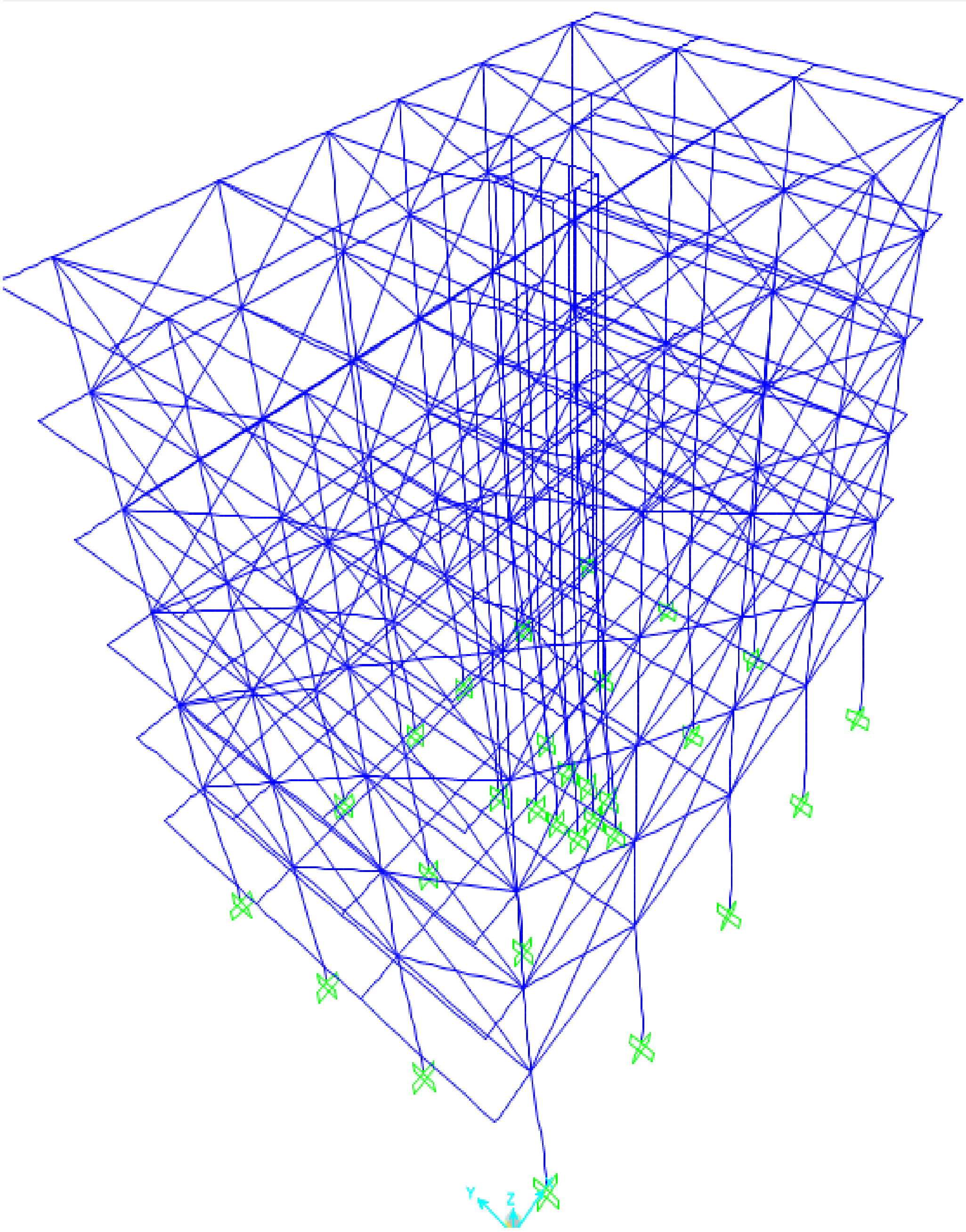

Figure A.1.1. Deformed 3D-model from gravity loads on SAP2000 software.

In order to verify the model elastically, you need to determine and calculate manually the following things:

a. Compatibility:

   Which means that all the structural members are connected well as assumed. In addition, it can be shown through the deformed shape and starting animation in the program.

b. Equilibrium:

   It can be approved by calculating the weight of structural elements and assigned loads then compare it with the base shear reaction.

c. Stress-strain relationship:

   It can be approved by calculating the moments and deformations manually and compare them to the program results.

d. Elastic period of the structure:

   This can be achieved by calculating effective mass and flexural stiffness for each floor. Then converting the MDOF system into Eq. SDOF system to form equation of motion then through Rayleigh method.

a. Compatibility is achieved through animating the model.

b. Equilibrium:

Weights of structural elements

Table A.1.1: Dimensions of structural elements

Name	Width (cm)	Depth (cm)
Columns of ground floor	75	30
Columns of 1st - 5th floor	70	25
Elevator shaft	Dim. C-C	Thickness
	1.8m X 1.8m	20cm

*Not: the beams weight are assumed to be part of the slab area self-load, which is mentioned in table (A.1.2), which is 4.22 kN/m^2

TableA.1.2. Loads ratios and their positions

Concrete	ɣ (kN/m3)	25
Slab	Self-Load (kN/m2)	4.22
	SIDL (kN/m2)	3.50
	Room & Corridors Area (m2) (14m*21m)	290.00
	Balconies Area (m2) (1.5m*14m+1.5m*14m)	42.00
	Roof Area (m2) (14m*24m)	332
Columns	No.	22
	GF col. Vol. (m3) (0.3*0.75*4.75)	1.07
	Above col. Vol. (m3) (0.25*0.7*3.25)	0.57

Elev. Shaft	Perimeter (m)	6.2
	GF Volume (6.2*0.2*4.75)	5.89
	Above Floors Volume (6.2*0.2*3.25)	4.03
Masonry wall (kN/m)	Room & Corridor Line Load (kN/m)	20.75
	Room & Corridor Perimeter (m) (14m*2+21m*2)	70
	Balconies. & Roof line load (kN/m)	10.375
	balconies Perimeter (m) (14m*2+1.5m*4)	34
	roof Perimeter (m) (14m*2+24m*2)	76
Live load	rooms & corridors wt. (kN/m2)	2
	Balc.& Roof wt. (kN/m2)	3

Table A.1.3. Manual calculation for weights and their positions in (kN) unit

	Self-weights and added loads					Service Dead Loads	Live load	Serv. Tot. Loads	Ult. Loads
	Slab self	Slab SID	Columns	Elev. Shaft	Masonry walls				
GF	1401.04	1162.00	587.81	147.25	1805.25	**5103.35**	**706.00**	**5809.35**	**7253.62**
F1	1401.04	1162.00	312.81	100.75	1805.25	**4781.85**	**706.00**	**5487.85**	**6867.82**
F2	1401.04	1162.00	312.81	100.75	1805.25	**4781.85**	**706.00**	**5487.85**	**6867.82**
F3	1401.04	1162.00	312.81	100.75	1805.25	**4781.85**	**706.00**	**5487.85**	**6867.82**
F4	1401.04	1162.00	312.81	100.75	1805.25	**4781.85**	**706.00**	**5487.85**	**6867.82**
F5 (Roof)	1401.04	1162.00	312.81	100.75	788.50	**3765.10**	**996.00**	**4761.10**	**6111.72**
SUM	**8406.24**	**6972.00**	**2151.88**	**651.00**	**9814.75**	**27995.87**	**4526.00**	**32521.87**	**40836.64**

Table A.1.4. Summary of loads in ND-A-B model (with slab area sections with bracing)

Base Reactions

File View Format-Filter-Sort Select Options

Units: As Noted

Filter:

OutputCase Text	CaseType Text	GlobalFX KN	GlobalFY KN	GlobalFZ KN
DEAD	LinStatic	-1.162E-11	-1.393E-10	11202.143
DEAD SIL	LinStatic	-9.241E-12	-1.107E-10	6972
DEAD Masonry	LinStatic	-1.043E-11	-1.325E-10	9814.75
Live	LinStatic	-6.729E-12	-8.025E-11	4526
Service dead loads	Combination	-3.13E-11	-3.825E-10	27988.893

Table A.1.5. Summary of loads in D-NA-B model (with diaphragm. constraints with bracing)

Base Reactions

File View Format-Filter-Sort Select Options

Units: As Noted

Filter:

OutputCase Text	CaseType Text	GlobalFX KN	GlobalFY KN	GlobalFZ KN
DEAD	LinStatic	-4.22E-14	9.516E-13	10999.307
DEAD SIL	LinStatic	-1.612E-14	7.183E-13	6821.402
DEAD Masonry	LinStatic	-9.772E-14	1.184E-12	10831.5
Live	LinStatic	-9.975E-15	6.576E-13	4484.012
Service DL	Combination	-1.56E-13	2.853E-12	28652.209

"DEAD" in table (A.1.4) and (A.1.5) in base reaction table represents self-weight of structural element that includes **slabs**, **columns**, **elevator shaft**. **"Service dead loads"** or **"Service DL"** in base reaction table represents service dead loads of structural element and added dead loads that includes "**DEAD" loads**, **super imposed dead load**, and **masonry walls loads.**

% Error in loads of ND-A-B model (with slab area sections and with bracing) with respect to manual calculations

$$\%\text{ error of service dead loads} = \frac{(27995.8 - 27988.9)}{27995.8} = 0\%$$

$$\%\text{ error of live loads} = \frac{(4526 - 4526)}{4526} = 0\%$$

% Error in loads of D-NA-B model (with diaphragm constrains and with bracing) with respect to manual calculations

$$\%\text{ error of service dead loads} = \frac{(28652.2 - 27988.9)}{27995.8} = 2.3\%$$

$$\%\text{ error of live loads} = \frac{(4526 - 4484)}{4526} = 0.9\%$$

The loads of the two model don't differ from calculated loads

c. Stress strain relationship will be verified in the next verification of plastic hinges.

d. Elastic period:

In order to calculate the period of the structure, the mass matrix of the structure floors are calculated in equilibrium step. But the Stiffness of each floor needs to be calculated and verified.

The manual calculations of stiffness for each floor will be compared to program SAP2000 results by these steps:

1) Calculate the sum of (I/L^3) for each floor in each direction.

2) In order to represent the masonry walls as a bracing elements, the bracing element must have the same stiffness of masonry wall. Bracing elements are verified through SAP 2000 by the following steps:

   Diagonal bracing elements will replace the masonry walls for these reasons:

- Program cannot show plastic stage in area elements.
- Reducing thousands of area elements and degrees of freedom by two bracing element elements, which increase efficiency.

  In order to achieve diagonal bracing elements, many unknowns need to be found and verified such as:

- Dimensions of beams.
- Characteristics of beams materials.
- Release conditions.
- Representative wall dimensions and window opening dimension.
- Represent plastic stage and failure of masonry wall.

  Since the masonry walls are unreinforced, many assumptions were adopted:

- According to (Figure 3.2.2), the concrete thickness was 18cm and 5cm for stone thickness. Therefore, 20cm beam thickness was adopted.
- Depth of bracing elements will be taken tenth the diagonal length (0.5m)
- The representative wall is 4m long and 3m height, and the window are 2m long and 1.5m height.
- The material of bracing elements are (f'c = 14 MPa) and elastic modulus will be equal to the value proposed by FEMA 356, which equals to (550*f'c), where f'c is limited to 6.5MPa (900psi) and Em = 3413MPa (495000psi). but in case study there are two reinforced lentils under and above window opening and f'c of used concrete is not less than (14 MPa). So the elastic modulus will be calculated through analytical tests.
- The main idea in representing reality is to show plastic hinge appearance in area elements. Therefore, many steps is done:

a) The last (0.5m) of the masonry area perimeter will be modeled as beams elements. Also, on the window opining perimeter.

b) Columns of (0.7*0.25m) and beams of (0.75*0.25m) forming closed frame columns are in the weak direction of frame.

c) Four models will be built to simulate reality:

1) Area elements of 0.2m thickness and 0.25m mesh with 0.25m beams boundary representing plastic stage by brittle hinges. With two square lentils of 0.2m.

2) Beams of 0.2m thickness and 0.25m width making 0.25m mesh. With two square lentils of 0.2m.

3) Bracing elements of 0.2m thickness and 0.5m depth with brittle hinges.

4) Without masonry wall effect.

Figure (A.2.1) shows frame dimensions.

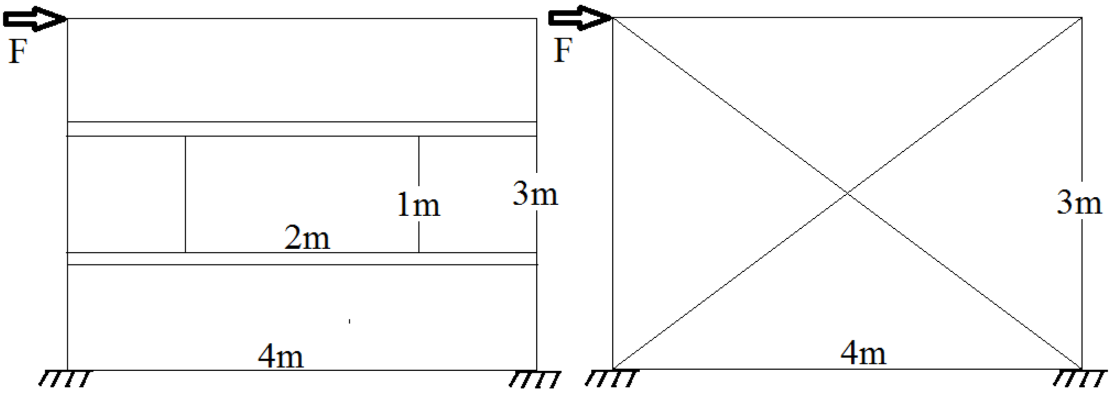

Figure A.1.2. Typical masonry wall dimensions

The results are:

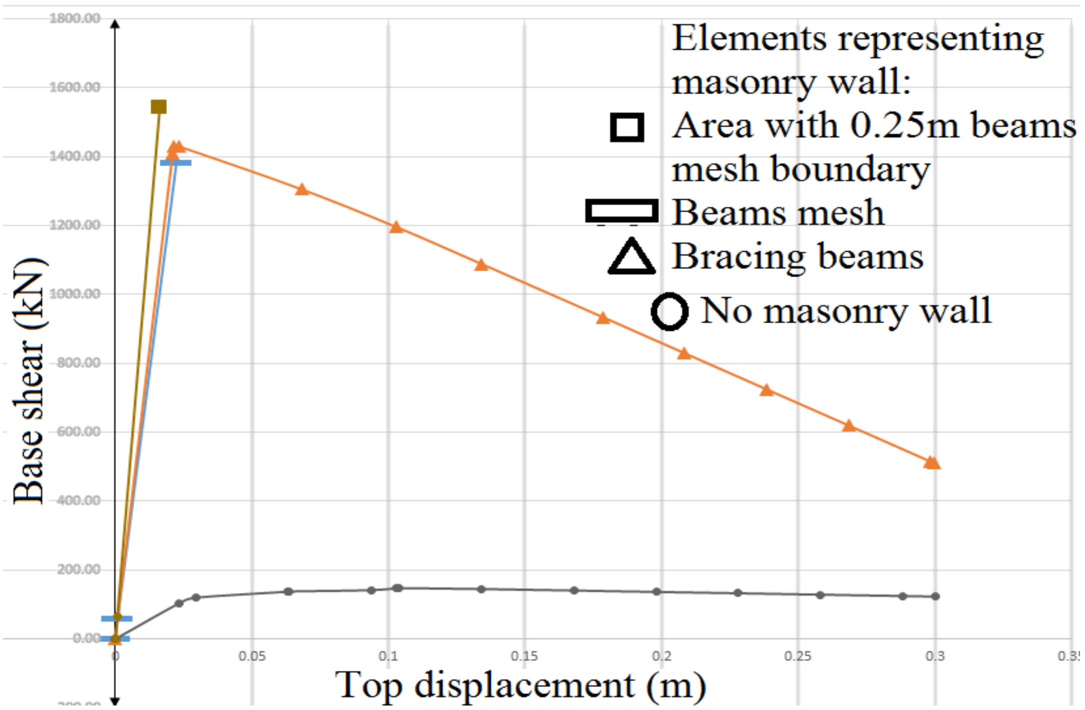

Figure A.1.3. Results of the four models

This means that the bracing elements can represent masonry walls

Also, the bracing elements will be considered in compression only. Its stiffness is considered in two forms and that are axial strain stiffness and side cross section stiffness and calculated like this:

Side cross section stiffness = I/L^3 * F^1

Axial strain stiffness = A/L * F^2

I: moment of inertia   L: bracing element length (6m) A: cross section area

F^1: sin33 = 0.54

F^2: cos33= 0.83

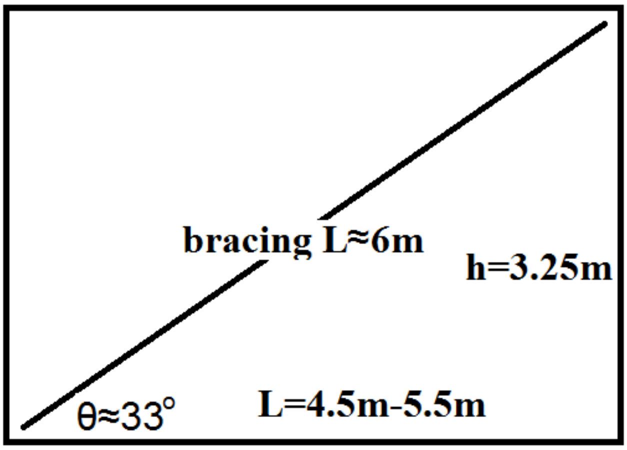

Fig. A.1.4. Bracing element assumption

3) The stiffness of the columns and elevator shaft parts in each floor range from $\frac{3EI}{L^3}$ to $\frac{12EI}{L^3}$.
4) In order to find the stiffness of each floor in the model, the joints of the first floor were restraint as fixed and the face column-beam-connection joints of the ground floor were subjected to (200 kN), once in X-direction and the second time in Y-direction and record the displacement of the center of mass.

Repeat the process but the joints of the ground floor and the second floor are restrained as fixed, and the (200 kN) are subjected to the face column-beam-connection joints of the first floor and record the displacement of the center of mass.

The approximate stiffness can be achieved by dividing the load (200 kN) over the displacement

5) Compare the results.

Table A.1.7 calculating stiffness for the floors of case study building, X-Direction.

Ground Floor								
X-Direction								
Name	Width	Length	Height	I	I/L^3	Factor	Item No.	∑(EI/L^3)
GF 1	0.3	0.75	4.5	0.0105	0.00012	1	18	47969.175
GF 2	0.75	0.3	4.5	0.0017	1.9E-05	1	4	1705.570667
Elev. Corn. 1	*	*	4.5	0.0031	3.4E-05	1	2	1580.242807
Elev. Corn. 2	*	*	4.5	0.0031	3.4E-05	1	2	1566.598242
Elev. Straight 1	0.2	1	4.5	0.0167	0.00018	1	1	4211.285597
Elev. Straight 2	1	0.2	4.5	0.0007	7.3E-06	1	2	336.9028477
Ecol.	23025204							
Ebr.	5.01E+06						SUM	57369.8
1st - 5th Floor with Bracing								
X-Direction								
Name	Width	Length	Height	I	I/L^3	Factor	Item No.	∑(EI/L^3)
GF 1	0.25	0.75	3	0.0088	0.00033	1	18	134913.3047
GF 2	0.75	0.25	3	0.0010	3.6E-05	1	4	3331.192708
Elev. Corn. 1	*	*	3	0.0031	0.00012	1	4	10666.63895
Elev. Corn. 2	*	*	3	0.0031	0.00011	1	4	10574.53813

Elev. Straight 1	0.2	1	3	0.0167	0.00062	1	1	14213.08889
Elev. Straight 2	1	0.2	3	0.0007	2.5E-05	1	2	1137.047111
Bracing	0.2	0.5	6	0.0021	9.6E-06	0.110	8	4.25E+01
							SUM	174878.4
E col.	23025204			A	A/L			EA/L
Axial	0.2	0.5	6	0.1000	0.01667	0.014	8	9.13E+03
E br.	5.01E+06							

Table A.1.8. Calculating stiffness for the floors of case study building, Y-Direction.

Ground Floor								
Y-Direction								
Name	Width	Length	Height	I	I/L^3	Factor	Item No.	∑(EI/L^3)
GF 1	0.75	0.3	4.5	0.0017	1.9E-05	1	18	7675.068
GF 2	0.3	0.75	4.5	0.0105	0.00012	1	4	10659.81667
Elev. Corn. 1	*	*	4.5	0.0031	3.4E-05	1	4	3160.485615
Elev. Corn. 2	*	*	4.5	0.0031	3.4E-05	1	4	3133.196484
Elev. Straight 1	1	0.2	4.5	0.0007	7.3E-06	1	1	168.4514239
Elev. Straight 2	0.2	1	4.5	0.0167	0.00018	1	2	8422.571193
Ecol.	23025204							
Ebr.	5.01E+06						SUM	33219.6
2nd - 5th Floor with Bracing								
Y-Direction								
Name	Width	Length	Height	I	I/L^3	Factor	Item No.	∑(EI/L^3)
GF 1	0.75	0.25	3	0.0010	3.6E-05	1	18	14990.36719
GF 2	0.25	0.75	3	0.0088	0.00033	1	4	29980.73438
Elev. Corn. 1	*	*	3	0.0031	0.00012	1	4	10666.63895
Elev. Corn. 2	*	*	3	0.0031	0.00011	1	4	10574.53813

Elev. Straight 1	1	0.2	3	0.0007	2.5E-05	1	1	568.5235556
Elev. Straight 2	0.2	1	3	0.0167	0.00062	1	2	28426.17778
Bracing	0.2	0.5	6	0.0021	9.6E-06	0.110	6	3.19E+01
							SUM	95238.9
E col.	23025204			A	A/L			
Axial	0.2	0.5	6	0.1000	0.01667	0.014	6	6.85E+03
E br.	5.01E+06							

The resultant stiffness:

Table A.1.9. Calculating stiffness rigid.

GF	Kx	Ky
12EI/L^3	688437.3	398635.1
1st-5th	Kx	Ky
12EI/L^3+EA/L	2107675.1	1149717.8

Table A.1.10. Calculating stiffness from the model.

SAP					
	F	Dx	Kx	Dy	Ky
GF+1st	200	0.000081	2469135.8	0.000146	1369863
1st+2nd	200	0.0000527	3795066.4	0.000088	2272727

Table A.1.11. Comparing the stiffness results.

	Kx			Ky		
	Man	SAP	Diff %	Man	SAP	Diff %
GF+1st	2796112.4	2469135.8	11.7	1548352.9	1369863.0	11.5
1st+2nd	4.22E+06	3795066.4	10.0	2.30E+06	2272727.3	1.2

The results of SAP2000 are less than the manual values, and that because:

1- Beams have rotated due to stiffness difference of columns on the ground floor.
2- The calculation of bracing maybe not accurate enough.
3- The difference in reinforcement of beams play major rule in floor rigidity against columns rigidity especially in the Y-direction.

Period verification using Rayleigh method:

Table A.1.12. Rayleigh method (1)

Rayleigh method (with bracing) (k from SAP)					
Floor No.	F. Disp. Shape	F. mass	F. k-X-dir	F. k-Y-dir	L
6	1.00	383.80	1897533.207	1136363.636	1
5	0.95	487.45	1.90E+06	1.14E+06	1
4	0.88	487.45	1.90E+06	1.14E+06	1
3	0.79	487.45	1.90E+06	1.14E+06	1
2	0.68	487.45	1.90E+06	1.14E+06	1
1	0.49	520.22	5.72E+05	2.33E+05	1
0	0.00				

Table A.1.13. Rayleigh method (2)

Rayleigh method			
m	1855.72		
k-X-dir	258114.65	k-Y-dir	128449.6
L	2247.28		
ω-X-dir	11.79	ω-Y-dir	8.320
Tn-X-dir	0.53	Tn-Y-dir	0.76
Γ(L/m)	1.21		

Table A.1.14. Comparing the results.

	Ray.	SAP	Diff. %
Tn-X-dir	0.53	0.72	35.85
Tn-Y-dir	0.76	0.89	17.11

According to the previous two tables, the model period is larger than the calculated boundaries for the following reasons:

1- Beams have rotated due to stiffness difference of columns on the ground floor.

2- The calculation of bracing maybe not accurate enough.

3- The difference in reinforcement of beams play major rule in floor rigidity against columns rigidity especially in the Y-direction.

4- The axial stiffness of structural elements play major rule in increasing the periods

In order to form equation of motion

Table A.1.15. Response spectrum data.

UBC97	
Ca	0.28
Cv	0.4
I	1
R	3.5
Sa	T
0.080	0.000
0.200	0.114
0.200	0.571
0.152	0.750
0.123	0.929
0.103	1.107
0.089	1.286
0.078	1.464
0.070	1.643
0.063	1.821
0.057	2.000
Stiff Soil	

Table A.1.16. Rayleigh method with base shear.

Rayleigh method			
m	1855.72		
k-X-dir	258114.65	k-Y-dir	128449.56
L	2247.28		
ω-X-dir	11.79	ω-Y-dir	8.320
ω-X-dir^2	139.09	ω-Y-dir^2	69.22
Tn-X-dir	0.53	Tn-Y-dir	0.76
Γ(L/m)	1.21	ζ	5%
Sax	0.20	Say	0.15
Vx	5339.53	Vy	4031.3463

Appendix B. Verification of plastic hinge

The used plastic hinges in this thesis are:

a- Concrete M hinge for beams

b- Concrete P-M-M hinge for columns

c- Concrete P hinge for bracing elements (representing unreinforced masonry walls)

These types of hinges will be verified in properties and formation conditions and performance criteria used. In addition, the auto hinge option in SAP2000 program will be verified.

The verifications will be done through two models, which are 1D cantilever model and 2D frame model. A reinforced concrete RC section will be used for the two models. The models section and dimensions are displayed in Figures (A.2.1) and (A.2.2) respectively.

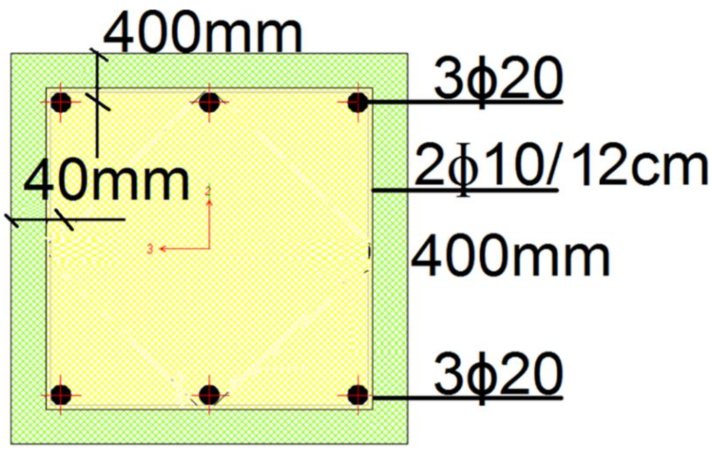

Fig. B.2.1. RC section used in the two models.

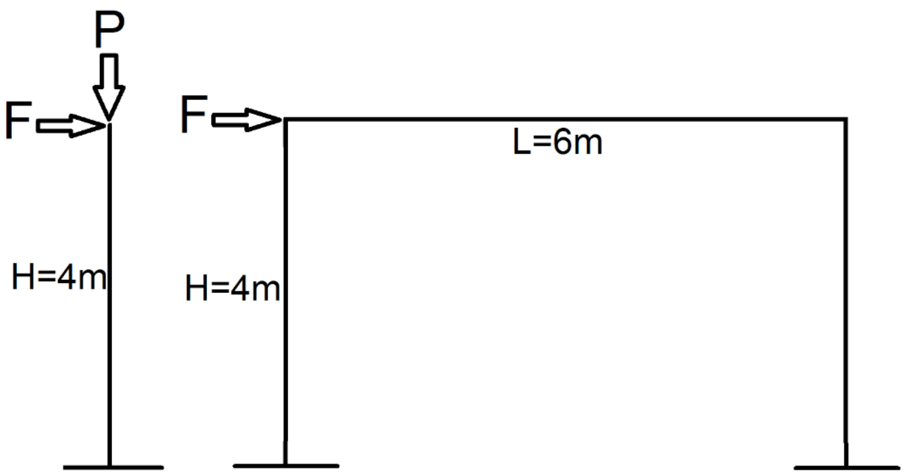

Figure B.2. 1D cantilever model and 2D frame model dimensions and loads directions.

The RC section properties are listed in Table (A.2.1).

Table B.1. RC section properties and capacities.

f'c MPa	**Fy MPa**	**Ec Mpa**	**Es Mpa**	**Es/Ec**	**β1**	**As (mm^2)**	**As' (mm^2)**	**b (mm)**	**h (mm)**
24	420	23025.2	200000	8.7	0.85	942.48	942.48	400	400
d (mm)	**d' (mm)**	**ρ**	**Mn (kN.m)**	**Pn (kN)**	**I (m^4)**	**Vc (kN)**			
360	40	0.0065	133	3213.6	2.133e9	117.6			

For more details about the values in table (A.2.1):

$$\mathbf{Ec} = 4700\sqrt{f'c} = 23025.2$$

$$\rho = \rho' = \frac{As}{b*d} = \frac{942.5}{400*360} = 0.00654$$

Compression steel can be neglected in calculations if $\rho < \rho_{0.005}$

$$\rho_{0.005} = \frac{0.375*\beta*0.85*f'c}{fy} = 0.015 > \rho \text{ compression steel can be neglected}$$

$$a = \frac{As*fy}{0.85*f'c*b} = \frac{942.5*420}{0.85*24*400} = 48.5\text{mm}$$

$$Mn = As * fy * (d – a/2) = 942.5*420 * (360 – 48.5/2) = 133 \text{ kN.m}$$

It is hard to calculate the yielding moment of reinforced concrete section, because there are many unknowns, such as fc, εc, and fs (for compression steel). Therefore, assumptions are adopted according to ACI code, where the compression strength of concrete at yielding point is about (0.7f'c), which equals to (C/(d-C)*0.0021). Also, the section here is tension controlled section ($\rho < \rho_b$) that tension steel starts yielding before compression concrete strain reaches (0.003) that means (fy = 420MPa, and εs value reaches (0.0021). see figure below.

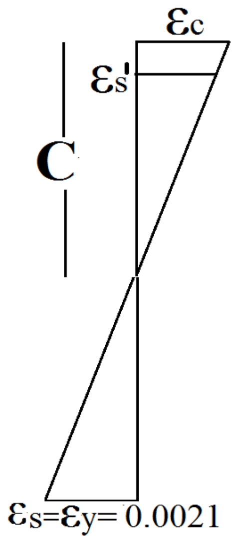

Figure B.3. Strain diagram at yielding point

From equilibrium:

$$Ts = Cc + Cs$$

$$Asfy = 0.5*0.7f'c*C*b + As'*fs$$

$$\frac{\varepsilon c}{C} = \frac{0.0021}{d-C} \rightarrow \varepsilon c = \left(\frac{0.0021}{d-C}\right) * C$$

$$\frac{\varepsilon c}{C} = \frac{\varepsilon s'}{C-d'} \rightarrow \varepsilon s' = \left(\frac{\varepsilon c}{C}\right) * (C-d')$$

$$fs = E * \varepsilon s'$$

By assuming C by trial, C = 93. Therefore, yielding moment is:

$$My = Cc * (d - C/3) + Cs * (d - d')$$

$$My = 127 \text{ kN.m}$$

Pn with accidental eccentricity factor (0.8)

$$Pn = 0.8(0.85*f'c*(Ag-As)+As*fy) = 0.8(0.85*24*(400^2-1880)+1880*420)$$

$$= 3213.6 \text{ kN}$$

$$I = 1/12*b*h^3 = 1/12 * 400^4 = 2.133e9 \text{ m}^4$$

$$Vc = 1/6 * \sqrt{f'c}\ b * d = 1/6 * \sqrt{24} * 400 * 360 = 117575N = 117.6 \text{ kN}$$

a- Concrete M hinge for beams (rotation hinge type)

In order to verify M3 hinge used in modeling case study by SAP2000 program, which was an automatic hinge selection option according to FEMA 356 tables, the following steps will be done to 1D cantilever model:

i. Manual selection of hinge properties from the table 2.4.3 in chapter two.

ii. Manual calculations of yielding and plastic moments will be done. Manual calculation of rotations at yielding, full plastic, FEMA 356 performance limits.

iii. Compare the results with program results.

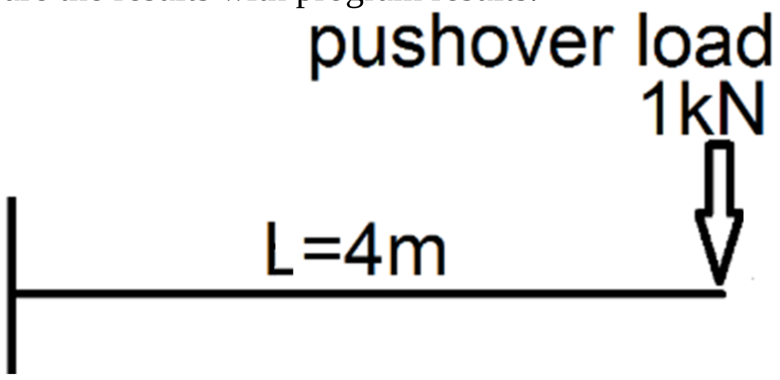

Figure B.4. 1D cantilever model for auto M3 hinge.

1- Manual selection of hinge properties from the table 2.4.3 according to certain limits

Since $\rho = \rho' = 0.00654$ then the first limit $\frac{\rho-\rho'}{\rho_{bal}} \leq 0.00$

The section is considered conforming because the hoops are spaced at $\leq$ d/3
The shear that causes yielding is

$$V = Mn/L = 127/4 = 32 \text{ kN}$$

$$\frac{V}{b_w d\sqrt{f_c'}} = \frac{32}{400*360*\sqrt{24}/1000} = 0.05 \leq 3$$

2- Manual calculations of yielding and plastic moments.

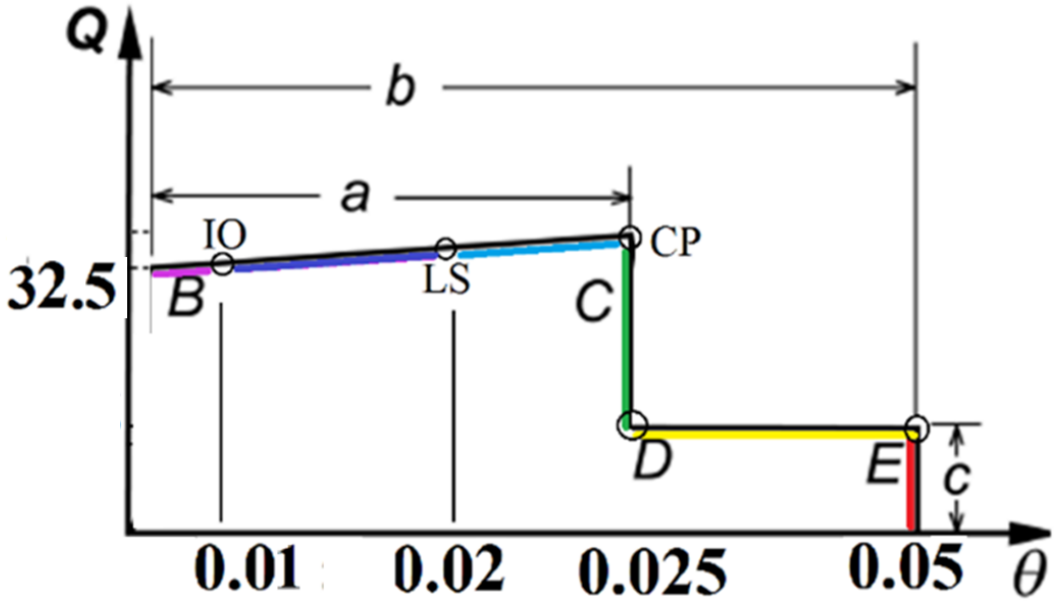

Figure B.5. Auto concrete M3 hinge from FEMA 356.

3- + 4- Manual calculation of rotations at yielding, full plastic, FEMA 356 performance limits

Table B.2. results of Manual vs. SAP2000 for auto concrete M3 hinge.

	Cantilever L=4m		Yielding	IO	LS	CP	C	D
	L (m)	My (kN.m)	Δ(mm)	0.01	0.02	0.025	0.025	0.05
Manual	4	128	14.1	54	94	114.46	114.46	214
SAP	4	128	14	54	94	115.4	115.4	--

$$\Delta \text{ (yielding)} = \frac{P*L^3}{3EI} = \frac{32000*4000^3}{323025.2*2.133e9} = 14 \text{ mm}$$

Δ (IO=0.01) = Δ (yielding) + L*(IO=0.01) = 14+4000*0.01 = 54 mm

b- Concrete P-M-M hinge for columns (rotation hinge type)

In order to verify P-M-M hinge used in modeling case study by SAP2000 program, which was automatic hinge selection option according to FEMA 356 tables, the following steps will be done to 1D cantilever model:

1- Manual selection of hinge properties from the table 2.4.4 in chapter two.
2- Manual calculations of yielding and plastic moments will be done.
3- Manual calculation of rotations at yielding, full plastic, and FEMA 356 performance steps.
4- Compare the results with program results.

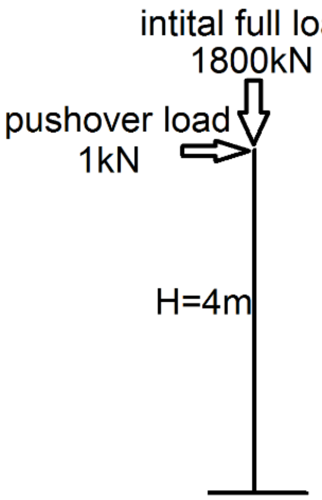

Figure B.6. 1D cantilever model for auto concrete P-M3 hinge.

1- Manual selection of hinge properties from the table 2.4.3 according to certain performance limits

In SAP program it takes two values for axial load to count the difference in axial load during pushover analysis once $\frac{P}{A_g f_c'} \leq 0.1$and the second $\frac{P}{A_g f_c'} \geq$ 0.4

The section is considered conforming because the hoops are spaced at ≤ d/3.

V is obtained from dividing the nominal moment by the column height, where Mn can be obtained from interaction diagram that is built by the program, which needs to be verified by three points, no axial load, no moments or eccentricity, and the balance point.

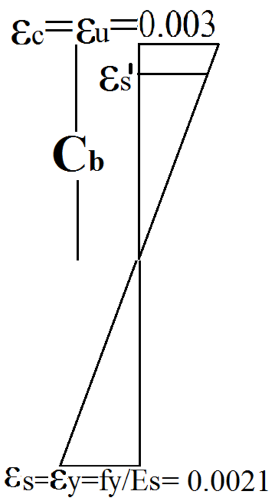

Figure B.7. strain distribution at balance point of interaction diagram.

The first and the second points are presented in Table A.1.1. The balance point can be obtained through the following equations:

$$Mn = Pn\ e = 0.85*fc'*a*b*(\frac{h}{2} - \frac{a}{2})+As'fs'(\frac{h}{2} - d')+Asfs(d - \frac{h}{2}) = 288 \text{ kN.m}$$

$$Pn= 0.85*fc'*a*b + As'fs' – Asfs = 1469$$

$$a = \beta_1 C_b = \beta_1*d*\frac{\varepsilon_u}{\varepsilon_u - \varepsilon_y} = 0.85*360*\frac{0.003}{0.003-0.0021} = 180mm$$

$$\varepsilon_y = fy/Es = 420/200000 = 0.0021$$

$$fs' = \varepsilon_u * Es * \frac{Cb-d'}{Cb} = 480 \leq 420 \text{ ??} \quad \text{take } fs' = 420MPa$$

SAP2000 interaction diagram:

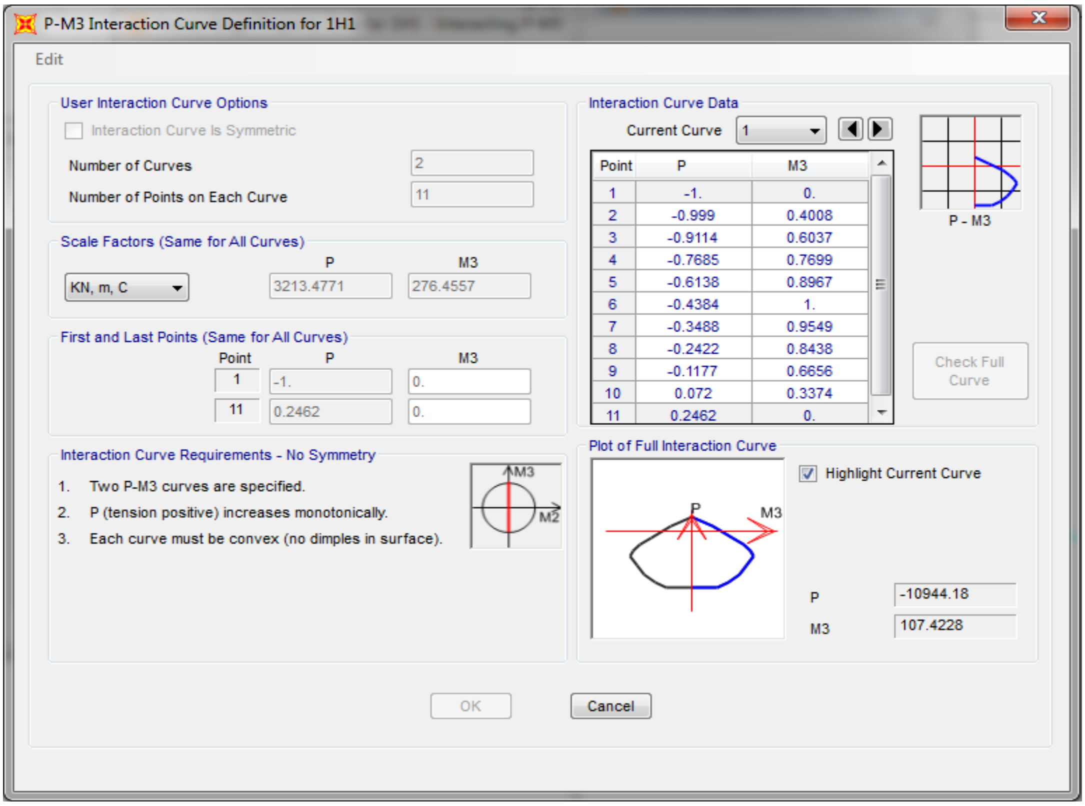

Figure B.8. interaction diagram presented in SAP2000.

The main three points of the interaction diagram in SAP2000 do not differ from the calculated points more than 5%, which means that the interaction curve can be accepted and the results are displayed as following:

Table B.3. The main three points on the interaction diagram.

	No moment or eccentricity		Balance point $\varepsilon_s = \varepsilon_y$		No axial force	
	Pn	Mn	Pn	Mn	Pn	Mn
Manual	3213.8	0	1469	288	0	128

SAP2000	3213.5	0	1409	276.5	0	128

The nominal moment that corresponds to the applied axial force (1800) is:

$$1800/3213.5=0.56$$

$$\frac{-0.6138+0.4384}{0.8967-1}=\frac{-0.6138+0.56}{X-1} \quad X= 0.9683 \quad Mn = 0.9683*276.5 = 267.7 \text{ kN.m}$$

The shear that causes yielding is smaller than

$$V = Mn/L = 267.7/4 = 66.9 \text{ kN}$$

$$\frac{V}{b_w d\sqrt{f_c'}}=\frac{66.9}{400*360*\sqrt{24}/1000}= 0.095 \leq 3$$

According to these conditions the performance limits are:

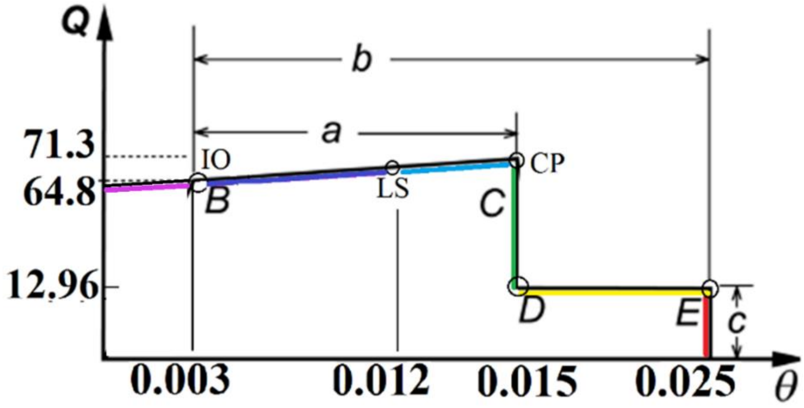

Figure B.9. Auto concrete P-M3 hinge from FEMA 356.

Table B.4. results of Manual vs. SAP2000 calculations for auto concrete P-M3 hinge.

	Cantilever L=4m		Yielding	IO	LS	CP	C	D
	L (m)	**Py from (My) (kN)**	**Δ(mm)**	**0.003**	**0.012**	**0.015**	**0.015**	**0.025**
Manual	**4**	**66.90**	**29.055**	**41.06**	**77**	**89.06**	**89.06**	**129**
SAP	**4**	**64.80**	**28.143**	**40.14**	**76**	**88.14**	**88.14**	**128**

c- Concrete P-Brittle hinge for bracing elements (representing unreinforced masonry walls)

The section is defined as unreinforced brittle section.

In order to verify P-brittle hinge used in modeling case study by SAP2000 program, which was a default hinge selection. The hinge performance levels starts with IO level when axial load reaches half of nominal load capacity, then LS level when axial load reaches (0.8) of the nominal load capacity, and CP level when axial load reaches the nominal capacity, the following steps will be done to 1D cantilever model:

1- Manual calculations of service load capacity, which is done in (Table B.1.).

2- Compare the results with program results.

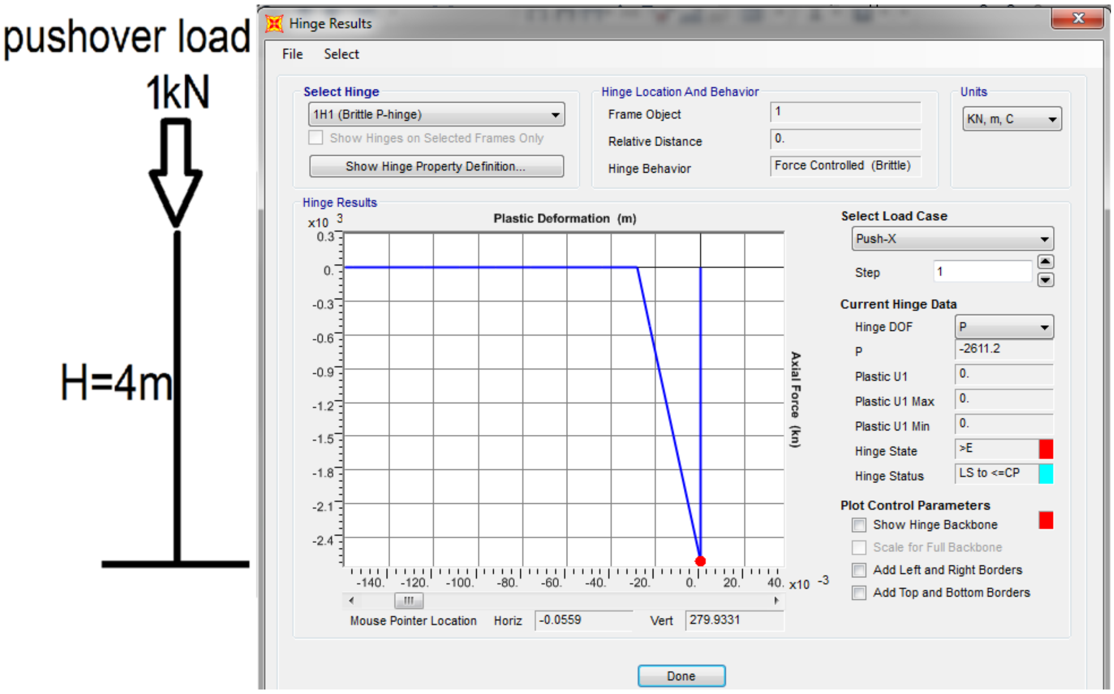

Figure B.10. 1D cantilever model and hinge result of P-brittle hinge.

For more detailed calculations

$$\text{IO performance limit} = 0.5*2089 = 1044.5 \text{ kN}$$

$$\text{LS performance limit} = 0.8*2611.2 = 2089 \text{ kN}$$

$$\text{CP performance limit } P_n = 0.8*0.85*f'c * Ag = 0.8*0.85*24*400^2 = 2611.2 \text{ kN}$$

The maximum deflection may occare at CP performance limit due to axial load, which is:

$$\Delta = \frac{P*L}{EA} = \frac{2611200*4000}{323025.2*400^2} = 2.84\text{mm}$$

$$\Delta(\text{SAP2000}) = 2.84\text{mm}$$

Appendix C. Verification of pushover procedure:

The procedure used by SAP2000 is hinge-to-hinge method. This method will be verified in the way of plastic hinge appearance sequence through 2D frame model presented in Fig. (A.1.10) with start pushover load of 50kN and assuming no hardening (SF=1), which means My=Mp=133kN.m.

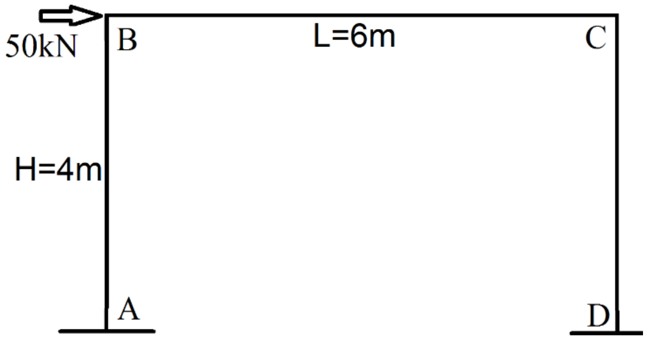

Figure C.1. 2D frame model and start pushover load.

Using moment distribution method:

a) K=$\frac{4EI}{L}$:

$$K_{BA} = K_{CD} = \frac{4EI}{4}$$
$$K_{BC} = K_{CB} = \frac{4EI}{6}$$

b) DF:

$$DF_{AB} = DF_{DC} = 1$$
$$DF_{BA} = DF_{CD} = \frac{1/4}{\frac{1}{4}+\frac{1}{6}} = 0.6$$
$$DF_{BA} = DF_{CD} = \frac{1/6}{\frac{1}{4}+\frac{1}{6}} = 0.4$$

c) Assume:

$$\Delta = \frac{-100}{EI} \quad \text{then } M_{BA} = M_{CD} = \frac{-6EI}{L} * \Delta = 150 \text{ kN.m}$$

d) Using moment distribution method

Table C.1. Moment distribution on the frame.

point	A	B		C		DC
member	AB	BA	BC	CB	CD	DC
DF	1	0.6	0.4	0.4	0.6	1
FEM	150	150	0	0	150	150
DIS	0	-90	-60	-60	-90	0
CO	-45	0	-30	-30	0	-45
DIS	0	18	12	12	18	0
CO	9	0	6	6	0	9
DIS	0	-3.6	-2.4	-2.4	-3.6	0
CO	-1.8	0	-1.2	-1.2	0	-1.8
DIS	0	0.72	0.48	0.48	0.72	0
CO	0.36	0	0.24	0.24	0	0.36
SUM	112.5	75	-75	-75	75	112.5

e) Calculation of horizontal load

$$F_A=F_D = \frac{112.5+75}{4} = 46.88 \text{ kN}$$

$$F_{total} = 46.88*2 = 93.8 \text{ kN}$$

f) Factor the results to the assumption loads

$$\text{Factor} = \frac{50}{93.8} = 0.53 \text{ kN}$$

$$M_{AB} = M_{DC} = 112.5*0.53 = 60 \text{ kN.m}$$

$$M_{BA} = M_{CD} = 40 \text{ kN.m}$$

$$M_{BC} = M_{CB} = -40 \text{kN.m}$$

g) Hinge to hinge method Stage one

Table C.2. Stage 1 hinge-to-hinge method.

Joint	Mo	Mp-Mi	α=Mp-Mi/Mo	Mi+1= Mi+αMo
A	60	128	2.133	128
B	40	128	3.2	85.32
C	40	128	3.2	85.32
D	60	128	2.133	128

Stage two

Analyze with pin supports instead of fixed because of plastic hinge appearance. Therefore, the moments on B and C joints are:

$$M_{AB} = M_{DC} = 0 \text{ kN.m}$$

$$M_{BA} = M_{CD} = \frac{50}{2} * 4m = 100 \text{ kN.m}$$

$$M_{BC} = M_{CB} = -100 \text{kN.m}$$

Table C.3. Stage 2 hinge-to-hinge method.

Joint	Mo	Mp-Mi	α=(Mp-Mi)/Mo	Mi+1= Mi+αMo
A	0	0	-	128
B	100	42.68	0.427	128
C	100	42.68	0.427	128
D	0	0	-	128

Through this procedure, plastic hinges sequence are known. In addition, the moments are known.

h) The pushover loads are

$$P_{stage1}=50*2.133 = 106.6\text{kN}$$

$$P_{stage2}=50*(2.133+0.427) = 128kN$$

The displacements are obtained through stiffness method

$$\Delta_{B(stage1)} = 0.0096m$$

$$\Delta_{B(stage2)} = 0.0184m$$

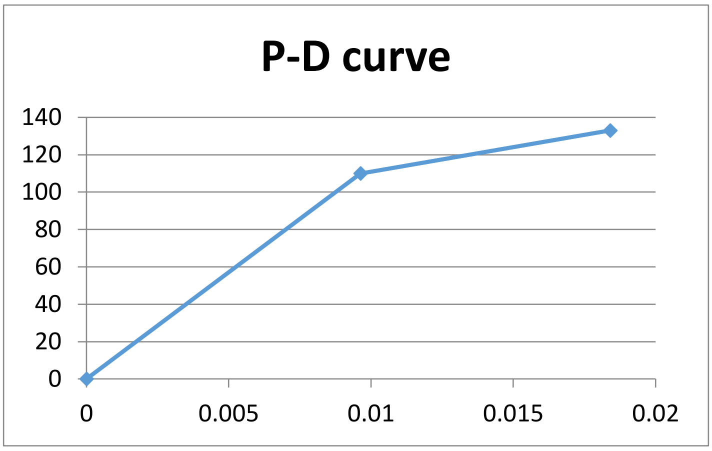

Figure C.2. Manual pushover load of 2D frame model.

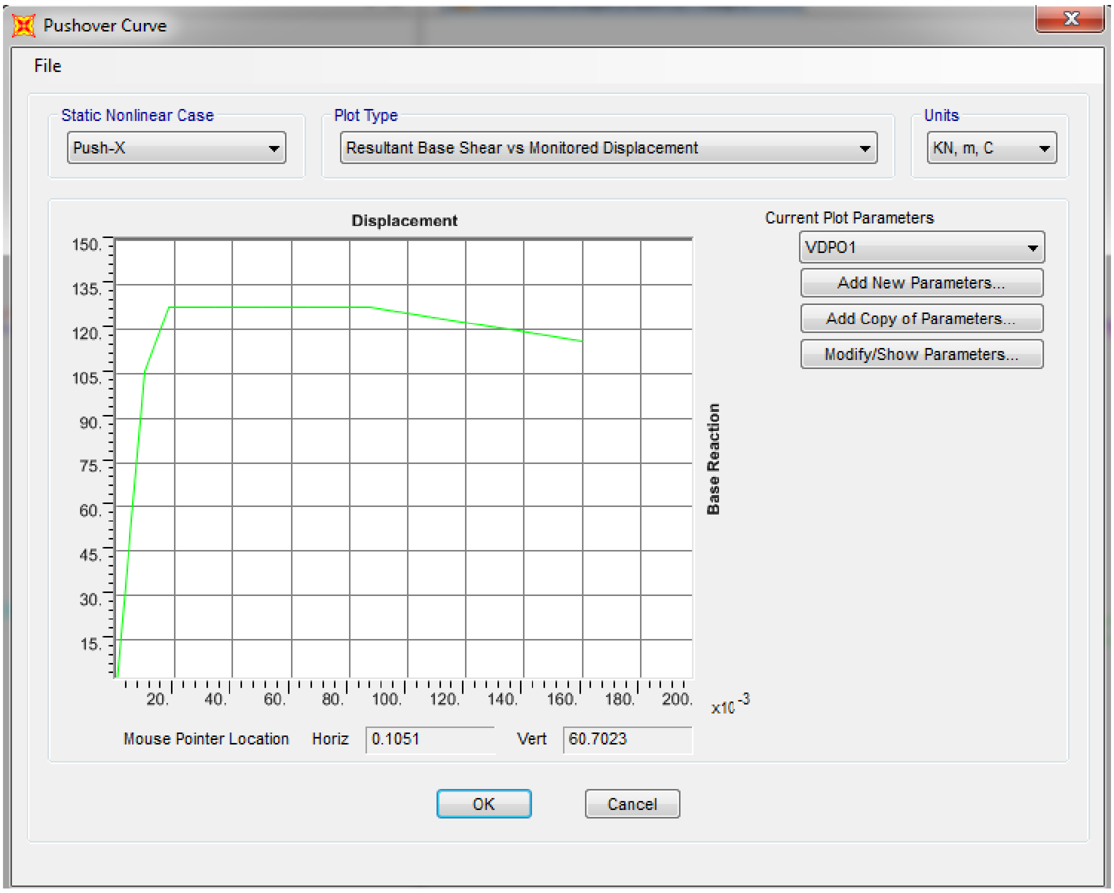

Figure C.3. SAP2000 pushover load of 2D frame model.

Table C.4. Manual vs. SAP2000 calculations.

	Stage 1		Stage 2	
	Pp	Δ_B	Pp	Δ_B
Manual	106.6	0.0096	128	0.0179
SAP2000	106	0.00945	128	0.0178

The resultant shows that SAP2000 is reliable in use.

Appendix D. Verification of CSM procedure:

The verification of CSM procedure includes:

a- MDOF to equivalent SDOF

b- RSC and Reduction of response spectrum and performance point

The model that will be used to verify the procedure are ND-A-B in X-direction

With Cv = 0.28 Ca= 0.4

a- Convert MDOF to equivalent SDOF

Table. D.1. Normalized shape factors matrix and mass matrix.

Φ	**M**					
0.47	**4834.201**	**0**	**0**	**0**	**0**	**0**

0.67	**0**	**4834.201**	**0**	**0**	**0**	**0**
0.79	**0**	**0**	**4834.201**	**0**	**0**	**0**
0.88	**0**	**0**	**0**	**4834.201**	**0**	**0**
0.96	**0**	**0**	**0**	**0**	**4834.201**	**0**
1	**0**	**0**	**0**	**0**	**0**	**3817.886**

Table. D.2. Generalized mass, modification factors to equivalent SDOF, base shear, and displacement of performance point of the structure.

$\Phi^T.M.\Phi$	**Γ**	**m***	**V-P.P.**	**D-P.P.**	**Say(m/sec2)**	**Say/g**	**Sd (m)**
18271.66	**1.21**	**22042.82**	**4964.000**	0.095	2.25	0.23	0.079
			SAP			0.182	.085

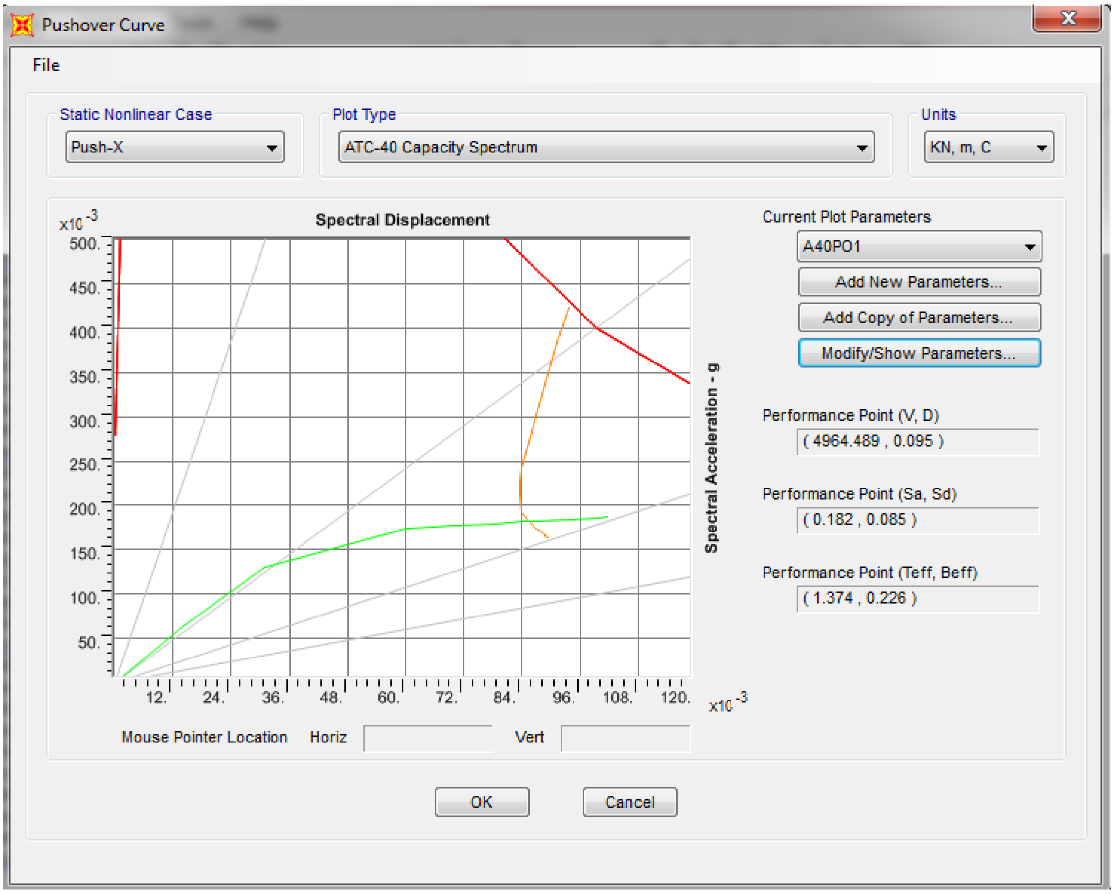

Figure D.1. Equivalent SDOF pushover curve, the demand curve, and performance point data

b- Reduce the response spectrum using CSM to find the manual performance point

Table. D.3. Response spectrum in both (Sa-T) and (Sa-Sd) formats and reduction factors and reduced response spectrum according to CSM method

Ca	0.28
Cv	0.4
SRv	0.624424

Reduced			SRa	0.513005	
Sa	T	Sd	Sa	T	Sd(cm)
0.143641	0	0	0.28	0	0.00
0.359104	0.139107	0.17285	0.7	0.114286	0.23
0.359104	0.695537	4.321251	0.7	0.571429	5.68
0.333026	0.858595	6.106671	0.533333	0.75	7.45
0.268983	1.021653	6.98362	0.430769	0.928571	9.23
0.225598	1.18471	7.876083	0.36129	1.107143	11.00
0.194265	1.347768	8.777595	0.311111	1.285714	12.78
0.170574	1.510826	9.684846	0.273171	1.464286	14.55
0.152034	1.673884	10.59596	0.243478	1.642857	16.33
0.137128	1.836942	11.50981	0.219608	1.821429	18.10
0.124885	2	12.42566	0.2	2	19.88

c- Get the data of the equivalent SDOF pushover curve from the program in order to be plotted with the reduced response spectrum to find plastic period, equivalent stiffness, Sa and Sd.

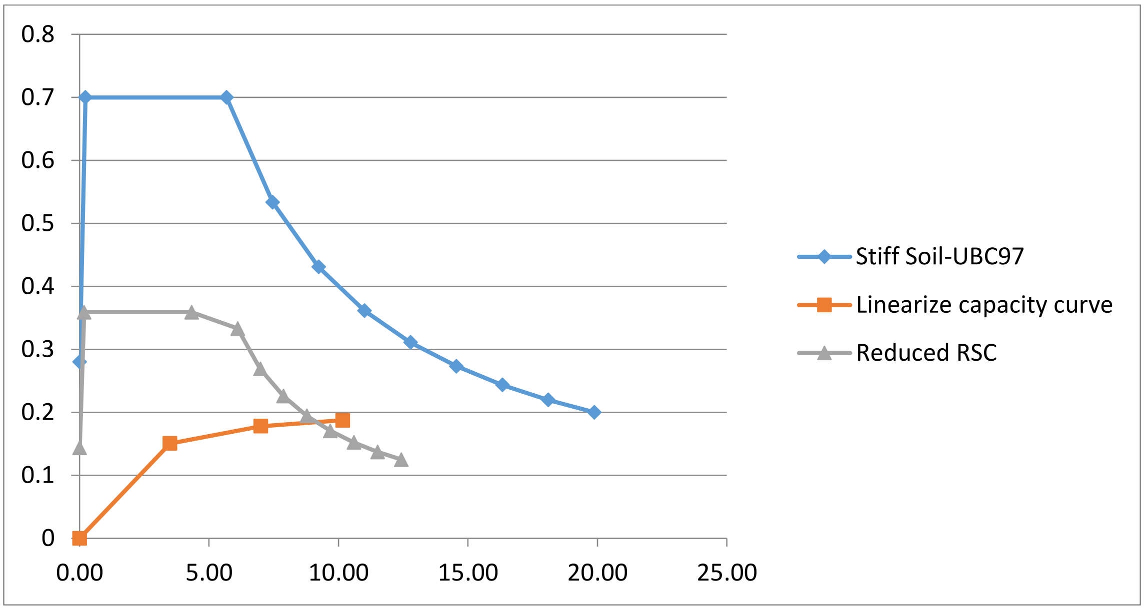

Figure D.2. Equivalent SDOF pushover curve with elastic and reduced response spectrum curve.

Table. D.4. Data of the resultant performance point.

Ts	Sd at Ts	Sa	SRa	SRv	β1	K	ay	dy	api	dpi
0.696	0.062	0.184	0.513	0.624	26.921	0.657	0.154	0.036	0.182	0.085

It can be seen that the results of the manual solution of converting from MDOF to equivalent SDOF and reducing the response spectrum gives the same results of the program, which means that the program is reliable.

Appendix E. Structural details:

The following details are not scaled and the dimensions shall be taken from the drawings.

The details shown in (Figure E.5- E.13), verify the assumption that the base joint can act as fixed supports can be adopted.

The details shown in (Figure E.17) are a part of the drawings of the case study building. Through these details the following assumptions can be inferred:

a- The sections are confined

b- The hooks indicates fixed end supports for beams

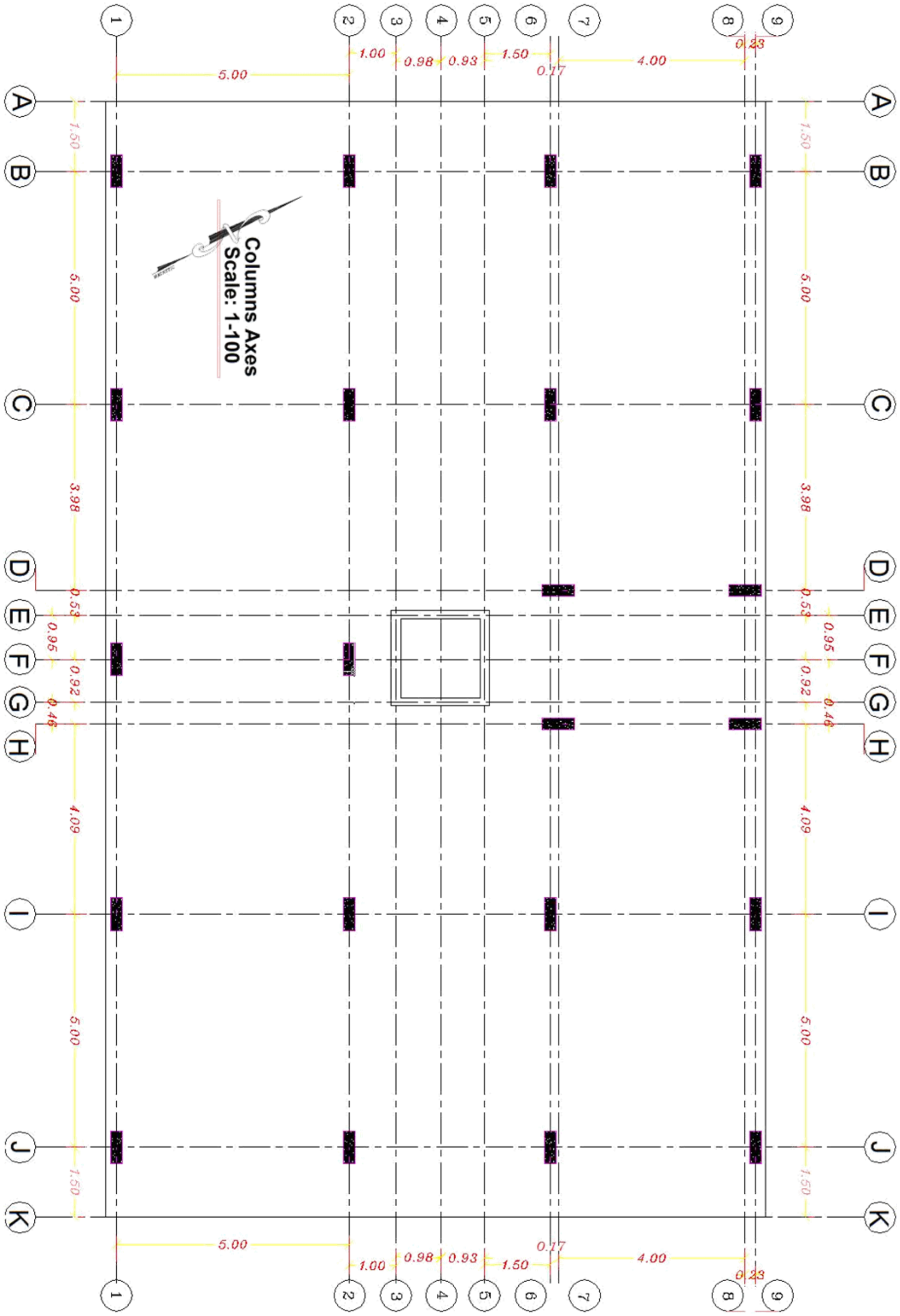

Figure E.1. Columns Grid Lines.

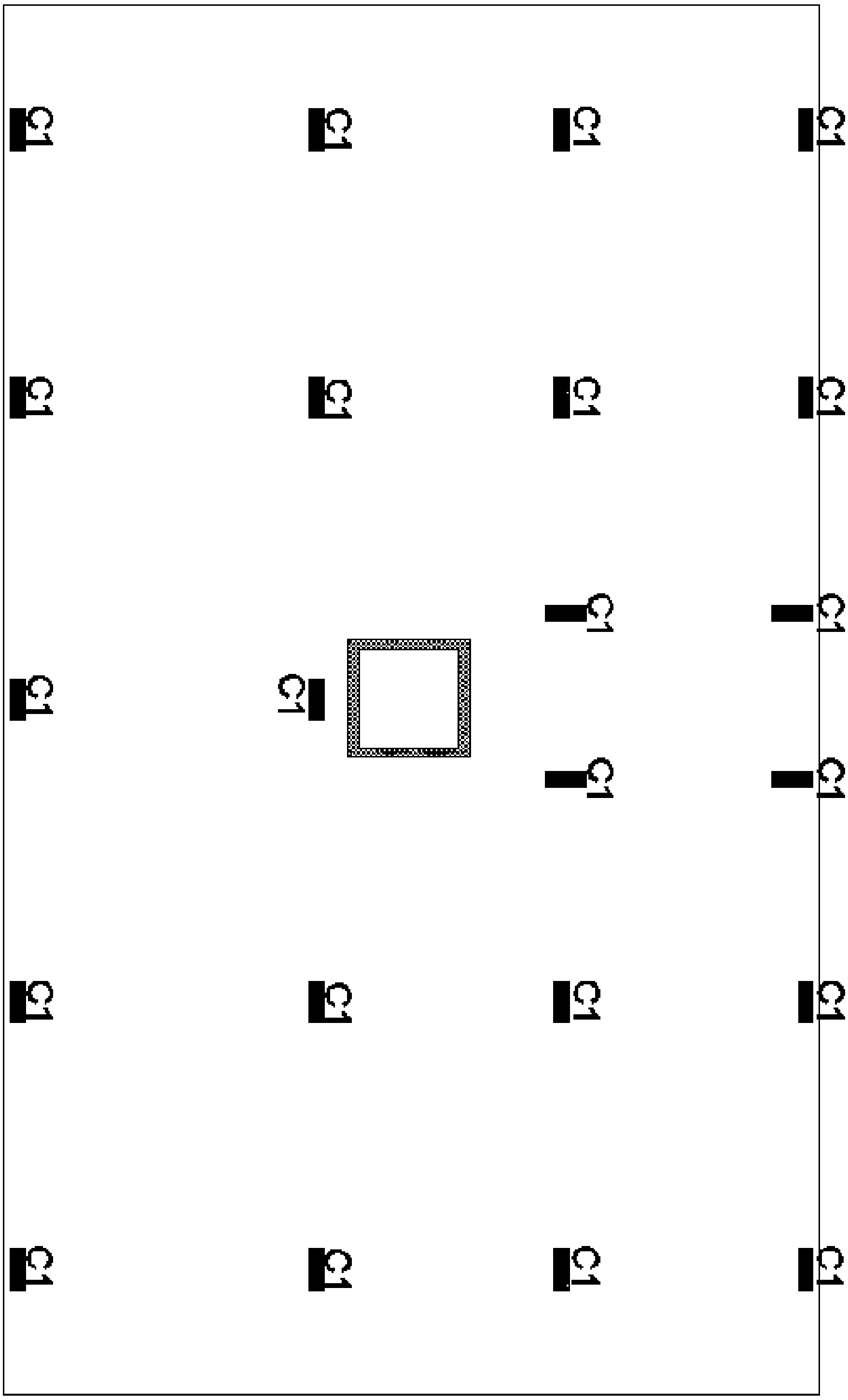

Figure E.2. Columns types.

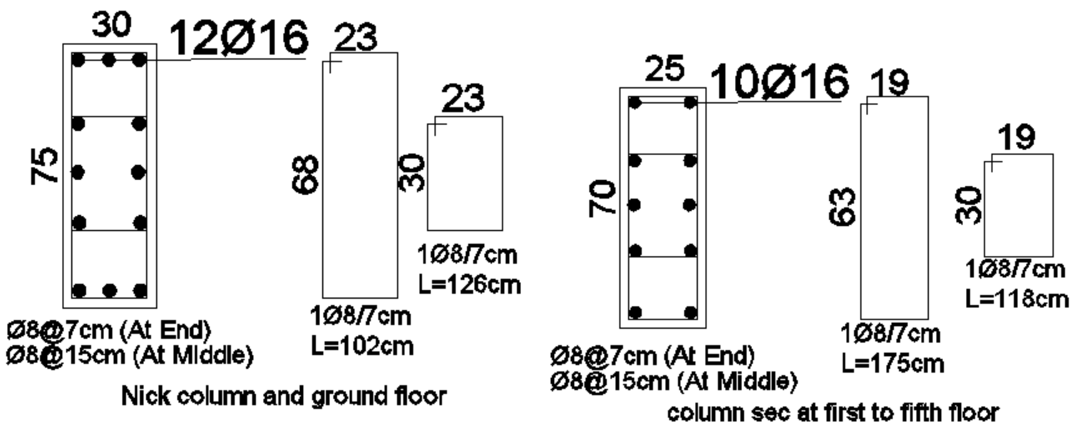

Figure E.3. Details of column sections.

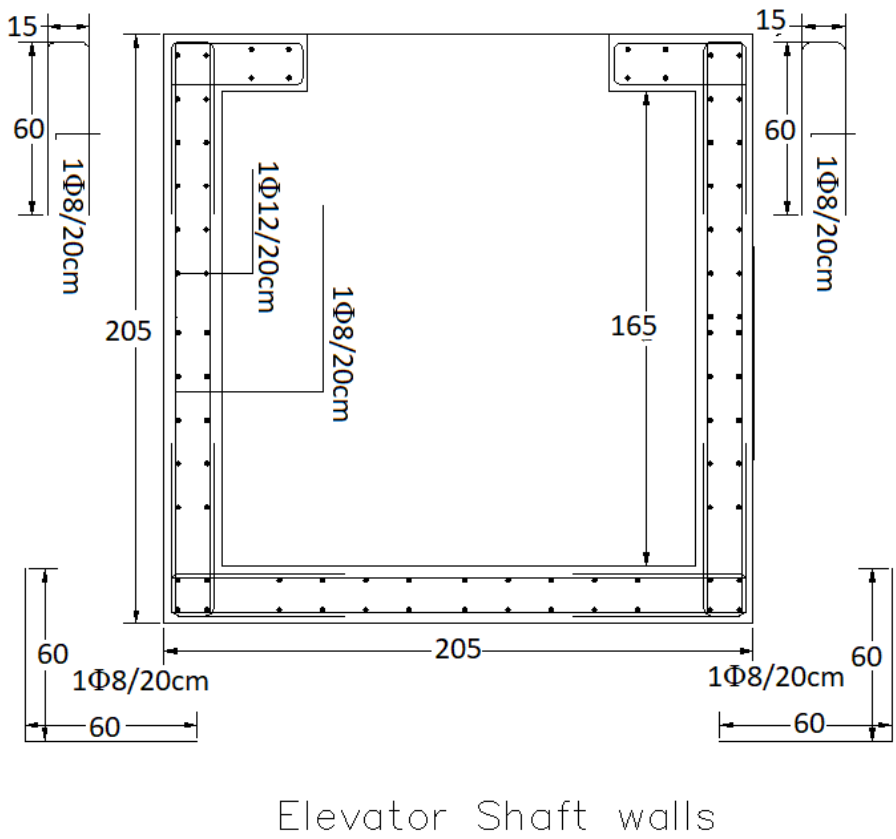

Figure E.4. Details of elevator shaft walls.

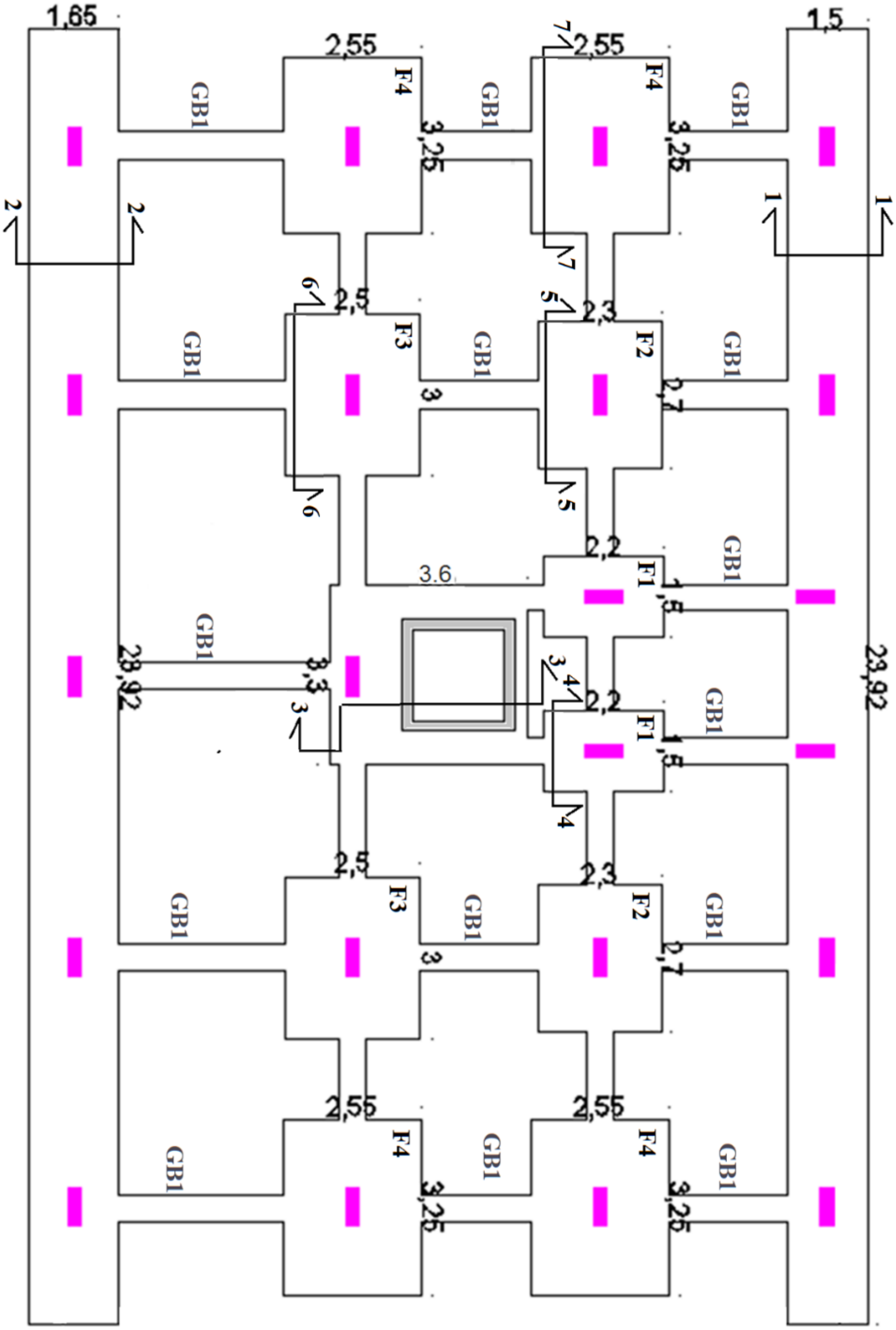

Figure E.5. Foundation system.

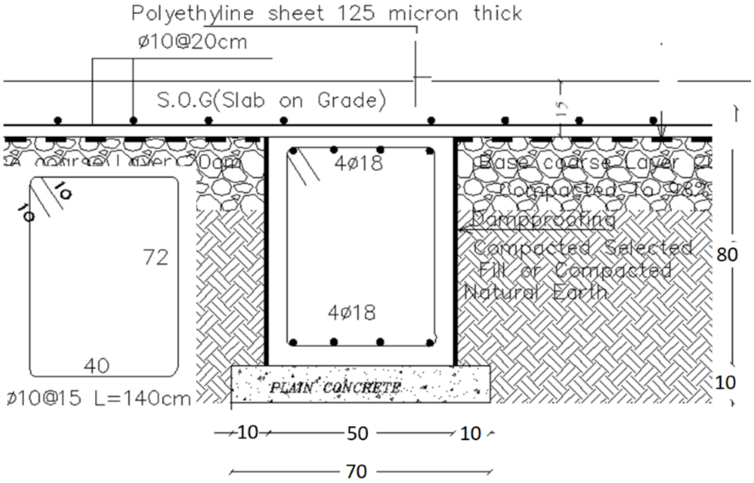

Figure E.6. Ground beam details.

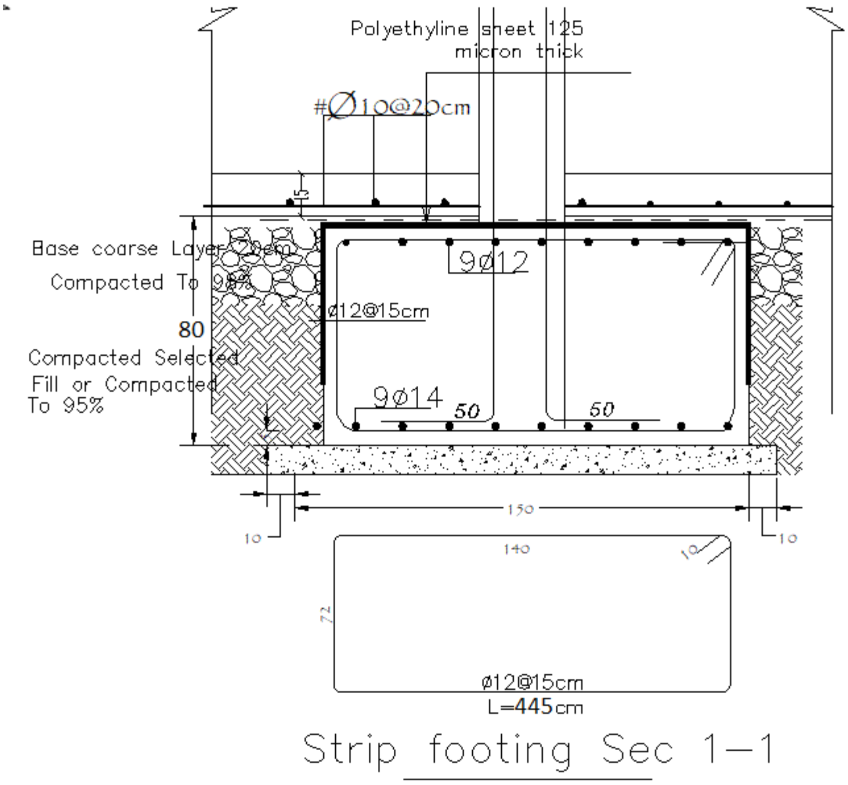

Figure E.7. Details of strip footing section 1-1.

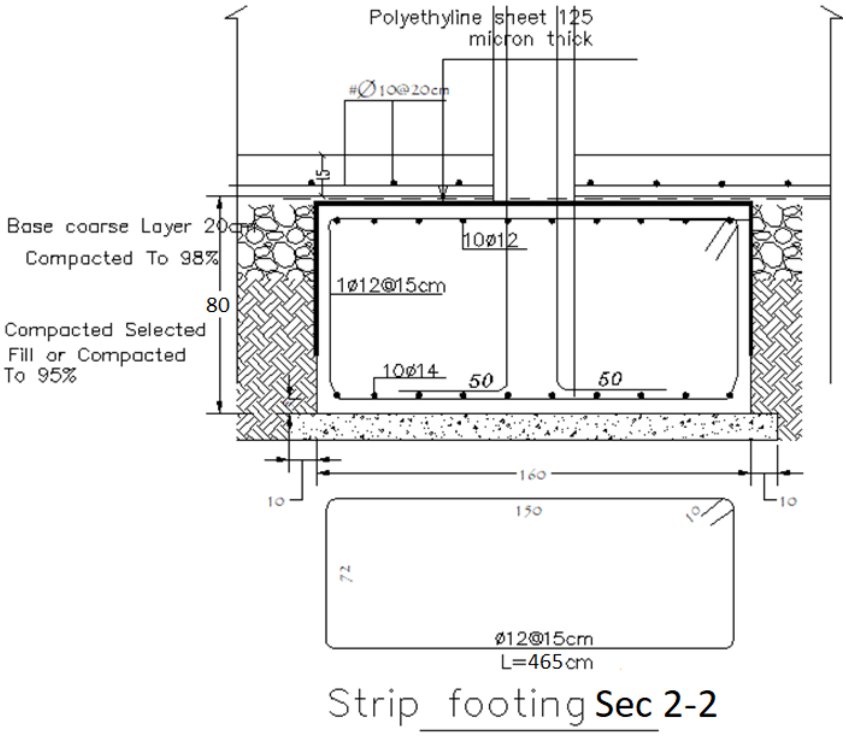

Figure E.8. Details of strip footing section 2-2.

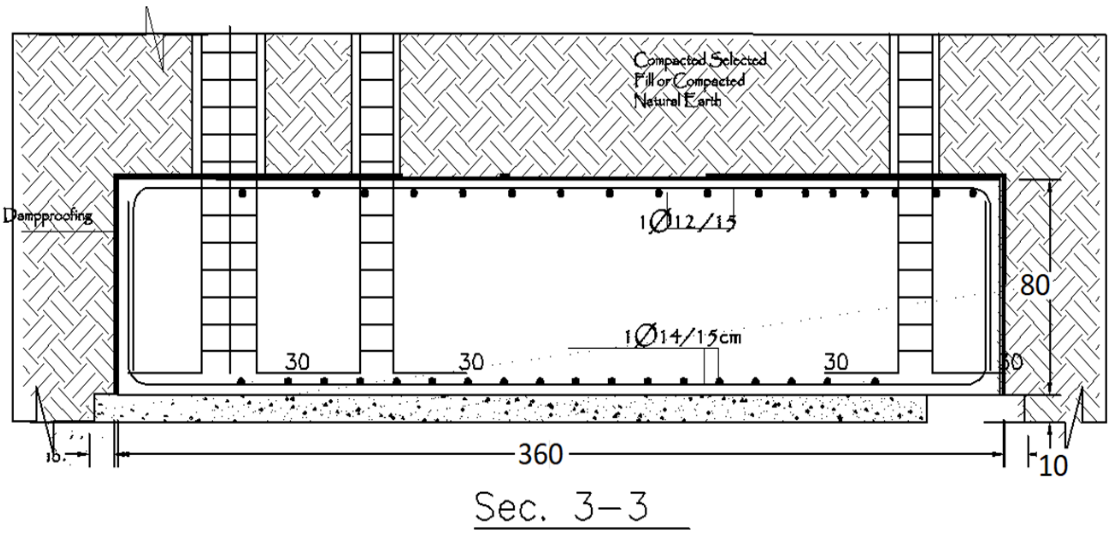

Figure E.9. Details of combined footing section 3-3.

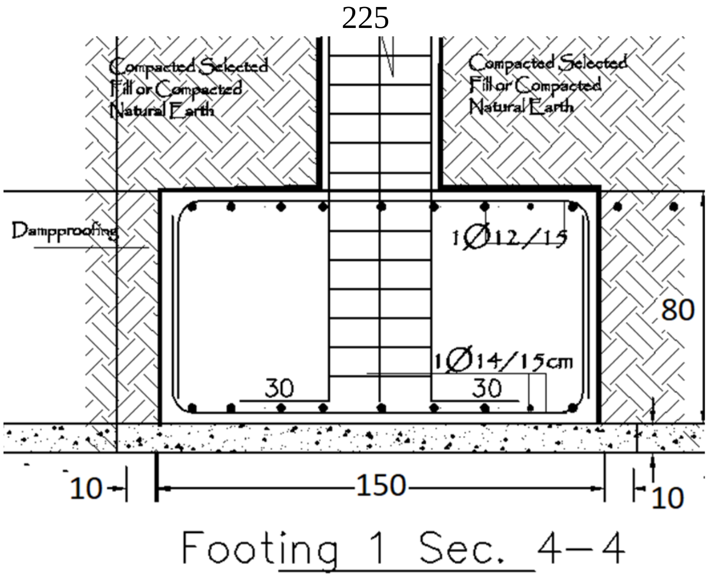

Figure E.10. Details of footing 1 section 4-4.

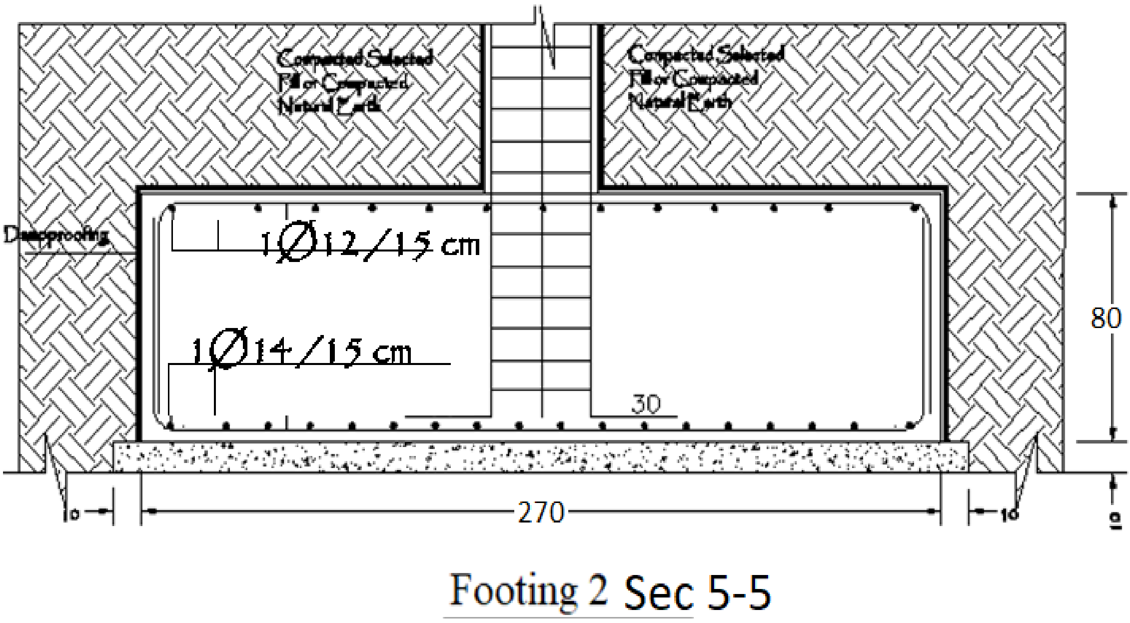

Figure E.11. Details of footing 2 section 5-5.

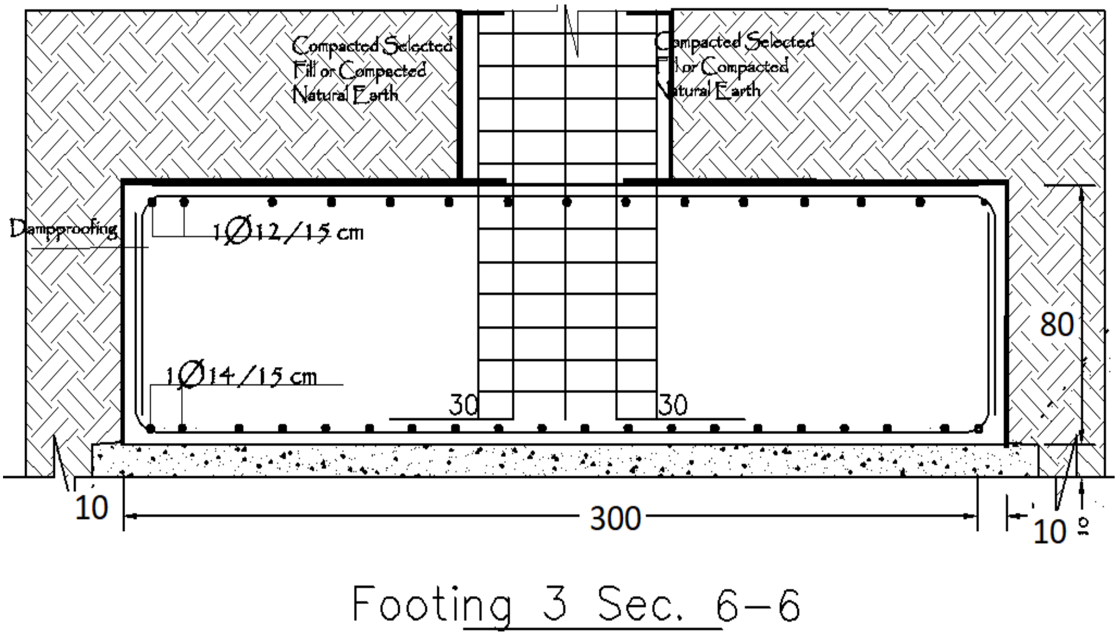

Figure E.12. Details of footing 3 section 6-6.

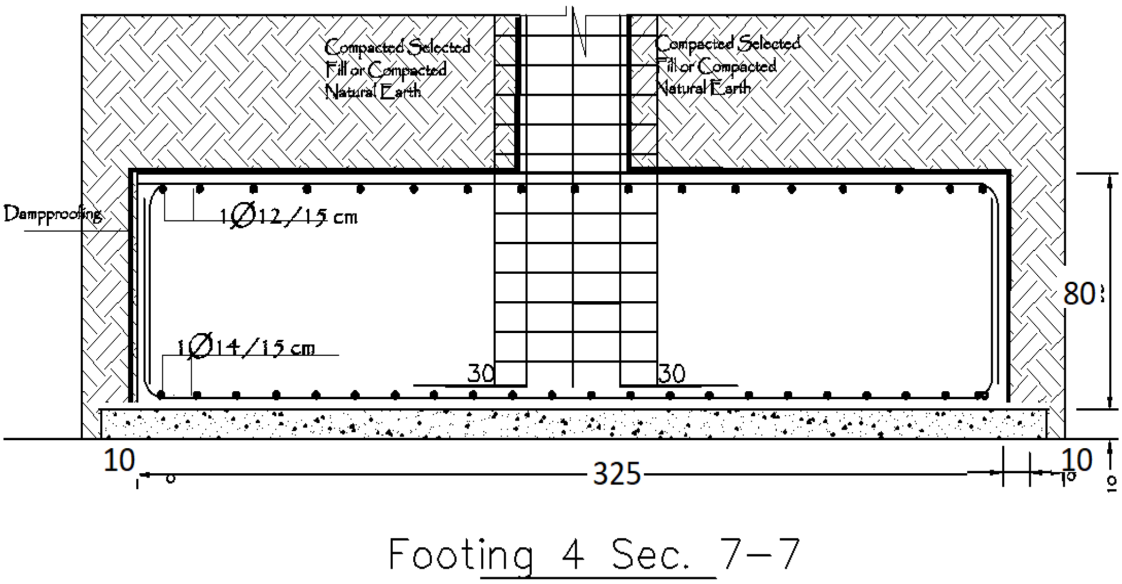

Figure E.13. Details of footing 4 section 7-7.

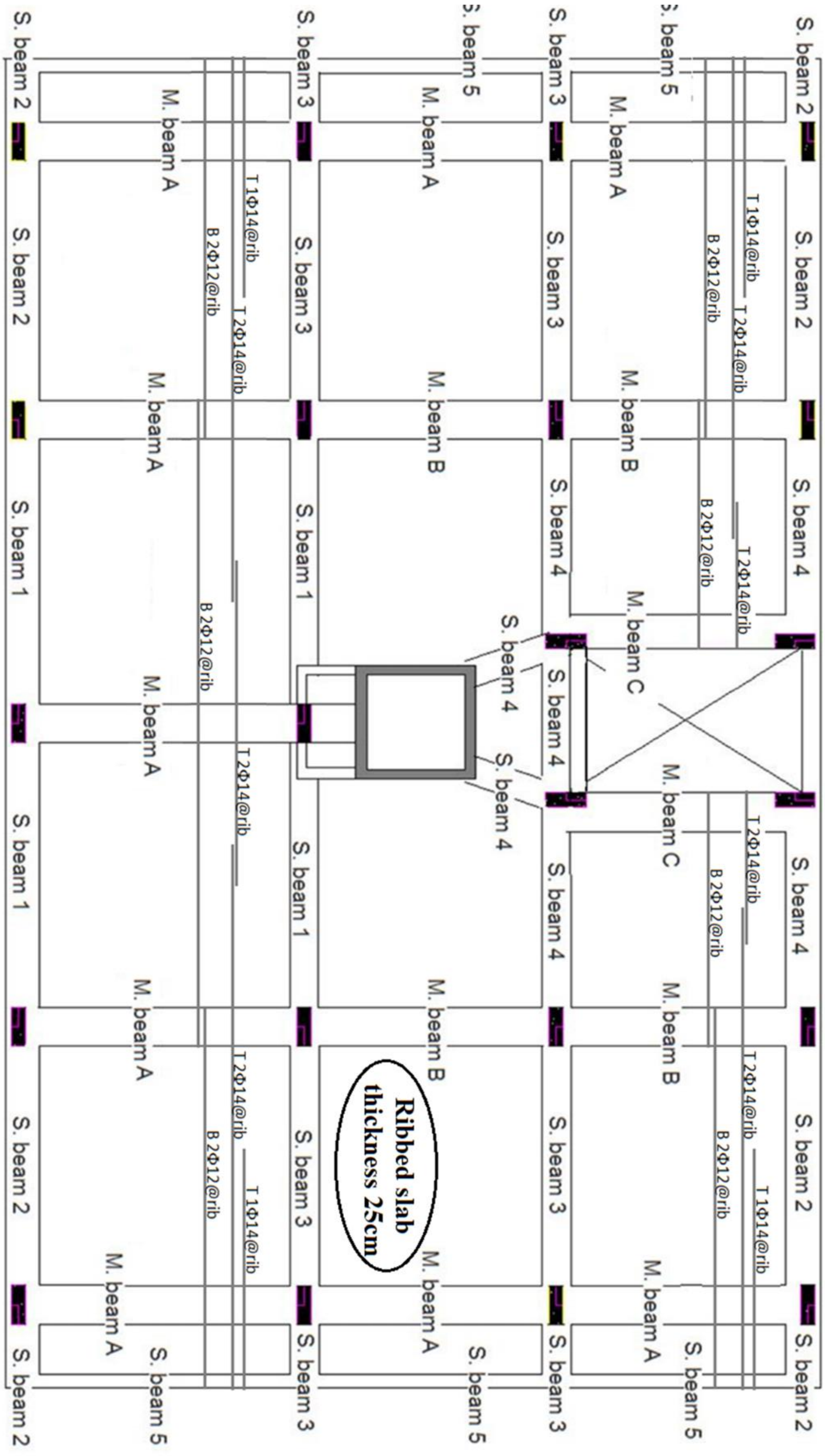

Figure E.14. Slab of GF, F1, F2, F3, F4, and F5.

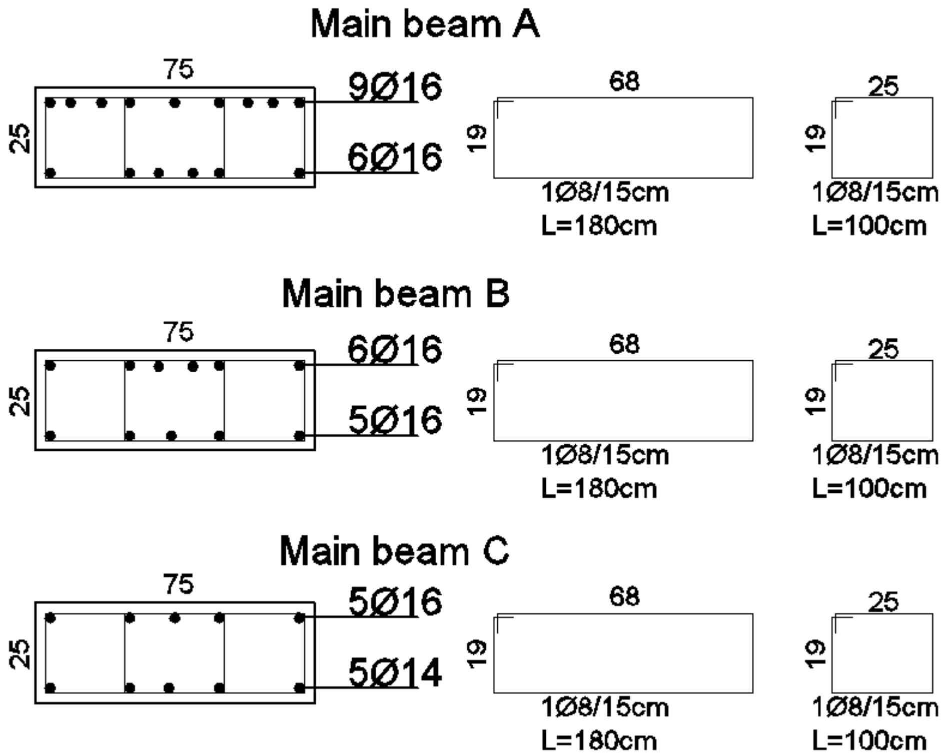

Figure E.15. Details of main beams.

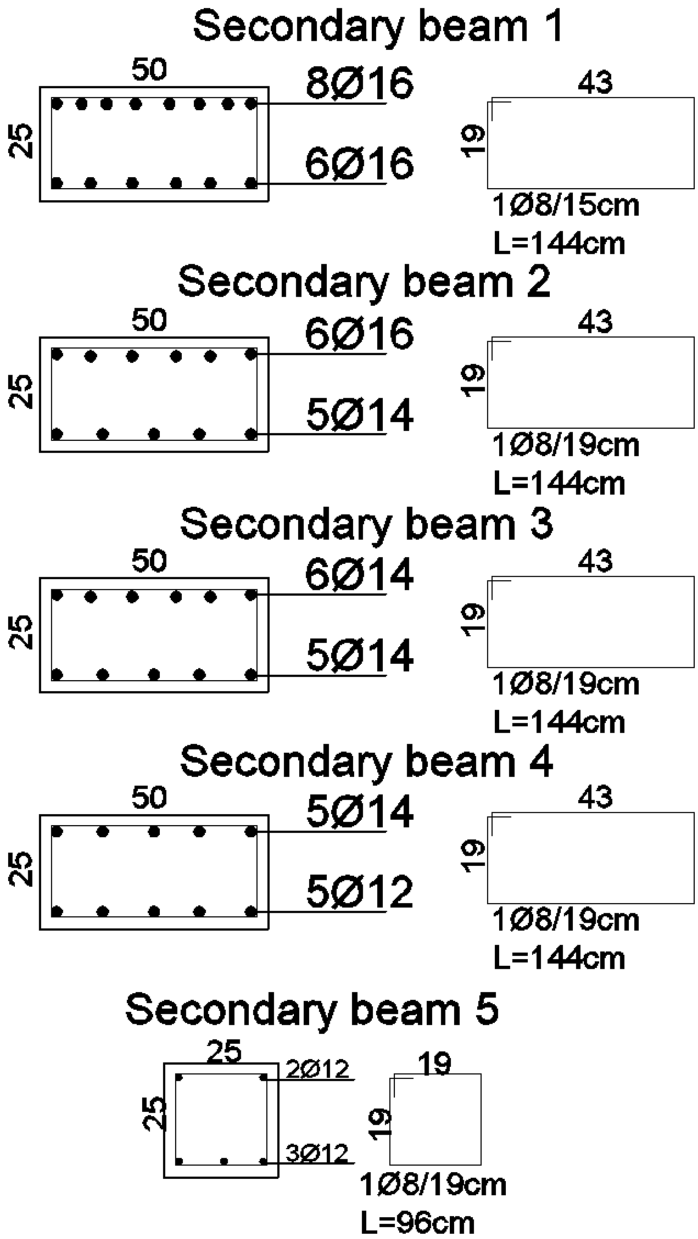

Figure E.16. Details of secondary beams.

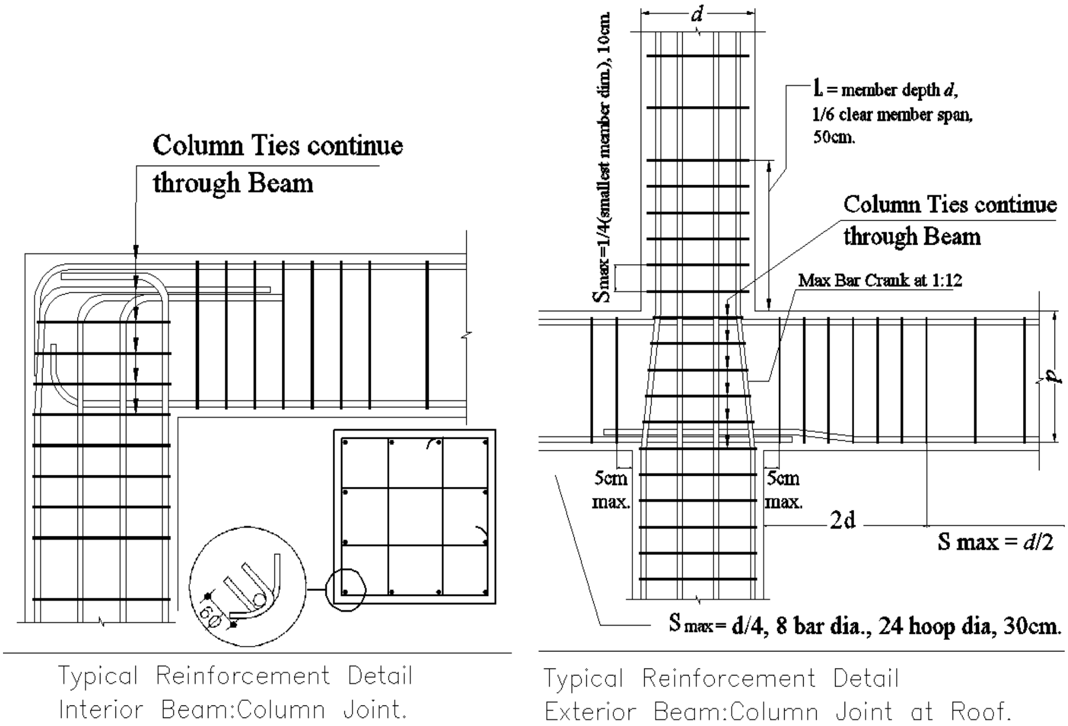

Figure E.17. Typical structural details.

جامعة النجاح الوطنية
كلية الدراسات العليا

تقييم وتأهيل المباني القائمة زلزالياً باستخدام طرق الأحمال الثابتة وغير الخطية
– طرق الدفع المتتالي –

إعداد
أنس شاهر عبد الحفيظ شحادة

إشراف
د. محمود دويكات
د. عبد الرزاق طوقان

قدمت هذه الاطروحة استكمالا لمتطلبات الحصول على درجة الماجستير في هندسة الإنشاءات في كلية الدراسات العليا في جامعة النجاح الوطنية
2017

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الملخص

لقد أصبح تصميم المباني لتحمل الاحمال الزلزالية إلزاميا في فلسطين، ولكن ماذا بالنسبة للمباني القائمة؟ لقد تم تصميم معظم المباني القائمة وخصوصا المباني القديمة لتحمل الاحمال الرأسية الثابتة (وزن المبنى والاوزان المضافة الى المبنى). ان هذه المباني تعتبر عرضة لخطر الزلازل ولهذا فهي بحاجة الى تقوية لكي تعتبر آمنة. وللوصول الى درجة التقوية المطلوبة، فإنه يتوجب استعمال طرق تحليل وتقييم حديثة.

نادرا ما يوجد دراسات توفر منهاجا عمليا للخطوط العريضة لكيفية التقييم والتحديث الزلزالي للمباني القائمة للمهندسين في فلسطين. بشكل عام، فإن الدراسات لهذا الموضوع مكتوبة حسب الكودات الاجنبية (مثل ASCEأو FEMA) وهي عامة وقد تشكل تحديا للمهندسين المحليين فيما يتعلق بطريقة تنفيذها. ان هذه الدراسة تجسر الفجوة بين المهندسين المحليين والكودات العالمية من خلال تطبيق هذه الكودات على حالة دراسية عملية.

بشكل عام، هناك أربع طرق متوفرة للتحليل الزالزالي للمباني: طريقتين خطيتان، وطريقتان غير خطيتان. طرق الاحمال غير الخطية تشمل طرق الاحمال الثابتة غير الخطية وطرق الاحمال الديناميكية غير الخطية. تعتبر طرق الاحمال الثابتة غير الخطية اداة عملية لتقييم الاداء غير الخطي. ومن ناحية اخرى فإن طرق الاحمال الديناميكية غير الخطية تتطلب إدخال خصائص مفصلة وبحاجة الى وقت أكثر، وهذه النقاط تعتبر عيوب بالنسبة الى مكاتب التصميم، من حيث قيود مواعيد التسليم. اضافة الى ذلك، فإنه لا يوجد في فلسطين سجلات للزلازل، ولا يوجد برامج

قوية لطرق الاحمال الديناميكية غير الخطية. وهذا يجعل من طرق الاحمال الثابتة غير الخطية الخيار الافضل للتقييم العملي للمباني.

ان هدف هذه الرسالة هو توضيح واظهار طريقة ومنهاج للتقييم من خلال دراسة المباني القائمة المحلية، والتي تم تصميمها على تحمل الاحمال الرأسية الثابتة (الاوزان) فقط، من ثم اقتراح طرق تحديث لحل واصلاح العيوب في المبنى.

اعتمادا على ما ذكر آنفا، فإنه تم تقييم الحالة الدراسية (المبنى) باستخدام أحد طرق الاحمال الثابتة غير الخطية والمعروفة باسم (capacity spectrum method (CSM)) وفقا للكود (ATC-40). وتم تحليل وإظهار التصرف اللدن وغير الخطي للمبنى باستخدام طريقة الدفع المتتالي.

وقد تم التقييم الزلزالي اعتمادا على معايير الاداء وفقا للكود (FEMA 356). ووفقا للكود (FEMA 356)، هناك نهجان للتقييم الزلزالي: على المستوى العام للمبنى وعلى مستوى العناصر الانشائية للمبنى مع ثلاثة مستويات للأداء، وهنّ مستوى الاشغال الفوري للمبنى ومستوى الحفاظ على الحياة، ومستوى الوقاية من الانهيار. إضافة الى ذلك، تم تطبيق متطلبات التصميم الزلزالي المذكورة في الكود (ASCE 7-10) من أجل تقييم المبنى من حيث انتظام المبنى.

واستنادا إلى التحليل غير الخطي وفق طريقة الدفع المتتالي، تبين أن المبنى يعاني من عدم انتظام عمودي وتركز المفاصل اللدنة في الطابق الأرضي.

ومن أجل تحسين أداء المبنى، تم تطبيق اثنين من التقنيات التحديثية الممكنة والتان تشملان إضافة تغليف خرسانة مسلحة للأعمدة وإضافة إطارات من الخرسانة المسلحة.

تمت مقارنة منحنيات القدرة للمبنى المعاد تهيئته مع تلك الخاصة بالمبنى قبل التعديل. وتمت مقارنة حد الازاحة العام حسب الكود (FEMA) مع حدود الازاحة لنقاط الأداء لكل التقنيات التحديثية على أساس معايير مستوى العناصر الانشائية حسب الكود (FEMA). إضافة الى ذلك، تم إعادة فحص حدود (ASCE) ومقارنتها مع النسب للمبنى غير المعدل. وقد ساعدت تقنيات التحديث والتهيئة على تحسين أداء المبنى.

لقد مهدت هذه الأطروحة الطريق لمزيد من الابحاث في التقييم الزلزالي للمباني القائمة مع أدوات فعالة للحكم على كفاءة وملاءمة تقنيات التحديث والتأهيل.