إقرار

أنا الموقع أدناه مقدم الرسالة التي تحمل العنوان:

Seismic Resistance of Reinforced Concrete Buildings Designed for Gravity Loads in Gaza Strip

المقاومة الزلزالية للمباني الخرسانية المصممة للأحمال الرأسية في قطاع غزة

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المقاومة الزلزالية للمباني الخرسانية المصممة للأحمال الرأسية في قطاع غزة

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This Thesis is submitted in partial fulfillment of the requirements for the degree of Master of Science in Civil Engineering - Design and Rehabilitation of Structures

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بسم الله الرحمن الرحيم

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نتيجة الحكم على أطروحة ماجستير

بناءً على موافقة شئون البحث العلمي والدراسات العليا بالجامعة الإسلامية بغزة على تشكيل لجنة الحكم على أطروحة الباحث/ محمد مهاجر سلمان التلباني لنيل درجة الماجستير في كلية *الهندسة* قسم الهندسة المدنية– تصميم وتأهيل المنشآت وموضوعها:

المقاومة الزلزالية للمباني الخرسانية المصممة للأحمال الرأسية في قطاع غزة

Seismic Resistance of Reinforced Concrete Buildings Designed for Gravity Loads in Gaza Strip

وبعد المناقشة التي تمت اليوم السبت 23 شوال 1436هـ، الموافق 2015/08/08م الساعة الثانية عشرة ظهراً، اجتمعت لجنة الحكم على الأطروحة والمكونة من:

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د. علي إبراهيم تايه	مناقشاً خارجياً	....

وبعد المداولة أوصت اللجنة بمنح الباحث درجة الماجستير في كلية *الهندسة*/ قسم الهندسة المدنية- تصميم وتأهيل المنشآت.

واللجنة إذ تمنحه هذه الدرجة فإنها توصيه بتقوى الله ولزوم طاعته وأن يسخر علمه في خدمة دينه ووطنه.

والله ولي التوفيق ،،،

مساعد نائب الرئيس للبحث العلمي والدراسات العليا

أ.د. فؤاد علي العاجز

DEDICATIONS

I dedicate this work

To my father, mother, brothers, and sisters for their endless support.

To my beloved wife for her encouragement.

To my son "Waseem" and daughter "Nadeen".

To my family, friends, and colleagues.

Hoping I have made all of them proud of me.

Mohammed Mohajer Al Telbani

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ABSTRACT

Palestine is located along the Dead Sea Transform fault and thus all areas including Gaza Strip are vulnerable to earthquakes. Despite this fact, unlike multistory buildings (locally called towers) most of residential reinforced concrete buildings of limited number of stories in Gaza Strip are designed and constructed to resist gravity loads only without any considerations to seismic resistance. It is generally assumed by designers that the effect of seismic forces on such buildings is low. The building frame structural system and infill walls are assumed to resist such loads. However, these assumptions are seldom verified by designers. So, the evaluation of the seismic resistance of such buildings is necessity in order to draw specific conclusions related to the design of new buildings and strengthening of existing ones, if necessary.

More specifically, this research aimed at evaluating the seismic resistance of the low-rise residential reinforced concrete buildings designed for gravity loads only, determining the contribution of infill walls to seismic resistance, assessing the performance of buildings with some irregularities, e.g. soft story, and draw conclusions related to design of new buildings and strengthening requirements for existing buildings.

The design and construction practices of buildings in Gaza Strip have been investigated with respect to resistance to earthquake forces. The investigation assisted in classification of buildings with respect to seismic resistance and determining the most used type of buildings to be assessed in the research.

Seismic parameters, assessment methodologies, analysis techniques have been determined for use in Gaza Strip buildings based on thorough review of relevant literature and practices. The assessment has been carried out using the static nonlinear (pushover) analysis procedure proposed by ATC-40 and FEMA-356 guidelines. IBC 2012 and ASCE/SEI 7-10 codes have been adopted where seismic parameters representing the study area have been used. SAP2000 software was used to perform the pushover analysis.

Eight real life case studies represent low-rise residential buildings that exist in Gaza Strip were assessed. The investigated variable parameters of the case studies included number of stories, infill walls, soft story irregularity, elements that may contribute to lateral load resistance, e.g. walls of elevator shafts and stair cases. For each case study the followings have been determined: Load- displacement (pushover) curve in x and y directions, deformation shape, number of plastic hinges related to each level of performance, performance point determined from Acceleration-Displacement Response-Spectra (ADRS) and accordingly the performance level of the whole building which indicates the adequacy of building to resist seismic forces in Gaza Strip.

Based on the results of this research, it is concluded that the regular low-rise residential buildings in Gaza Strip designed for gravity loads only are considered to be seismically safe.

The presence of infill walls positively affect the performance of the buildings since it increases the lateral stiffness and thus enhances the seismic resistance. Presence of soft stories decreases the lateral stiffness of buildings significantly and thus reduces the seismic resistance and may lead to structural damages and failures, especially in relatively high buildings of five stories or more. The structural walls of the elevator shafts or the stair cases enhance the seismic resistance if their locations were carefully selected such that not to produce horizontal irregularities related to torsion.

Although the buildings of Gaza Strip are evaluated, the conclusion of this research can be readily available for utilization in other areas with similar buildings.

ARABIC ABSTRACT

تعتبر كافة مناطق فلسطين بما فيها قطاع غزة معرضة للزلازل وذلك لأن فلسطين تقع على امتداد صدع البحر الميت. وبالرغم من هذه الحقيقة، فإن غالبية المباني السكنية الخرسانية محدودة الطوابق في قطاع غزة وعلى عكس المباني متعددة الطوابق (والتي تسمى محليًا بالأبراج) يتم تصميمها وإنشاؤها لمقاومة الأحمال الرأسية فقط دون الأخذ في الاعتبار مقاومة هذه المباني للزلازل. حيث يفترض المهندسون المصممون في قطاع غزة أن تأثير القوى الزلزالية على مثل هذه المباني منخفضًا. كذلك يفترضون أن النظام الإنشائي وحوائط البلوك هي التي تقاوم مثل هذه القوى الأفقية. وبالرغم من ذلك فإنه وحتى هذه اللحظة لم يقم أحد في قطاع غزة بالتحقق من هذه الافتراضات ليتم تأكيدها. لذا، فإن تقييم المقاومة الزلازلية لهذا النوع من المباني هو ضروري من أجل الوصول لاستنتاجات تتعلق بتصميم المباني الجديدة وتأهيل وتقوية المباني القائمة إن لزم الأمر.

وبشكل أكثر تحديدا وبناءًا على ما سبق، فإن هذا البحث يهدف إلى: تقييم المقاومة الزلزالية للمباني السكنية الخرسانية منخفضة الارتفاع والمصممة للأحمال الرأسية فقط، تحديد مساهمة حوائط البلوك في المقاومة الزلزالية لهذه المباني، تقييم الأداء الزلزالي للمباني الغير منتظمة مثل المباني التي تحوي الطابق الرخو (المظلة)، والوصول لاستنتاجات تتعلق بتصميم المباني الجديدة و تقوية المباني القائمة.

وقد تمت مراجعة ودراسة ممارسات تصميم وتشييد المباني في قطاع غزة فيما يتعلق بمقاومة الزلازل. وقد ساعدت هذه الدراسة في تصنيف المباني زلزاليًا وتحديد نوع المباني الأكثر استخدامًا والتي سيتم تقييمها في هذا البحث.

ومن خلال دراسة ومراجعة شاملة للمراجع والممارسات والخبرات ذات الصلة تم تحديد كل من: المعاملات الزلزالية، منهجيات التقييم، وطرق التحليل التي سيتم استخدامها لتقييم المباني في قطاع غزة. في هذا البحث تم استخدام طريقة التحليل الاستاتيكي اللاخطي (Pushover Analysis) المقترحة في الأكواد ATC-40 و FEMA-356 لتقييم أداء المباني خلال الزلازل. وقد تم اعتماد الأكواد IBC 2012 و ASCE/SEI 7-10 كمرجع للتصميم الزلزالي حيث تم استخدام المعاملات الزلزالية التي تخص منطقة الدراسة. وقد تم استخدام برنامج SAP2000 لتطبيق الطريقة المقترحة على عدد من الحالات الدراسية في قطاع غزة.

ثماني حالات دراسية تمثل المباني السكنية منخفضة الارتفاع الموجودة في قطاع غزة قد تم تقييمها في هذا البحث. وقد شملت الحالات الدراسية عدة عوامل متغيرة مثل: عدد الطوابق، وجود حوائط البلوك، وجود طابق المظلة، ووجود العناصر الإنشائية التي يمكن أن تساهم في مقاومة الأحمال الأفقية مثل الجدران الخرسانية للمصاعد والسلالم. من خلال هذا البحث ولكل حالة دراسية تم الحصول على: العلاقة بين قوى القص والإزاحة الأفقية للمبنى في كلا الاتجاهين، الشكل المتشوه للمبنى، عدد plastic hinges المتكونة في المبنى ومستوى أدائها، نقطة أداء المبنى ومستوى الأداء للمبنى ككل والذي من خلاله نحدد مدى كفاءة هذا المبنى لمقاومة الزلازل.

استنادًا إلى نتائج هذا البحث، فإن المباني السكنية المنتظمة منخفضة الارتفاع في قطاع غزة والمصممة للأحمال الرأسية فقط تعتبر آمنة زلزاليًا. كذلك فإن وجود حوائط البلوك يؤثر بشكل إيجابي على أداء المباني خلال الزلازل حيث أنه يزيد من الصلابة الجانبية للمبنى وبالتالي يعزز مقاومته للزلازل. وجود طابق المظلة يقلل من الصلابة الجانبية للمباني بشكل كبير، وبالتالي يقلل من مقاومتها للزلازل وربما يؤدي إلى أضرار إنشائية تؤدي إلى الانهيار وخاصة في المباني العالية نسبيًا والتي تتكون من خمسة طوابق فأكثر. الحوائط الخرسانية للمصاعد أو السلالم تعزز المقاومة الزلزالية للمباني إذا تم اختيار مواقعها بعناية بحيث لا ينتج عنها عدم انتظام أفقي يؤدي إلى حصول التواء في المبنى.

على الرغم من أن هذا البحث شمل تقييم المباني السكنية الخرسانية في قطاع غزة فقط، فإن استنتاجات هذا البحث يمكن أن تكون متاحة بسهولة لاستخدامها في مناطق أخرى مع مبان مشابهة.

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ABBREVIATIONS

ACI	:	American Concrete Institute
ADRS	:	Acceleration-Displacement Response-Spectra
ASCE	:	American Society for Civil Engineers
ATC	:	Applied Technology Council
CP	:	Collapse Prevention
CSM	:	Capacity Spectrum Method
DBA	:	Displacement-Based Assessment
DCR	:	Demand/Capacity Ratio
DPM	:	Damage Probability Matrices
DST	:	Dead Sea Transform
EMS-98	:	European Macroseismic Scale 1998
FEMA	:	Federal Emergency Management Agency
IBC	:	International Building Code
IO	:	Immediate Occupancy
LS	:	Life Safety
MCE	:	Maximum Considered Earthquake
MoLG	:	Ministry of Local Government
MoPWH	:	Ministry of Public Works and Housing
MSK	:	Medvedev–Sponheuer–Karnik Scale
PBSD	:	Performance Based Seismic Design
PGA	:	Peak Ground Acceleration
R.C.	:	Reinforced Concrete
SEI	:	Structural Engineering Institute
UBC	:	Uniform Building Code

CHAPTER 1

INTRODUCTION

1 INTRODUCTION

1.1 General

Earthquakes over the ages killed large number of people and destroyed large number of buildings. Thus, ensuring the safety of people and buildings during earthquakes is a matter of concern. The experience gained from past earthquakes demonstrates that damages are occurred to buildings that do not meet the requirements of seismic resistance design, e.g. buildings designed to resist gravity loads only. Various codes and regulations have been developed all around the world to design new structures to have adequate reinforcement detailing to provide an adequate ductile behavior necessary to resist a targeted earthquake. For existing buildings that were not designed to resist seismic loads, seismic evaluation and rehabilitation guidelines need to be developed to assess the behavior of those buildings in order to propose the required strengthening.

Palestine is vulnerable to earthquakes due to its location between the Arabian and African tectonic plates. During the last two millenniums, Palestine exposed to a number of earthquakes that killed thousands of people and destroyed thousands of buildings. Due to these facts, the need for an evaluation of the seismic resistance of buildings in Palestine is a necessity.

The reinforced concrete building frame (not moment resisting) with masonry infill is the most common type of construction of buildings in Gaza Strip. This system is generally consisted of frame system providing support to vertical loads and a lateral load resisting system such as shear walls, moment frames, etc. In Gaza Strip, this system consists of one-way or two-way ribbed slabs supported on columns which in turn transfer the loads to footings which are supported on the soil. The design and construction practice in Gaza Strip show that most of reinforced concrete buildings having up to 7 stories are designed to resist gravity loads only, without any considerations to seismic resistance design. It is generally assumed by designers that the seismic forces on such buildings are low. The building frame structural system and non-structural elements, e.g. partitions are assumed to resist such loads. These assumptions are seldom verified by designers. Seismic resistance assessment of those buildings is the verification tool for those assumptions.

1.2 Research Problem

Most of typical reinforced concrete buildings in Gaza strip are designed and constructed to resist gravity loads without any considerations to seismic resistance. In addition, such buildings may have typical deficiencies such as:

1. In-adequate column-beam joint detailing.
2. Presence of soft-story.
3. Presence of cantilevers.
4. Presence of various horizontal and vertical irregularities, etc.

Normally, design and general non-seismic reinforcement detailing provisions of ACI 318 "*Building Code Requirements for Reinforced Concrete*" are used.

Therefore, it is necessary to evaluate the adequacy of such buildings to resist seismic forces. Conclusion of the evaluation would guide engineers in designing new buildings. Also, it will help in determining possible strengthening of existing buildings.

1.3 Research Aim and Objectives

The ultimate aim of this research is to reduce the seismic risk in existing and new reinforced concrete buildings in Gaza Strip. This aim is intended to be achieved by accomplishing the following objectives:

1. Investigate the performance and identify the structural deficiencies of the typical reinforced concrete buildings in Gaza Strip during earthquakes.
2. Determine the contribution of infill walls on the overall strength of the building.
3. Assess the performance of buildings with soft stories.
4. Outline guidelines for designing similar new buildings.
5. Assist in determining strengthening techniques to increase the ability of existing buildings to withstand earthquakes.

1.4 Research Scope and Limitations

This research is concerned with the evaluation of low-rise reinforced concrete buildings that are designed only for gravity loads in resisting seismic forces in Gaza Strip. Low-rise buildings are buildings with height not exceed 21 meters, i.e. number of stories does not exceed 7 stories. Multi-story buildings are out of the scope of this research (locally referred to as towers).

The contribution from the building frame structural system, infill masonry walls, etc. in resisting seismic forces will be considered. The analysis and evaluation will be carried out using the pushover analysis procedure proposed by ATC 40 "*Seismic evaluation and retrofit of concrete buildings*" and FEMA 356 "*Pre-standard and commentary for seismic rehabilitation of buildings*". Other documents may also be used. SAP2000 software is used to perform the pushover analysis in this research.

Buildings of normal use, e.g. residential buildings will be considered in this research. Regular reinforced concrete buildings are analyzed in this research. Although, most used vertical and horizontal irregularities in the targeted buildings such as soft story and cantilevers will be evaluated.

This research utilized the seismic parameters of IBC 2012 "*International Building Code*".

Although the buildings of Gaza Strip will be evaluated, the conclusion of the research can be utilized in other locations having similar buildings.

1.5 Research Methodology

The research objectives are intended to be achieved by conducting the following activities shown in Fig. (1.1):

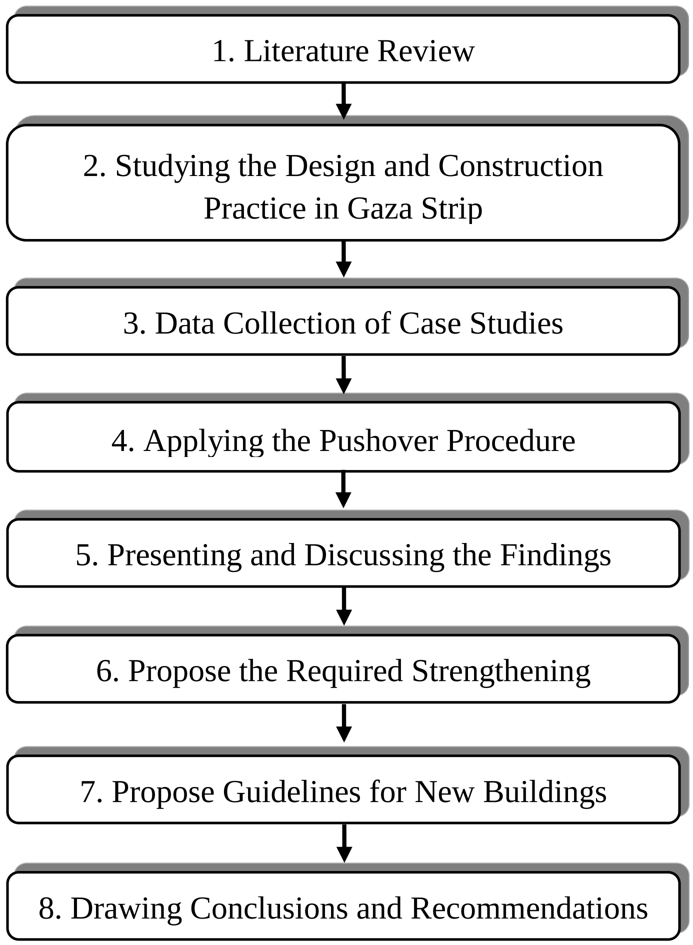


Fig. (1.1): Research Methodology.

1.6 Structure of the Research

This research consists of 7 chapters and references as follows:

Chapter 1 (Introduction): This chapter includes the research problem, aim and objectives, scope and limitations, and research methodology.

Chapter 2 (Literature Review): This chapter includes a review of the earthquake mechanism and a historical background about the seismic condition of Palestine. It also includes a review of the available seismic evaluation methodologies. The used seismic evaluation methodology in this research is determined in this chapter.

Chapter 3 (Design and Construction Practices in Gaza Strip): This chapter includes a classification of Gaza Strip Buildings according to their construction materials, structural systems, and use. The type of buildings that will be analyzed in this research is identified in this chapter. The design and construction practices of Gaza Strip related to the topic of this research which may affect the results of analysis are reviewed in this chapter.

Chapter 4 (Pushover Analysis): This chapter includes an overview and implementation of the pushover analysis procedures.

Chapter 5 (Implementation of Pushover Analysis by SAP2000): This chapter includes the modelling issues related to the implementation of pushover analysis with SAP2000 such as the modelling of frame elements, infill walls, earthquake, and plastic hinges. It also includes a description of the general steps of performing the pushover analysis using SAP2000.

Chapter 6 (Application of Pushover Analysis to Gaza Strip Buildings): This chapter includes the application of pushover analysis to case studies from Gaza Strip. Results of analysis and discussion of these results are presented in this chapter. Conclusions regarding the condition of Gaza Strip buildings during earthquakes are drawn.

Chapter 7 (Conclusion and Recommendations): This chapter includes the conclusion of this research and recommendations for existing buildings, new designs, concerned public authorities and for future researches.

References.

CHAPTER 2

LITERATURE REVIEW

2 LITERATURE REVIEW

2.1 Introduction

This chapter presents a summary of the literature review which includes a review of earthquake phenomenon and the buildings behavior during earthquakes. It also includes a description of the seismic condition of Palestine and the historical records of earthquakes that occurred in Palestine. Furthermore, this chapter includes a review and discussion about some of the available seismic evaluation methodologies for existing buildings and outlines the findings of recent researches utilizes these methodologies from different parts of the world. It also outlines the findings of the recent researches carried out in the field of seismic evaluation of existing buildings in Palestine.

The main purpose of this review is to identify the most suitable seismic evaluation methodology to be used for the evaluation of the seismic resistance of the existing reinforced concrete buildings in Gaza Strip that are designed for gravity loads only.

2.2 Earthquakes Mechanism

Several theories explained the mechanism of earthquakes. Plate tectonics theory visualizes the earth as consisting of a viscous, molten magma core with a number of lower-density rock plates floating on it called tectonic plates shown in Fig. (2.1). The exposed surfaces of the plates form the continents and the bottoms of the oceans. As time goes by, the plates move relative to each other, breaking apart in some areas and jamming together in others. Where the plates are moving apart, this movement causes cracks (or rifts) to form, generally in the ocean beds. The regions where the plates are either moving into each other or are sliding adjacent to each other are referred to as fault zones. Compression and shear stresses are generated in the plates and strain energy builds up in at the edges of the plates. At some point in time, the stresses and strain energy at a locked fault exceeds the limiting resistance to rupture or slip along the fault. Once started, energy is released rapidly, causing intense vibrations to propagate out from the fault. Three main types of stress waves travel through the rock layers: primary (compression) waves, secondary (shear) waves, and surface waves-each at different speeds. As a result, the effects of these seismic waves and local soil conditions will lead to different ground motions at various sites which called earthquake [Wight and MacGregor, 2012].

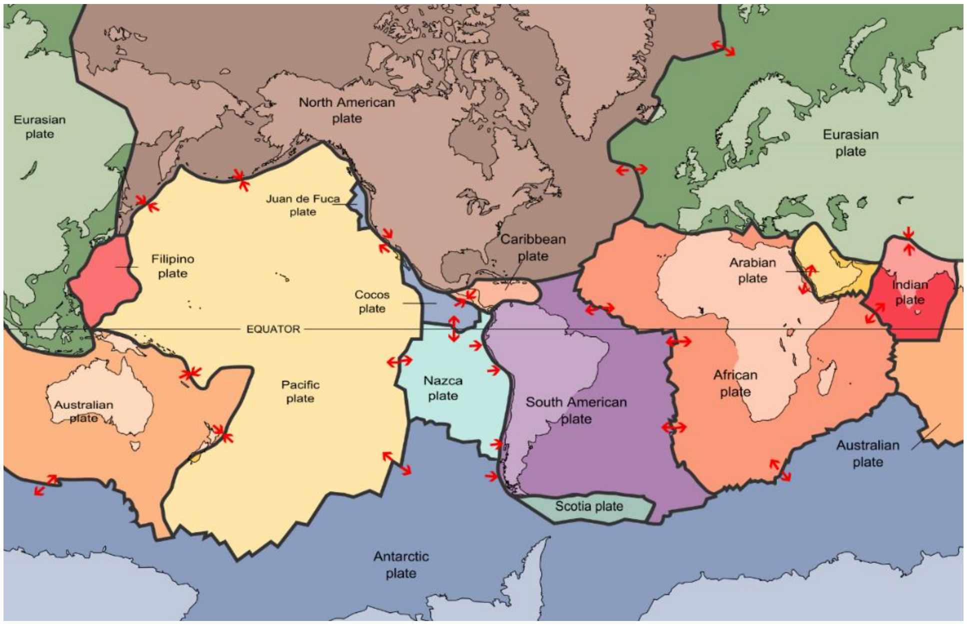


Fig. (2.1): World Tectonic Plates. [Source: http://pubs.usgs.gov**]**

2.3 Building Behavior during Earthquakes

The behavior of a building during an earthquake is a vibration problem. The seismic motions of the ground do not damage a building by impact or by externally applied pressure such as wind, but by internally generated inertial forces caused by vibration of the building mass as shown in Fig. (2.2). An increase in mass has two undesirable effects on the earthquake design. First, it results in an increase in the force, and second, it can cause buckling or crushing of columns and walls when the mass pushes down on a member bent or moved out of plumb by the lateral forces. This effect is known as the P-Δ effect and the greater the vertical forces, the greater the movement due to P-Δ. The magnitude of inertia forces induced in an earthquake depends on the building mass, ground acceleration, the nature of the foundation, and the dynamic characteristics of the structure [Taranath, 2005].

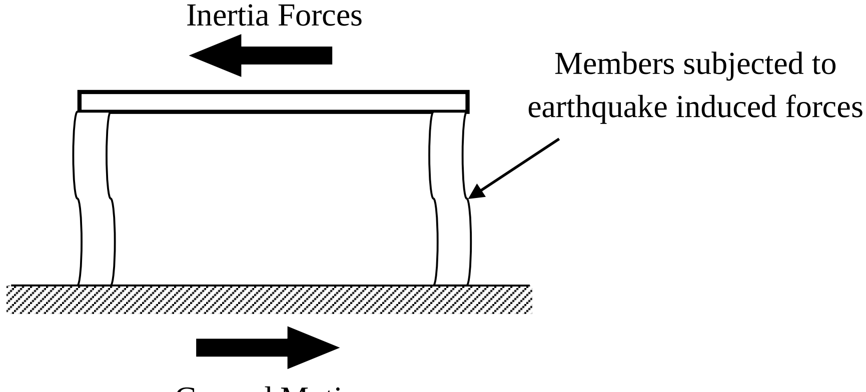


Fig. (2.2): Inertia Forces on a Structure.

2.4 Seismic Condition in Palestine

Palestine is located along the Dead Sea Transform fault (DST) which is considered as one of the most active faults in the eastern Mediterranean. The DST extends from Gulf of Aqaba in the northern part of the Red Sea to the Alpine convergence zone in the Taurus Mountains, where the Arabian plate separates from the Africa plate a distance of some 1000 km. It forms the boundary between the Arabian plate and the Sinai Palestine sub-plate. Studies of historical earthquakes occurred in Palestine and vicinity countries for the past few hundred years demonstrate that the damaging earthquakes were located along this fault as shown in Fig. (2.3). The largest destructive recorded earthquake (Nablus Earthquake) occurred on 11 July 1927 north to Jericho at the boundary between the Arabian and the Sinai plates and had a magnitude of about 6.3 resulting in 500 deaths. An earthquake in 1837 killed 5,000 people. In 31 B.C. Earthquake, 30,000 people lost their lives. Studies of instrumental earthquakes reflect also the ongoing seismic activity of the DST [Dabeek, 2008].

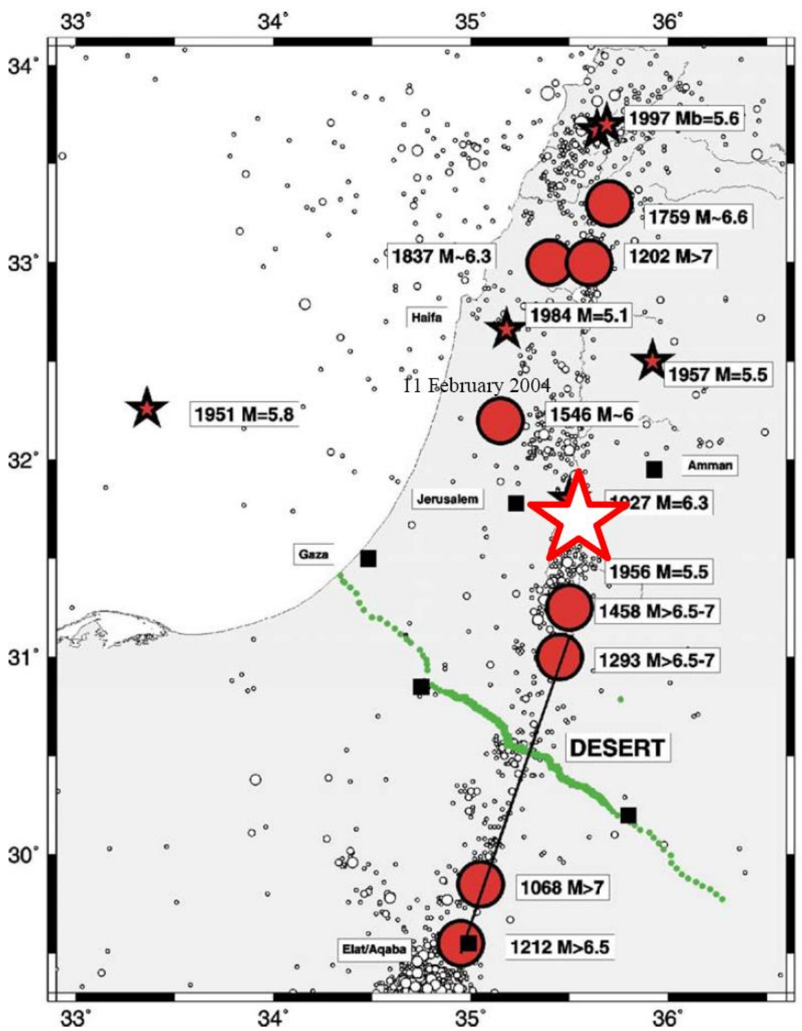


Fig. (2.3): Seismicity Map of the Dead Sea Transform Region. [Source: Dabeek, 2008]

2.5 Seismic Design Procedures

Seismic design procedures can be classified into two types: force-based or performance-based seismic design.

2.5.1 Force-Based Seismic Design

Current seismic design in most countries in the world is carried out in accordance with force-based design methodology. The force-based design sequence is given in Fig. (2.4).

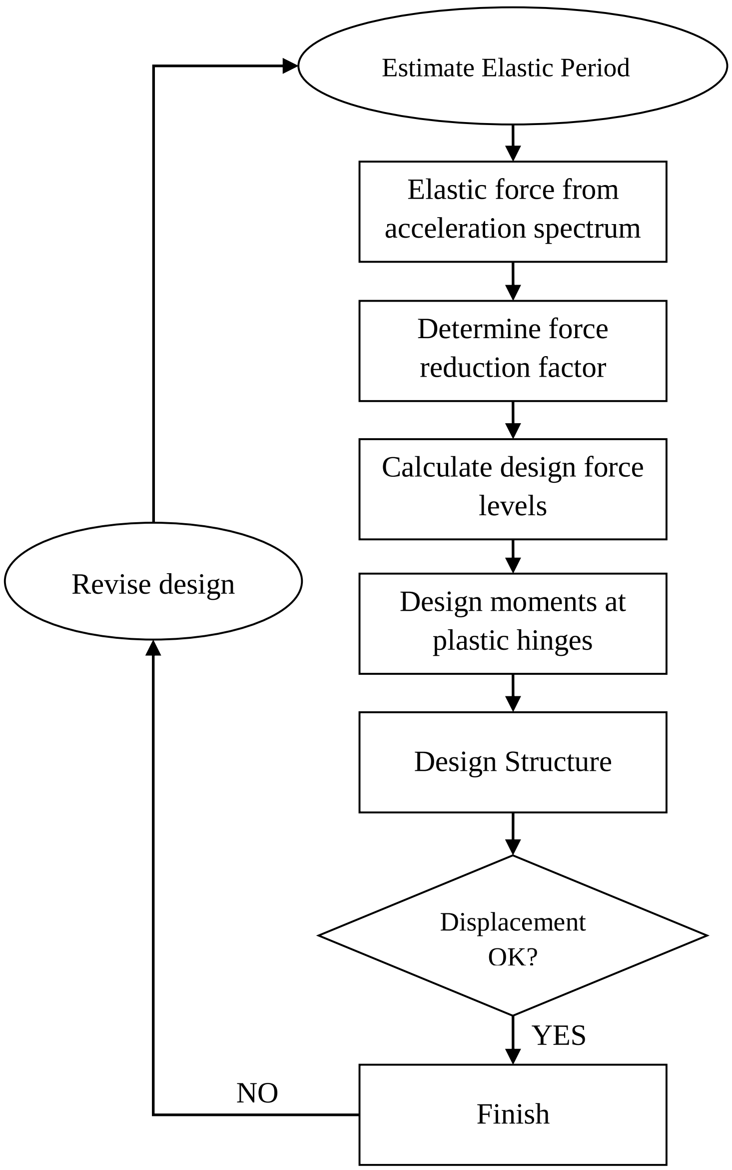


Fig. (2.4): Design Sequence of Force-Based Design.

Fig. (2.4) briefly shows the process of determining design base shear as used in most of the current practices around the world. The force reduction factor (R) depending upon assumed ductility of the structural system, and the importance factor (*I*) represents occupancy factor to increase the design force for more important buildings. Lateral design forces at the floor levels (along the building height) are then determined according to the prescribed formulas to represent dynamic characteristics of the structure. Elastic analysis is performed to determine the required member strengths. After member section design for strength, a deflection amplification factor, C_d, is then used to multiply the calculated drift obtained from elastic analysis to check the specified limits. The process is repeated in an iterative manner until the strength and drift requirements are satisfied. Proper detailing provisions are followed in order to meet the expected ductility demands.

In summary, the major weaknesses of the current code procedure are:

1. Assuming safety could be guaranteed by increasing the design base shear: it has been observed in many past earthquakes that collapse occurred due to local column damage.
2. Assuming design lateral force distribution along the building height based on elastic behavior: Nonlinear dynamic analyses 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.
3. Proportioning member sizes based on initial stiffness (i.e. elastic analysis): The magnitude of individual member forces from elastic analysis is obtained based on relative elastic stiffness of structural members. However, when subjected to major earthquakes, stiffness of many members changes significantly due to concrete cracking or yielding in steel, while that of others may remain unchanged. This alters the force distribution in the structural members. Proper proportioning of member sizes cannot be achieved without using a more representative force distribution which takes into account the expected inelastic behavior.
4. Attempting to predict inelastic displacements by using approximate factors and analysis behavior: This has been shown by many prior investigations to be unrealistic, especially for structures having degrading hysteretic behavior and energy dissipation characteristics [Liao, 2010].

2.5.2 Performance Based Seismic Design (PBSD)

It is an iterative process that begins with the selection of performance objectives, followed by the development of a preliminary design, an assessment as to whether or not the design meets the performance objectives, and finally redesign and reassessment, if required, until the desired performance level is achieved. Fig. (2.5) shows a flowchart that presents the key steps in the performance-based design process.

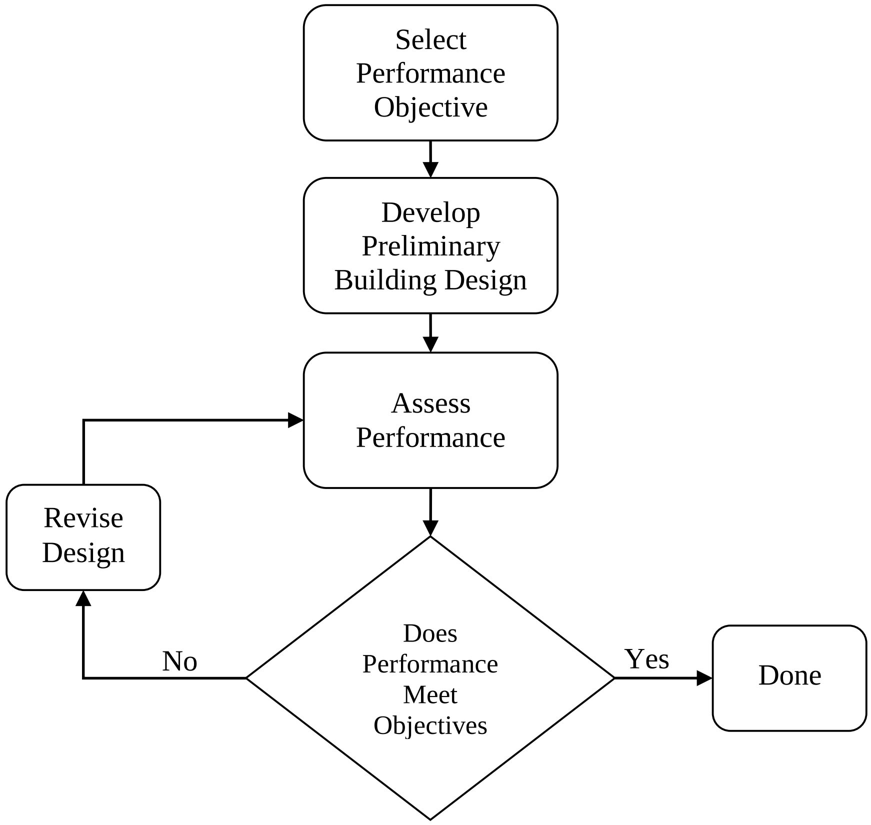


Fig. (2.5): Performance-Based Design Flow Chart.

Performance-based design begins with the selection of design criteria stated in the form of one or more performance objectives. Each performance objective is a statement of the acceptable risk of incurring specific levels of damage, and the consequential losses that occur as a result of this damage, at a specified level of seismic hazard.

Once the performance objectives are set, a series of simulations (analyses of building response to loading) are performed to estimate the probable performance of the building under various design scenario events. In the case of extreme loading, as would be imparted by a severe earthquake, simulations may be performed using nonlinear analysis techniques. If the simulated performance meets or exceeds the performance objectives, the design is complete. If not, the design is revised in an iterative process until the performance objectives are met. In some cases it may not be possible to meet the stated objective at reasonable cost, in which case, some relaxation of the original objectives may be appropriate [FEMA 445, 2006].

2.6 Seismic Evaluation Methodologies

Damages of existing buildings and loss of lives during a large number of earthquakes in different parts of the world has demonstrated the need for seismic resistance evaluation of the existing buildings especially those that are not designed to resist seismic loads. Based on that need, various organizations in various countries have introduced methodologies and guidelines for the seismic evaluation of existing buildings.

Calvi et al. (2006) classified the available seismic evaluation methodologies into two main categories: empirical (qualitative) methods and analytical (quantitative) methods. The empirical seismic evaluation methodologies are based on identifying damage patterns suffered during past seismic effects to assess the expected damage for a given building typology during future earthquakes. In another way, it tried to find damage in a building type due to a predetermined earthquake. This damage was then extrapolated to evaluate city based or region based damage. The analytical seismic evaluation methodologies are based on a more detailed seismic evaluation with a complete numerical analysis of the building to express the relationship between seismic intensity and expected damage [Calvi et al., 2006].

Rai (2003) classified the available seismic evaluation procedures into two categories: (a) configuration-related and (b) strength-related checks. The configuration-related checks involve a quick assessment of the earthquake resistance of the building by assessing the configurationally induced deficiencies known for unsatisfactory performance along with a few global level strength checks. Typical building configuration deficiencies include an irregular geometry, a weakness in a given story, a concentration of mass, or a discontinuity in the lateral force resisting system. The objective of the configuration-related checks is to screen out the significantly vulnerable structures for the detailed analysis and evaluation. The strength-related checks consist of proper force and displacement analysis to assess structural performance at both global and/or component level. Number of the available seismic evaluation methodologies are a combination of configuration-related checks and strength-related checks [Rai, 2003].

2.6.1 Empirical Evaluation Methodologies

According to Calvi et al. (2006), the use of empirical methods in the seismic assessment of buildings in the early 70’s of the past century is came as a result of the fact that seismic hazard maps were defined in terms of a macroseismic intensity scales such as the MSK scale [Medvedev and Sponheuer, 1969], the Modified Mercalli scale [Wood and Neumann, 1931] and the EMS98 scale [Grünthal, 1998].

Empirical methods of seismic assessment of buildings can be classified into three main types: damage probability matrices (DPM), vulnerability index methods, and screening methods.

It should be noted that the word “vulnerability” is used to express differences in the way that buildings respond to earthquake shaking. If two groups of buildings are subjected to the same earthquake shaking, and one group performs better than the other, then it can be said that the buildings that were less damaged had lower earthquake vulnerability than the ones that were more damaged, and vice versa.

2.6.1.1 Damage Probability Matrices

DPM are based on the concept that a given structural typology will have the same probability of being in a given damage state for a given earthquake intensity. The first DPMs have been proposed by Whitman et al. [Whitman et al., 1973]. For a given structural typology, the probability of being in a given state of structural and non-structural damage is provided. For each damage state, the damage ratio is provided too, representing the ratio between the cost of repair and the cost of replacement. These DPMs are compiled for different structural typologies based on the damage observed in over 1600 buildings after the 1971 San Fernando earthquake. Table (2.1) presents the DBM proposed by Whitman et al.

Table (2.1): Format of the DBM proposed by Whitman et al. (1973).

Damage State	Structural Damage	Non-structural Damage	Damage Ratio (%)	Intensity of Earthquake				
				V	VI	VII	VIII	IX
0	None	None	0-0.05	10.4	-	-	-	-
1	None	Minor	0.05-0.3	16.4	0.5	-	-	-
2	None	Localized	0.3-1.25	40.0	22.5	-	-	-
3	Not noticeable	Widespread	1.25-3.5	20.0	30.0	2.7	-	-
4	Minor	Substantial	3.5-4.5	13.2	47.1	92.3	58.8	14.7
5	Substantial	Extensive	7.5-20	-	0.2	5.0	41.2	83.0
6	Major	Nearly total	20-65	-	-	-	-	2.3
7	Building condemned		100	-	-	-	-	-
8	Collapse		100	-	-	-	-	-

Braga et al. proposed the first European version of DPMs based on the damage observed after the 1980 Irpinia earthquake. Three vulnerability classes (A, B and C) corresponding to different building typologies are defined, and the seismic intensity measure is based on the MSK scale [Braga et al., 1982].

The DPMs from Braga et al. are adapted for the town of Potenza by Dolce et al. adding the vulnerability class D, which represents the buildings constructed since 1980, and expressing the seismic intensity according to the European Macroseismic Scale (EMS-98) [Dolce et al., 2003].

The use of EMS-98 scale in the vulnerability assessment of a structure in the field include two main steps, the first step is to determine the building type in order to determine the basic vulnerability class from the vulnerability table. Table (2.2) shows the vulnerability table.

The second step is to assign an earthquake intensity (EMS-98 include 12 intensity degrees) to the region under consideration. Based on the intensity and the vulnerability class of the building, the grade of damage is assigned to each building. The EMS-98 scale classified damages to 6 grades. Table (2.3) shows classification of damage to masonry buildings and Table (2.4) shows classification of damage to reinforced concrete buildings.

Table (2.2): EMS-98 Vulnerability Table.

	Type of Structure	A	B	C	D	E	F
MASONRY	rubble stone, fieldstone	O					
	adobe (earth brick)	O	—				
	simple stone	·····	O				
	massive stone		—	O	·····		
	unreinforced, with manufactured stone units	·····	O	·····			
	unreinforced, with RC floors		—	O	·····		
	reinforced or confined			·····	O	—	
REINFORCED CONCRETE (RC)	frame without earthquake-resistant design (ERD)	·····	—	O	·····		
	frame with moderate level of ERD		·····	—	O	—	
	frame with high level of ERD			·····	—	O	—
	walls without ERD		·····	O	—		
	walls with moderate level of ERD			·····	O	—	
	walls with high level of ERD				·····	O	—
STEEL	steel structures			·····	—	O	—
WOOD	timber structures		·····	—	O	—	

O most likely vulnerability class; — probable range;
····· range of less probable, exceptional cases

The definition of the EMS-98 intensity degrees in regard to buildings damage is:

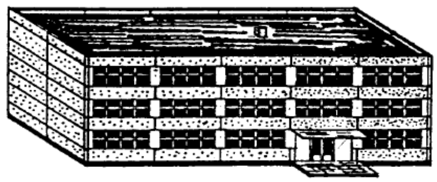
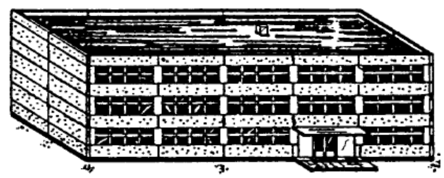
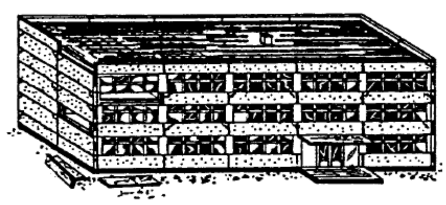
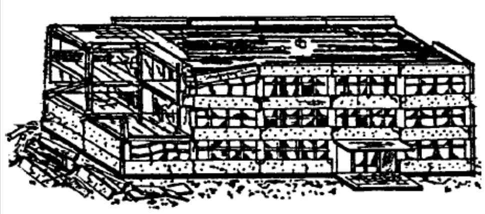
- **I. Not felt**: No damage.
- **II. Scarcely felt**: No damage.
- **III. Weak**: No damage.
- **IV. Largely observed**: No damage.

- **V. Strong**: Damage of grade 1 to a few buildings of vulnerability class A and B.
- **VI. Slightly damaging**: Damage of grade 1 is sustained by many buildings of vulnerability class A and B; a few of class A and B suffer damage of grade 2; a few of class C suffer damage of grade 1.
- **VII. Damaging**: Many buildings of vulnerability class A suffer damage of grade 3; a few of grade 4. Many buildings of vulnerability class B suffer damage of grade 2; a few of grade 3. A few buildings of vulnerability class C sustain damage of grade 2. A few buildings of vulnerability class D sustain damage of grade 1.
- **VIII. Heavily damaging**: Many buildings of vulnerability class A suffer damage of grade 4; a few of grade 5. Many buildings of vulnerability class B suffer damage of grade 3; a few of grade 4. Many buildings of vulnerability class C suffer damage of grade 2; a few of grade 3. A few buildings of vulnerability class D sustain damage of grade 2.
- **IX. Destructive**: Many buildings of vulnerability class A sustain damage of grade 5. Many buildings of vulnerability class B suffer damage of grade 4; a few of grade 5. Many buildings of class C suffer damage of grade 3; a few of grade 4. Many buildings of vulnerability class D suffer damage of grade 2; a few of grade 3. A few buildings of vulnerability class E sustain damage of grade 2.
- **X. Very destructive**: Most buildings of vulnerability class A sustain damage of grade 5. Many buildings of vulnerability class B sustain damage of grade 5. Many buildings of vulnerability class C suffer damage of grade 4; a few of grade 5. Many buildings of vulnerability class D suffer damage of grade 3; a few of grade 4. Many buildings of vulnerability class E suffer damage of grade 2; a few of grade 3. A few buildings of vulnerability class F sustain damage of grade 2.
- **XI. Devastating**: Most buildings of vulnerability class B sustain damage of grade 5. Most buildings of class C suffer damage of grade 4; many of grade 5. Many buildings of class D suffer damage of grade 4; a few of grade 5. Many buildings of vulnerability class E suffer damage of grade 3; a few of grade 4. Many buildings of vulnerability class F suffer damage of grade 2; a few of grade 3.
- **XII. Completely devastating**: All buildings of vulnerability class A, B and practically all of vulnerability class C are destroyed. Most buildings of vulnerability class D, E and F are destroyed. The earthquake effects have reached the maximum conceivable effects.

Table (2.3): Classification of Damage to Masonry Buildings.

Classification of damage to masonry buildings	
	**Grade 1: Negligible to slight damage** **(no structural damage,** **slight non-structural damage)** Hair-line cracks in very few walls. Fall of small pieces of plaster only. Fall of loose stones from upper parts of buildings in very few cases.
	**Grade 2: Moderate damage** **(slight structural damage, moderate non-structural damage)** Cracks in many walls. Fall of fairly large pieces of plaster. Partial collapse of chimneys.
	**Grade 3: Substantial to heavy damage** **(moderate structural damage,** **heavy non-structural damage)** Large and extensive cracks in most walls. Roof tiles detach. Chimneys fracture at the roof line; failure of individual non-structural elements (partitions, gable walls).
	**Grade 4: Very heavy damage** **(heavy structural damage,** **very heavy non-structural damage)** Serious failure of walls; partial structural failure of roofs and floors.
	**Grade 5: Destruction** **(very heavy structural damage)** Total or near total collapse.

Table (2.4): Classification of Damage to Buildings of Reinforced Concrete.

Classification of damage to buildings of reinforced concrete	
	**Grade 1: Negligible to slight damage** **(no structural damage,** **slight non-structural damage)** Fine cracks in plaster over frame members or in walls at the base. Fine cracks in partitions and infills.
	**Grade 2: Moderate damage** **(slight structural damage,** **moderate non-structural damage)** Cracks in columns and beams of frames and in structural walls. Cracks in partition and infill walls; fall of brittle cladding and plaster. Falling mortar from the joints of wall panels.
	**Grade 3: Substantial to heavy damage** **(moderate structural damage,** **heavy non-structural damage)** Cracks in columns and beam column joints of frames at the base and at joints of coupled walls. Spalling of conrete cover, buckling of reinforced rods. Large cracks in partition and infill walls, failure of individual infill panels.
	**Grade 4: Very heavy damage** **(heavy structural damage,** **very heavy non-structural damage)** Large cracks in structural elements with compression failure of concrete and fracture of rebars; bond failure of beam reinforced bars; tilting of columns. Collapse of a few columns or of a single upper floor.
	**Grade 5: Destruction** **(very heavy structural damage)** Collapse of ground floor or parts (e. g. wings) of buildings.

DPMs methodology is not fully suitable to be used in the seismic assessment of Gaza Strip buildings for the following reasons:

1. There is no observed damage data from previous earthquakes for Gaza Strip buildings to predict effects of future earthquakes. This methodology can be used in Gaza Strip in case of observed damage data for a region of similar characteristics is available.
2. Seismic hazard maps are no longer defined in terms of macroseismic intensities. Seismic hazard maps are now defined in terms of PGA.
3. This methodology doesn't include the evaluation of retrofit options.

2.6.1.2 Vulnerability Index Methods

The "Vulnerability Index" method is first proposed by Benedetti and Petrini. The index I_v is evaluated by means of a field survey form where "scores" K_i (from A to D) are assigned to eleven parameters having a high influence on building vulnerability (e.g., plan and elevation configuration, type of foundation, structural and non-structural elements); then, the index is defined as the weighted sum according to the importance assigned to each parameter [Benedetti and Petrini, 1984].

$$I_v = \sum_{i=1}^{11} K_i W_i$$

Based on observed damage data from past earthquakes, for different values of this vulnerability index a relationship can be calibrated between seismic intensity and damage ratio (see Fig. 2.6).

The main advantage of vulnerability index methods is that they allow the vulnerability characteristics of the building stock under consideration to be determined, rather than base the vulnerability definition on the typology alone as in DPMs methodology.

This methodology is also not suitable to be used in the seismic assessment of Gaza Strip buildings because it requires an observed damage data from previous earthquakes.

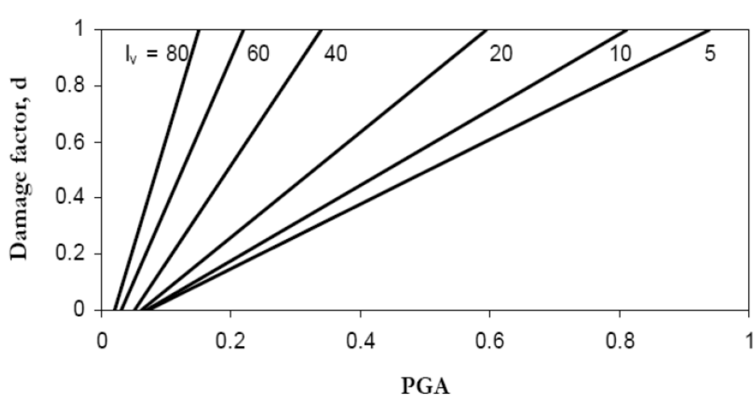


Fig. (2.6): Vulnerability functions to relate damage factor (*d*) and peak ground acceleration (PGA) for different values of vulnerability index (I_v).

2.6.1.3 Screening Methods

This method is adopted in the Japanese Seismic Index Method [JBDPA, 1990]. The seismic performance of the building in this method is represented by a seismic performance index (I_S), evaluated by means of a screening procedure. (I_S) is calculated for each story in every frame direction according to the following expression:

$$I_S = E_0 S_D T$$

E_0, S_D and T correspond to the basic structural performance, to the structural design and to the time-dependent deterioration of the building, respectively. E_0 is given by the product between C and F, respectively representing the ultimate strength and the ductility of the building, depending on the failure mode, the total number of stories and the position of the considered story. S_D accounts for irregularity in stiffness and/or mass distribution. A field survey is needed to define T. The calculated seismic performance index (I_S) is compared with the seismic judgment index I_{S0} to determine the degree of safety of the building. I_{S0} represents a story shear force and is given by:

$$I_{S0} = E_S Z G U$$

E_S conservatively increases with the decreasing accuracy of the screening procedure, Z is a zone index modifying the ground motion intensity assumed at the site of the building, G accounts for local effects such as ground-building interaction or stratigraphic and topographic amplification and U is a kind of importance factor of the building.

Preliminary assessment methods based on screening procedures have been proposed in Turkey, during last years. Some methods require the dimensions of the lateral load resisting elements to be defined: the "Priority Index" proposed by Hassan and Sozen is a function of a wall index (area of walls and infill panels divided by total floor area) and a column index (area of columns divided by total floor area); the "Capacity Index" proposed by Yakut depends on orientation, size and material properties of the lateral load-resisting structural system as well as the quality of workmanship and materials and other features such as short columns and plan irregularities [Hassan and Sozen, 1997] [Yakut, 2004].

The use of screening methods has an important role to play in the definition of prioritization of buildings for seismic retrofit, but the use of such methods in large-scale seismic risk models is limited due to the need to consider buildings individually, and thus this would not be economically feasible.

2.6.2 Analytical Evaluation Methodologies

With the advancement of computational techniques, more complicated methods of seismic evaluation have been suggested. The overall objective of this type of methods are to determine the capacity of the inspected buildings to bear the seismic loads.

Analytical methods can be carried out in absence of past earthquake damage records for similar type of buildings. It also used to evaluate a specific building or type of buildings have the same structural characteristics. Based on that facts, analytical methods have been used to evaluate the seismic resistance of Gaza Strip buildings in the undertaken research.

Analytical methods can be classified into two main types: capacity spectrum-based methods and displacement-based methods.

2.6.2.1 Capacity Spectrum-Based Methods

The Capacity Spectrum Method (CSM), a performance-based seismic analysis technique, can be used for a variety of purposes such as rapid evaluation of a large inventory of buildings, design verification for new construction of individual buildings, evaluation of an existing structure to identify damage states, and correlation of damage states of buildings to various amplitudes of ground motion. The procedure compares the capacity of the structure (in the form of a pushover curve) with the demands on the structure (in the form of response spectra). The graphical intersection of the two curves approximates the response of the structure. In order to account for non-linear inelastic behavior of the structural system, effective viscous damping values are applied to linear-elastic response spectra similar to inelastic response spectra [Freeman, 2004]. This method is also known as pushover analysis. Fig. (2.7) shows the principle of capacity spectrum method.

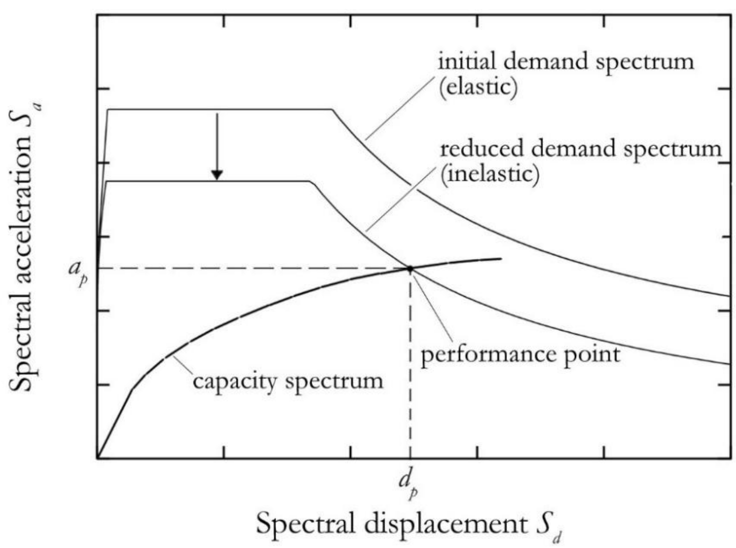


Fig. (2.7): Capacity Spectrum-Based Method.

CSM has been adopted in several guidelines for seismic evaluation of reinforced concrete buildings around the world. ATC-40, 1996 "*Seismic Evaluation and Retrofit of Concrete Buildings*" is one of the most popular document using the CSM. CSM also has been employed in ASCE 31-03 "*Seismic Evaluation of Existing Buildings*"

ASCE 31-03 provides a process for seismic evaluation of existing buildings. This standard has evolved from and is intended to replace FEMA 310 "*Handbook for the Seismic Evaluation of*

Buildings" which considered as the most advanced seismic evaluation procedure for buildings developed in USA [Rai, 2003]. The ASCE 31 standard has incorporated many recent developments in performance based design. [Kehoe, 2004].

The analysis methodology of ASCE 31-03 employs three tiers: the quick check (Tier 1 analysis), a more accurate and calculation intensive (Tier 2 analysis), and a very detailed component evaluation (Tier 3 analysis) involving advance computational methods including non-linear analysis. Fig. (2.8) shows the ASCE 31-03 methodology.

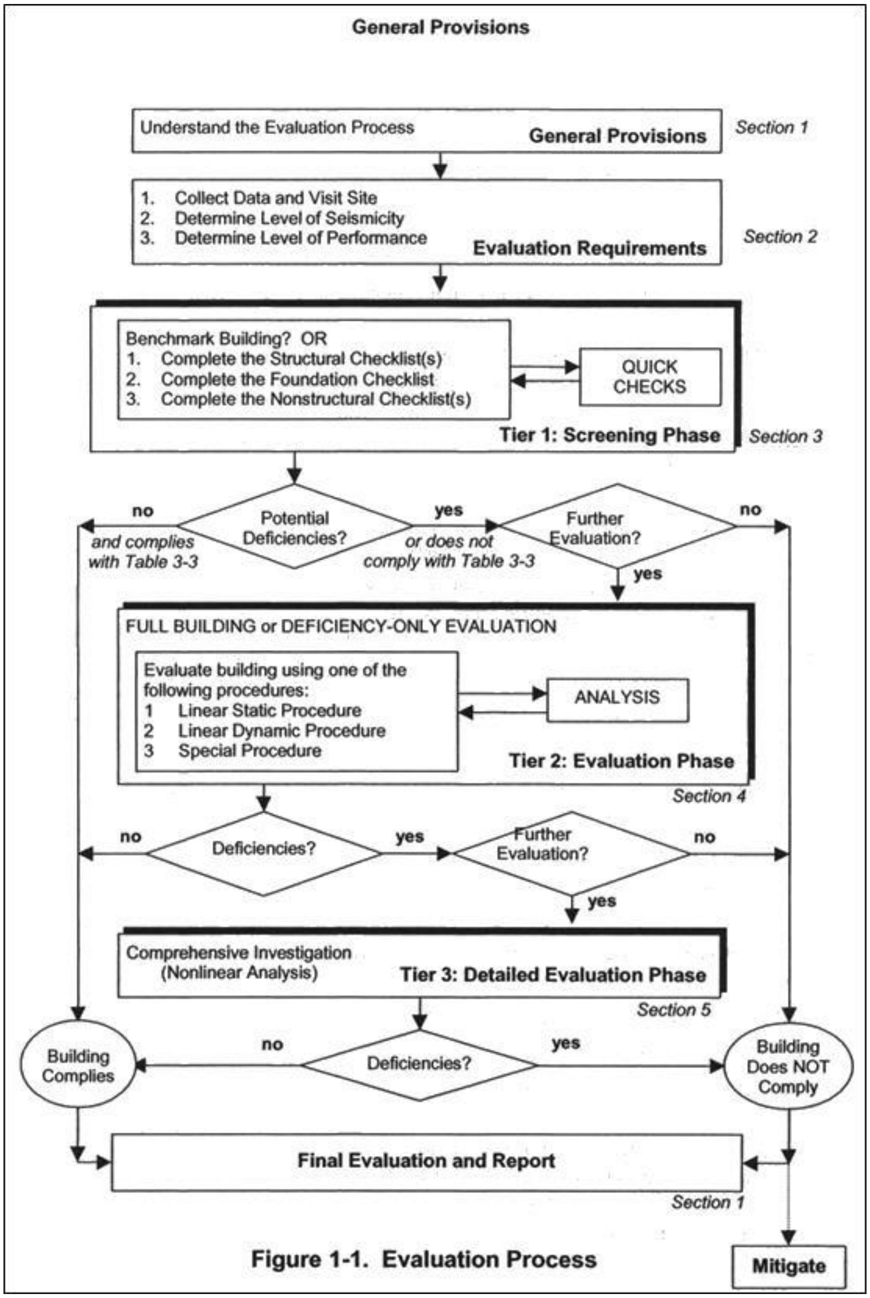


Fig. (2.8): ASCE 31 Evaluation Process.

2.6.2.2 Displacement-Based Methods

The displacement-based assessment (DBA) procedure compares the lateral displacement capacity of a building with the expected lateral displacement demand. The substitute structure single-degree-of-freedom (SDOF) approximation is used to characterize a building as an equivalent linear system responding to the displacement capacity [Kam, 2013]. It is noted that displacement-based assessment may be achieved using direct hand calculation methods [Priestley, 1996, Priestley et al. 2007] or sophisticated non-linear computer analysis as illustrated in ASCE 41-06 "*Seismic Rehabilitation of Existing Buildings*".

In the undertaken research, capacity spectrum method (pushover analysis) has been used since it provides a graphical representation of the demand and capacity of the building and directly identify the performance point of the building.

2.7 Review of Previous Researches on Pushover Analysis

Many researches have been conducted to evaluate the seismic behavior of existing reinforced concrete buildings using pushover analysis.

Ismaeil M. A. (2014) presents a research paper on "Pushover Analysis of Existing 3 Stories RC Flat slab Building". This paper is focused on the study of seismic performance of the existing hospital buildings in the Sudan. The pushover analysis was performed on the building using SAP2000 software. The principles of Performance Based Seismic Engineering are used to govern the analysis. The evaluation has proved that the three stories hospital building is seismically safe [Ismaeil, M. A 2014].

Raju et al. (2015) presents a research paper on "Effective location of shear wall on performance of building frame subjected to earthquake load". This paper deals with the non-linear pushover analysis of building frame for various positions of shear walls. The analysis has been carried out using ETABS software. Pushover curves have been developed and compared for various models. It has been observed that structure with shear wall at appropriate location is more significant in case of displacement and base shear [Raju et al., 2015].

Babu et al., presents a research paper on "Pushover Analysis of Unsymmetrical Framed Structures on Sloping Ground". The paper deals with non-linear analysis of various symmetric and asymmetric structures constructed on plain as well as sloping grounds subjected to various kinds of loads .The analysis has been carried out using SAP2000 and ETABS software. The paper concluded that the structure with vertical irregularity is more critical than a structure with plan irregularity [Babu, et al., 2012].

2.8 Relevant Researches in Palestine

Several researches have been carried out in the field of seismic evaluation of existing buildings in Palestine. The following sub-sections include review of some of these researches and its findings.

2.8.1 Evaluating Seismic Performance of Existing School Buildings in Gaza Strip

This research [Shurrab, 2013] includes a seismic assessment of the existing school buildings in Gaza Strip. The assessment has been carried out based on the EMS-98 scale. The research also includes a seismic evaluation of three samples of the dominant structural systems of school buildings in Gaza Strip by following the guidelines of ASCE 31-03. A comparative study has been carried out on the results obtained from the two approaches.

The results of applying the EMS-98 approach on more than 54 case studies showed that about 60% the school buildings in Gaza Strip is assigned to vulnerability class A and B in which it might expose to full or partial damages (3, 4 and 5 degree of damages as defined by EMS-98) during a specified earthquake scenarios.

The assessment of schools is out of this research scope. It is recommended to evaluate school buildings by using analytical methods to verify this study results.

2.8.2 Structural Needs of Existing Buildings in Gaza for Earthquake Resistance

In this research [Qandil, 2009], a new seismic evaluation method was developed to evaluate the buildings in Gaza Strip in regard to its seismic resistance. The new method has been developed by combining an Israeli method [Scalat, 2007] and a Turkish method [Yakut et al., 2005].

The developed approach was applied on thirty three different Gaza buildings which include: residential housing buildings, tower buildings, schools, health clinic and asbestos shelters. It was found that the structural system used on Gaza Strip which is Skelton type is an appropriate system to resist earthquakes of high intensity. The weakness of this system appeared in the case of the presence of soft story. Tower buildings are classified as intermediate and weak in resisting earthquakes, according to the area of shear walls in the building. The reinforced concrete frame system which is used in public buildings is suitable and adequate to resist earthquakes of high intensity. The asbestos buildings are weak and unsuitable in resisting earthquakes forces.

In the undertaken research, the findings of this research will be verified by using analytical methods of analysis.

2.8.3 Vulnerability, and Expected Seismic Performance of Buildings in West Bank.

This study [Dabeek, 2007] aims to determine the seismic vulnerability of common buildings in Palestine and to estimate the range of damages when exposed to earthquakes. The study was carried out in according to the European Macroseismic Scale EMS-98 and calibrated by using Japanese qualitative method (JBDPA, 1990).

The results of this research indicated that the Palestinian cities could exposed to huge losses due to the damage and full or partial collapse of buildings in the event of a strong or relatively strong earthquakes. The results also showed that one third of the investigated buildings belong to seismic vulnerability of class A (Many buildings of class A will suffer heavy damage); whereas about 40 percent of the buildings indicate class B (Many buildings of class B will suffer moderate damage). The researcher provide several recommendations to decision makers include avoiding the use of seismically unsafe building types, legislating a special laws to enforce engineers to design and construct building according to seismic requirements, and developing plans to rehabilitate and strengthen existing buildings to resist earthquakes.

2.9 Concluded Remarks

A review of existing seismic evaluation methodologies has been presented. The findings of some researches carried out in the field of seismic evaluation of existing buildings in Palestine have been outlined.

It can be concluded that the most suitable seismic evaluation methodology to be used in Gaza strip is the analytical methodologies since they do not require an observed damage data from previous earthquakes. Capacity spectrum method, which is one of the analytical methods, has been used in the undertaken research. Several computer software provide tools for seismic design and evaluation of reinforced concrete buildings such as SAP2000, ETABS, PERFORM 3D, etc. SAP2000 program is used in this research.

CHAPTER 3

DESIGN AND CONSTRUCTION PRACTICES IN GAZA STRIP

3 DESIGN AND CONSTRUCTION PRACTICES IN GAZA STRIP

3.1 Introduction

Earthquakes cause different kinds and levels of damages on buildings. The level of damage depends on several factors. The main factors are the intensity of the earthquake, type of the buildings in terms of the construction material, structural system, use, and the quality of seismic design of the building. Since these factors are significant in determining the behavior of buildings during earthquakes, a detailed study and investigation on the design and construction practices in Gaza Strip was conducted as part of the undertaken research. Information about the types of buildings, construction materials, and design and construction regulations that exist in Gaza Strip was collected. The collected information is important in determining the type of Gaza Strip buildings to be seismically evaluated in this study.

The findings of the study related to the design and the construction practice in Gaza Strip are as follows:

3.2 Building Types

The seismic behavior of buildings during earthquakes depends on their construction materials, structural systems and their use. Accordingly, Gaza strip buildings can be classified as follows:

3.2.1 Classification According to Construction Materials

It is known that different materials behave differently during earthquakes based on their engineering properties such as strength and ductility. This leads us to classify Gaza Strip buildings according to their construction materials in order to understand the behavior of each type of buildings during earthquake. Different types of materials are used in the construction of buildings in Gaza Strip. Gaza Strip buildings can be classified according to their construction materials as follows:

3.2.1.3 Unreinforced Masonry Buildings

This type of buildings is constructed by using an individual blocks bonded to each other by mortar. Sand and rock natural stones and concrete blocks are the most common types of blocks used in the construction of buildings in Gaza Strip. Concrete blocks are commonly used as bearing walls for one story buildings. It also used as an infill walls for reinforced concrete buildings. Concrete blocks buildings can be found mainly in the refugee camps of Gaza Strip as a residential units. Sand and rock natural stones are used in some of old buildings and as cladding in new buildings. The photos in Fig. (3.1) show deferent types of masonry buildings in Gaza Strip.

This type of buildings is seismic vulnerable because it is constructed without following any engineering design principles as well as the brittle behavior of the blocks makes it unfavorable seismic resistant material. Natural stone cladding is considered as a source of danger during earthquakes because it is exposed to fall due to the lack of sufficient attachment to the buildings.

The undertaken research is not concerned with the seismic resistance of this type of buildings. Concrete blocks walls which are used as partitions in reinforced concrete buildings designed for gravity loads only are assumed by Gaza Strip designers as a contributor to the seismic resistance of the buildings. This assumption is investigated in this research.

(a) (b)

Fig. (3.1): Unreinforced Masonry Buildings, (a) Concrete block building, (b) Sand stone building.

3.2.1.4 Reinforced Concrete Buildings

Concrete was used in Palestine for thousands of years [BCA, 1999]. Reinforced concrete is the most widely used material for construction of buildings in Gaza Strip. Reinforced concrete consists mainly of two materials: concrete and reinforcing steel bars. Concrete is a brittle material. This fact makes concrete non seismic-resistant material. Concrete is provided by reinforcing steel bars which enhances its ductility which in turn converts it to a seismic-resistant material. Steel reinforcement also resist tensile stresses that concrete cannot resist.

Since the vast majority of Gaza Strip buildings are constructed by using reinforced concrete, the need for evaluating the seismic resistance of this type of buildings is an important issue. This reason justifies carrying out the undertaken research. The photo in Fig. (3.2) shows reinforced concrete buildings in Gaza Strip.

Fig. (3.2): Reinforced Concrete Buildings in Gaza.

3.2.1.5 Steel Buildings

Structural steel is used in a special type of structures in Gaza Strip. The use of steel is limited to the construction of warehouses, petrol stations, school sheds, etc. Due to the high ductility of the steel material, these structures behave in a good manner during earthquakes. This type of buildings is not within the scope of the undertaken research. The photo in Fig. (3.3) shows a steel structure in Gaza strip.

Fig. (3.3): Steel Structure in Gaza Strip.

3.2.2 Classification According to Structural System

Structural system of buildings plays an important role in the seismic behavior of these buildings during earthquakes. Gaza Strip buildings can be classified according to their structural systems as follows:

3.2.2.1 Masonry Bearing Walls

This type of structures consists mainly of a thin reinforced concrete two-way solid slab supported on concrete blocks bearing walls which in turn supported on either reinforced concrete beams or concrete blocks which transfer loads to the soil. Most of Gaza Strip buildings that were constructed by this system are relatively old because it is linked to the appearance of reinforced concrete in Gaza Strip in the fifties of the last century. This type of structural systems is normally used in buildings which have one or two stories. The photo in Fig. (3.4) shows this type of structures.

Masonry bearing wall buildings are brittle structures and don't follow any design and construction guidelines. So, these buildings may behave in a bad manner during earthquakes. Based on previous studies and available guidelines [Dabeek 2007 and EMS 98], this type of buildings is assigned to class B of seismic vulnerability classes which include masonry structures, i.e. it is considered as high vulnerable to seismic risks. The evaluation of the seismic resistance of this type of buildings is not within the scope of the undertaken research.

Fig. (3.4): Masonry Bearing Walls Buildings.

3.2.2.2 Building Frame System

Building frame system is the most widely used structural system in the construction of reinforced concrete buildings in Gaza Strip. This system is generally consisted of a space frame skeleton system (non-moment resistance) providing support to vertical loads and in some cases it is provided with a lateral load resisting system such as shear walls and moment resisting frames. The space frame system is consisted of one-way or two-way solid or ribbed slabs supported on columns which in turn transfers the loads to isolated, combined, or raft footings which transfer loads to the soil. Concrete block infill walls are used in this system as internal and external walls. The Photo in Fig. (3.5) shows the building frame system.

Fig. (3.5): Building Frame System.

According to the bylaw of urban planning of the Palestinian National Authority and the [System of Multi-Story Buildings] issued by the Palestinian Authority, reinforced concrete buildings in Gaza Strip can be classified into two major types: low-rise buildings and multi-story buildings. Multi-story buildings are buildings with height exceeds 15 meter measured from the level of the road to the floor level of the last story. The total number of stories shall not be less than 5 stories. In another way, it can be said that the multi-story buildings are the buildings with total height exceeds 21 meters by taking into account the height of the last floor and the mezzanine floor. All other buildings are considered as low-rise buildings.

The multi-story buildings shall be designed and constructed to resist lateral loads as well as gravity loads. In Gaza Strip, multi-story buildings are commonly provided with shear walls as a lateral load-resisting system. It was found that this type of buildings is considered as sufficient to resist earthquake loads unless seismic deficiencies are existed [Qandil 2009]. The undertaken research is not concern with this type of buildings.

Low-rise buildings are normally designed in Gaza Strip to resist gravity loads only, without any considerations to seismic resistance design. It is generally assumed by designers that the seismic forces act on such buildings are low. It is also assumed that the building frame structural system and non-structural elements, e.g. partitions are able to resist such low lateral loads. These and other assumptions are investigated in this research. The seismic resistance evaluation of this type of buildings is the main subject of this research. Several configurations of this type of buildings in Gaza Strip make the building irregular in seismic resistance design. These configurations may represent seismic deficiencies. Common types of irregularities in Gaza Strip buildings are as follows:

1- Presence of soft-story. The ground floor have no infill walls which makes it soft story. Usually, soft-story is used as a parking or for social activities. In the case of soft-story, the stiffness of the ground story is less than that for upper stories since they have infill walls. This leads to the occurrence of large deformations in ground story columns during earthquakes which may lead to the collapse of the structure. The influence of soft-story on the seismic behavior of buildings is investigated in this research. The photo in Fig. (3.6) shows a soft-story case.

Fig. (3.6): Soft Story.

2- Wide use of long cantilevers. Cantilevers are usually used to increase the area of stories above the ground story. Cantilevers are vibrated during earthquake which decreases significantly the strength of the cantilever and thus may lead to failure. Also, cantilevers form eccentricity in buildings which increases as the cantilever span increases. The influence of cantilevers on the seismic behavior of buildings is investigated in this research. The photo in Fig. (3.7) show a building with cantilever slab.

Fig. (3.7): Building with Cantilever Slab.

3- Presence of horizontal and vertical irregularities. Horizontal irregularities are present due to the complex and non-symmetrical plans of buildings. Vertical irregularities are present due to the discontinuity of infill walls in some stories as in the case of soft-story. These irregularities are the main reason of building torsion under earthquake loads. Torsion may cause failure or heavy damage to columns. The influence of horizontal and vertical irregularities on the seismic behavior of buildings is investigated in this research.
4- Formation of short column. This case is found when the column is supported by walls in both sides. These walls do not cover the whole height of the column so that a part of the column remains exposed. This type of columns behaves as a short column during earthquakes where it is exposed to shear forces higher than the other long columns and may fail in shear. The photo in Fig. (3.8) shows the short-column phenomenon.

Fig. (3.8): Short Column.

5- Presence of adjacent buildings. This case is found widely in Gaza Strip where inadequate distance is maintained between adjacent buildings. Such buildings vibrate laterally during earthquakes which may make them hitting each other which may lead to severe damages or collapse. Seismic codes give limitations for separation distance between buildings by limiting the deflection to a specific values to avoid pounding between buildings. The undertaken research will determine the maximum lateral displacement of the investigated buildings which may lead to identify the suitable separation distance between adjacent buildings to prevent pounding. The photo in Fig. (3.9) shows two adjacent buildings in Gaza Strip.

Fig. (3.9): Adjacent Buildings.

3.2.2.3 Reinforced Concrete Moment Resisting Frames

Reinforced concrete moment resisting frames consist of beams and columns that are rigidly connected. This type of systems is used to resist lateral forces. It resists lateral forces by flexure and shear in beams, columns and joints. It is used in Gaza Strip in some large span buildings such as schools, conference rooms, mosques, warehouses, industrial buildings, etc. The photos in Fig. (3.10) shows the moment resisting frame system.

The behavior and resistance of this type of structural systems during earthquakes is mainly depending on the level of system ductility which in turn depending on the level of seismic design and detailing of the system components. Vast majority of this type of buildings in Gaza Strip are designed and constructed to resist gravity loads only because it is used in the construction of low-rise buildings. This type of buildings is out of this research scope.

Fig. (3.10): Moment Resisting Frame.

3.2.3 Classification According to Use

Building use is important in the seismic design as well as in the field of seismic evaluation and strengthening of existing buildings since it is used to determine the importance factor. Generally, buildings can be classified according to their use as essential facilities, structures of low risk to human life, and normal buildings. Residential buildings only are considered in this research.

3.3 Design and Construction Practice in Gaza Strip

As a part of the undertaken research, a detailed investigation on the design and construction practice of reinforced concrete buildings in Gaza Strip was carried out in order to collect information required for the seismic resistance evaluation process. Design codes, building characteristics and construction practices that affect the behavior of buildings during earthquakes were also investigated. In order to collect this information, several meetings have been conducted with relevant regulatory bodies such as Ministry of Public Works and Housing (MoPWH), Ministry of Local Government (MoLG), Gaza Municipality and Association of Engineers. Design requirements and construction regulations of these bodies have been discussed in these meetings. Site visits to several existing buildings have been conducted to collect relevant information. The following points outline the collected information and the main characteristics of reinforced concrete design and construction practice in Gaza Strip:

1. Until now, there is no special building code in Palestine for the structural design of reinforced concrete structures. Also, there is no obligatory law for designing all reinforced concrete buildings to resist lateral loads. Many efforts have been made by the MoPWH towards the development of a building code for Palestine but these efforts were not successful. Instead of that, they recommended the use of the available building codes but not obligatory. This is adequate since the development of a building code for is not easy and needs a huge efforts.
2. The unique official document in Gaza Strip that contains obligations to the seismic design of reinforced concrete buildings is the "*System for Multi-Story Buildings*" issued by the Palestinian Authority. This system require the designers to design reinforced concrete building with total height exceeds 21 meters, i.e. multi-story buildings, for seismic and wind loads. This system is adopted by all bodies relevant to the construction of buildings such as ministries, municipalities and Association of Engineers.
3. Other reinforced concrete buildings, i.e. low-rise buildings, are normally designed only for gravity loads without any consideration to seismic design and detailing. Normally, non-seismic design provisions and reinforcement detailing of ACI 318 are used. This makes the behavior of this type of buildings during earthquakes is a matter of concern. So, this research will evaluate the seismic behavior of this type of buildings.

4. Municipalities' regulations allow the owners to add a mezzanine floor and a roof floor to the low-rise buildings without any additional requirements for seismic design. This result in buildings with 7 stories designed and constructed for gravity loads only. This type of buildings is found widely in Gaza Strip which triggered this research.

5. It is generally assumed by designers that the seismic forces act on low-rise buildings are low. Also, the building frame system and non-structural elements, e.g. partitions are assumed to resist such loads. These assumptions are investigated in this research.

6. Most of Gaza Strip engineers carry out the seismic analysis of multi-story buildings according to the provisions of the 1997 Uniform Building Code (UBC 97 or other versions). Although an official seismic zone map corresponding to UBC 97 for Palestine does not exist, several seismic zone maps for Palestine are produced by using peak ground acceleration (PGA) attenuation relationships. Fig. (3.11) is the seismic zone map for Palestine produced by using Boore et al. PGA attenuation relationship. This map contains PGA values for 10% probability of exceedance in 50 years (475 years return period) [Boore et al., 1997].

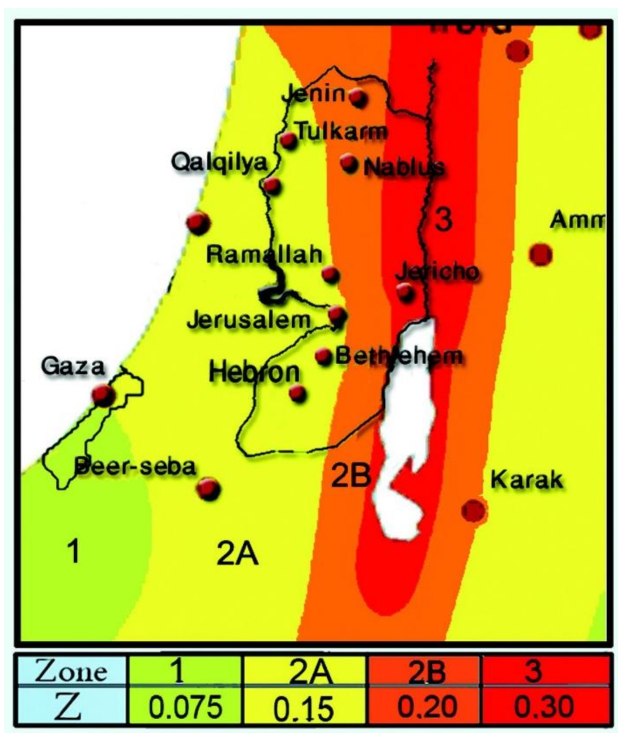


Fig. (3.11): Seismic Zone Map for Palestine [Boore et al., 1997].

7. According to the seismic zone map in Fig. (3.11), Gaza Strip falls within the weakest seismic zones. The northern part of Gaza Strip falls within zone 2A with seismic zone coefficient equal to 0.15 and the southern part falls within zone A with seismic zone coefficient equal to 0.075. Part of Gaza Strip engineers found that it is reasonable to

assume an intermediate zone between zone 1 and 2A to be considered in the seismic design.

8. Till now, a few Gaza Strip engineers carry out the seismic analysis of multi-story buildings according to the provisions of different versions of the International Building Code (IBC) and ASCE/SEI 7 standard (*Minimum Design Load for Buildings and other Structures*). Official seismic hazard map corresponding to IBC and ASCE 7 codes for Palestine does not exist also. Probabilistic seismic hazard maps for short and 1 Sec. periods at 2% probability of exceedance in 50 years (2500 years return period) for the region were proposed by Jordanian researchers [Jaradat et al., 2008]. These maps are shown in Fig. (3.12) and (3.13) for the short and 1 second periods, respectively. To obtain the *Maximum Considered Earthquake* (MCE_R), the values in these maps should be multiplied by the risk coefficient (C_R) which is close to 1.0. Thus, in the absence of official MCE_R maps for Palestine, these maps have been proposed to correspond to MCE_R and thus to give directly S_s and S_1.

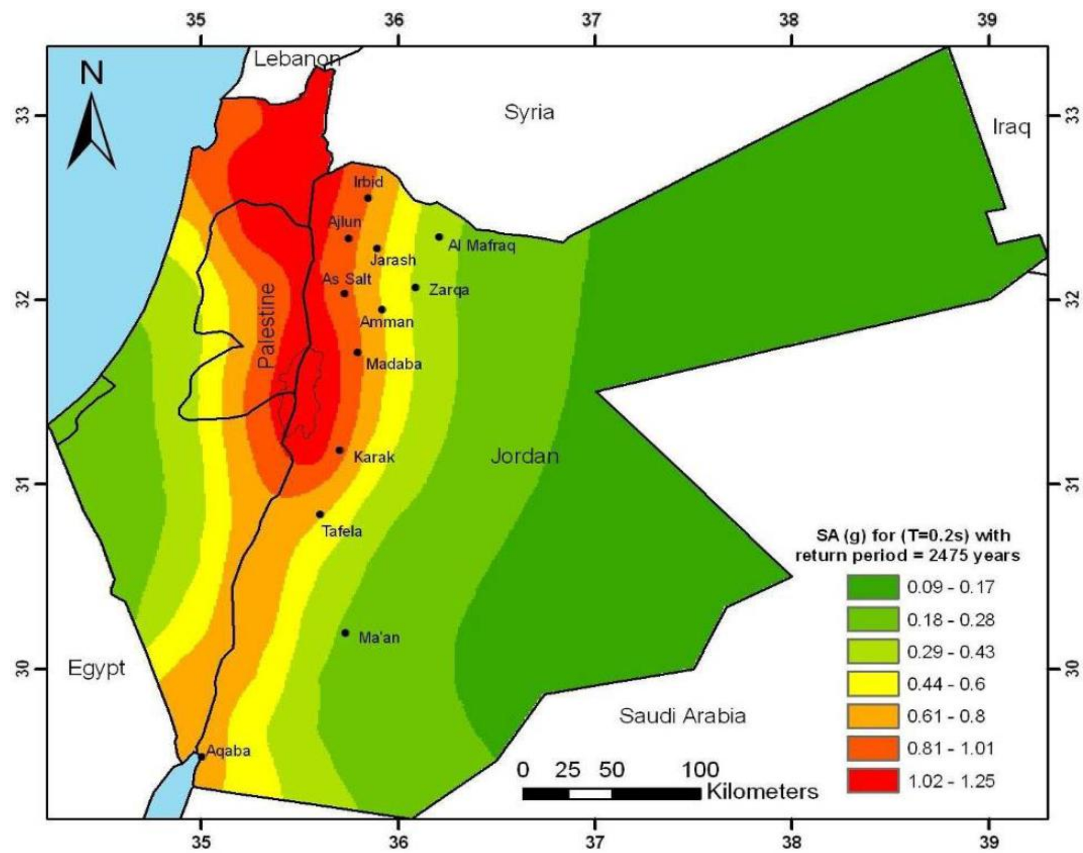


Fig. (3.12): Probabilistic seismic hazard map for spectral acceleration (T=0.2 sec.) at 2% probability of exceedance in 50 years (once in about 2500 years) on firm-rock site conditions [Source: Jaradat et al., 2008].

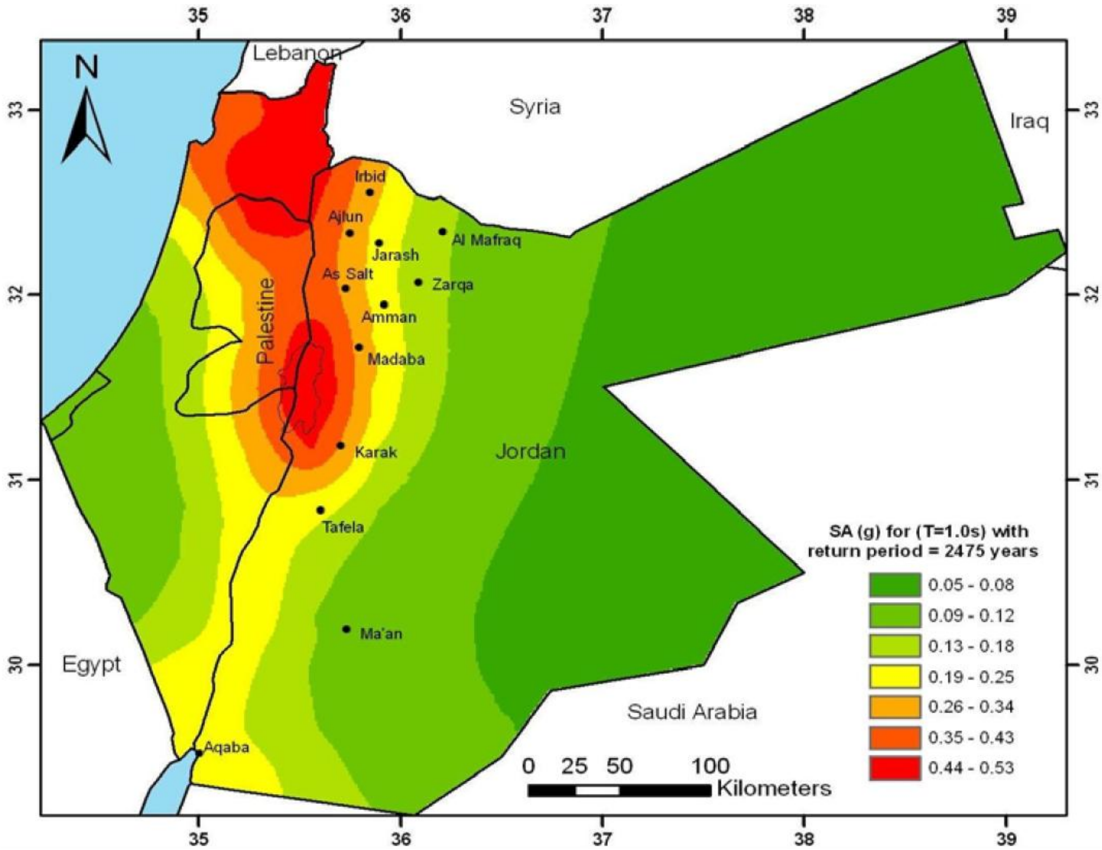


Fig. (3.13): Probabilistic seismic hazard map for spectral acceleration (T=1.0 sec.) at 2% probability of exceedance in 50 years (once in about 2500 years) on firm-rock site conditions [Source: Jaradat et al., 2008].

9. According to the maps in Fig. (3.12) and (3.13), the value of MCE_R spectral response acceleration at short period (S_s)can be taken as a value between 0.09 – 0.17g and the value of MCE_R spectral response acceleration at a period of 1 sec. (S_1)can be taken as a value between 0.09 – 0.12g.

10. In this research, the latest version of the International Building Code and ASCE/SEI 7 standard, i.e. IBC 2012 and ASCE 7-10 are considered.

11. Shear walls are the most widely used lateral load resisting system for multi-story buildings in Gaza Strip. Seismic design of shear walls is carried out in according to the provisions of ACI code. Multi-story buildings that were designed according to the seismic design provisions of the building codes will normally have sufficient resistance to earthquake forces, so it will not be seismically evaluated in this research.

12. Although the low-rise buildings are not designed for seismic loads, several design and construction practices in Gaza Strip are considered as a good practices for seismic resistance such as:

a. Most of designers use closely spaced stirrups over the ends of beams and columns. This practice increases the ductility of beams and columns which in turn enhances the behavior of buildings during earthquakes.

b. Many of buildings, especially those have more than 4 stories, are provided with elevators. In most cases, the elevator walls are constructed as a reinforced concrete walls. These walls are considered by the designers as shear walls which increase the seismic resistance of the building.

c. Column Necks are connected with ground beams. These ground beams enhance the behavior of buildings during earthquakes by decreasing the height-to-width ratio of the ground floor columns. Also it prevents the differential movement of columns and foundations during earthquakes which increase the stability of the structure.

d. In many buildings, internal and external partition walls are connected to columns by a concrete lintel with steel anchors. This makes the walls to act with the columns as one unit. This action enhances the stiffness and seismic resistance of the structure.

e. The column and beam reinforcement is continuous at the joint. Thus, the joint provide certain level of rigidity of the structural system which may resist lateral loads.

13. On the another hand, there are many bad design and construction practices that adversely affect the behavior of Gaza Strip buildings during earthquakes such as:

a. Presence of bad geometrical configurations such as soft story, long cantilevers, horizontal and vertical irregularities related to both geometry and stiffness, adjacency of buildings, etc. These configurations have bad effects on the building behavior during earthquakes. These assumptions are evaluated in this research.

b. Most of Gaza Strip buildings are designed and constructed without carrying out soil investigation for the construction site so that the soil properties are not identified and the water table level not determined. This practice may result in the selection of improper type of foundation. Also the soil liquefaction phenomenon is not surely prevented especially in areas in which the ground table is shallow.

c. Most of Gaza Strip buildings are constructed without professional engineering supervision. This may lead to bad construction quality and materials which in turn reduces the strength of the building during earthquakes. Results of a previous research by Ziara et al. indicated that there is a strong and clear relation between the deterioration of the physical condition of housing units and the lack of professional involvement in either the design or the supervision of the construction [Ziara et al., 1997].

3.4 Concluded Remarks

This chapter includes the findings of the detailed investigation which was carried out on the design and construction practices in Gaza strip with regard to seismic resistance and evaluation. These findings can be summarized as follows:

1. Types of Gaza Strip buildings have been classified according to their construction materials, structural systems and their use. The behavior of each type during earthquake is identified.
2. The type of Gaza Strip buildings that will be seismically evaluated in this research has been identified. This type is the low-rise reinforced concrete buildings designed for gravity loads only. The contribution of infill walls to the overall seismic resistance of the low-rise buildings will be investigated. The influence of the common types of irregularities such as soft story and large cantilevers on the seismic resistance of buildings will be also investigated. The suitable separation distance between adjacent buildings will be determined.
3. Unreinforced masonry building, steel structures, and moment resisting reinforced concrete frame buildings are out of this research scope.
4. Information about the design codes and construction regulations that are used in Gaza Strip has been collected. Latest versions of IBC, ASCE/SEI codes and other codes are considered in this research.
5. Parameters of seismic design that are used in Gaza Strip have been identified. For the seismic design according to UBC 97 code, zone 2A with seismic zone coefficient equal to 0.15 is used for northern part of Gaza Strip and zone 1 with seismic zone coefficient equal to 0.075 is used for southern parts. For the seismic design according to IBC code, values of 0.17 and 0.12 are used for S_s and S_1, respectively.
6. Design and construction practices in Gaza Strip buildings that may affect the behavior of buildings during earthquakes either positively or adversely have been outlined. Good practices are concentrating stirrups over the ends of columns and beams, use of reinforced concrete elevator walls, construction of ground beam, presence of partition walls, and the continuous beam and column reinforcement at the joints. The bad practices are presence of geometrical irregularities, lack of soil tests, and constructing buildings without professional engineering supervision.

CHAPTER 4

PUSHOVER ANALYSIS

4 PUSHOVER ANALYSIS

4.1 Introduction

Since the buildings behavior is inelastic when subjected to significant earthquake loading, the use of inelastic nonlinear analyses is essential to account for this behavior when carrying out seismic assessment for reinforced concrete buildings. Nonlinear static procedure (NSP), which is known as (Pushover Analysis), is the most popular inelastic analysis procedure in the world due to its simplicity and accuracy.

This chapter includes description of the pushover analysis and description of different procedures of pushover analysis which are the capacity spectrum method and the displacement coefficient method. It also outlines the advantages and disadvantages of the pushover analysis and the general steps of performing pushover analysis.

4.2 Methods of Analysis

Various analysis methods, both elastic (linear) and inelastic (nonlinear), are available for the seismic evaluation of reinforced concrete buildings [Oğuz, 2005]. A summarized review of these methods in relation to the undertaken research is as follows:

4.2.1 Elastic (Linear) Methods of Analysis

The force demand on each component of the structure is obtained and compared with available capacities by performing an elastic analysis. Elastic analysis methods include code static lateral force procedure, code dynamic procedure and elastic procedure using demand-capacity ratios. These methods are also known as force-based procedures which assume that structures respond elastically to earthquakes.

In code static lateral force procedure, a static analysis is performed by subjecting the structure to lateral forces obtained by scaling down the smoothened soil-dependent elastic response spectrum by a structural system dependent force reduction factor, "R". In this approach, it is assumed that the actual strength of structure is higher than the design strength and the structure is able to dissipate energy through yielding.

In code dynamic procedure, force demands are determined by an elastic dynamic analysis. The dynamic analysis may be either a response spectrum analysis or an elastic time history analysis.

In demand/capacity ratio (DCR) procedure, the force actions are compared to corresponding capacities as demand/capacity ratios. Demands for DCR calculations must include gravity effects. While code static lateral force and code dynamic procedures reduce the full earthquake demand by an R-factor, the DCR approach takes the full earthquake demand without reduction and adds it to the gravity demands. DCRs approaching 1.0 (or higher) may indicate potential deficiencies.

Although force-based procedures are well known by engineering profession and easy to apply, they have certain drawbacks. Structural components are evaluated for serviceability in the elastic range of strength and deformation. Post-elastic behavior of structures could not be identified by an elastic analysis. However, post-elastic behavior should be considered as almost all structures are expected to deform in inelastic range during a strong earthquake. The seismic force reduction factor "*R*" is utilized to account for inelastic behavior indirectly by reducing elastic forces to inelastic. Force reduction factor, "*R*", is assigned considering only the type of lateral system in most codes, but it has been shown that this factor is a function of the period and ductility ratio of the structure as well.

Elastic methods can predict elastic capacity of structure and indicate where the first yielding will occur, however they don’t predict failure mechanisms and account for the redistribution of forces that will take place as the yielding progresses. Real deficiencies present in the structure could be missed. Therefore, elastic analysis methods are not utilized in this research.

4.2.2 Inelastic (Nonlinear) Methods of Analysis

Structures suffer significant inelastic deformation under a strong earthquake and dynamic characteristics of the structure change with time. So, investigating the performance of a structure requires inelastic analytical procedures accounting for these features.

Inelastic analytical procedures help to understand the actual behavior of structures by identifying failure modes and the potential for progressive collapse. Inelastic analysis procedures basically include inelastic time history analysis and inelastic static analysis which is also known as pushover analysis.

The inelastic time history analysis is the most accurate method to predict the force and deformation demands at various components of the structure. However, the use of inelastic time history analysis is limited because dynamic response is very sensitive to modeling and ground motion characteristics. It requires proper modeling of cyclic load-deformation characteristics considering deterioration properties of all important components. Also, it requires availability of a set of representative ground motion records that accounts for uncertainties and differences in severity, frequency and duration characteristics. Moreover, computation time, time required for input preparation and interpreting voluminous output make the use of inelastic time history analysis impractical for seismic performance evaluation.

Inelastic static analysis has been the preferred method for seismic performance evaluation due to its simplicity. It is a static analysis that directly incorporates nonlinear material characteristics. Therefore, this type of methods is utilized in this research. The following sections include a detailed discussion about the inelastic static analysis procedures.

4.3 Pushover Analysis

Pushover analysis consists of a series of sequential elastic analyses, superimposed to approximate a force-displacement curve of the overall structure. A two or three dimensional model which includes bilinear or trilinear load-deformation diagrams of all lateral force resisting elements is first created and gravity loads are applied initially. A predefined lateral load pattern which is distributed along the building height is then applied. The lateral forces are increased until some members yield. The structural model is modified to account for the reduced stiffness of yielded members and lateral forces are again increased until additional members yield. The process is continued until a control displacement at the top of building reaches a certain level of deformation or structure becomes unstable. The roof displacement is plotted with base shear to get the global capacity curve as shown in Fig. (4.1).

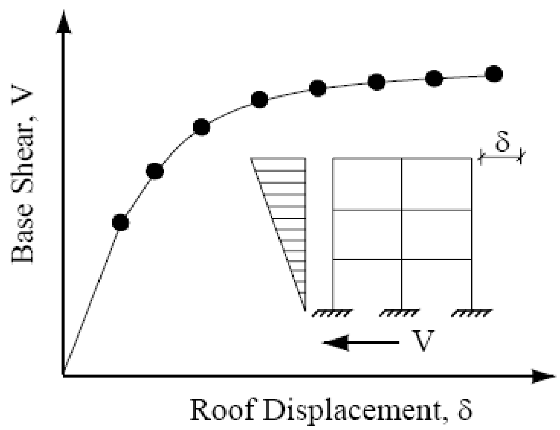


Fig. (4.1): Global Capacity (Pushover) Curve of a Structure [Source: Oğuz, 2005**].**

Pushover analysis can be performed as force-controlled or displacement controlled. In force-controlled pushover procedure, full load combination is applied as specified, i.e, force-controlled procedure should be used when the load is known (such as gravity loading).

Generally, pushover analysis is performed as displacement-controlled. In displacement-controlled procedure, specified drifts are sought where the magnitude of applied load is not known in advance (as in seismic loading). The magnitude of load is increased or decreased as necessary until the control displacement reaches a specified value. Generally, roof displacement at the center of mass of structure is chosen as the control displacement.

The internal forces and deformations computed at the target displacement are used as estimates of inelastic strength and deformation demands that have to be compared with available capacities for a performance check. [Oğuz, 2005].

4.4 Purpose of Pushover Analysis

The purpose of the pushover analysis is to evaluate the expected performance of a structural system by estimating its strength and deformation demands by means of a static inelastic analysis, and comparing these demands to available capacities at the performance levels of interest [Ismail, A. 2014].

Mouzzoun et al. outlines the characteristics that the pushover analysis is very useful in estimating it as follows:

- The capacity of the structure as represented by the base shear versus roof- displacement graph.
- Maximum rotation and ductility of critical members.
- The distribution of plastic hinges at the ultimate load.
- The distribution of damage in the structure, as expressed in the form of load damage indices, at the ultimate load.
- Estimates of inter-story drifts and its distribution along the height.
- Determination of force demands on members, such as axial force demands on columns, moment demands on beam-column connections.
- As an alternative to the design based on linear analysis.
- To assess the structural performance of existing or retrofitted buildings [Mouzzoun et al., 2013].

4.5 General Steps of Pushover analysis

Pambhar (2012) provides a general steps to perform pushover analysis as follows:

1. Form the analytical model of the nonlinear structure.
2. Set the performance criteria, like drift at specific floor levels, limiting plastic hinge rotation at specific plastic hinge points, etc.
3. Apply the gravity load and analyze for the internal forces.
4. Assign the equivalent static seismic lateral load to the structure incrementally.
5. Select a control point to see the displacement.
6. Apply the lateral load gradually using incremental iteration procedure.
7. Draw the “Base Shear vs. Controlled Displacement” curve, which is called “pushover curve”.
8. Convert the pushover curve to the Acceleration-Displacement Response-Spectra (ADRS) format.
9. Obtain the equivalent damping based on the expected performance level.

10. Get the design Response Spectra for different levels of damping and adjust the spectra for the nonlinearity based on the damping in the Capacity Spectrum.
11. The capacity spectrum and the design response spectra can be plotted together when they are expressed in the ADRS format.
12. The intersection of the capacity spectrum and the response spectra defines the performance level [Pambhar, 2012].

4.6 Pushover analysis Procedures

In order to determine compliance with a given performance level, the probable maximum global displacement of the structure when exposed to the design earthquake must be determined. Two methodologies for determining this displacement, Capacity Spectrum Method (ATC-40) and Displacement Coefficient Method (FEMA-356 and ASCE 41-06), are presented in the following sections.

4.6.1 Capacity Spectrum Method

The capacity spectrum method (CSM) was initially proposed by Freeman. The method compares the capacity of a structure to resist lateral forces to the demand given by a response spectrum. The response spectrum represents the demand while the pushover curve (or the 'capacity curve') represents the available capacity [Freeman, 1998].

ATC-40 presents three procedures of the Capacity Spectrum Method to estimate the earthquake induced displacement demand of inelastic systems. All three procedures are based on the same underlying principles that these procedures are approximate since they avoid the dynamic analysis of inelastic system.

Procedures A and B are analytical and suitable to computer implementation while Procedure C is graphical and more suitable for hand analysis. In the undertaken research, Procedure A has been used. The procedure consists of the following steps:

1. Develop a capacity curve (base shear versus roof displacement) of the overall structure by pushover analysis.
2. Construct a bilinear representation of capacity curve. A line representing the average post-elastic stiffness, K_s, of capacity curve is first drawn by judgment. Then, a secant line representing effective elastic stiffness, K_e, is drawn such that it intersects the capacity curve at 60% of the yield base shear. The yield base shear, V_y, is defined at the intersection of K_e and K_s lines. The process is iterative because the value of yield base shear is not known at the beginning. An illustrative capacity curve and its bilinear representation are shown in Fig. (4.2).

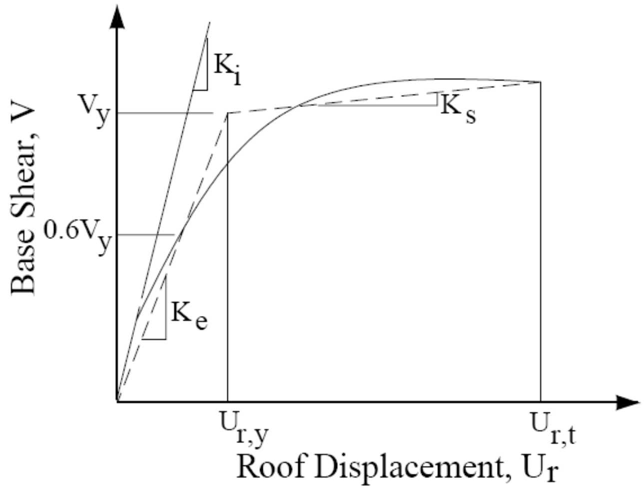


Fig. (4.2): Bilinear Representation of Capacity (pushover) Curve.

3. Convert the bilinear capacity curve into acceleration-displacement response spectrum (ADRS) format using the Eq. (4.1) and Eq. (4.2) (See figure 4.3):

$$S_a = \frac{V/W}{\alpha_1} \qquad \text{Eq. (4.1)}$$

$$S_d = \frac{U_r}{\Gamma_1 . \phi_{1,r}} \qquad \text{Eq. (4.2)}$$

where W: total weight of building (kN).
V: base shear (kN).
U_r: roof displacement (m).
α_1: modal mass coefficient for the fundamental mode.
Γ_1: modal participation factor for the fundamental mode.
$\O_{1,r}$: amplitude of first mode at roof level.
S_a: spectral acceleration (m/s^2).
S_d: spectral displacement (m).

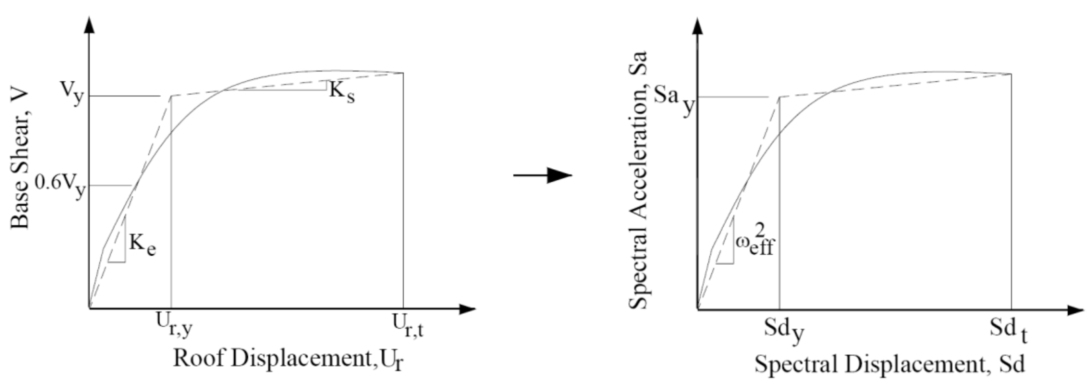


Fig. (4.3): Conversion of Capacity (Pushover) Curve to Capacity Spectrum.

4. Convert 5% elastic response (demand) spectrum from standard S_a vs T format to S_a vs S_d (ADRS) format. For this purpose, the spectral displacement, S_d, can be computed using the Eq. (4.3) for any point on standard response spectrum (See Fig. 4.4).

$$S_d = \frac{1}{4\pi^2} S_a T^2 \qquad \text{Eq. (4.3)}$$

where S_a is the spectral acceleration (m/s^2), S_d is the spectral displacement (m).

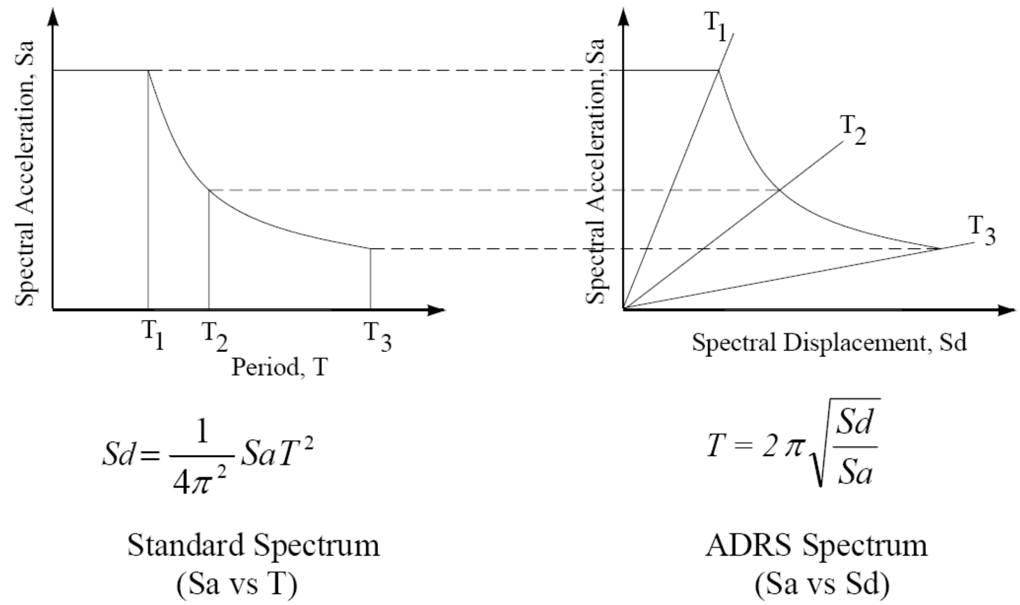


Fig. (4.4): Response Spectrum in Standard and ADRS Formats.

5. Initially, assume a peak spectral displacement demand $Sd_i = Sd\,(T_1, \xi = 5\%)$ determined for period T_1 from the elastic response spectrum.
6. Compute displacement ductility ratio $\mu = Sd_i / Sd_y$
7. Compute the equivalent damping ratio ξ_{eq} from Eq. (4.4):

$$\xi_{eq} = 0.05 + \kappa . \xi_0 \qquad \text{Eq. (4.4)}$$

where ξ_{eq}: equivalent damping ratio.

0.05: 5% viscous damping inherent in the structure (assumed to be constant).

κ: damping modification factor to simulate the probable imperfections in actual building hysteresis loops

ξ_0: hysteretic damping ratio represented as equivalent viscous damping ratio.

The most common method for defining equivalent viscous damping ratio is to equate the energy dissipated in a vibration cycle of the inelastic system and of the equivalent linear system. Based on this concept, [Chopra, 1995] defines equivalent viscous damping ratio as given in Eq. (4.5):

$$\xi_0 = \frac{1}{4\pi}\frac{E_D}{E_S} \qquad \text{Eq. (4.5)}$$

where E_D: the energy dissipated in the inelastic system given by the area enclosed by the hysteresis loop.

E_S: maximum strain energy.

Substituting E_D and E_S in Eq. (4.5) leads to Eq. (4.6):

$$\xi_0 = \frac{2}{\pi}\frac{(\mu-1)(1-\alpha)}{\mu(1+\alpha\mu-\alpha)} \qquad \text{Eq. (4.6)}$$

where μ: displacement ductility ratio.

α: ratio of average post-elastic stiffness of capacity curve to effective elastic stiffness of the capacity curve.

The κ-factor depends on the structural behavior of the building which in turn depends on the quality of seismic resisting system and the duration of ground shaking. ATC-40 defines three different structural behavior types. Type A represents hysteretic behavior with stable, reasonably full hysteresis loops while Type C represents poor hysteretic behavior with severely pinched and/or degraded loops. Type B denotes hysteresis behavior intermediate between Type A and Type C (see Table 4.1)

Table (4.1): Structural Behavior Types (ATC-40).

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

The ranges and limits for the values of κ assigned to the three structural behavior types are given in Table (4.2).

Table (4.2): Values for Damping Modification Factor, κ (ATC-40).

Structural Behavior Type	ξ_0 (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

8. Plot elastic demand spectrum for ζ_{eq} and bilinear capacity spectrum on same chart and obtain the spectral displacement demand Sd_j at the intersection. (Fig. 4.5).

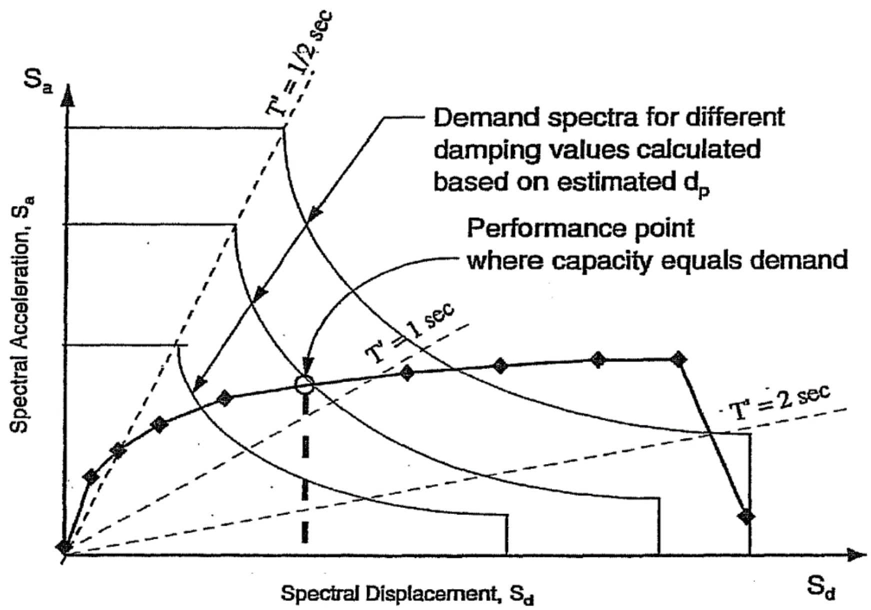


Fig. (4.5): Capacity Spectrum Method [Source: ATC-40].

9. Check for convergence. If $\frac{(Sd_j - Sd_i)}{Sd_j} \leq \text{tolerance} (= 0.05)$, then earthquake induced spectral displacement demand is $Sd = Sd_j$. Otherwise, set $Sd_i = Sd_j$ (or another estimated value) and repeat Steps 6-9.

10. Convert the spectral displacement demand determined in Step 9 to global (roof) displacement by multiplying estimated spectral displacement demand of equivalent SDOF system with first modal participation factor at the roof level.

4.6.2 Displacement Coefficient Method

The Displacement Coefficient Method described in FEMA-356 and adopted in ASCE 41-06 is an approximate method which provides a direct numerical calculation of maximum global displacement demand of structures. Inelastic displacement demand, δ_t, is calculated by modifying elastic displacement demand with a series of displacement modification factors.

Bilinear representation of capacity curve is required to be used in the procedure.

The procedure described in Capacity Spectrum Method is recommended for bilinear representation. After the construction of bilinear curve, effective fundamental period (T_e) of the structure is calculated using Eq. (4.7):

$$T_e = T_i \sqrt{\frac{K_i}{K_e}} \qquad \text{Eq. (4.7)}$$

where T_e: effective fundamental period (in seconds).

T_i: elastic fundamental period (in seconds) in the direction under consideration.

K_i: elastic lateral stiffness of the structure in the direction under consideration.

K_e: effective lateral stiffness of structure in the direction under consideration

The target displacement, δ_t, is computed by modifying the spectral displacement of an equivalent SDOF system using the coefficients as shown in Eq. (4.8):

$$\delta_t = C_0 C_1 C_2 C_3 S_a \frac{T_e^2}{4\pi^2} g \qquad \text{Eq. (4.8)}$$

where

C_0: modification factor to relate spectral displacement and likely roof displacement of the structure. The first modal participation factor at the roof level is used.

C_1: modification factor to relate expected maximum inelastic displacements to displacements calculated for linear elastic response.

C_2: modification factor to represent the effect of hysteresis shape on the maximum displacement response.

C_3: modification factor to represent increased displacements due to second-order effects.

S_a: response spectrum acceleration at the effective fundamental period of the structure.

T_e: effective fundamental period of the structure.

4.7 Advantages of Pushover Analysis

1. It allows us to evaluate overall structural behaviors and performance characteristics.
2. It enables us to investigate the sequential formation of plastic hinges in the individual structural elements constituting the entire structure.
3. When a structure is to be strengthened through a rehabilitation process, it allows us to selectively reinforce only the required members maximizing the cost efficiency.
4. The pushover analysis provides good estimate of global and local inelastic deformation demands for structures that vibrate primarily in the fundamental mode [Khan and Vyawahare, 2013].

4.8 Limitations of Pushover Analysis

1. Deformation estimates obtained from a pushover analysis may be grossly inaccurate for structures where higher mode effects are significant. The undertaken research deals with low-rise buildings with short periods and vibrate in the fundamental mode. Thus, this limitation is not applicable in the undertaken research.
2. In most cases it will be necessary to perform the analysis with displacement rather than force control, since the target displacement may be associated with very small positive or even a negative lateral stiffness because of the development of mechanisms and P-delta effects. In the undertaken research, pushover analysis is performed as displacement-control.
3. Pushover analysis implicitly assurances that damage is a function only of the lateral deformation of the structure, neglecting duration effects, number of stress reversals and cumulative energy dissipation demand.
4. The procedure does not take into account for the progressive changes in modal properties that take place in a structure as it experiences cyclic non-linear yielding during earthquake [Khan and Vyawahare, 2013].

4.9 Concluded Remarks

Various methods of analysis including linear and nonlinear methods have been discussed in this chapter. It has been concluded that the most suitable method for seismic evaluation of existing buildings is the nonlinear methods since it accounts for the inelastic behavior of structures during earthquakes.

Within the nonlinear methods, static nonlinear (pushover) analysis is considered as the preferred method of seismic evaluation due its simplicity and because it easily calculate the capacity of existing buildings and check it against the demand. Thus, pushover analysis has been used in the undertaken research.

Different procedures of pushover analysis including Capacity Spectrum Method and Coefficient Method have been discussed. In the undertaken research, Capacity Spectrum Method of ATC-40 is used to evaluate the seismic resistance of Gaza Strip buildings because it gives a visual representation of capacity-demand equation, could suggests possible remedial action if the equation is not satisfied and easily incorporates several limit states, expressed as point on the load displacement curve of the structure.

It has been concluded that pushover analysis is suitable for seismic evaluation of the targeted buildings of the undertaken research which are the low-rise buildings designed for gravity loads only because these buildings have short periods and vibrate in the fundamental mode.

CHAPTER 5

IMPLEMENTATION OF PUSHOVER ANALYSIS WITH SAP2000

5 IMPLEMENTATION OF PUSHOVER ANALYSIS WITH SAP2000

5.1 Introduction

SAP2000 is one of the most famous programs for linear and nonlinear analysis of structures. It provides a powerful features for performing pushover analysis according to various codes and procedures. Thus, it is used in the implementation of pushover analysis as demonstrated in this chapter.

This chapter also identifies load patterns and general steps for performing pushover analysis with SAP2000 version 16 (2013).

5.2 Modeling Parameters and Acceptance Criteria

In the undertaken research, 3-D structural models are created using SAP2000. Beam and column elements are modeled as a frame element having linear elastic properties. Nonlinear characteristics of these frame elements are modelled using nonlinear load-deformation or moment-rotation relationship at both ends of the element which called plastic hinges. Fig. (5.1) illustrates a typical representation of the load-deformation relationship.

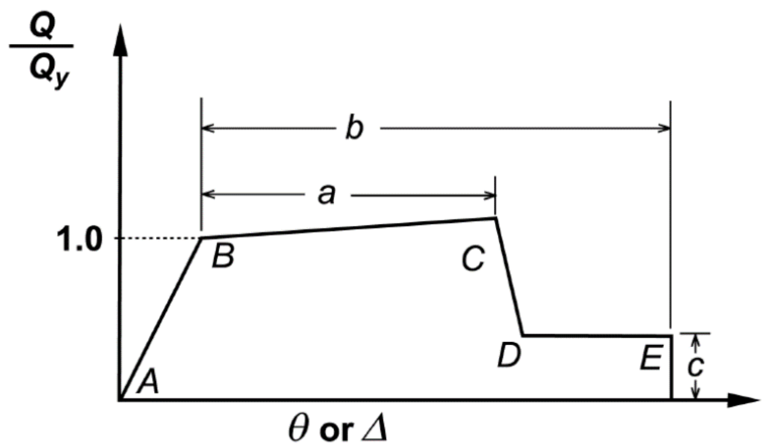


Fig. (5.1): Generalized Load-Deformation Relations [Source: FEMA 356].

In Fig. (5.1), Q_y refers to the strength of the component and Q refers to the demand imposed by the earthquake. Five points labelled A, B, C, D, and E defines the force-deformation behavior of the plastic hinge. Point A shows the unloaded state, Point B shows yielding state of an element, point C represents nominal strength and coordinate of point C on displacement axis shows deformation at which significant amount of strength degradation occurs. The part from C to D shows the starting failure of an element and the strength of the element to resist lateral forces is unreliable after point C. The portion D to E on the curve shows that only the gravity loads are sustained by the frame elements. After point E, the structure has no more capacity to sustain gravity loads.

The parameters (*a* and *b*) refer to those portions of the deformation that occur after yield (from B to D on the curve). The parameter (*c*) is the reduced resistance after the sudden reduction from C to D. Parameters (*a*, *b*, and *c*) which called modeling parameters are defined numerically in various tables for various structural elements in ATC-40 and FEMA-356.

ATC-40 and FEMA-356 define the acceptance criteria depending on the plastic hinge rotations by considering various performance levels as shown in Fig. (5.2). Three points labeled *IO*, *LS* and *CP* are used to define the acceptance criteria or performance level for the plastic hinge. *IO*, *LS* and *CP* stand for Immediate Occupancy, Life Safety and Collapse Prevention, respectively. The values assigned to each of these points vary depending on the type of member. Tables (5.1) and (5.2) show the values of modeling parameters and acceptance criteria for both beams and columns.

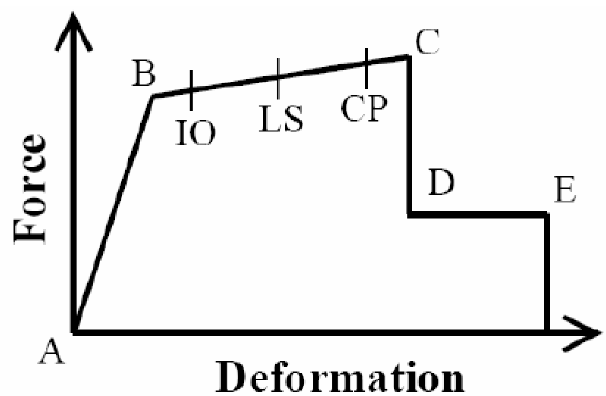


Fig. (5.2): Acceptance Criteria on a Force-Deformation Diagram.

Table (5.1): Modeling Parameters and Numerical Acceptance Criteria for Nonlinear Procedures-Reinforced Concrete Beams. [FEMA-356]

			Modeling Parameters[3]			Acceptance Criteria[3]				
						Plastic Rotation Angle, radians				
						Performance Level				
							Component Type			
			Plastic Rotation Angle, radians		Residual Strength Ratio		Primary		Secondary	
Conditions			a	b	c	IO	LS	CP	LS	CP
i. Beams controlled by flexure[1]										
$\frac{\rho-\rho'}{\rho_{bal}}$	Trans. Reinf.[2]	$\frac{V}{b_w d\sqrt{f'_c}}$								
≤ 0.0	C	≤ 3	0.025	0.05	0.2	0.010	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 ≤ d/2			0.0030	0.02	0.2	0.0015	0.0020	0.0030	0.01	0.02
Stirrup spacing > d/2			0.0030	0.01	0.2	0.0015	0.0020	0.0030	0.005	0.01
iii. Beams controlled by inadequate development or splicing along the span[1]										
Stirrup spacing ≤ d/2			0.0030	0.02	0.0	0.0015	0.0020	0.0030	0.01	0.02
Stirrup spacing > d/2			0.0030	0.01	0.0	0.0015	0.0020	0.0030	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

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. Linear interpolation between values listed in the table shall be permitted.

Table (5.2): Modeling Parameters and Numerical Acceptance Criteria for Nonlinear Procedures-Reinforced Concrete Columns. [FEMA-356]

			Modeling Parameters[4]			Acceptance Criteria[4]				
						Plastic Rotation Angle, radians				
						Performance Level				
							Component Type			
			Plastic Rotation Angle, radians		Residual Strength Ratio		Primary		Secondary	
Conditions			a	b	c	IO	LS	CP	LS	CP
i. Columns controlled by flexure[1]										
$\frac{P}{A_g f'_c}$	Trans. Reinf.[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]			—	—	—	—	—	—	.0030	.0040
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	0.01	0.2	0.0	0.0	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	0.005	0.01	0.01	0.02
All other cases			0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0

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.

The performance level as a limiting damage state or condition described by the physical damage within the building, the threat to life safety of the building's occupants due to the damage, and the post-earthquake serviceability of the building.

FEMA 356 defines the performance levels as follows:

- Immediate Occupancy (IO), means the post-earthquake damage state in which only very limited structural damage has occurred. The basic vertical- and lateral-force-resisting systems of the building retain nearly all of their pre-earthquake strength and stiffness. The risk of life-threatening injury as a result of structural damage is very low, and although some minor structural repairs may be appropriate.
- Life Safety (LS), means the post-earthquake damage state in which significant damage to the structure has occurred, but some margin against either partial or total structural collapse remains. Some structural elements and components are severely damaged, but this has not resulted in large falling debris hazards, either within or outside the building. Injuries may occur during the earthquake; however, the overall risk of life-threatening injury as a result of structural damage is expected to be low.
- Collapse Prevention (CP), means the post-earthquake damage state in which the building is on the verge of partial or total collapse. Substantial damage to the structure has occurred, potentially including significant degradation in the stiffness and strength of the lateral-force-resisting system, large permanent lateral deformation of the structure, and degradation in vertical-load-carrying capacity. However, all significant components of the gravity-load- resisting system must continue to carry their gravity load demands. Fig. (5.3) shows a graphical representation of the performance levels.

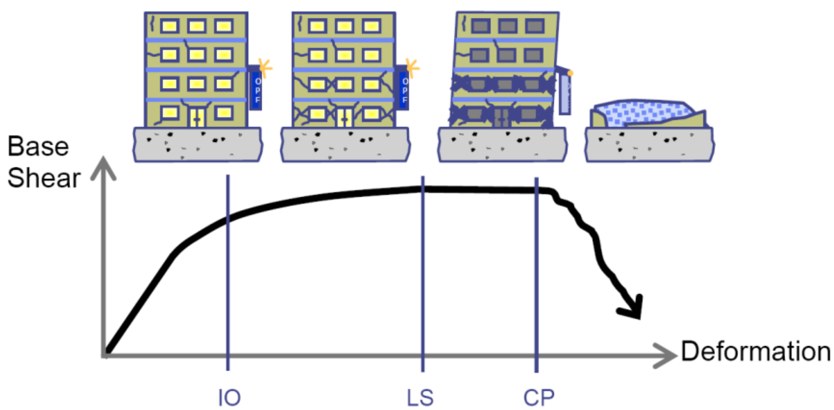


Fig. (5.3): Performance Levels. [Source: Deierlein, 2004]

5.3 Modeling of Masonry Infill Walls

One of the main objectives of this research is to determine the contribution of infill walls on the overall strength of reinforced concrete buildings in Gaza Strip during earthquakes. Masonry infill walls are usually considered as non-structural elements. The masonry infill walls can increase the overall strength of the buildings. Masonry infill walls interfere with lateral deformation of beams and columns of buildings during earthquake and significantly influence the seismic behavior of buildings by participating in lateral force transfer mechanism which changes from a predominant frame action to predominant truss action as shown in Fig. (5.4) and (5.5). However, under seismic loading it can also cause some unfavorable effects like torsion, short-column effect, and soft-story effect.

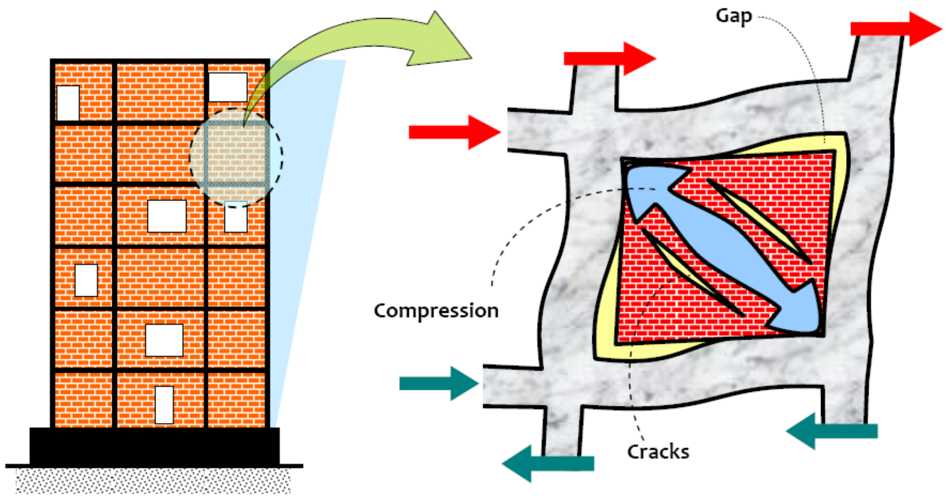


Fig. (5.4): Deformation of R.C. Frame Building with Masonry Infill Walls [Source: Murty et al.].

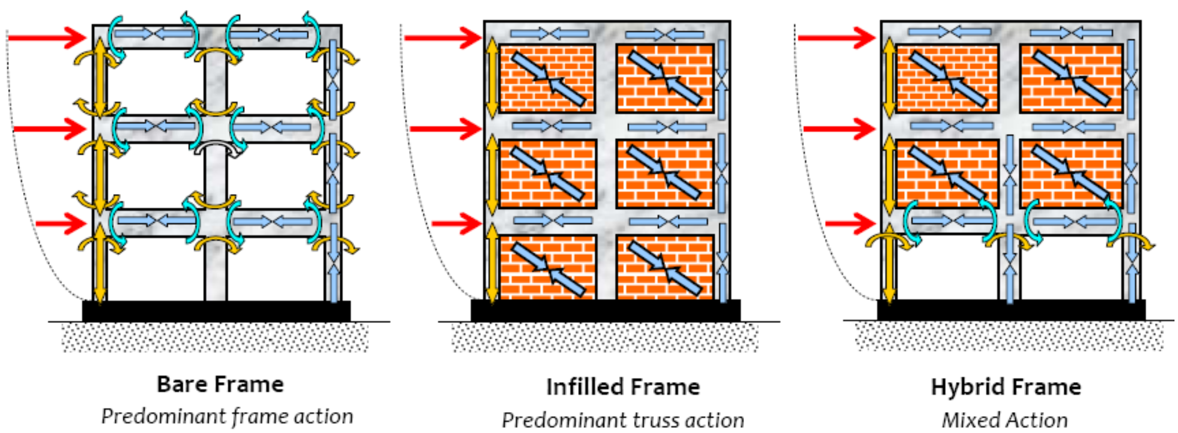


Fig. (5.5): Lateral Force Transfer Mechanism in R.C. Frame Buildings [Source: Murty et al.].

Proper modeling of masonry infill walls is necessary to account for its lateral resistance. Modelling procedures of masonry infill walls can be classified into two groups namely micro-models and macro-models.

Micro-modeling is a complex method of analysis and it is always done by using finite element method. The benefits of using finite element approach is that, all possible modes of failure are discussed in detail but its use is limited due to the greater computational effort and time-requirement.

Macro-models, which have been used in the undertaken research, are the ones in which the masonry infill is replaced by an equivalent pin-jointed diagonal strut system. The basic parameter which affects the stiffness and strength of these struts is their equivalent width which depends on the relative infill-frame stiffness.

Mainstone relationship for calculating the width of the equivalent diagonal compression strut (*a*) which was included in FEMA 356 is used in the undertaken research [Mainstone, 1971]. The relationship is shown in Eq. (5.1):

$$a = 0.175(\lambda_1 h_{col})^{-0.4} r_{\inf} \qquad \text{Eq. (5.1)}$$

Where,

$$\lambda_1 = \left[\frac{E_{me} t_{\inf} \sin 2\theta}{4E_{fe} I_{col} h_{\inf}}\right]^{\frac{1}{4}} \qquad \text{Eq. (5.2)}$$

and

h_{col} = Column height between centerlines of beams, in.

h_{inf} = Height of infill panel, in.

E_{fe} = Expected modulus of elasticity of frame material, ksi.

E_{me} = Expected modulus of elasticity of infill material, ksi.

I_{col} = Moment of inertia of column, in^4.

L_{inf} = Length of infill panel, in.

r_{inf} = Diagonal length of infill panel, in.

t_{inf} = Thickness of infill panel and equivalent strut, in.

θ = Angle whose tangent is the infill height-to-length aspect ratio, radians.

λ_1 = Coefficient used to determine equivalent width of infill strut.

Based on ACI 318-11 code, modulus of elasticity of concrete (E_{fe}) is equal to $57{,}000\sqrt{f_c^{'}}$ (psi), where $f_c^{'}$ is the compressive strength of concrete. Based on FEMA 356, modulus of elasticity of infill (E_{me}) is equal to $550 f_m^{'}$, where $f_m^{'}$ is the compressive strength of infill in ksi units.

Fig. (5.6) shows the equivalent diagonal compression strut.

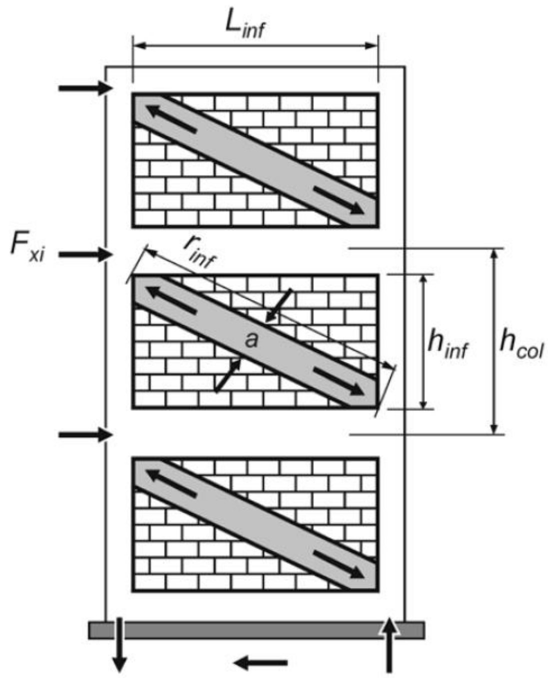


Fig. (5.6): Equivalent Diagonal Compression Struts [Source: FEMA 356].

5.4 Loads

The loads that are considered in the undertaken research are as follows:

5.4.1 Gravity Loads

Gravity loads are dead and live loads. Dead load is taken as the calculated structure self-weight plus the loads of covering materials and partitions. Live loads are taken from relevant tables of ASCE/SEI 7-10.

5.4.2 Lateral Loads

In order to perform a pushover analysis, a pattern of increasing lateral load is applied to the structure. Different lateral load patterns results in different capacity curves. If the curve is over-or-underestimates the seismic capacity of the building, then the estimate of displacement response would not be realistic. Therefore, the selection of lateral load pattern is important in pushover analysis.

The following code lateral load pattern is used in the undertaken research. This load pattern is defined in ASCE/SEI 7-10. The lateral seismic force (F_x) induced at any level is determined from Eq. (5.3) and Eq. (5.4):

$$F_x = C_{vx} V \qquad \text{Eq. (5.3)}$$

$$C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k} \qquad \text{Eq. (5.4)}$$

where

C_{vx} = vertical distribution factor.

V = total design lateral force or shear at the base of the structure.

w_i and 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 from the base to Level *i* or *x*.

k = an exponent related to the structure 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.

It should be mentioned that the equivalent lateral load procedure of ASCE/SEI 7-10 is mainly used for the calculation of base shear (V).

5.4.2.1 Equivalent Lateral Load Procedure of ASCE/SEI 7-10

The seismic base shear (V) is calculated as shown in Eq. (5.5).

$$V = C_s W \qquad \text{Eq. (5.5)}$$

Where:

C_s = seismic response coefficient for the building.

W = effective seismic weight of the building.

The seismic response coefficient is given by Eq. (5.6) to Eq. (5.10)

$$C_s = \frac{S_{DS}}{\left(\frac{R}{I}\right)} \qquad \text{Eq. (5.6)}$$

Where:

S_{DS} = the design spectral response acceleration parameter in the short period.

R = the response modification factor.

I = the importance factor.

C_s need not exceed:

$$C_s = \frac{S_{D1}}{T\left(\frac{R}{I}\right)} \quad \text{for } T \leq T_L \qquad \text{Eq. (5.7)}$$

$$C_s = \frac{S_{D1}T_L}{T^2\left(\frac{R}{I}\right)} \quad \text{for } T > T_L \qquad \text{Eq. (5.8)}$$

C_s need not be less than:

$$C_s = 0.044 D_{DS} I \geq 0.01 \qquad \text{Eq. (5.9)}$$

For structures located where S_1 is equal to or greater than 0.6g, C_s shall not be less than:

$$C_s = 0.5 \times S_1 / (R/I) \qquad \text{Eq. (5.10)}$$

Where:

S_{D1} = the design spectral response acceleration parameter at a period of 1.0s.

T = the fundamental period of the structure(s).

T_L = long-period transition period(s).

S_1 = the mapped maximum considered earthquake spectral response acceleration.

5.5 Pushover Analysis with SAP2000

The following are the general steps followed in the undertaken research to perform pushover analysis using SAP2000:

1. Create a model of the structure. Columns and beams are modeled as line objects and slabs are modeled as shell elements.
2. Define linear static load cases for dead and live loads then assign these loads to the structure.
3. Design the structure by linear analysis cases using predefined load combinations. Check the frame sections to be adequate.
4. Unlock the model after completing the design of the building.
5. Define a linear static load case to represent seismic loads.
6. Define a nonlinear static load case for gravity loads consisting of dead load and portion of live load. This load case need to be applied as force-controlled case.
7. Define the pushover load case (nonlinear static load case). This case should continue from state at end of the nonlinear gravity load case. The load applied to this load case is the pre-defined seismic load case. This load case need to be applied as displacement-controlled case.
8. Define frame hinge properties and then assign it to frame elements.

9. Run the analysis.
10. Graphically review the pushover analysis results.

The following sections describe in details the main steps of performing pushover analysis with SAP2000.

5.5.1 Definition of Seismic loads

Fig. (5.7) shows the SAP2000 dialog box for the definition of seismic load. Seismic loads is defined for each direction separately.

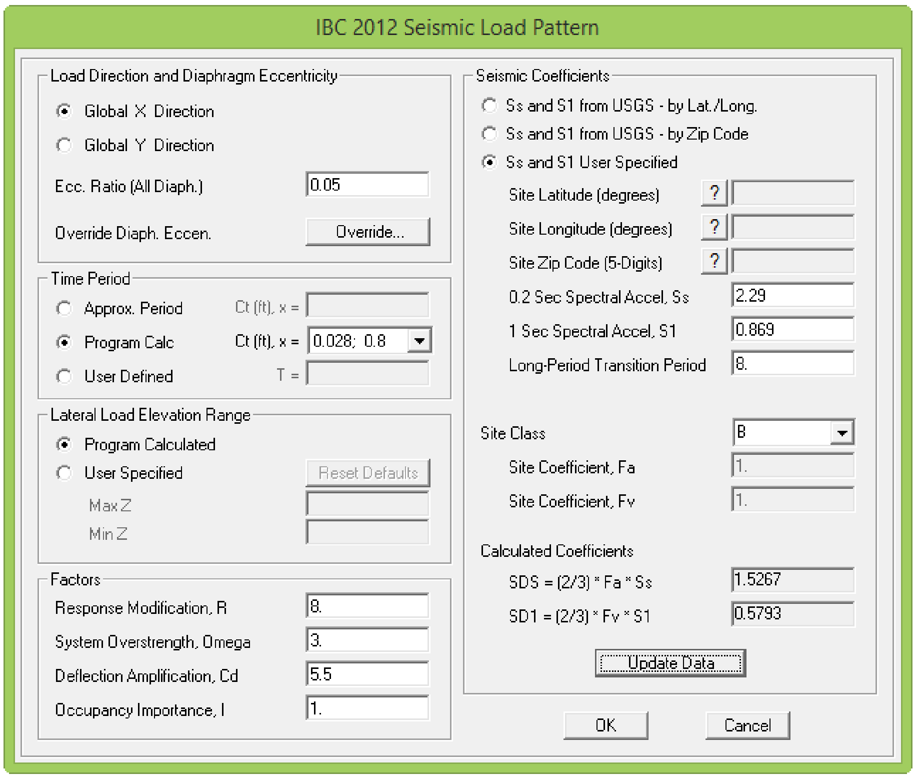


Fig. (5.7): Seismic Load Pattern Dialog Box (SAP2000).

5.5.2 Definition of Nonlinear Gravity Load Case

This load case includes the gravity loads that exist during the seismic action. It include the dead load and a portion of live load. Fig. (5.8) shows the SAP2000 dialog box for the definition of nonlinear static load case (gravity).

In the “load Case Type” and “Analysis Type” boxes, “Static” and “Nonlinear” options are selected respectively. In the “Initial Condition” box, the option of “Zero Initial Condition” is selected. In the “Loads Applied” box, pre-defined gravity loads are selected with specified scale factors. In the “Other Parameters” box, the “Load Application” option is set to “Full Load” in order to perform a force-controlled analysis and other parameters are left as default values. In the “Geometric Nonlinearity Parameters” box, the option of “P-Delta” is selected.

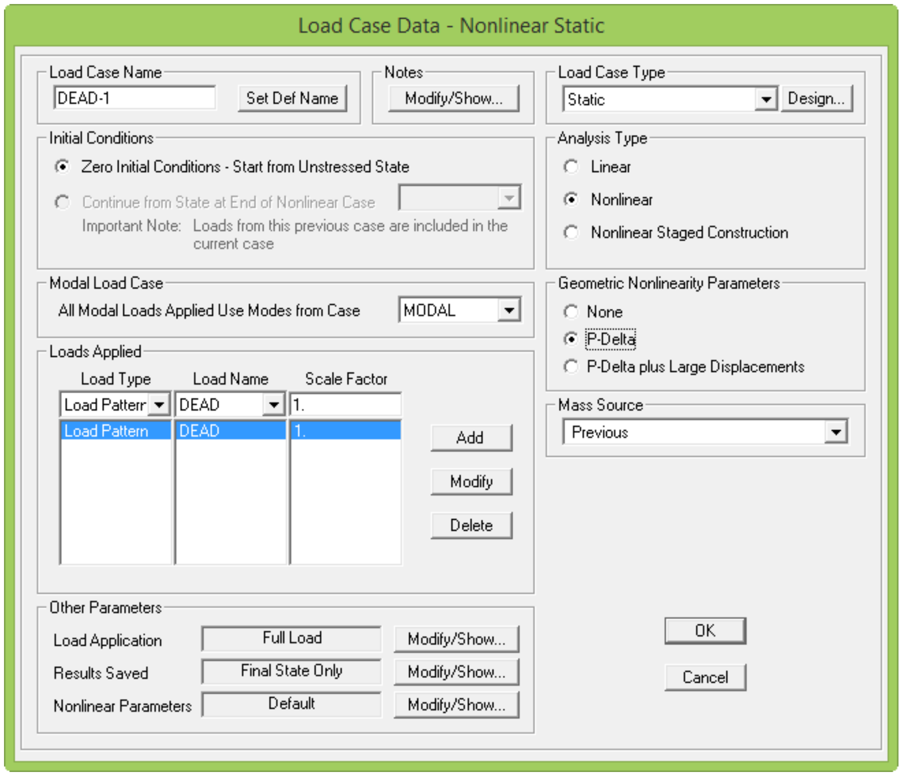


Fig. (5.8): Nonlinear Gravity Load Case Dialog Box (SAP2000).

5.5.3 Definition of Pushover Load Case

This load case includes the seismic loads that will push the building to the target displacement. Fig. (5.9) shows the SAP2000 dialog box for the definition of nonlinear static load case (pushover).

In the “load Case Type” and “Analysis Type” boxes, “Static” and “Nonlinear” options are selected respectively. In the “Initial Condition” box, the option of “Continue from State at End of Nonlinear Case” is selected. The load case selected for this option is the pre-defined nonlinear gravity load case. In the “Loads Applied” box, pre-defined seismic load case is selected with scale factor of 1. In the “Geometric Nonlinearity Parameters” box, the option of “P-Delta” is selected. In the “Other Parameters” box:

- The “Load Application” option dialog box is shown in Fig. (5.10a). The “Load Application Control” is set to “Displacement Control” in order to perform a deformation-controlled analysis. In the “Control Displacement” box, the option of “Use Monitored Displacement” is selected and the magnitude of displacement is set to a specified value. In the “Monitored Displacement” box, a degree of freedom is selected which represent the direction of displacement and the joint that will be monitored on the roof of building is selected.

- The "Results Saved" option dialog box is shown in Fig. (5.10b). In the "Results Saved" box, the option of "Multiple States" is selected.
- The "Nonlinear Parameters" option dialog box is shown in Fig. (5.11). All values of this dialog box are set to the default values.

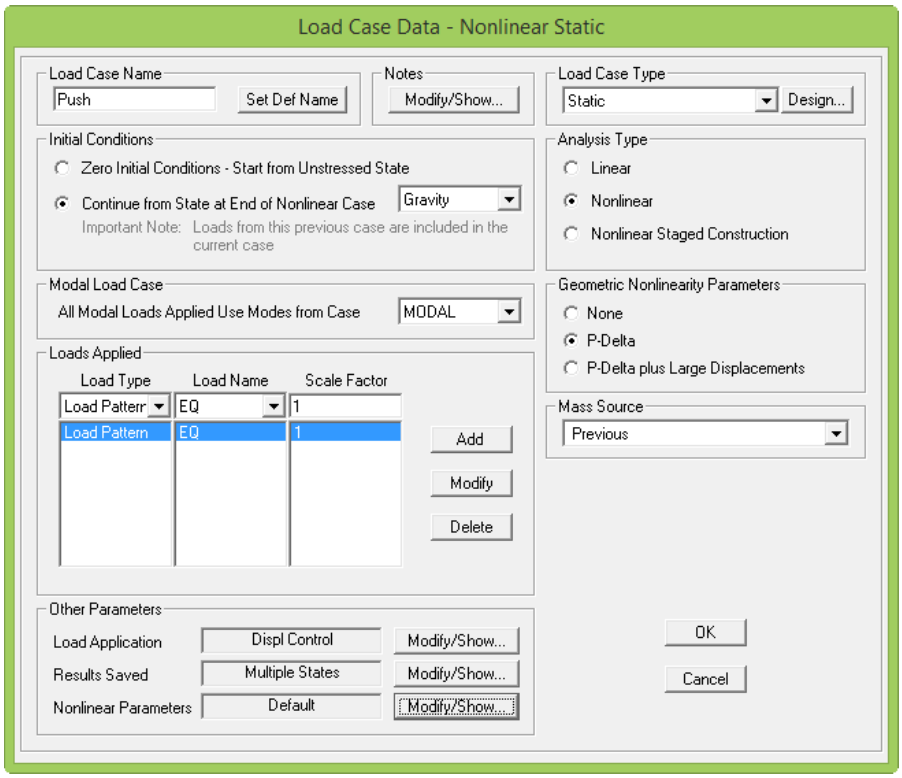
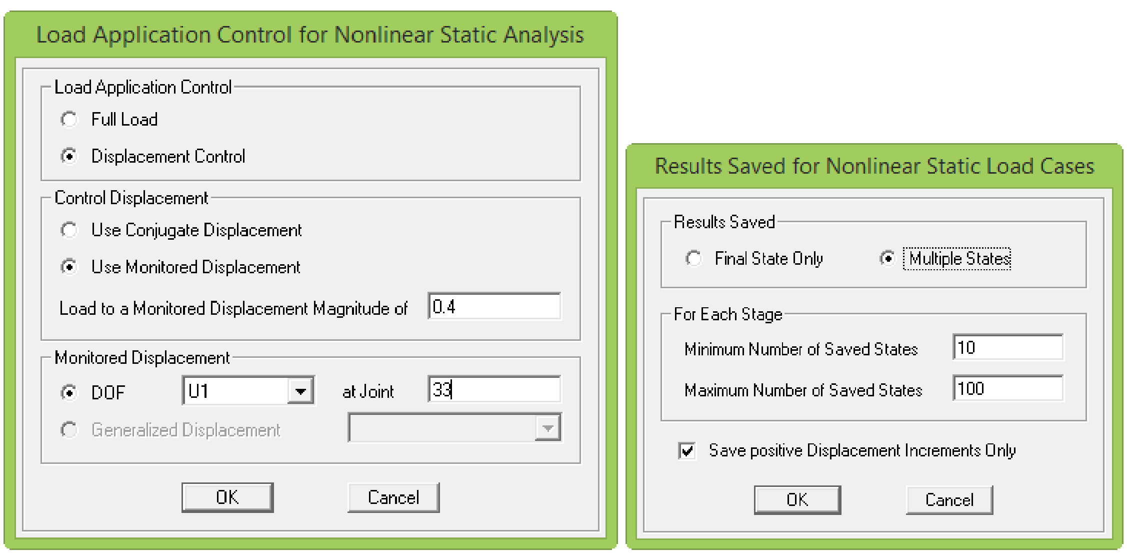


Fig. (5.9): Pushover Load Case Dialog Box (SAP2000).



(a) (b)

Fig. (5.10): (a) Dialog Box for "Load Application" Option, (b) Dialog Box for "Results Saved" Option.

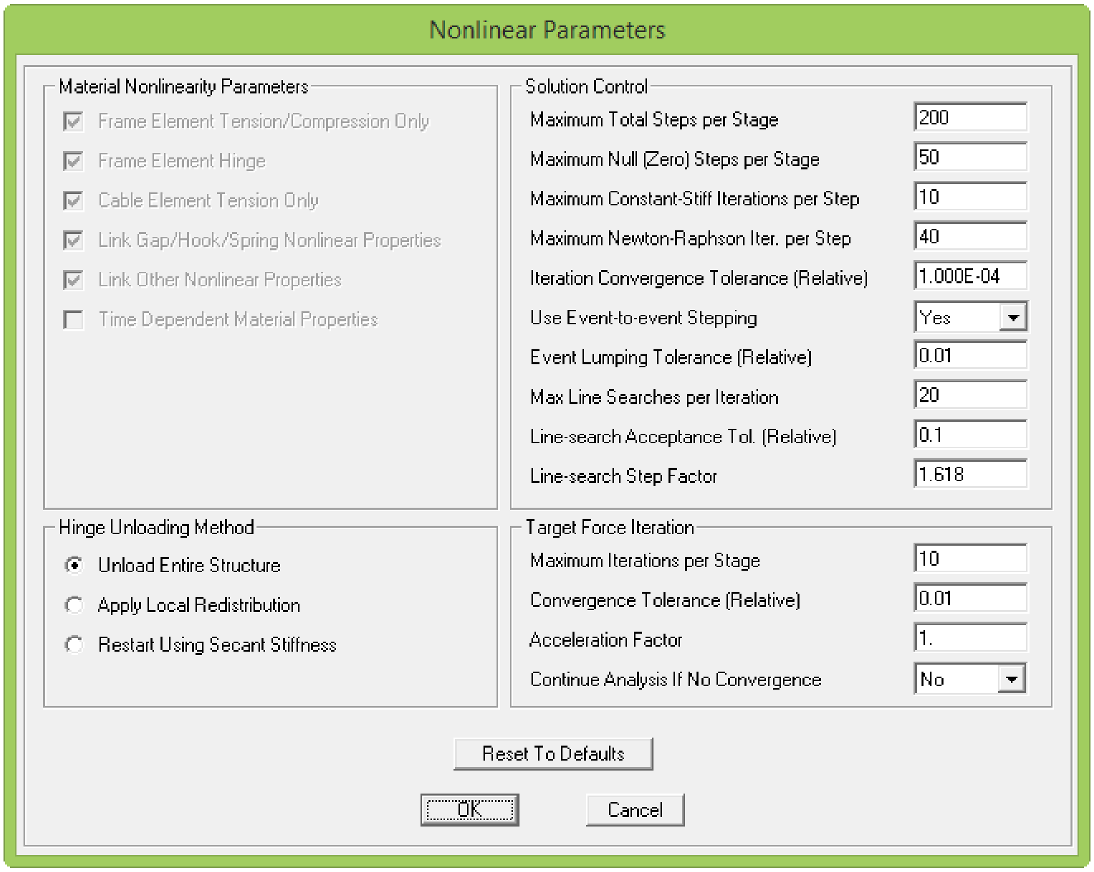


Fig. (5.11): Dialog Box for "Nonlinear Parameters" Option.

5.5.4 Definition of Frame Hinges

Nonlinear behavior of a frame element is represented by hinges in SAP2000. Hinges are assigned at any number of locations (potential yielding points) along the span of the frame element as well as element ends. There are three types of hinge properties in SAP2000. They are default hinge properties, user-defined hinge properties and generated hinge properties. When default and user-defined hinge properties are assigned to a frame element, the program automatically creates a new generated hinge property for each hinge.

Default hinge properties could not be modified and they are section dependent. When default hinge properties are used, the program combines its built-in default criteria with the defined section properties for each element to generate the final hinge properties. The built-in default hinge properties for concrete members are based on FEMA 356 criteria. Fig. (5.12) and (5.13) show the SAP2000 dialog boxes for assigning default hinge properties for columns and beams respectively.

User-defined hinge properties can be based on default properties or they can be fully user-defined. When user-defined properties are not based on default properties, then the properties can be viewed and modified. The generated hinge properties are used in the analysis. They could be viewed, but they could not be modified. Fig. (5.14) and (5.15) shows the dialog box for generated hinge properties for columns and beams respectively.

In the undertaken research, default hinge properties are assigned to both ends of columns and beams.

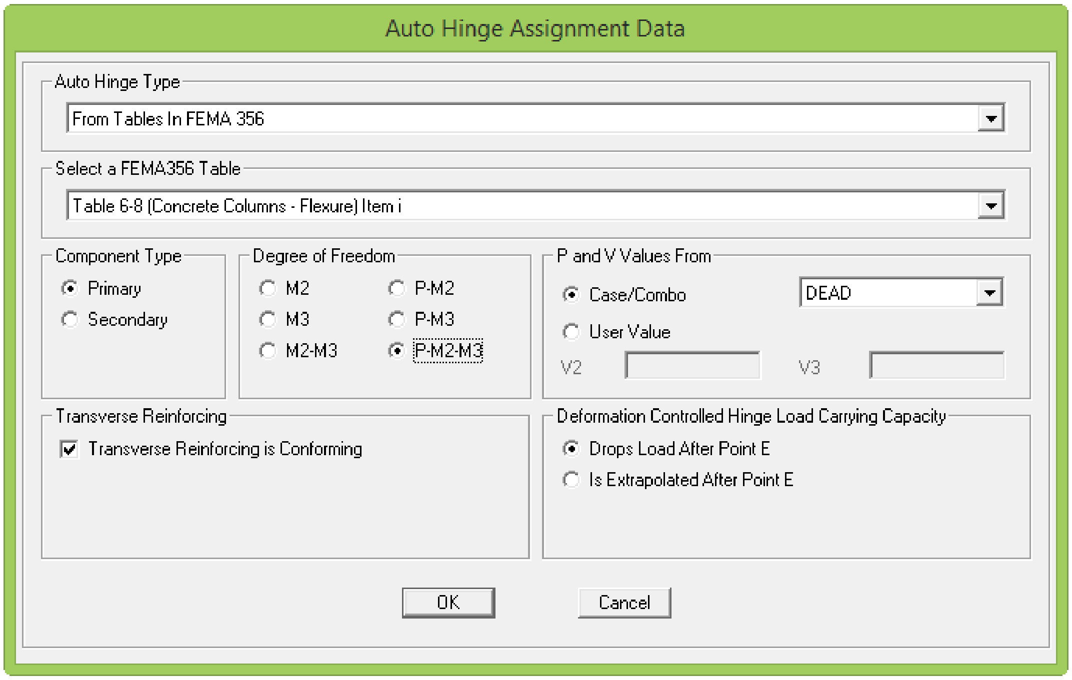


Fig. (5.12): Dialog Box for Default Hinge Properties for Column Elements.

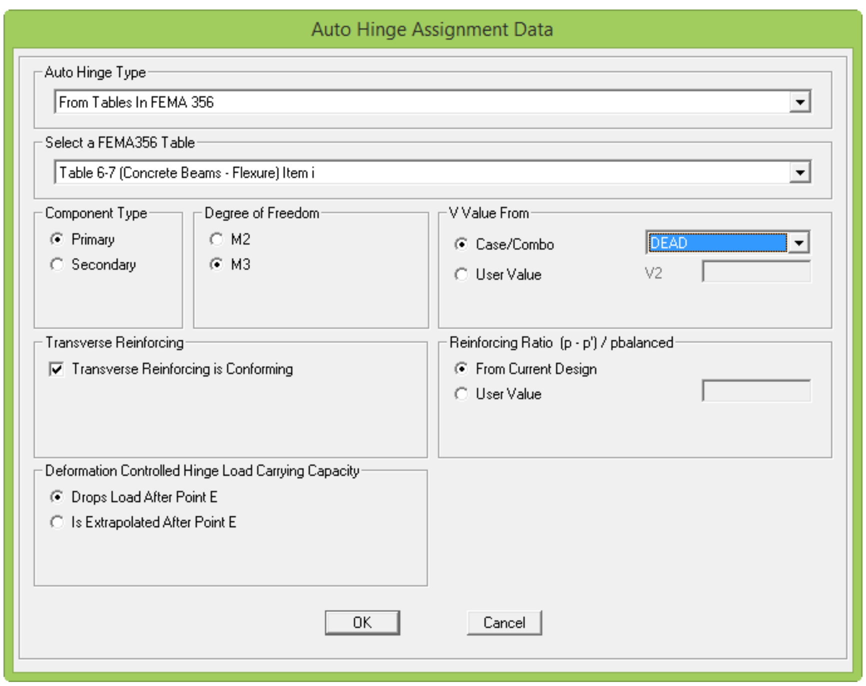


Fig. (5.13): Dialog Box for Default Hinge Properties for Beam Elements.

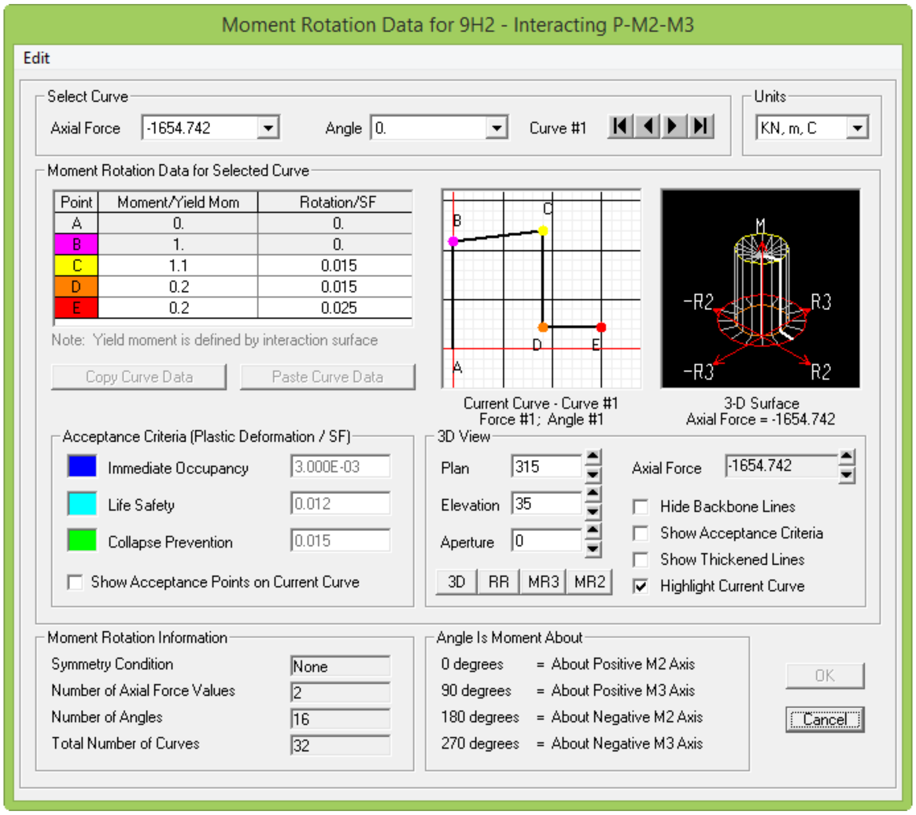


Fig. (5.14): Dialog Box for Generated Hinge Properties for Column Element.

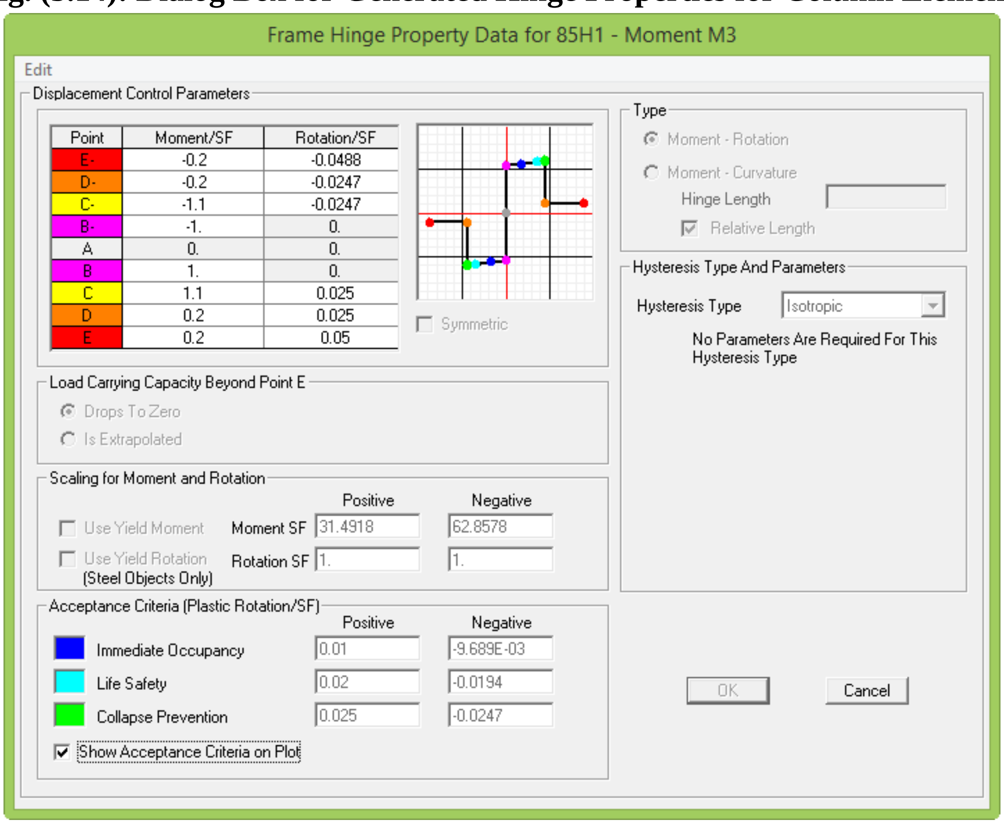


Fig. (5.15): Dialog Box for Generated Hinge Properties for Beam Element.

5.5.5 Review Pushover analysis Results

SAP2000 provides the following results regarding the pushover analysis:

1. Deformation shape of the structure at any step of pushover analysis as shown in Fig. (5.16). Hinge locations at any step are presented. Hinge colors represent the performance level that the hinge reached based on FEMA 356 criteria.
2. Pushover curve in terms of base shear and monitored displacement as shown in Fig. (5.17).
3. Pushover curve intersected with demand curve which show the performance point as shown in Fig. (5.18). Green line represents the pushover curve, blue line represents the demand curve, red lines represent the family of demand spectra of different damping ratios, and gray lines represent the period lines at different values.

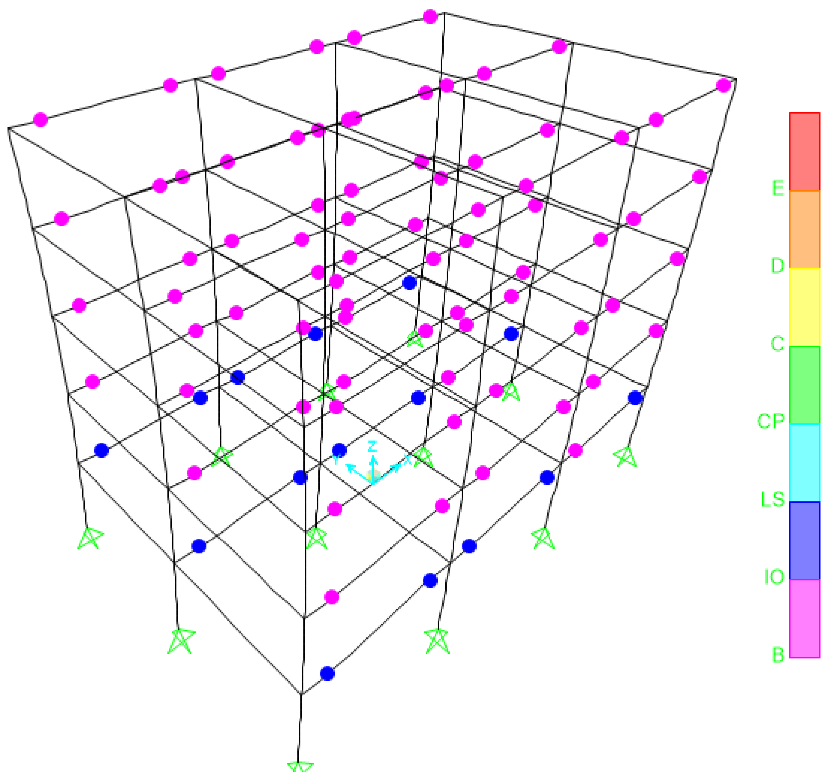


Fig. (5.16): Deformation Shape and Yielding Pattern.

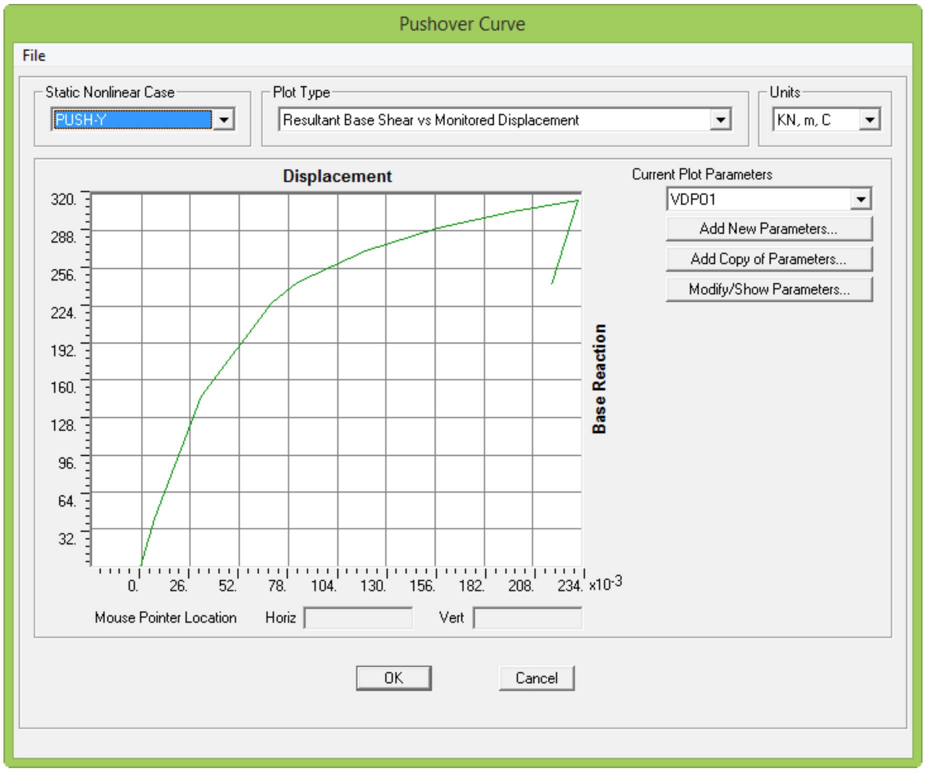


Fig. (5.17): Pushover Curve.

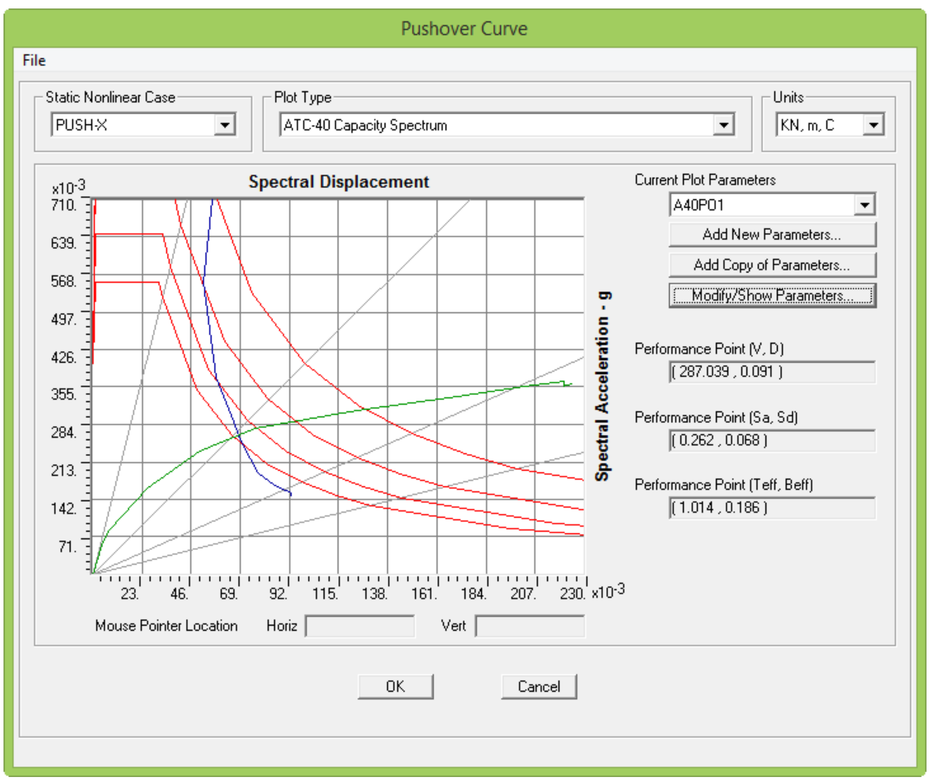


Fig. (5.18): Pushover Curve and Performance Point.

5.6 Concluded Remarks

This chapter deals with the issues related to the implementation of pushover analysis using SAP2000. The followings are the conclusion of this chapter:

1. SAP2000 provides a powerful tools for performing pushover analysis.
2. SAP2000 version 16 (2013) has been used in the undertaken research.
3. Load-deformation relationship proposed by ATC 40 and FEMA 356 has been used to model the nonlinear characteristics of structural and nonstructural elements. Plastic hinges represent the load-deformation relationship in SAP2000.
4. Equivalent diagonal strut method proposed by FEMA 356 for modelling infill walls has been used in the undertaken research.
5. Equivalent lateral load procedure of ASCE/SEI 7/10 has been used to model the lateral load pattern.
6. Default hinge properties in SAP2000 have been used for columns and beams.

CHAPTER 6

APPLICATION OF PUSHOVER ANALYSIS TO GAZA STRIP BUILDINGS

6 APPLICATION OF PUSHOVER ANALYSIS TO GAZA STRIP BUILDINGS

6.1 Introduction

The main objective of this research is to evaluate the seismic performance of the existing R.C. buildings in Gaza Strip. So, the case studies in this research were selected to represent the majority of the existing low-rise buildings that were designed for gravity loads only. Pushover analysis methodology was used to check the performance of the selected case studies. SAP2000 was used as the analysis tool. Results of analysis and discussion of these results are presented in this chapter.

6.2 Selection of Case Studies

Eight case studies were selected carefully to represent the majority of existing residential reinforced concrete buildings of Gaza Strip. The 8 case studies are divided to two building configurations, i.e. B1 and B2.

Since large number of existing reinforced concrete buildings are regular in plan and elevation, regular buildings were selected as case studies. Although, irregular buildings having vertical and horizontal irregularities such as soft story, cantilevers, and irregular plan were also considered.

Each building configuration is analyzed several times separately as follows:

1. Building frame system with no infill walls in all stories.
2. Building frame system with infill walls in all stories.
3. Building frame system with a soft ground story.
4. One of the three previous cases which perform within the damage performance level with the proposed strengthening.

6.3 Building Configuration (B1)

6.3.1 General Description of Building Configuration (B1)

This building is an existing R.C. building located in Gaza city. The building consists of ground floor of 4m height and 4 typical floors of 3m height (i.e. the building height is 16m). The building dimensions are 12m x 7.5m in plan as shown in Fig. (6.1).

The gravity load carrying system is a typical skeleton system comprises of 25cm one way ribbed slab supported on hidden beams which are supported on columns which in turn transfers the loads to the soil through isolated footings. Infill walls thickness is 20 cm for external walls and 10 cm for internal walls.

The building is designed to resist gravity loads only according to ACI 318 provisions. The cross section and reinforcement details of columns are shown in Table (6.1). The arrangement and cross section of beams are shown in Fig. (6.2).

Table (6.1): Dimensions and Reinforcement of (B1) Columns.

Col. No.	Ground+1st Floors		Other Floors	
	Dim. (cm)	Reinf.	Dim. (cm)	Reinf.
C1	20x40	6Φ14	20x40	6Φ14
C2	20x50	8Φ14	20x40	6Φ14
C3	20x70	10Φ14	20x50	8Φ14

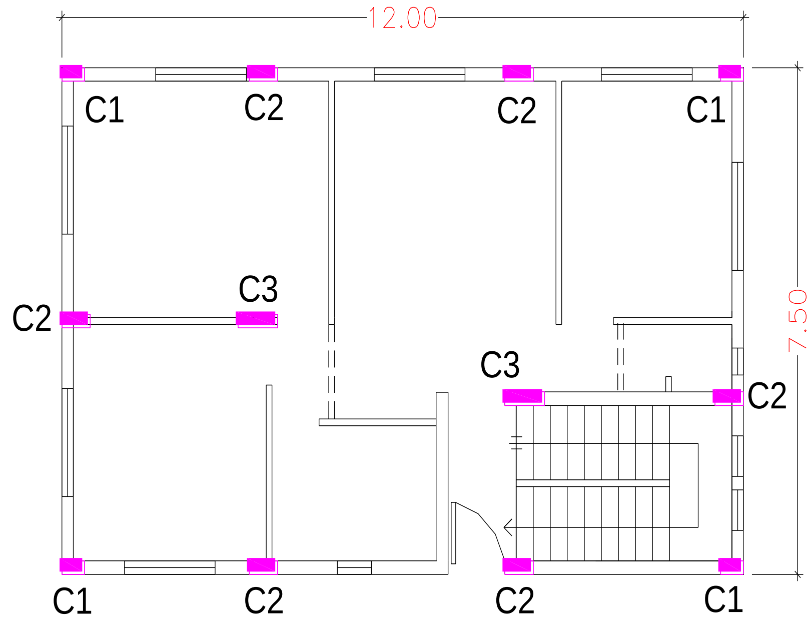


Fig. (6.1): Floor Plan and Columns Location for Building (B1).

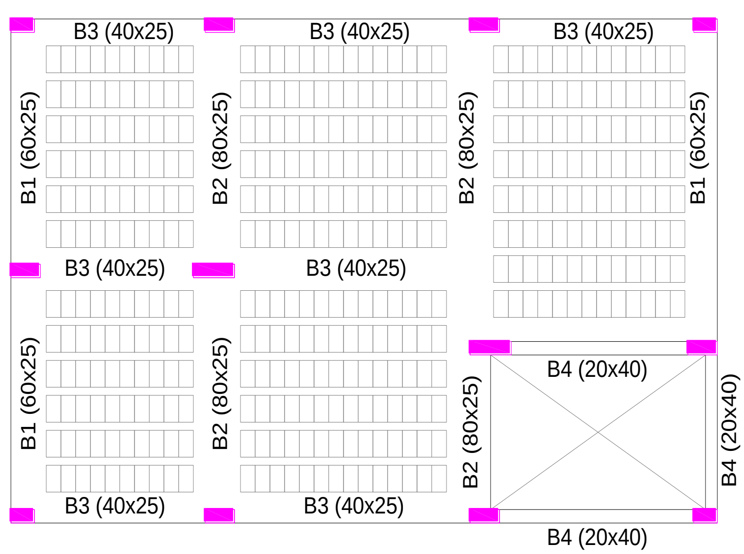


Fig. (6.2): Beams Arrangement and Dimensions for Building (B1).

6.3.2 Structural Modelling and Analysis of (B1)

A 3D structural model is created for the building using SAP2000. The concrete compressive strength of the structural elements is taken as $f_c^{'} = 21$ MPa. The design distributed dead load is taken as 10 KN/m^2 including the own weight of the slab and the superimposed dead loads such as covering materials and partitions loads. The design distributed live load is taken as 2.5 KN/m^2.

The mapped spectral response acceleration at short period is taken as $S_s = 0.17g$ and at period of 1s is taken as $S_1 = 0.12g$. The building is modelled as bare frame (columns and beams only). In the analysis, the structural system of the building is considered as ordinary moment resisting frame in order to take into account the rigidity of the joint between beams and columns. The response modification coefficient is taken as R=3. The importance factor is taken as I=1.0 since the building is residential building.

ASCE/SEI 7-10 equivalent lateral load pattern is used as the lateral load for the pushover analysis. Pushover analysis is performed in X and Y directions separately. Plastic hinges are assigned to each beams and columns in order to model the inelastic behavior of elements during earthquakes. The targeted displacement is taken by trial and error as 350mm in each direction.

This building is analyzed 4 times separately: (1) without infill walls (B1-1), (2) with infill walls in all stories (B1-2), (3) with infill walls and soft ground story (B1-3), and (4) with infill walls, soft ground story, and shear walls in X-direction (B1-4).

6.3.2.1 Analysis Results for Case Study 1: (B1-1)

(B1-1) case study is (B1) building analyzed without infill walls. After running the analysis, the analysis is completed in 15 steps in X-direction and in 9 steps in Y-directions. The capacity curve (pushover curve) in terms of base shear and monitored displacement of the building is obtained for each direction. Fig. (6.3) shows the pushover curve for X-direction and Fig. (6.4) shows the pushover curve for Y-direction. The pushover curve can be obtained in a tabular format as shown in Table (6.2) for X-direction and in Table (6.3) for Y-direction. These tables display the base shear and the corresponding displacement at each step of the pushover analysis. It also display the number and type of the formed plastic hinges in each step.

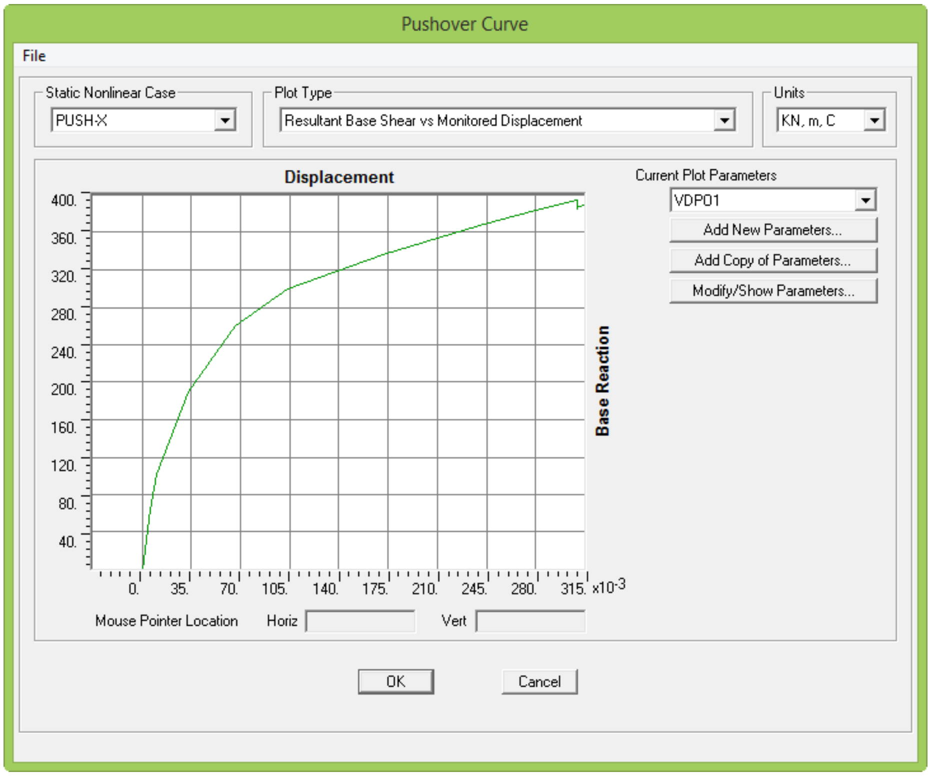


Fig. (6.3): Pushover Curve in X-Direction for Building (B1-1).

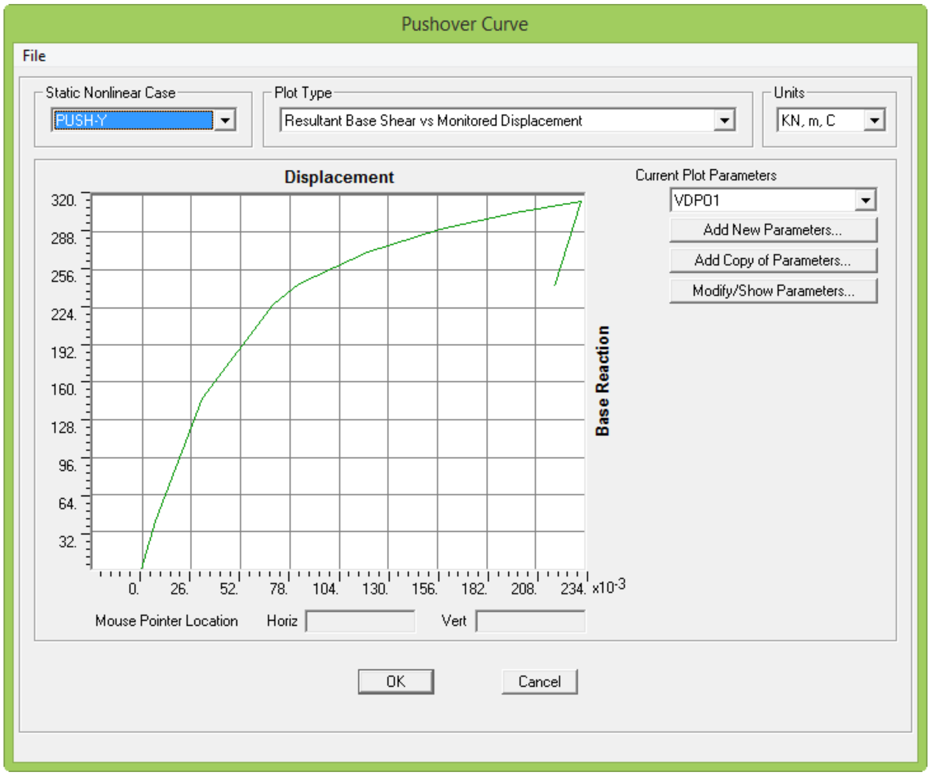


Fig. (6.4): Pushover Curve in Y-Direction for Building (B1-1).

Table (6.2): Pushover Curve Data in X-Direction for Building (B1-1).

Step	Displ. (m)	Base Force (KN)	A to B	B to IO	IO to LS	LS to CP	CP to C	C to D	D to E	Beyond E	Total
0	0.000	0.0	290	0	0	0	0	0	0	0	290
1	0.005	64.3	288	2	0	0	0	0	0	0	290
2	0.010	101.3	258	32	0	0	0	0	0	0	290
3	0.033	189.6	225	65	0	0	0	0	0	0	290
4*	0.066	260.3	207	83	0	0	0	0	0	0	290
5*	0.103	299.7	198	91	1	0	0	0	0	0	290
6	0.171	335.6	190	42	58	0	0	0	0	0	290
7	0.240	366.9	189	9	89	3	0	0	0	0	290
8	0.283	384.9	184	9	61	36	0	0	0	0	290
9	0.309	394.9	184	6	52	47	0	1	0	0	290
10	0.309	384.5	184	6	52	47	0	0	1	0	290
11	0.310	385.6	184	6	52	47	0	0	1	0	290
12	0.310	386.4	184	6	52	47	0	0	1	0	290
13	0.310	386.8	184	6	52	47	0	0	1	0	290
14	0.311	387.3	184	6	50	49	0	0	1	0	290
15	0.315	389.4	184	6	48	47	0	4	1	0	290

* Performance point falls between the yellow shaded steps.

Table (6.3): Pushover Curve Data in Y-Direction for Building (B1-1).

Step	Displ. (m)	Base Force (KN)	A to B	B to IO	IO to LS	LS to CP	CP to C	C to D	D to E	Beyond E	Total
0	0.000	0.0	290	0	0	0	0	0	0	0	290
1	0.007	41.2	289	1	0	0	0	0	0	0	290
2	0.032	146.3	263	27	0	0	0	0	0	0	290
3	0.070	226.9	243	47	0	0	0	0	0	0	290
4	0.083	244.4	230	59	1	0	0	0	0	0	290
5	0.119	271.7	222	53	15	0	0	0	0	0	290
6	0.157	290.2	209	48	33	0	0	0	0	0	290
7	0.197	304.5	201	47	37	5	0	0	0	0	290
8	0.232	314.8	192	48	36	13	0	1	0	0	290
9	0.218	242.0	191	49	34	14	0	1	1	0	290

Deformation shape of the structure at any step of pushover analysis is obtained for each direction. Fig. (6.5) and (6.6) show the deformation shapes for X-direction and Y-direction respectively. The deformation shape also shows the hinge locations at any step of analysis. Hinge colors represent the performance level that the hinge reached.

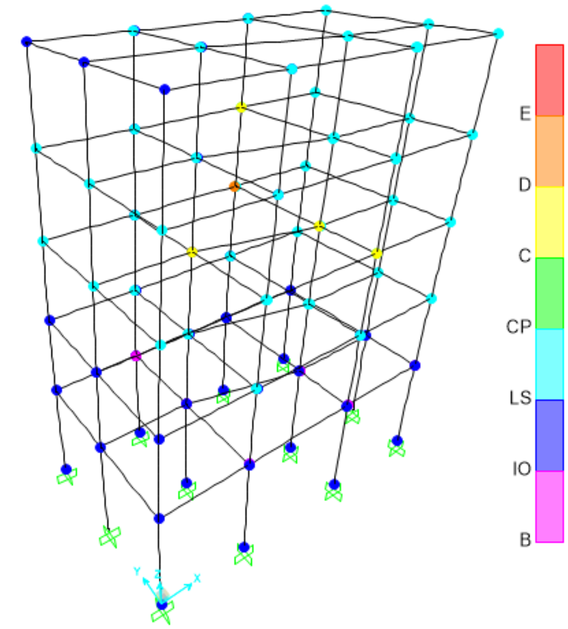


Fig. (6.5): Deformation Shape at Step 15 in X-Direction for Building (B1-1).

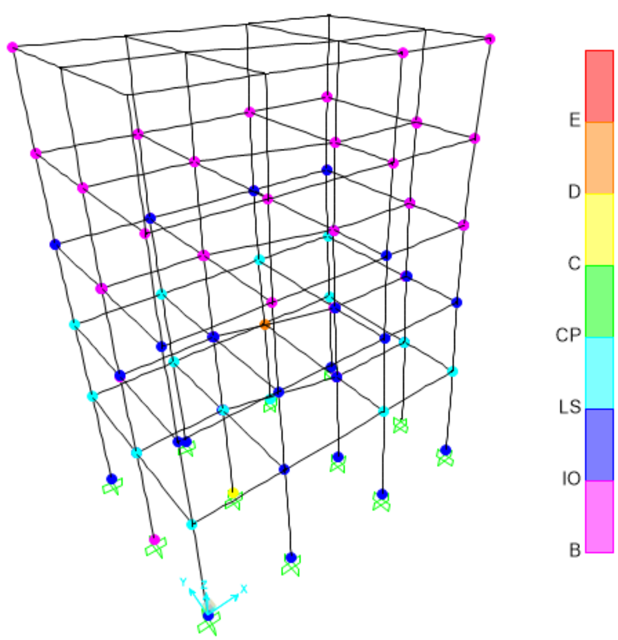


Fig. (6.6): Deformation Shape at Step 9 in Y-Direction for Building (B1-1).

Pushover curve intersected with demand curve show the performance point as shown in Fig. (6.7) for X-direction and Fig. (6.8) for Y-direction. The performance point for (B1-1) in X-direction is at base shear *V*=287 KN and displacement *D*=0.091m, and in Y-direction is at base shear *V*=262 KN and displacement *D*=0.106m.

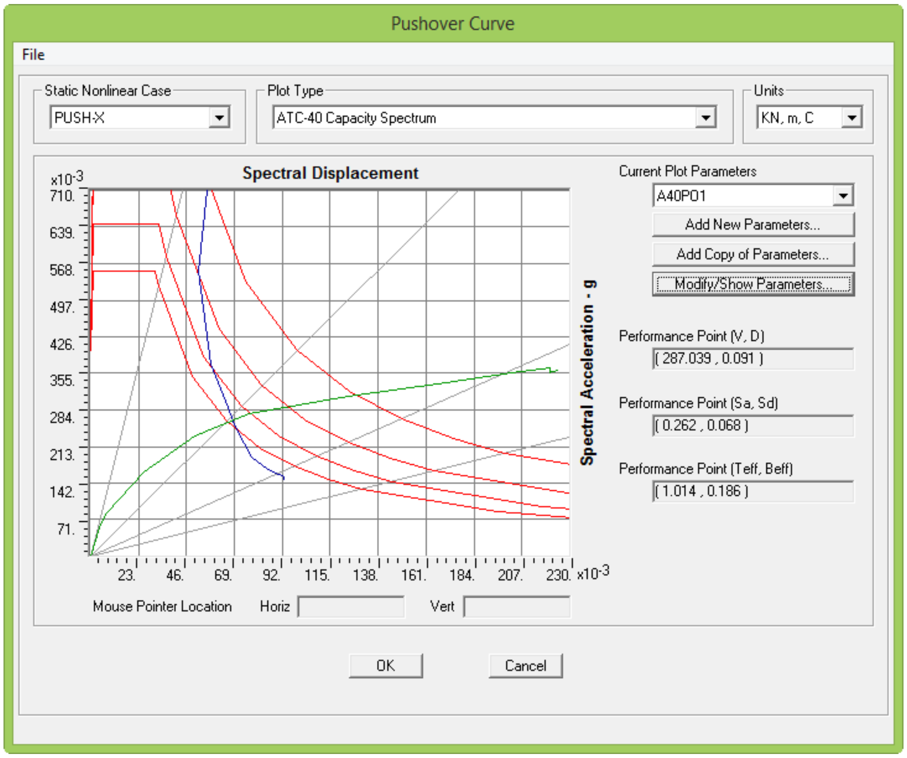


Fig. (6.7): Performance Point in X-direction for Building (B1-1).

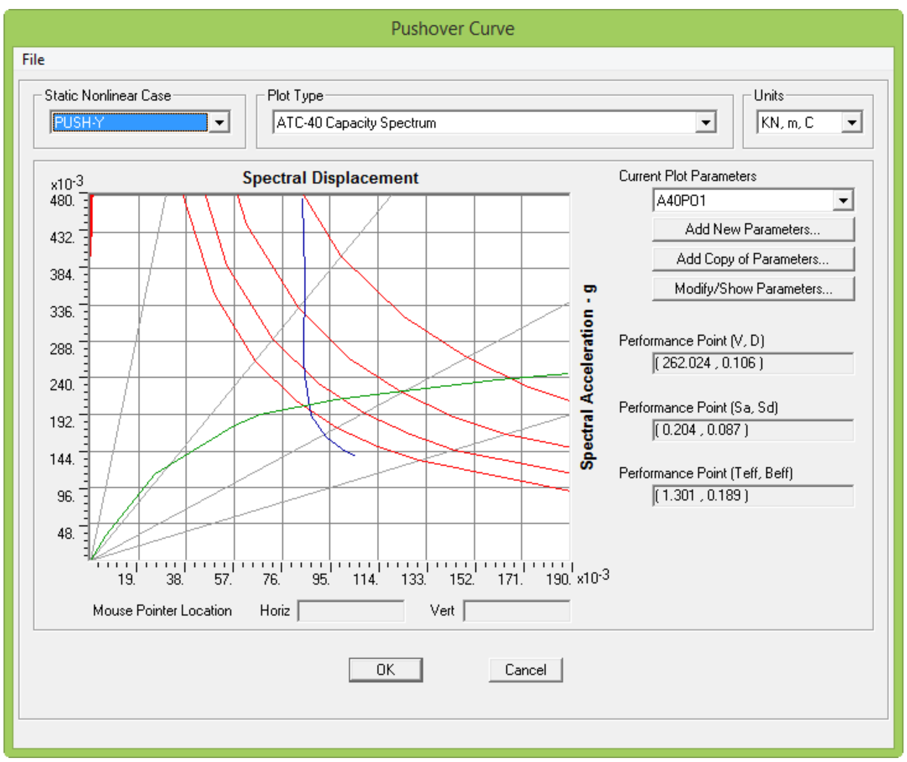


Fig. (6.8): Performance Point in Y-direction for Building (B1-1).

The green line represents the pushover curve, the blue line represents the demand curve, the red lines represent the family of demand spectra of different damping ratios (0.05, 0.10, 0.15, and 0.20), and gray lines represent the period lines at different values (0.5, 1.0, 1.50, and 2.0 sec).

6.3.2.2 Analysis Results for Case Study 2: (B1-2)

(B1-2) case study is (B1) building analyzed with infill walls in all stories. Infill walls width is calculated by Mainstone relationship and taken as 40cm. The analysis is completed in 5 steps in X-direction and in 6 steps in Y-directions. Tables (6.4) and (6.5) show the pushover curve data for X and Y-directions respectively.

Table (6.4): Pushover Curve Data in X-Direction for Building (B1-2).

Step	Displ. (m)	Base Force (KN)	A to B	B to IO	IO to LS	LS to CP	CP to C	C to D	D to E	Beyond E	Total
0	0.001	0.0	290	0	0	0	0	0	0	0	290
1	0.005	138.8	289	1	0	0	0	0	0	0	290
2	0.042	1284.3	223	67	0	0	0	0	0	0	290
3	0.078	2220.0	197	91	2	0	0	0	0	0	290
4	0.113	3091.6	186	88	16	0	0	0	0	0	290
5	0.115	3134.6	184	89	16	0	0	0	0	1	290

Table (6.5): Pushover Curve Data in Y-Direction for Building (B1-2).

Step	Displ. (m)	Base Force (KN)	A to B	B to IO	IO to LS	LS to CP	CP to C	C to D	D to E	Beyond E	Total
0	0.002	0.0	290	0	0	0	0	0	0	0	290
1	0.006	88.7	289	1	0	0	0	0	0	0	290
2	0.042	741.9	256	34	0	0	0	0	0	0	290
3	0.078	1320.8	222	67	1	0	0	0	0	0	290
4	0.104	1696.3	207	76	7	0	0	0	0	0	290
5	0.104	1697.3	207	74	9	0	0	0	0	0	290
6	0.109	1765.5	203	76	10	0	0	1	0	0	290

Fig. (6.9) and (6.10) shows the deformation shapes and hinge locations for X-direction and Y-direction respectively.

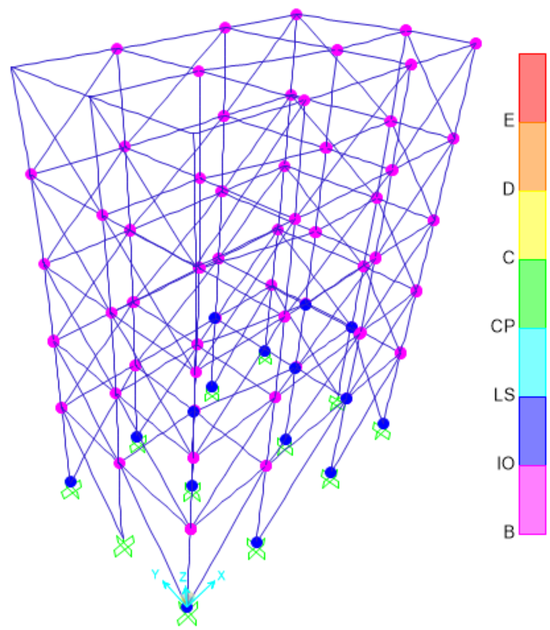


Fig. (6.9): Deformation Shape at Step 5 in X-Direction for Building (B1-2).

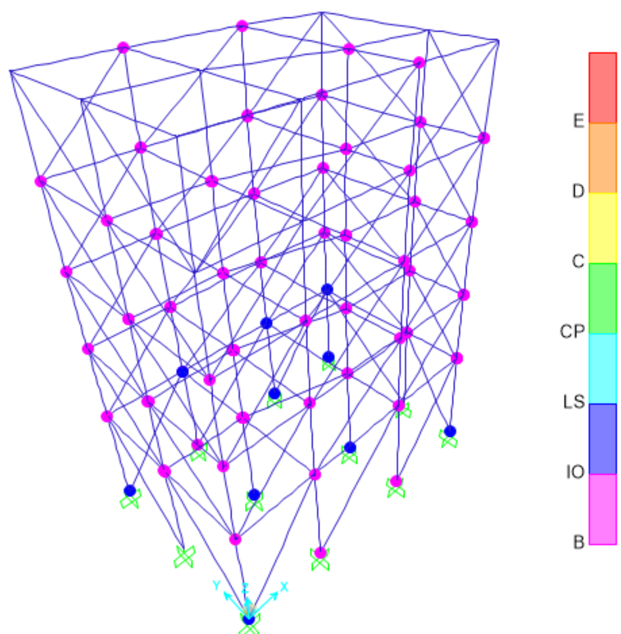


Fig. (6.10): Deformation Shape at Step 6 in Y-Direction for Building (B1-2).

The performance point is shown in Fig. (6.11) and (6.12) for X and Y-directions respectively. The performance point for (B1-2) in X-direction is at base shear V=1296 KN and displacement D=0.043m, and in Y-direction is at base shear V=1079 KN and displacement D=0.063m.

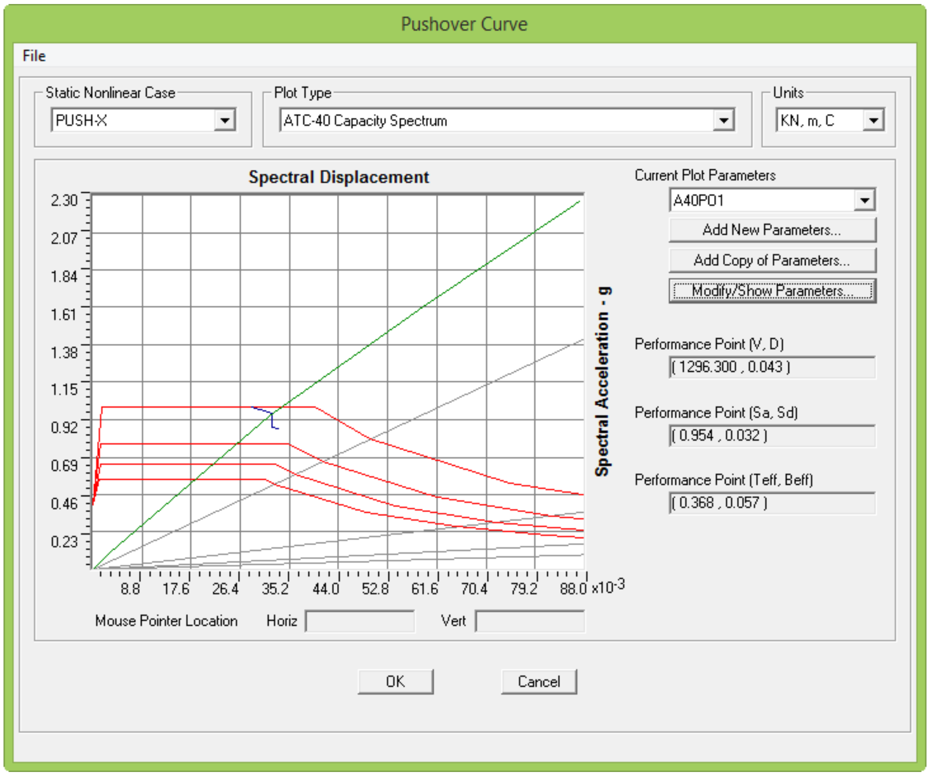


Fig. (6.11): Performance Point in X-direction for Building (B1-2).

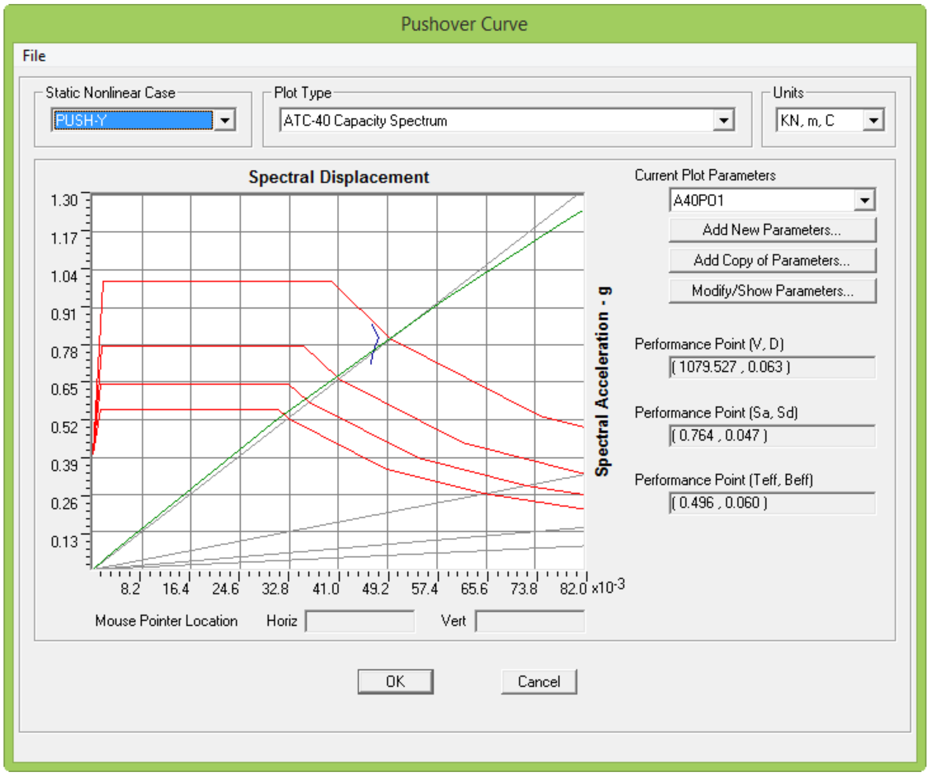


Fig. (6.12): Performance Point in Y-direction for Building (B1-2).

6.3.2.3 Analysis Results for Case Study 3: (B1-3)

(B1-3) case study is (B1) building analyzed with infill walls and soft ground story. The analysis is completed in 8 steps in X-direction and in 6 steps in Y-directions. Tables (6.6) and (6.7) show the pushover curve data for X and Y-directions respectively.

Table (6.6): Pushover Curve Data in X-Direction for Building (B1-3).

Step	Displ. (m)	Base Force (KN)	A to B	B to IO	IO to LS	LS to CP	CP to C	C to D	D to E	Beyond E	Total
0	0.001	0.0	290	0	0	0	0	0	0	0	290
1	0.005	123.1	288	2	0	0	0	0	0	0	290
2	0.028	627.6	239	51	0	0	0	0	0	0	290
3	0.064	1047.4	214	67	9	0	0	0	0	0	290
4	0.077	1169.5	198	81	11	0	0	0	0	0	290
5	0.093	1264.7	189	82	19	0	0	0	0	0	290
6	0.093	1249.0	188	82	18	2	0	0	0	0	290
7	0.106	1309.4	186	75	23	5	0	1	0	0	290
8	0.106	1297.5	186	75	23	5	0	1	0	0	290

Table (6.7): Pushover Curve Data in Y-Direction for Building (B1-3).

Step	Displ. (m)	Base Force (KN)	A to B	B to IO	IO to LS	LS to CP	CP to C	C to D	D to E	Beyond E	Total
0	0.001	0.0	290	0	0	0	0	0	0	0	290
1	0.003	24.5	289	1	0	0	0	0	0	0	290
2	0.038	300.5	268	22	0	0	0	0	0	0	290
3	0.065	402.1	251	29	10	0	0	0	0	0	290
4	0.077	422.3	244	32	14	0	0	0	0	0	290
5	0.094	437.7	242	28	19	1	0	0	0	0	290
6	0.107	457.2	241	22	20	6	0	1	0	0	290

Fig. (6.13) and (6.14) shows the deformation shapes and hinge locations for X-direction and Y-direction respectively.

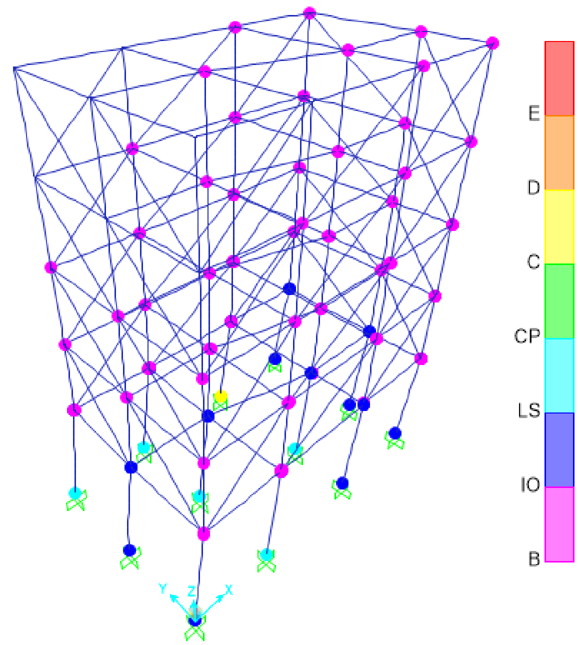


Fig. (6.13): Deformation Shape at Step 8 in X-Direction for Building (B1-3).

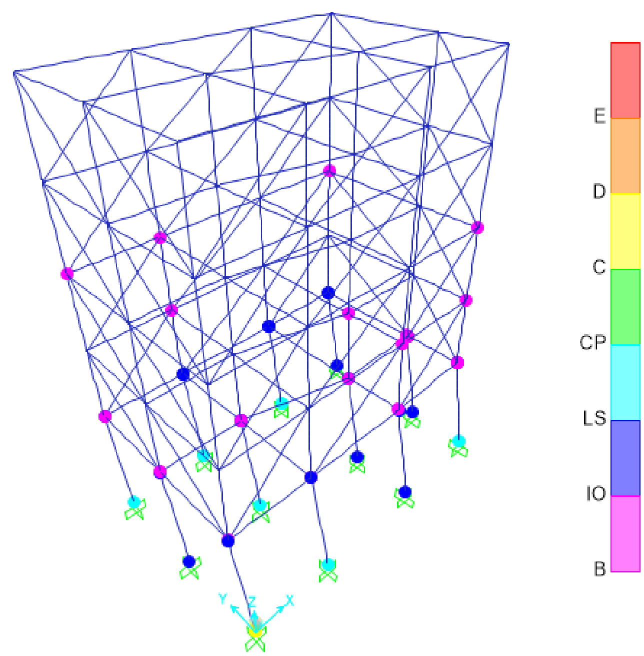


Fig. (6.14): Deformation Shape at Step 6 in Y-Direction for Building (B1-3).

The performance point is shown in Fig. (6.15) and (6.16) for X and Y-directions respectively. The performance point for (B1-3) in X-direction is at base shear *V*=892 KN and displacement *D*=0.051m, and in Y-direction is at base shear *V*=425 KN and displacement *D*=0.08m.

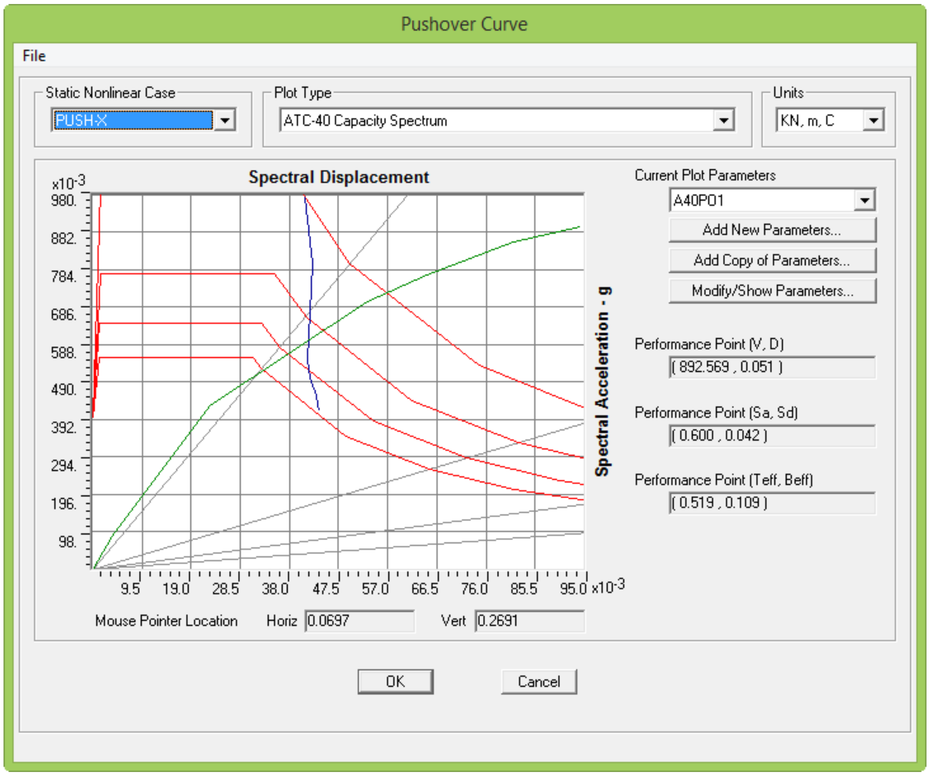


Fig. (6.15): Performance Point in X-direction for Building (B1-3).

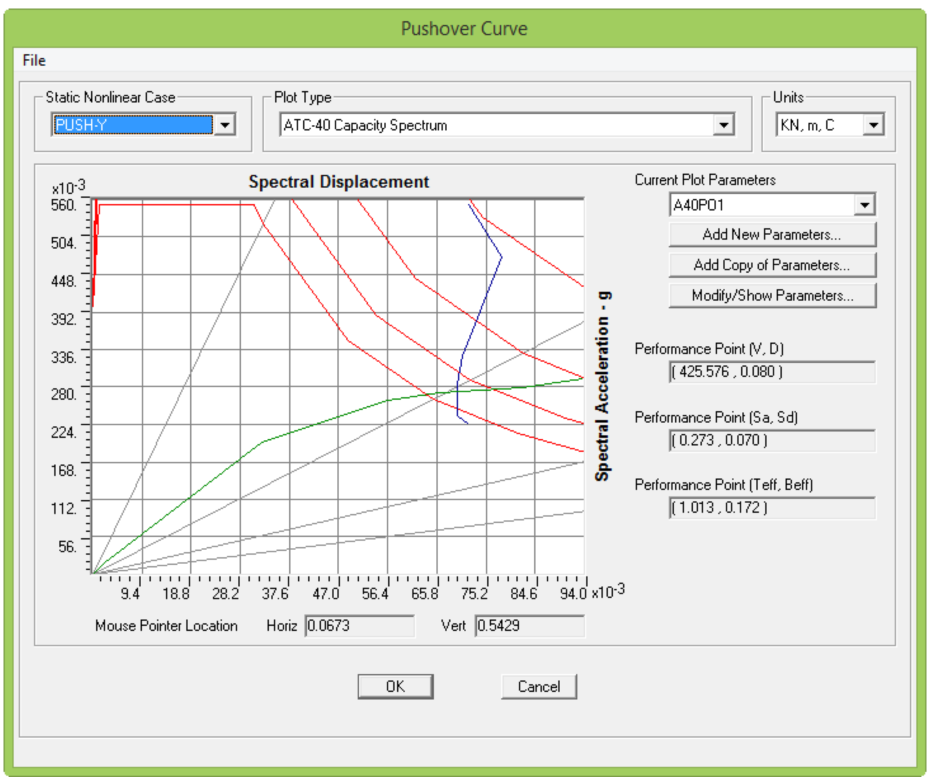


Fig. (6.16): Performance Point in Y-direction for Building (B1-3).

6.3.2.4 Analysis Results for Case Study 4: (B1-4)

(B1-4) case study is (B1) building analyzed with infill walls and soft ground story and two shear walls of 20cm thickness are inserted around the staircases in X-direction. The analysis is completed in 11 steps in X-direction and in 5 steps in Y-directions. Tables (6.8) and (6.9) show the pushover curve data for X and Y-directions respectively.

Table (6.8): Pushover Curve Data in X-Direction for Building (B1-4).

Step	Displ. (m)	Base Force (KN)	A to B	B to IO	IO to LS	LS to CP	CP to C	C to D	D to E	Beyond E	Total
0	0.000	0.0	230	0	0	0	0	0	0	0	230
1	0.000	147.1	229	1	0	0	0	0	0	0	230
2	0.027	2234.8	168	62	0	0	0	0	0	0	230
3	0.027	2231.0	168	62	0	0	0	0	0	0	230
4	0.031	2512.7	163	66	1	0	0	0	0	0	230
5	0.031	2511.5	163	66	1	0	0	0	0	0	230
6	0.032	2600.1	163	65	2	0	0	0	0	0	230
7	0.032	2597.3	163	65	2	0	0	0	0	0	230
8	0.037	2943.4	158	69	3	0	0	0	0	0	230
9	0.037	2942.3	158	69	3	0	0	0	0	0	230
10	0.073	5219.9	120	97	8	5	0	0	0	0	230
11	0.074	5286.3	118	99	6	6	0	1	0	0	230

Table (6.9): Pushover Curve Data in Y-Direction for Building (B1-4).

Step	Displ. (m)	Base Force (KN)	A to B	B to IO	IO to LS	LS to CP	CP to C	C to D	D to E	Beyond E	Total
0	0.002	0.0	230	0	0	0	0	0	0	0	230
1	0.005	38.0	229	1	0	0	0	0	0	0	230
2	0.041	429.3	199	31	0	0	0	0	0	0	230
3	0.079	715.4	178	42	10	0	0	0	0	0	230
4	0.117	954.8	156	53	18	3	0	0	0	0	230
5	0.128	1014.1	153	54	19	3	0	1	0	0	230

Fig. (6.17) and (6.18) shows the deformation shapes for X and Y-directions respectively.

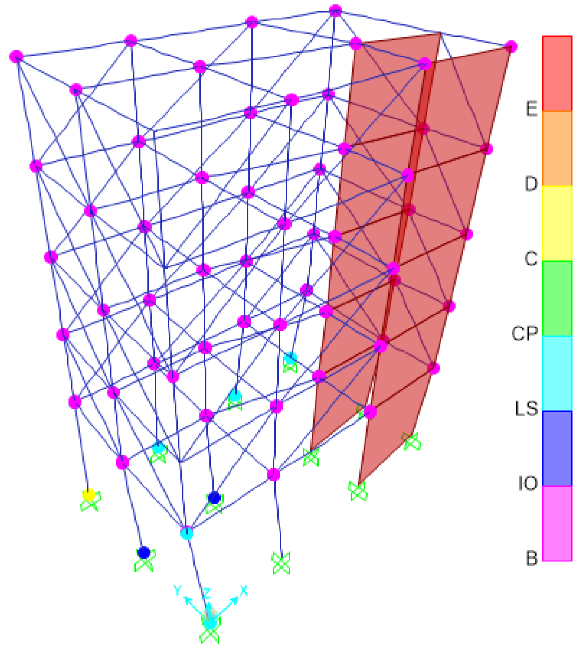


Fig. (6.17): Deformation Shape at Step 11 in X-Direction for Building (B1-4).

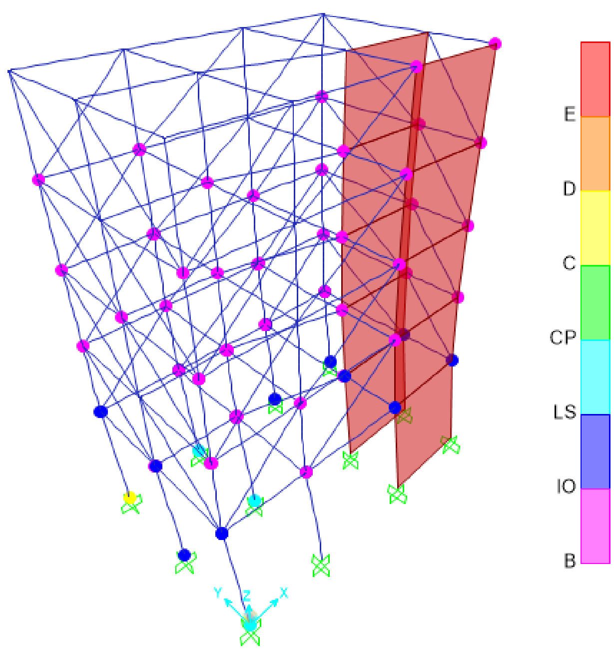


Fig. (6.18): Deformation Shape at Step 5 in Y-Direction for Building (B1-4).

The performance point is shown in Fig. (6.19) and (6.20) for X and Y-directions respectively. The performance point for (B1-4) in X-direction is at base shear *V*=1174 KN and displacement *D*=0.013m, and in Y-direction is at base shear *V*=762 KN and displacement *D*=0.087m.

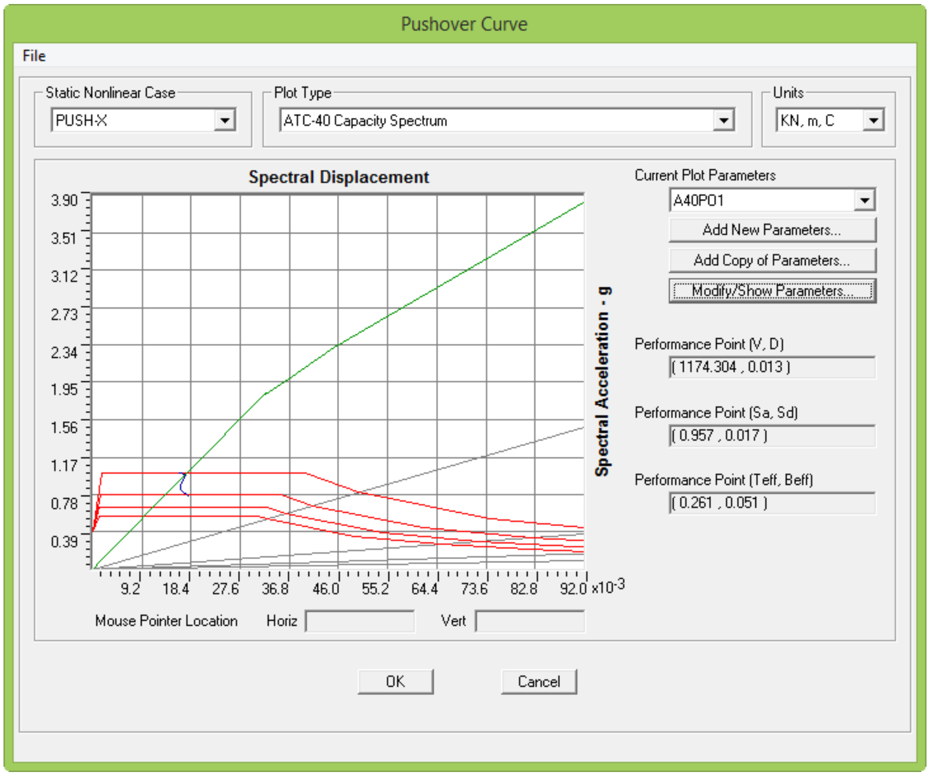


Fig. (6.19): Performance Point in X-direction for Building (B1-4).

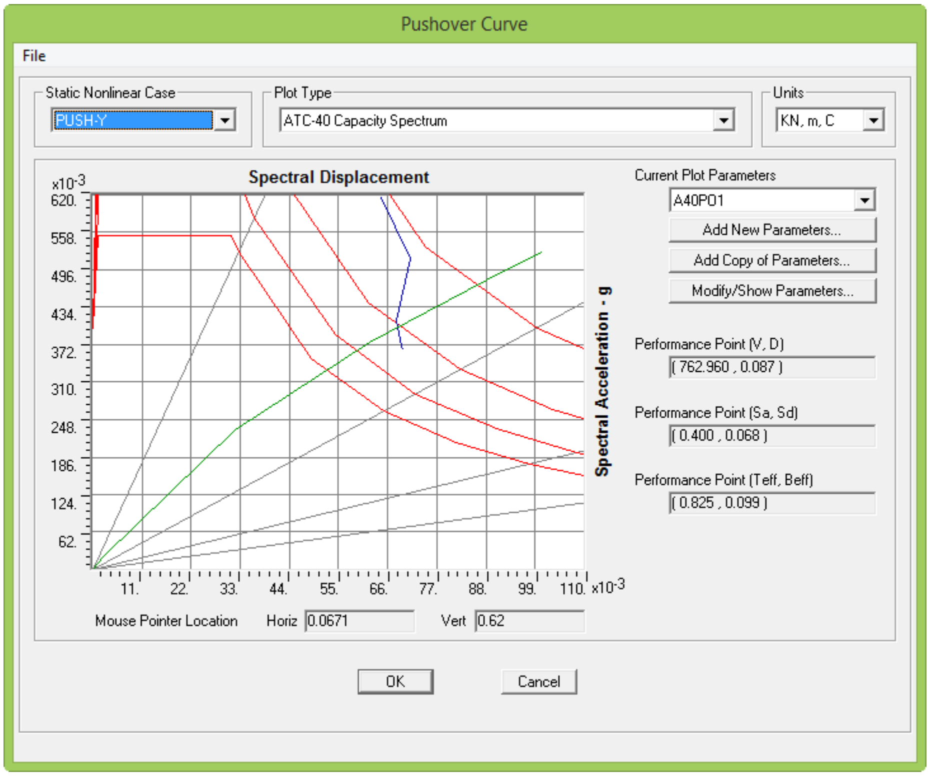


Fig. (6.20): Performance Point in Y-direction for Building (B1-4).

6.3.3 Discussion of Results for (B1) Building Configuration

Table (6.10) includes the maximum roof displacement and the corresponding base shear for each case study in each direction of analysis.

Table (6.10): Maximum Displacements and Base Shear for B1 Case Studies.

Displ. & Shear	(B1-1)		(B1-2)		(B1-3)		(B1-4)	
	X-Dir	Y-Dir	X-Dir	Y-Dir	X-Dir	Y-Dir	X-Dir	Y-Dir
Max. Displ. (m)	0.309	0.218	0.115	0.109	0.106	0.107	0.074	0.128
Base Shear (KN)	394.90	242.0	3134.6	1765.5	1297.5	457.2	5286.3	1014.1

From Table (6.10), it has been observed the followings:

1. Building (B1-1) experiences lateral displacements in X and Y-directions larger than the displacements of the other buildings in the same directions. Also, the maximum base shear resisted by building (B1-1) is less than the base shear resisted by the other building. These observations prove that infill walls increase the lateral stiffness of buildings and thus reduce the lateral displacement and enhance the seismic resistance of it. Also, the insertion of shear walls in X-direction of building (B1-4) decreases significantly the lateral displacement and increase the base shear. Y-direction of building (B1-4) is not affected by the insertion of shear walls since the stiffness of shear walls in Y-direction is very small.

2. The base shear resisted by each building in Y-direction is less than the base shear resisted in X-direction. This is because the long dimension of all of the building columns is oriented in X-direction which increase the stiffness of the building in this direction.

3. The presence of soft story in building (B1-3) decreases significantly the base shear that the building can resist. This proves that the presence of soft story decreases the seismic resistance of buildings.

4. The insertion of shear walls in X-direction of building (B1-4) solve the problem of the soft story.

From the deformation shape and the tables of pushover curves data at the performance point of each building, it has been observed the followings:

1. For building (B1-1) in X-direction, hinges of first yielding condition are formed in two columns only in the ground floor. All other formed hinges are in the beams of upper floors and are also in the first yielding condition. This means that the building is seismically safe in X-direction taking into account the rigidity of the joints between beams and columns only. In Y-direction, 7 hinges of first yielding condition and 3 hinges of immediate occupancy level are formed in the ground floor columns. All other

formed hinges are in the beams of other floors and are in the state of first yielding and immediate occupancy level. The building is safe in Y-direction but less than X-direction.

2. For building (B1-2) in X and Y-directions, all the formed hinges in the beams and columns are in the state of first yielding condition except 2 columns in the ground floor in X- direction which are in the state of immediate occupancy. The building is considered as seismically safe in each direction since the infill walls increase the stiffness and the strength of the building.
3. For building (B1-3) in X-direction, 9 hinges of immediate occupancy level are formed in the ground floor columns. All other formed hinges are in the beams of other floors and are in the state of first yielding condition. This means that the building is safe in the X-direction despite of the presence of the soft story but less than the same direction of building (B1-2). This is because the larger stiffness of all columns is in the X-direction. In Y-direction, plastic hinges of different performance levels ranges from immediate occupancy to collapse levels are formed in all of the ground floor columns. This proves that the building is not safe in Y-direction in case of the presence of soft story and emphasizes that the soft story floor needs to be seismically strengthened.
4. For building (B1-4) in X-direction, hinges of first yielding condition are formed in 4 columns only in the ground floor. All other formed hinges are in the beams of upper floors and are also in the first yielding condition. This means that the building is seismically safe in X-direction. Although the lateral displacement in X-direction is very small at the performance point (0.013m), the plastic hinges are formed in columns due to the torsion of the building. Torsion is formed because the inserted shear walls are not symmetric along the direction of concern.

The estimation of inter-story drift ratio of the buildings at the performance point is essential for seismic performance evaluation since the structural damage is directly related to the inter-story drift ratio as shown in Table (6.11) taken from ATC-40 code.

Table (6.11): Deformation Limits of ATC-40.

Inter-story Drift Limit	Performance Level			
	Immediate Occupancy	Damage Control	Life Safety	Structural Stability
Maximum Total Drift	0.01	0.01-0.02	0.02	$0.33\frac{V_i}{P_i}$ *

* V_i is the total lateral shear force in story *i* and P_i is the total gravity loads in the same story.

The inter-story drift at the performance point of each building is shown in Table (6.12).

Table (6.12): Inter-story Drift at the Performance Point for B1 Case Studies.

Floor	Inter-story Drift (X-Direction)				Inter-story Drift (Y-Direction)			
	B1-1	B1-2	B1-3	B1-4	B1-1	B1-2	B1-3	B1-4
4th	0.017	0.007	0.003	0.005	0.005	0.006	0.002	0.005
3rd	0.023	0.012	0.005	0.006	0.013	0.010	0.004	0.007
2ed	0.026	0.016	0.008	0.007	0.025	0.014	0.005	0.009
1st	0.021	0.020	0.012	0.008	0.035	0.017	0.010	0.013
Gr.	0.015	0.022	0.033	0.019	0.043	0.026	0.088	0.087

From Table (6.12), it has been observed the followings:

1. The performance level of building (B1-1) is met the criteria of structural stability in X-direction and exceed the criteria of structural stability in Y-direction. Thus, the building is seismically safe in X-direction and seismic vulnerable in Y-direction.
2. The performance level of building (B1-2) is met the criteria of structural stability in X and Y-directions. Thus, presence of infill walls in all stories makes the building safe during earthquakes.
3. The performance level of building (B1-3) is met the criteria of structural stability in X-direction and exceed the criteria of structural stability in Y-direction. Thus, the building is seismically safe in X-direction and may exposed to damage in Y-direction. Thus, the presence of soft story decreases the seismic resistance of the buildings significantly.
4. The performance level of building (B1-4) is met the criteria of damage control in X-direction and exceed the criteria of structural stability in Y-direction. Thus, the building is seismically safe in X-direction and may exposed to damage in Y-direction. The insertion of shear walls in X-direction only does not improve the performance in Y-direction. So, the building should also be strengthened in Y-direction.

6.4 Building Configuration (B2)

6.4.1 General Description of Building Configuration (B2)

This building is an existing R.C. building located in Gaza city. The building consists of ground floor, mezzanine floor, 4 typical floors, and roof floor. All floors are of 3m in height (i.e. the building height is 21m). The building dimensions are 20.9m x 14.85m in plan as shown in Fig. (6.21).

The gravity load carrying system is the same system of (B1). Also the building is designed to resist gravity loads only. The cross section and reinforcement details of columns are shown in Table (6.13). The arrangement and cross section of beams are shown in Fig. (6.22).

Table (6.13): Dimensions and Reinforcement of (B2) Columns.

Col. No.	Ground+1st + Mezzanine Floors		Other Floors	
	Dim. (cm)	Reinf.	Dim. (cm)	Reinf.
C1	20x50	8Φ14	20x40	6Φ14
C2	20x70	10Φ14	20x50	8Φ14
C3	20x80	12Φ14	20x60	8Φ14

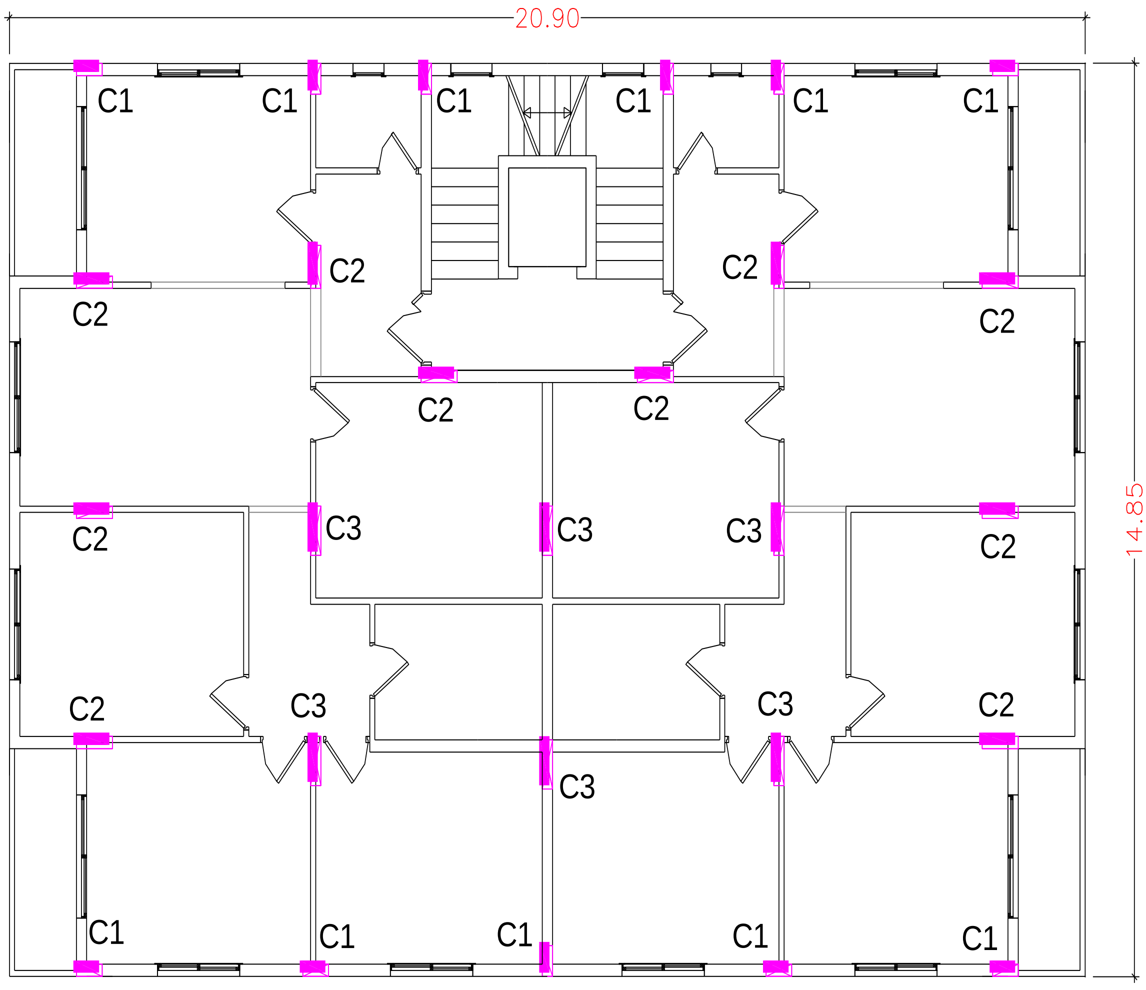


Fig. (6.21): Floor Plan and Columns Location for Building (B2).

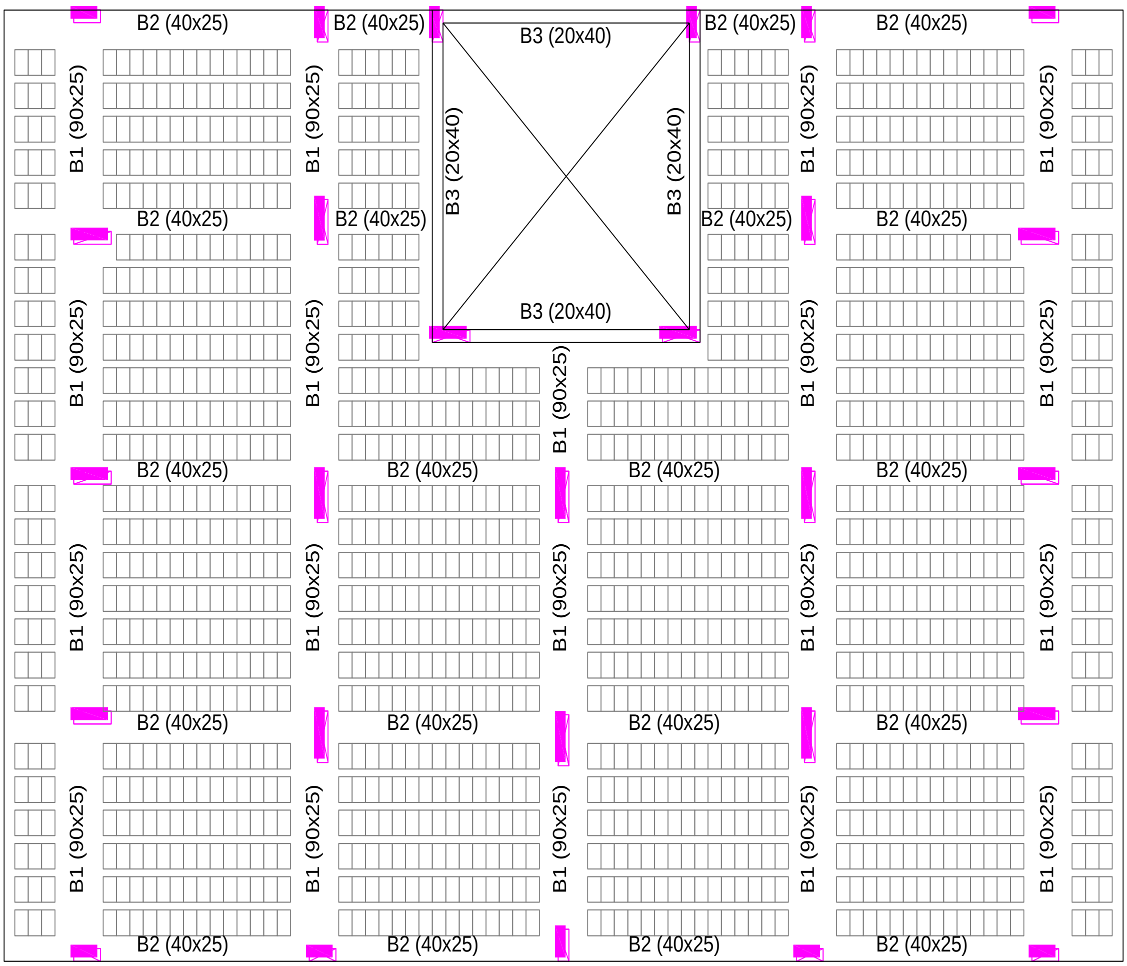


Fig. (6.22): Beams Arrangement and Dimensions for Building (B2).

6.4.2 Structural Modelling and Analysis of (B2)

All modelling, design, and analysis parameters for this building is the same as for building (B1). The targeted displacement is taken as 300mm in each direction.

This building is also analyzed 4 times separately: (1) without infill walls (B2-1), (2) with infill walls in all stories (B2-2), (3) with infill walls and soft ground story (B2-3), and (4) with infill walls, soft ground story, and shear walls in around the elevator (B1-4).

6.4.2.1 Analysis Results for Case Study 5: (B2-1)

(B2-1) case study is (B2) building analyzed without infill walls. After running the analysis, the analysis is completed in 9 steps in X-direction and in 10 steps in Y-directions. Tables (6.14) and (6.15) show the pushover curve data for X and Y-directions respectively.

Table (6.14): Pushover Curve Data in X-Direction for Building (B2-1).

Step	Displ. (m)	Base Force (KN)	A to B	B to IO	IO to LS	LS to CP	CP to C	C to D	D to E	Beyond E	Total
0	0.000	0.0	1162	0	0	0	0	0	0	0	1162
1	0.003	46.9	1160	2	0	0	0	0	0	0	1162
2	0.029	327.7	1044	118	0	0	0	0	0	0	1162
3	0.060	525.6	998	164	0	0	0	0	0	0	1162
4	0.091	654.6	923	238	1	0	0	0	0	0	1162
5	0.121	732.5	885	273	4	0	0	0	0	0	1162
6	0.152	797.8	860	250	50	2	0	0	0	0	1162
7	0.183	847.7	841	235	82	4	0	0	0	0	1162
8	0.215	892.7	828	222	106	4	0	2	0	0	1162
9	0.192	546.3	828	218	110	4	0	0	2	0	1162

Table (6.15): Pushover Curve Data in Y-Direction for Building (B2-1).

Step	Displ. (m)	Base Force (KN)	A to B	B to IO	IO to LS	LS to CP	CP to C	C to D	D to E	Beyond E	Total
0	0.000	0.0	1162	0	0	0	0	0	0	0	1162
1	0.002	59.5	1161	1	0	0	0	0	0	0	1162
2	0.021	513.5	1099	63	0	0	0	0	0	0	1162
3	0.051	886.4	992	170	0	0	0	0	0	0	1162
4	0.084	1099.6	930	232	0	0	0	0	0	0	1162
5	0.116	1221.0	898	256	8	0	0	0	0	0	1162
6	0.151	1327.8	873	238	51	0	0	0	0	0	1162
7	0.185	1412.7	850	224	86	2	0	0	0	0	1162
8	0.224	1487.9	827	200	129	6	0	0	0	0	1162
9	0.251	1534.3	811	174	163	12	0	2	0	0	1162
10	0.236	1127.1	811	174	163	12	0	2	0	0	1162

Fig. (6.23) and (6.24) show the deformation shapes for X-direction and Y-direction respectively.

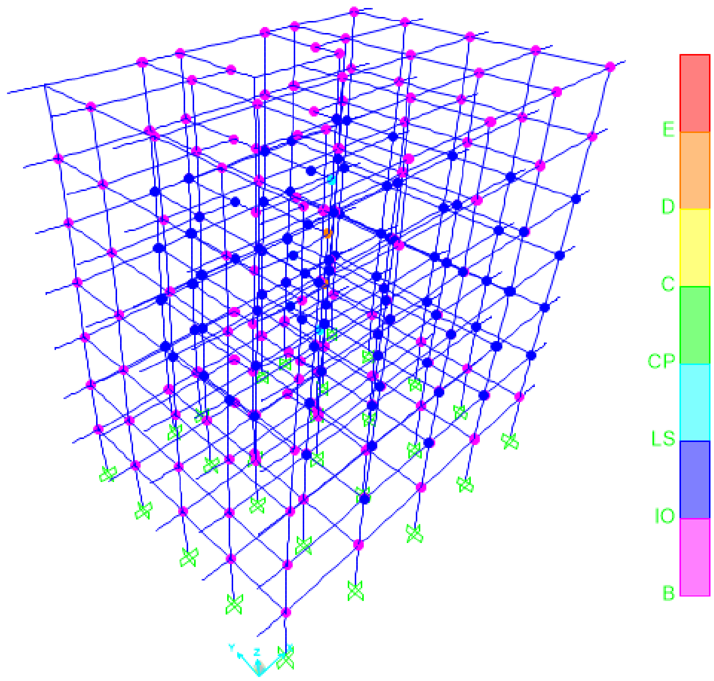


Fig. (6.23): Deformation Shape at Step 9 in X-Direction for Building (B2-1).

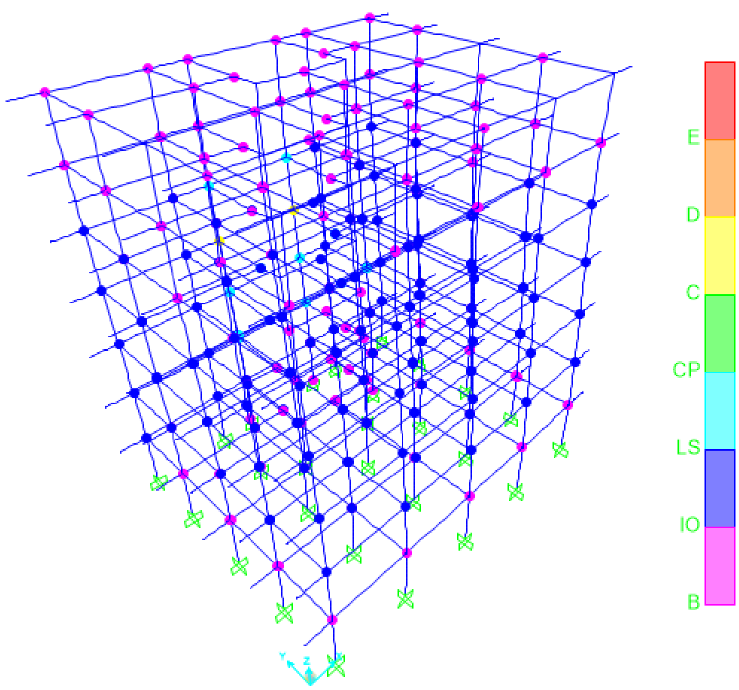


Fig. (6.24): Deformation Shape at Step 10 in Y-Direction for Building (B2-1).

The performance point is shown in Fig. (6.25) and (6.26) for X and Y-directions respectively. The performance point for (B2-1) in X-direction is at *V*=776 KN and *D*=0.142m, and in Y-direction is at *V*=1142 KN and *D*=0.096m.

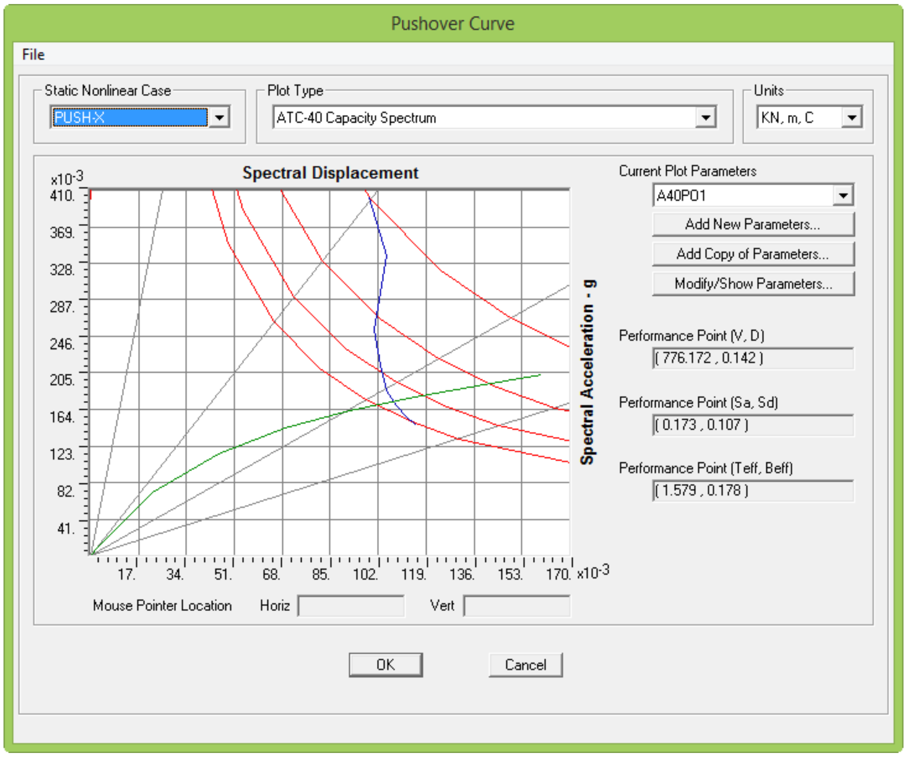


Fig. (6.25): Performance Point in X-direction for Building (B2-1).

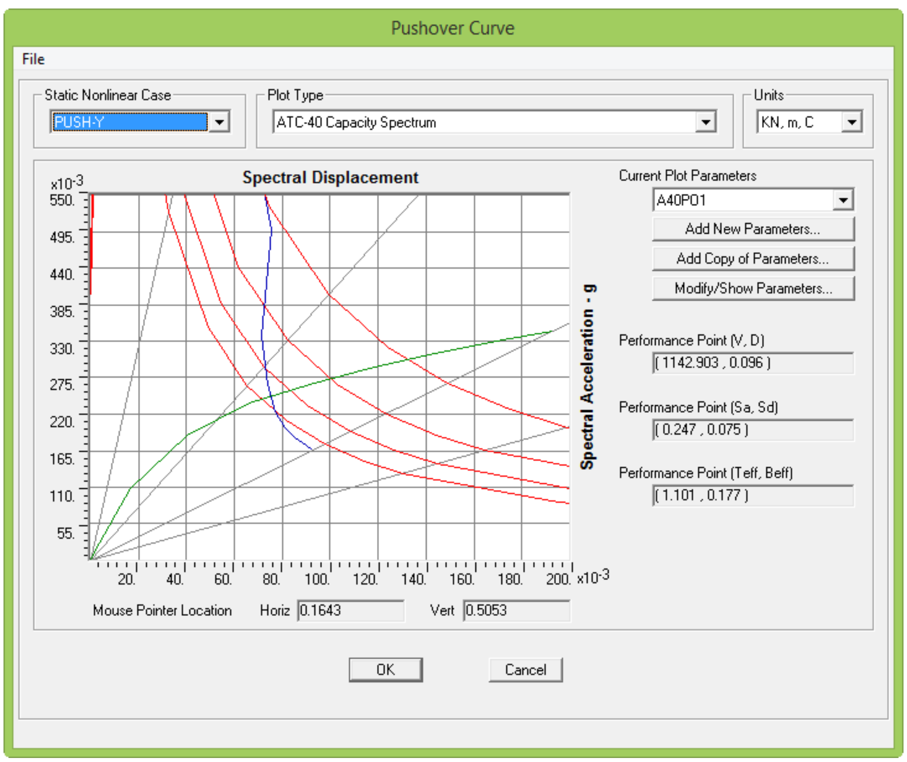


Fig. (6.26): Performance Point in Y-direction for Building (B2-1).

6.4.2.2 Analysis Results for Case Study 6: (B2-2)

(B2-2) case study is (B2) building analyzed with infill walls in all stories. Infill walls width is taken as 40cm. The analysis is completed in 9 steps in X and Y-directions. Tables (6.16) and (6.17) show the pushover curve data for X and Y-directions respectively.

Table (6.16): Pushover Curve Data in X-Direction for Building (B2-2).

Step	Displ. (m)	Base Force (KN)	A to B	B to IO	IO to LS	LS to CP	CP to C	C to D	D to E	Beyond E	Total
0	0.002	0.0	1162	0	0	0	0	0	0	0	1162
1	0.004	91.9	1161	1	0	0	0	0	0	0	1162
2	0.035	1283.8	1043	119	0	0	0	0	0	0	1162
3	0.065	2352.3	988	174	0	0	0	0	0	0	1162
4	0.096	3391.1	912	248	2	0	0	0	0	0	1162
5	0.131	4458.1	882	271	8	1	0	0	0	0	1162
6	0.162	5421.2	853	264	43	2	0	0	0	0	1162
7	0.193	6339.5	829	255	73	5	0	0	0	0	1162
8	0.204	6669.5	820	254	83	4	0	1	0	0	1162
9	0.189	6046.4	820	254	83	4	0	1	0	0	1162

Table (6.17): Pushover Curve Data in Y-Direction for Building (B2-2).

Step	Displ. (m)	Base Force (KN)	A to B	B to IO	IO to LS	LS to CP	CP to C	C to D	D to E	Beyond E	Total
0	0.002	0.0	1162	0	0	0	0	0	0	0	1162
1	0.004	102.0	1161	1	0	0	0	0	0	0	1162
2	0.034	1501.7	1062	100	0	0	0	0	0	0	1162
3	0.066	2591.4	968	194	0	0	0	0	0	0	1162
4	0.096	3529.0	930	232	0	0	0	0	0	0	1162
5	0.127	4434.4	895	253	14	0	0	0	0	0	1162
6	0.156	5253.5	867	257	38	0	0	0	0	0	1162
7	0.156	5246.8	864	260	38	0	0	0	0	0	1162
8	0.184	5998.5	836	264	60	2	0	0	0	0	1162
9	0.183	5950.3	836	263	61	2	0	0	0	0	1162

Fig. (6.27) and (6.28) shows the deformation shapes and hinge locations for X-direction and Y-direction respectively.

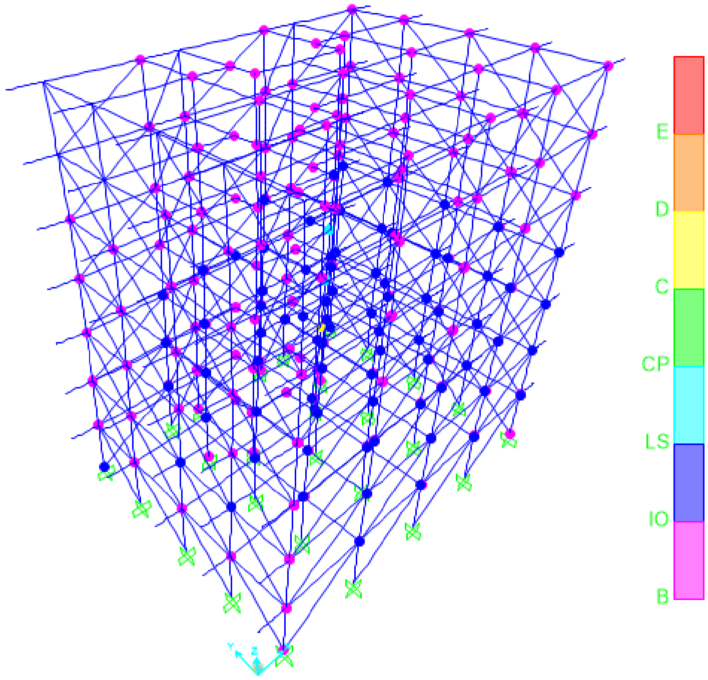


Fig. (6.27): Deformation Shape at Step 9 in X-Direction for Building (B2-2).

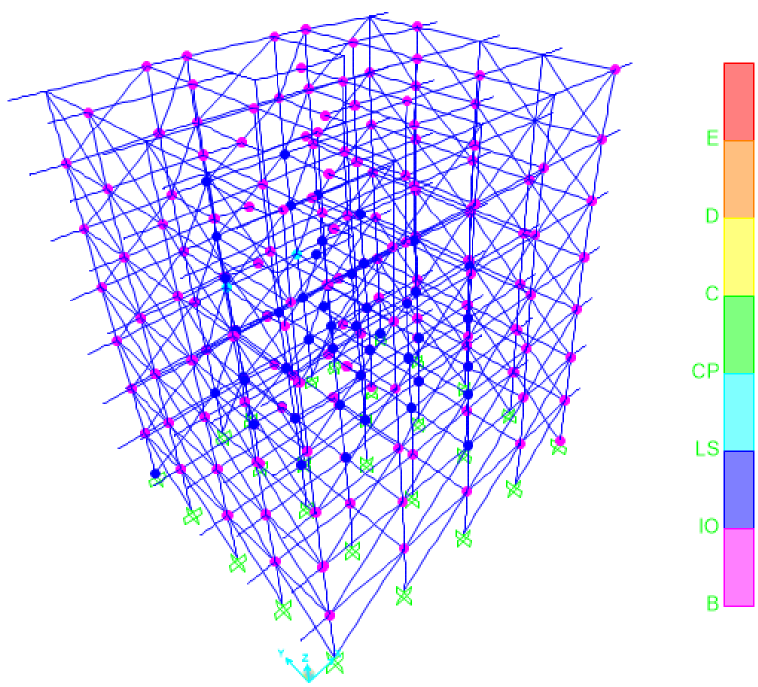


Fig. (6.28): Deformation Shape at Step 9 in Y-Direction for Building (B2-2).

The performance point is shown in Fig. (6.29) and (6.30) for X and Y-directions respectively. The performance point for (B2-2) in X-direction is at *V*=2949 KN and *D*=0.083m, and in Y-direction is at *V*=2873 KN and *D*=0.075m.

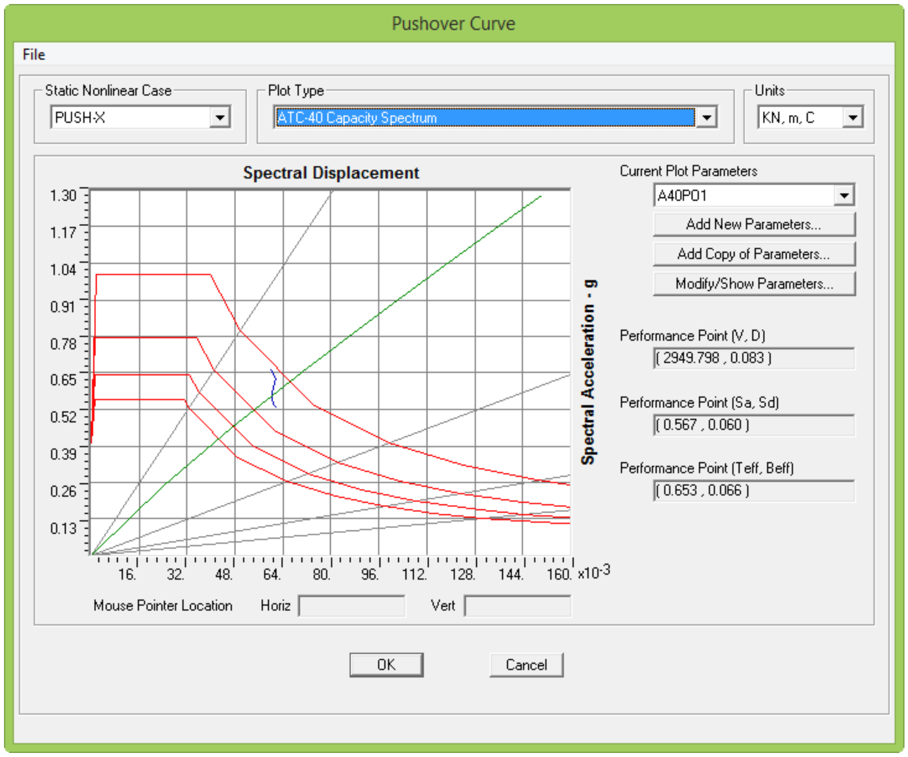


Fig. (6.29): Performance Point in X-direction for Building (B2-2).

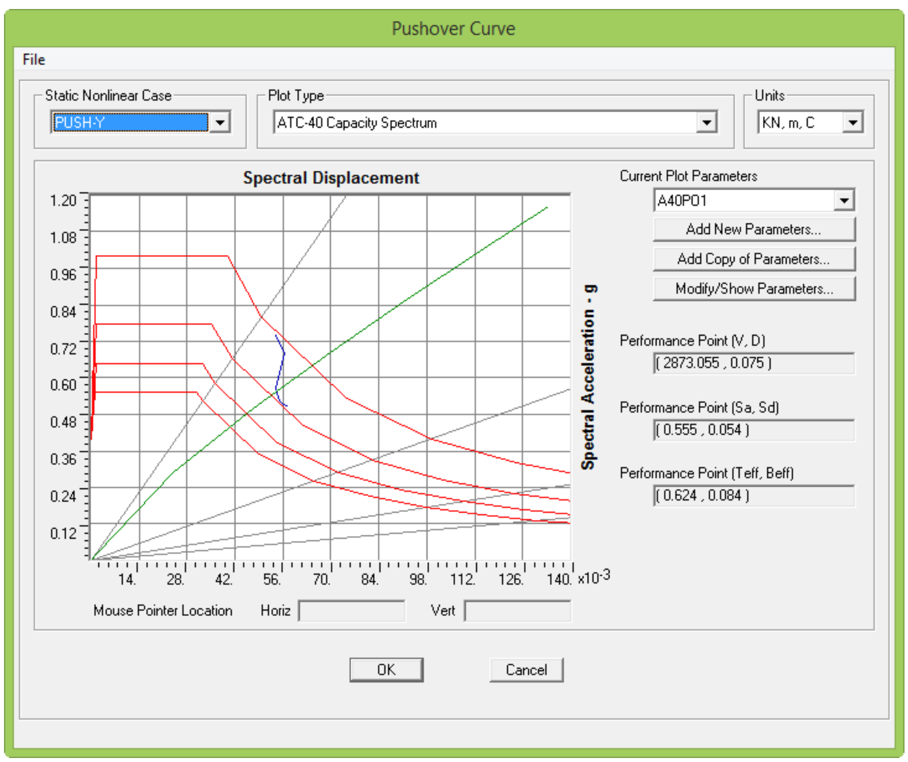


Fig. (6.30): Performance Point in Y-direction for Building (B2-2).

6.4.2.3 Analysis Results for Case Study 7: (B2-3)

(B2-3) case study is (B2) building analyzed with infill walls and soft ground story. The analysis is completed in 7 steps in X and Y-directions. Tables (6.18) and (6.19) show the pushover curve data for X and Y-directions respectively.

Table (6.18): Pushover Curve Data in X-Direction for Building (B2-3).

Step	Displ. (m)	Base Force (KN)	A to B	B to IO	IO to LS	LS to CP	CP to C	C to D	D to E	Beyond E	Total
0	0.002	0.0	1162	0	0	0	0	0	0	0	1162
1	0.004	86.9	1161	1	0	0	0	0	0	0	1162
2	0.035	1143.4	1050	112	0	0	0	0	0	0	1162
3	0.066	2083.7	981	181	0	0	0	0	0	0	1162
4	0.098	2905.9	918	239	5	0	0	0	0	0	1162
5	0.122	3513.0	891	249	21	1	0	0	0	0	1162
6	0.122	3511.5	891	248	22	1	0	0	0	0	1162
7	0.134	3788.9	883	240	38	1	0	0	0	0	1162

Table (6.19): Pushover Curve Data in Y-Direction for Building (B2-3).

Step	Displ. (m)	Base Force (KN)	A to B	B to IO	IO to LS	LS to CP	CP to C	C to D	D to E	Beyond E	Total
0	0.001	0.0	1162	0	0	0	0	0	0	0	1162
1	0.003	66.8	1161	1	0	0	0	0	0	0	1162
2	0.034	1237.0	1077	85	0	0	0	0	0	0	1162
3	0.066	2116.6	984	178	0	0	0	0	0	0	1162
4	0.096	2809.2	948	210	4	0	0	0	0	0	1162
5	0.129	3502.0	918	198	46	0	0	0	0	0	1162
6	0.159	4082.0	883	203	74	2	0	0	0	0	1162
7	0.189	4634.3	858	203	91	8	0	2	0	0	1162

Fig. (6.31) and (6.32) shows the deformation shapes and hinge locations for X-direction and Y-direction respectively.

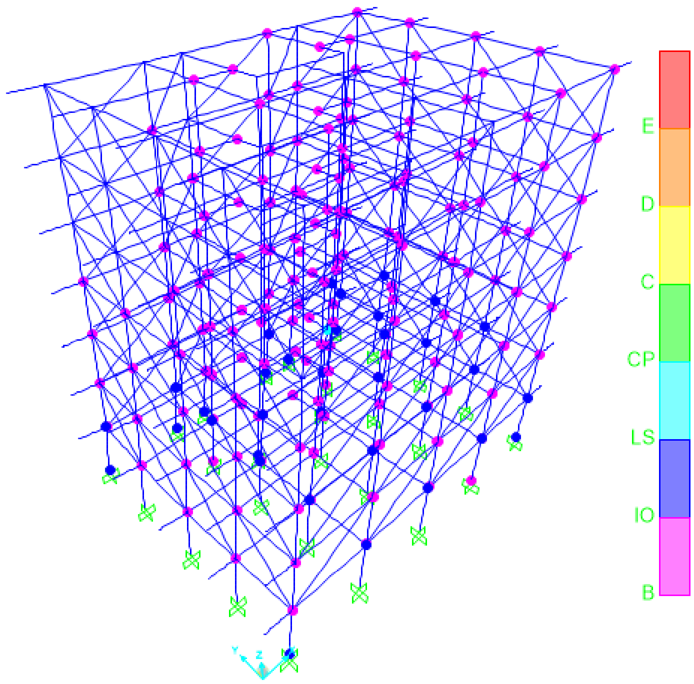


Fig. (6.31): Deformation Shape at Step 7 in X-Direction for Building (B2-3).

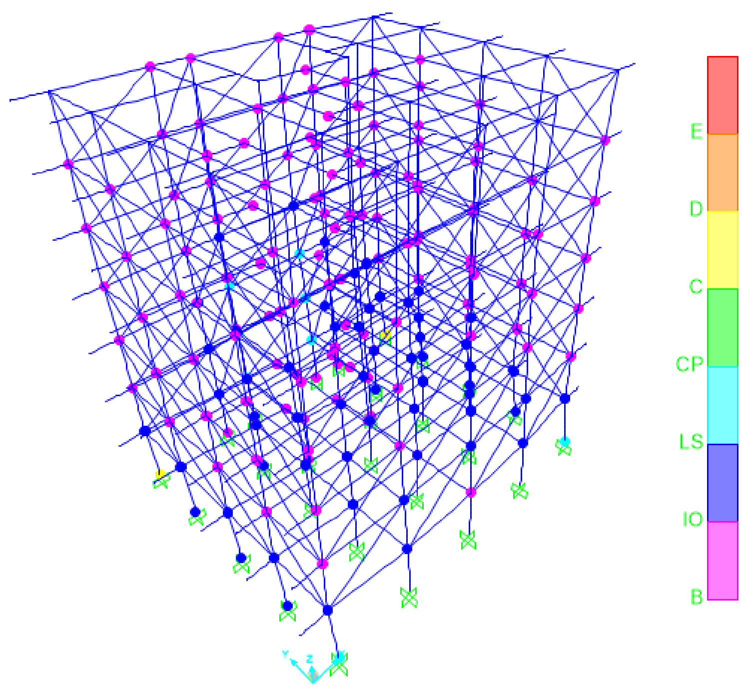


Fig. (6.32): Deformation Shape at Step 7 in Y-Direction for Building (B2-3).

The performance point is shown in Fig. (6.33) and (6.34) for X and Y-directions respectively. The performance point for (B2-3) in X-direction is at *V*=2640 KN and *D*=0.087m, and in Y-direction is *V*=2475 KN and *D*=0.082m.

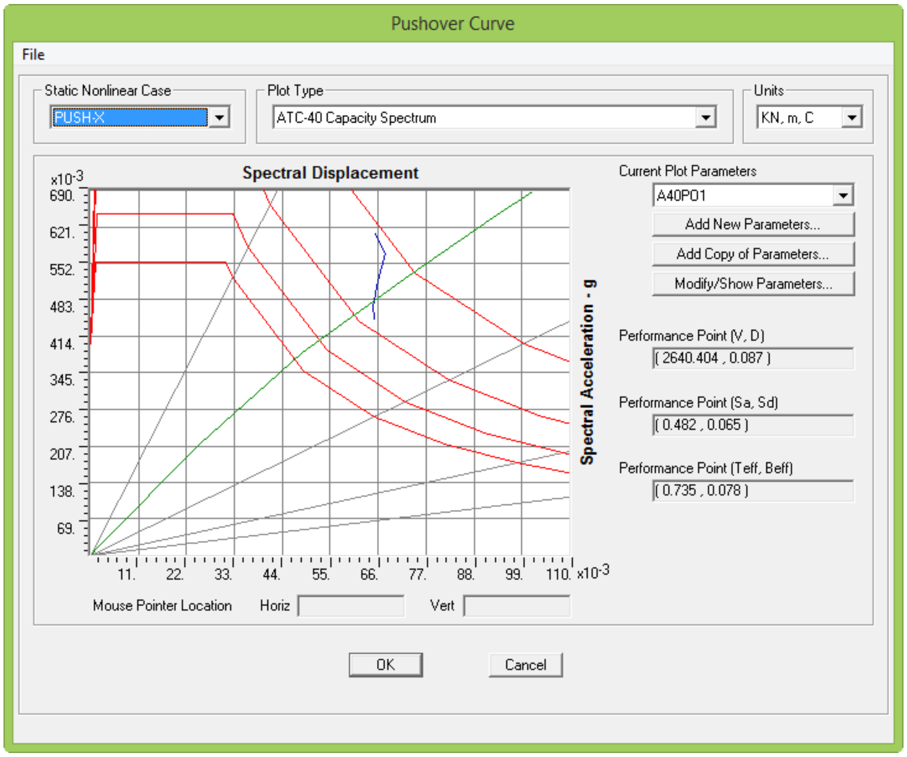


Fig. (6.33): Performance Point in X-direction for Building (B2-3).

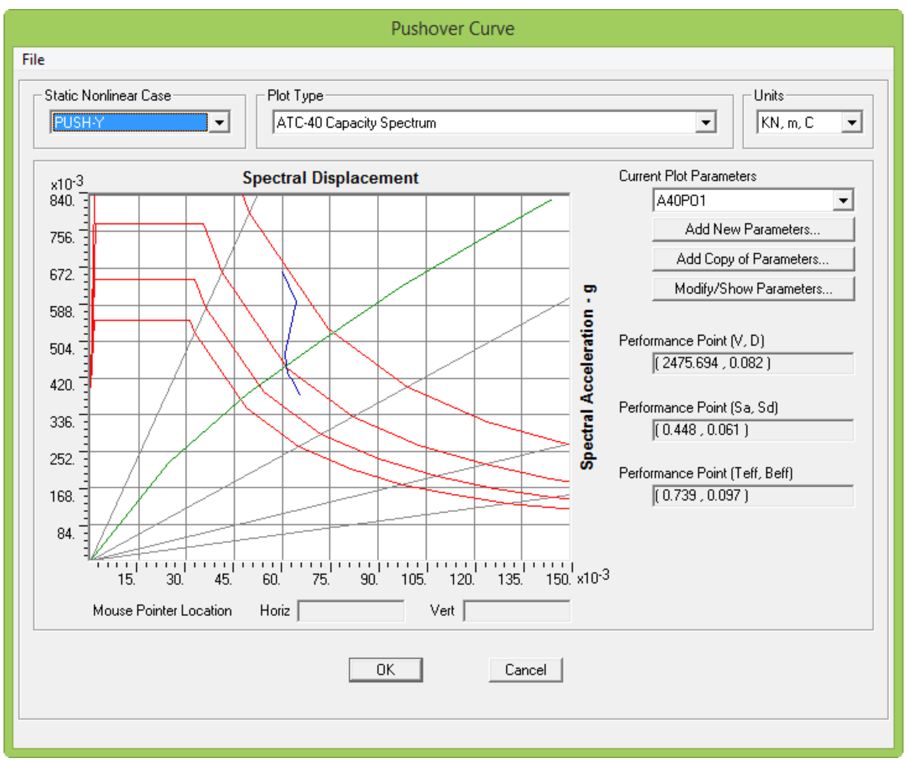


Fig. (6.34): Performance Point in Y-direction for Building (B2-3).

6.4.2.4 Analysis Results for Case Study 8: (B2-4)

(B2-4) case study is (B2) building analyzed with infill walls, soft ground story, and inserted shear walls around the elevator. The analysis is completed in 7 steps in X and Y-directions. Tables (6.20) and (6.21) show the pushover curve data for X and Y-directions respectively.

Table (6.20): Pushover Curve Data in X-Direction for Building (B2-4).

Step	Displ. (m)	Base Force (KN)	A to B	B to IO	IO to LS	LS to CP	CP to C	C to D	D to E	Beyond E	Total
0	0.001	0.0	1162	0	0	0	0	0	0	0	1162
1	0.003	87.8	1161	1	0	0	0	0	0	0	1162
2	0.034	1619.4	1018	144	0	0	0	0	0	0	1162
3	0.065	2967.7	955	207	0	0	0	0	0	0	1162
4	0.097	4281.8	863	297	2	0	0	0	0	0	1162
5	0.128	5460.0	795	354	13	0	0	0	0	0	1162
6	0.147	6126.9	778	352	32	0	0	0	0	0	1162
7	0.130	5281.9	777	350	35	0	0	0	0	0	1162

Table (6.21): Pushover Curve Data in Y-Direction for Building (B2-4).

Step	Displ. (m)	Base Force (KN)	A to B	B to IO	IO to LS	LS to CP	CP to C	C to D	D to E	Beyond E	Total
0	0.000	0.0	1162	0	0	0	0	0	0	0	1162
1	0.008	705.8	1160	2	0	0	0	0	0	0	1162
2	0.039	2290.8	1045	117	0	0	0	0	0	0	1162
3	0.071	3602.7	934	228	0	0	0	0	0	0	1162
4	0.077	3823.3	913	249	0	0	0	0	0	0	1162
5	0.077	3815.6	911	251	0	0	0	0	0	0	1162
6	0.078	3867.8	903	259	0	0	0	0	0	0	1162
7	0.054	2374.7	900	262	0	0	0	0	0	0	1162

Fig. (6.35) and (6.36) shows the deformation shapes and hinge locations for X-direction and Y-direction respectively.

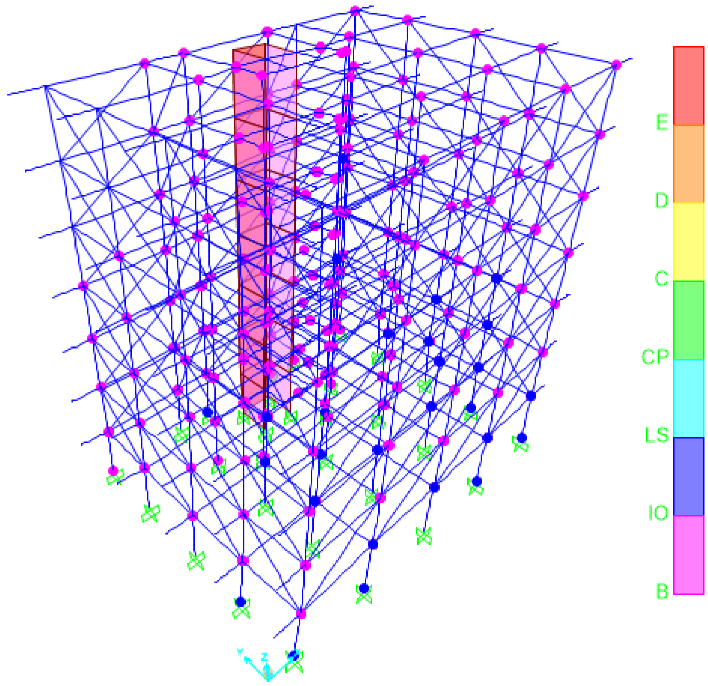


Fig. (6.35): Deformation Shape at Step 7 in X-Direction for Building (B2-4).

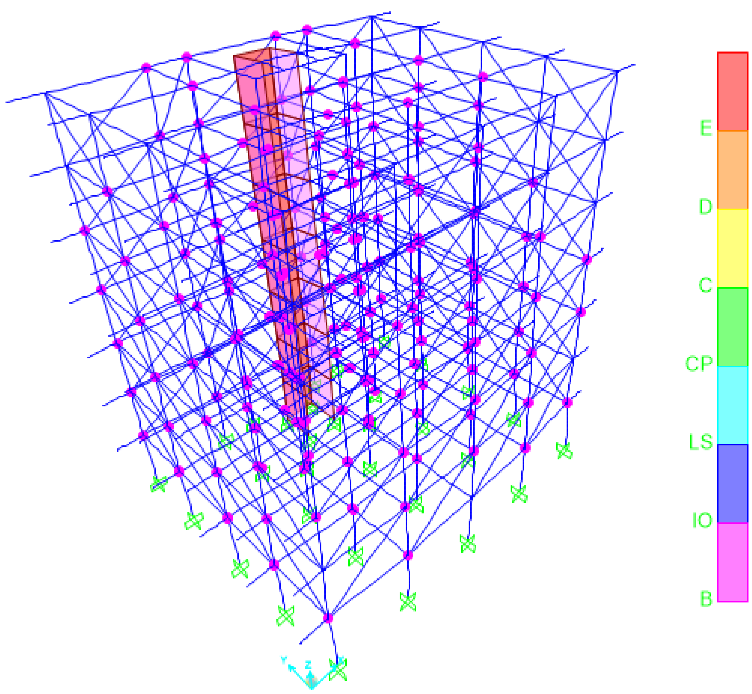


Fig. (6.36): Deformation Shape at Step 7 in Y-Direction for Building (B2-4).

The performance point is shown in Fig. (6.37) and (6.38) for X and Y-directions respectively. The performance point for (B2-4) in X-direction is at *V*=3388 KN and *D*=0.075m, and in Y-direction is *V*=3557 KN and *D*=0.07m.

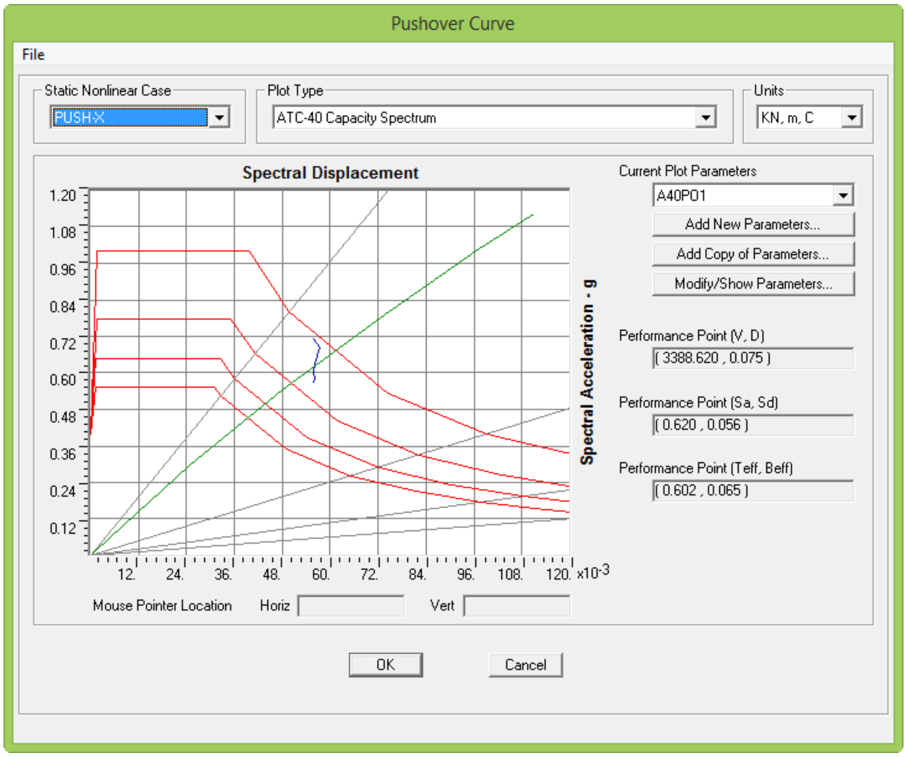


Fig. (6.37): Performance Point in X-direction for Building (B2-4).

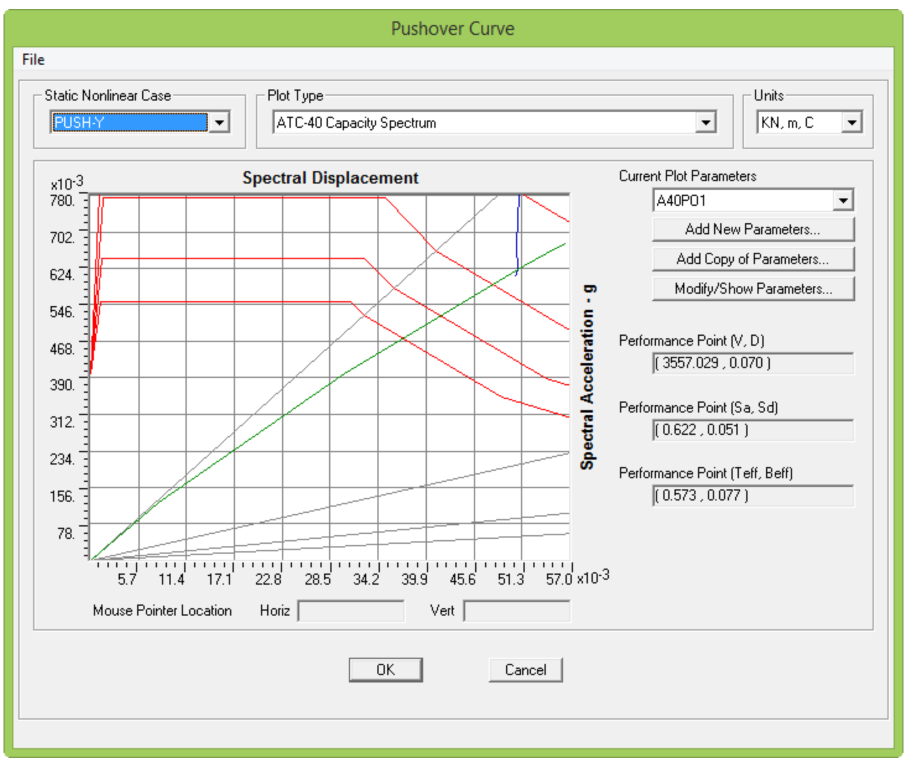


Fig. (6.38): Performance Point in Y-direction for Building (B2-4).

6.4.3 Discussion of Results for (B2) Building Configuration

Table (6.22) include the maximum roof displacement and the corresponding base shear for each case study in each direction of analysis.

Table (6.22): Maximum Displacements and Base Shear for B2 Case Studies.

Displ. & Shear	(B2-1)		(B2-2)		(B2-3)		(B2-4)	
	X-Dir	Y-Dir	X-Dir	Y-Dir	X-Dir	Y-Dir	X-Dir	Y-Dir
Max. Displ. (m)	0.215	0.251	0.204	0.184	0.134	0.189	0.147	0.078
Base Shear (KN)	892.7	1127	6669.5	5998.5	3788.9	4634.3	6126.9	3867.8

From Table (6.22), it has been observed the followings:

1. Building (B2-1) experiences lateral displacements in X and Y-directions larger than the displacements of the other buildings in the same directions. Also, the maximum base shear resisted by building (B2-1) is less than the base shear resisted by the other building. Infill walls is the reason of these observations. Also, the insertion of shear walls around the elevator decreases the lateral displacement in Y-direction and does not affect the displacement of X-direction because the formation of torsion.
2. The base shear resisted by each building in the two directions is almost the same. This is because the long dimension of columns is well distributed in X and Y-directions and the building is almost square in plan.
3. The presence of soft story in building (B2-3) decreases significantly the base shear that the building can resist in X-direction.
4. The insertion of shear walls in X-direction of building (B2-4) solve the problem of the soft story.

From the deformation shape and the tables of pushover curves data at the performance point of each building, it has been observed the followings:

1. For building (B2-1) in X and Y-directions, no hinges are formed in the ground floor columns. Hinges are formed in the beams of upper floors and are in the first yielding condition and immediate occupancy level. This means that the building is seismically safe in X and Y-directions taking into account the rigidity of the joints between beams and columns only.
2. For building (B2-2), the building condition is the same as building (B2-1).
3. For building (B2-3), 12 hinges of first yielding and immediate occupancy levels are formed in the ground floor columns in X-direction and 6 hinges of first yielding and immediate occupancy levels in Y-direction. All other formed hinges are in the beams of other floors and are in the state of first yielding and immediate occupancy level. This

means that the building is safe in the X and Y-directions despite of the presence of the soft story.

4. For building (B2-4), 5 hinges of first yielding and immediate occupancy levels are formed in the ground floor columns in X-direction and 4 hinges of first yielding levels in Y-direction. All other formed hinges are in the beams of other floors and are in the state of first yielding and immediate occupancy level. The insertion of shear walls enhances the performance of the building in the two directions.

The inter-story drift at the performance point of each building is shown in Table (6.23).

Table (6.23): Inter-story Drift at the Performance Point for B2 Case Studies.

Floor	Inter-story Drift (X-Direction)				Inter-story Drift (Y-Direction)			
	B2-1	B2-2	B2-3	B2-4	B2-1	B2-2	B2-3	B2-4
Roof	0.011	0.007	0.005	0.006	0.005	0.004	0.003	0.006
4th	0.019	0.009	0.008	0.010	0.011	0.009	0.006	0.009
3rd	0.028	0.014	0.012	0.014	0.020	0.014	0.011	0.011
2ed	0.032	0.018	0.016	0.017	0.026	0.019	0.017	0.012
1st	0.030	0.020	0.021	0.017	0.024	0.021	0.021	0.012
Mezz.	0.023	0.019	0.023	0.019	0.021	0.017	0.022	0.012
Gr.	0.010	0.010	0.013	0.023	0.009	0.010	0.014	0.007

From Table (6.23), it has been observed the followings:

1. The performance level of building (B2-1) is met the criteria of structural stability in X and Y-directions. Thus, the building is seismically safe in the two directions
2. The performance level of building (B2-2) is met the criteria of life safety in X and Y-directions. Thus, presence of infill walls in all stories makes the building safe during earthquakes.
3. The performance level of building (B2-3) is met the criteria of life safety in X and Y-directions. Thus, the building is seismically safe in the two directions.
4. The performance level of building (B2-4) is met the criteria of life safety in X and Y-directions. Thus, the building is seismically safe in the two directions.

6.5 Conclusions for Gaza Strip Buildings

Based on the results of the seismic assessment of B1 and B2 building configurations and assuming that these configurations represent the majority of existing low-rise residential reinforced concrete buildings in Gaza Strip, the following conclusions have been drawn:

1. Regular low-rise residential reinforced concrete buildings of Gaza Strip designed for gravity loads only are considered to be seismically safe taking into account the rigidity of the joints between beams and columns only.
2. Buildings have horizontal and vertical irregularities may be exposed to local damages in the ground floor columns during earthquakes which could lead to failures.
3. Presence of infill walls has beneficial effects on the performance of buildings during earthquakes as long as horizontal and vertical irregularities such as soft story do not exists. Infill walls increases the lateral stiffness of buildings and thus enhances its seismic resistance.
4. The bad effect of the presence of soft ground stories depends on several factors such as number of stories, building irregularity, etc. Soft stories have no significant adverse effects on regular and symmetric buildings despite the number of stories. Buildings consist of 3 stories or less will not be affected by the presence of soft stories. Soft story decreases the lateral stiffness of irregular buildings significantly and thus reduces the seismic resistance.
5. The orientation of the long dimension of columns is an important factor in the seismic resistance of buildings. The direction contains the long dimension of columns have a seismic resistance larger than the other direction.
6. Buildings having structural walls behave better than other buildings during earthquakes as long as the location of these walls does not form horizontal irregularities.

6.6 Concluded Remarks

This chapter includes the application of pushover analysis to several low-rise residential reinforced concrete buildings in Gaza Strip. It has been concluded that the pushover analysis is a simple and effective procedure to assess the seismic resistance of buildings during earthquake. The findings of this research are obtained based on eight case studies assuming they represent typical residential buildings in Gaza strip. However, different findings may be obtained if different buildings have been considered, e.g. building with other veridical and horizontal irregularities. In conclusion this research is the first of its kind in assessing the seismic resistance of typical residential buildings in Gaza Strip. Further research is recommended in the future to assess all types of buildings and determine strengthening techniques for existing buildings. Concerned official authorities in Palestine are encouraged to take actions and draw regulations related to design of low rise buildings to ensure their adequacy to resist seismic forces.

CHAPTER 7

CONCLUSIONS AND RECOMMENDATIONS

7 CONCLUSIONS AND RECOMMENDATIONS

7.1 Introduction

In this research, the seismic resistance assessment of the low-rise reinforced concrete residential buildings of Gaza Strip which designed for gravity loads only has been carried out using pushover analysis methodology. SAP2000 has been used to perform the analysis of the eight case studies. This chapter includes the main conclusions drawn from the research and includes recommendations for existing buildings, new buildings, concerned authorities, and for future researches.

7.2 Conclusions

1. Palestine is exposed to significant earthquakes since it is located along the Dead Sea Transform fault (DST). Thus, buildings should be designed and constructed to resist seismic forces. This is not always the case.
2. Most of typical reinforced concrete residential buildings in Gaza strip are designed and constructed to resist gravity loads only without any considerations to seismic resistance.
3. It is generally assumed by designers that the seismic forces on low-rise buildings are low. The building frame structural system and infill walls are assumed to resist such loads. There has been no verification to these assumptions by designers.
4. Several seismic evaluation methodologies exist over around the world including qualitative (empirical) and quantitative (analytical) methodologies.
5. The most suitable seismic evaluation methodology to be used in Gaza Strip is the analytical methodology since it does not require an observed damage data from previous earthquakes.
6. The pushover analysis is a simple and effective procedure to assess the nonlinear behavior of building during earthquakes.
7. Pushover analysis identifies the weak structural elements by predicting the failure mechanism and account for the redistribution of forces during progressive yielding. It helps engineers to take action for rehabilitation work.
8. Regular low-rise residential reinforced concrete buildings of Gaza Strip designed for gravity loads only are considered to be seismically safe taking into account the rigidity of the joints between beams and columns only.
9. Buildings have horizontal and vertical irregularities may exposed to local damages in the ground floor columns during earthquakes and may lead to overall failure of the whole buildings especially in relatively high buildings of 5 or more stories.

10. The masonry infill walls positively affect the seismic resistance of buildings and thus their contribution should be considered in the assessment.
11. Presence of infill walls has a beneficial effects on the performance of buildings during earthquakes as long as horizontal and vertical irregularities does not exists such as soft story.
12. The adverse effect of the presence of soft ground stories depends on several factors such as number of stories, building irregularity, etc. Soft stories have no significant bad effects on regular and symmetric buildings despite the number of stories. Buildings consist of 3 stories or less will not be affected by the presence of soft stories. Soft story decreases the lateral stiffness of irregular buildings significantly and thus reduce the seismic resistance.
13. The orientation of the long dimension of columns is an important factor in the seismic resistance of buildings. The direction contains the long dimension of columns have a seismic resistance larger than the other direction.
14. Buildings having structural walls behave better than other buildings during earthquakes as long as the location of these walls does not form a horizontal irregularities.

7.3 Recommendations

Based on the results of the undertaken research, the following recommendations were made for existing buildings, new buildings, concerned authorities, and for future researches.

7.3.1 Recommendations for Existing Buildings

Buildings that are venerable to seismic forces such as buildings with soft stories need to be strengthened using proper techniques.

7.3.2 Recommendations for New Buildings

New residential buildings in Gaza Strip are to be designed for earthquake utilization the existing rigidity of the beam column connection, infill walls, and proper orientation of columns to enhance stiffness in the two directions. Special attention should be given to irregularities, if exist.

7.3.3 Recommendations for Concerned Public Authorities

1. Legal authorities should legislate special bylaws to enforce engineers to design and construct building according to seismic requirements.
2. Plans for rehabilitation and strengthening of existing buildings to resist earthquakes should be developed and enforced.
3. Seismically unsafe building types and practices should be prevented.

7.3.4 Recommendations for Future Researches

1. It would be desirable to study more case studies with more variables and irregularities before reaching definite general conclusions about the behavior of reinforced concrete frame buildings in Gaza Strip.
2. Residential buildings only have been seismically evaluated in this research. It is recommended to evaluate other types of buildings such as public and commercial buildings.
3. Code lateral load pattern is used in this research to represent the earthquake, the effect of using other load patterns such as uniform load pattern, first mode load pattern, etc. on the analysis results can be evaluated and compared to the results of this research.
4. Static nonlinear analysis is used in this research. Dynamic analysis methods can be used in future researches.
5. This research focuses on seismic evaluation of buildings. The pushover analysis can be utilized also in the design of new buildings.
6. Other available seismic evaluation methodologies can be used and the results can be compared to the results of this research.
7. Strengthening techniques for existing buildings need to be investigated.
8. Arrangement of infill walls affect the post yield behavior and has an influence on distribution and sequence of damage formation. To generalize this, more infill arrangements should be investigated.
9. This research investigated the behavior of the superstructure of Gaza Strip buildings. So, the behavior of the building foundations during earthquakes need to be investigated.

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