Palestine Polytechnic University

Deanship of Graduate Studies and Scientific Research

Master of Civil Engineering

Seismic Assessment of Existing Building in Israel

(By means of Pushover Analysis)

Student Name:

Sammy Al Asam

Supervisor:

Dr. Maher Amro

Thesis submitted in partial fulfillment of requirements of the degree

Master of Civil Engineering

May, 2023

The undersigned hereby certify that they have read, examined and recommended to the Deanship of Graduate Studies and Scientific Research at Palestine Polytechnic University:

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Graduate Advisory Committee:

Assistant Prof./Dr Maher Amro (Supervisor), Palestine Polytechnic University

Signature:______________________________ Date:____________________

Assistant Prof./Dr (External committee member),

Signature:______________________________ Date:____________________

Assistant Prof./Dr (Internal committee member), Palestine Polytechnic University

Signature:______________________________ Date:____________________

Thesis Approved by:

Name: ____

Dean of Graduate Studies & Scientific Research

Palestine Polytechnic University

Signature:……………..…………………

Date:…..……………...…………………

DECLARATION

I declare that the Master Thesis entitled" Seismic Assessment of Existing Building in Israel

(By means of Pushover Analysis)" is my own original work, and herby certify that unless stated, all work contained within this thesis is my own independent research and has not been submitted for the award of any other degree at any institution, except where due acknowledgement is made in the text.

Student Name: Sammy Al Asam

Signature:——————————— Date:———————

Acknowledgement

All praise be to ALLAH, for bestowing us health, patience, and knowledge to complete this report. May the peace and blessings of Allah be upon Prophet Mohammed (peace be upon him).

I would like to dedicate this project to:

Our homeland which attends our efforts to build the glorious future,

For my parents' sisters, brothers & friends for their constant prayers, guidance, encouragement and support, they are the source of power, inspiration, and confidence for me.

I acknowledge with deep gratitude and appreciation the remarkable assistance and continuous support, which are given to us by our project advisor, **Dr. Maher Amro**, who gave us the opportunity of great learning and experience through working with him.

To my great university staff who expertly gave us all the knowledge we needed.

To all who contributed to the success of this report and make it appear in a good shape.

For their contributions to the achievement of this report and providing us with all the facilities and the continued support

Abstract

Many existing buildings in high seismic hazard zones (including buildings and structures in Israel) have inadequate capacity to resist earthquakes loads. This is because they may have been designed and constructed according to earlier codes, which did not satisfy modern seismic design requirements and current engineering standards, faulty design and improper construction, alteration of building functions, changes of seismic load characteristics in the area, etc.

Israel code (as many international codes) adopted the conventional earthquake-resistant design philosophy (force based); the structures are designed for forces which are much less than the expected design earthquake forces.

These methods usually don’t consider the expected performance level and seismic risk levels of the structure after an earthquake event. So, need of new method comes which would give the actual performance of the structure after an earthquake event.

Performance based analysis and evaluation procedures are used to investigate seismic inadequacy of existing buildings. The improvements of seismic performance of these buildings may be achieved using retrofit systems such as steel bracing, shear walls, concrete and steel jacketing.

In this study, extensive literature study of most using retrofit schemes, which improve seismic performance of existing buildings, was introduced including their features.

Procedures of performance-based analysis using nonlinear static analysis (pushover) was used to study the performance of G+6 story reinforced concrete existing buildings without/and with different retrofit systems. The seismic response and performance of these cases were discussed.

The results of evaluation and analyses of the study cases showed necessity of 3D analysis, steel jacking system was proposed to bypass collapse plastic hinges and to reduce displacements and drifts, and concrete and steel jacketing bypass collapse hinges and keep ductility.

الملخص

العديد من المباني القائمة في مناطق الخطر الزلزالي العالية (بما في ذلك المباني والهياكل في إسرائيل) لديها قدرة غير كافية لمقاومة أحمال الزلازل. هذا لأنه ربما تم تصميمها وإنشائها وفقًا للرموز السابقة ، والتي لم تفي بمتطلبات التصميم الزلزالي الحديث والمعايير الهندسية الحالية ، والتصميم الخاطئ والبناء غير المناسب ، وتغيير وظائف المبنى ، والتغييرات في خصائص الحمل الزلزالي في المنطقة ، إلخ.

اعتمد الكود الاسرائيلي (مثل العديد من الاكواد الدولية) فلسفة التصميم التقليدية المقاومة للزلازل (القائمة على القوة) ؛ تم تصميم الهياكل للقوى التي هي أقل بكثير من قوى الزلزال التصميمية المتوقعة.عادة لا تأخذ هذه الأساليب في الاعتبار مستوى الأداء المتوقع ومستويات المخاطر الزلزالية للهيكل بعد وقوع الزلزال. لذلك ، تأتي الحاجة إلى طريقة جديدة من شأنها أن تعطي الأداء الفعلي للهيكل بعد حدث الزلزال.

تُستخدم إجراءات التحليل والتقييم القائمة على الأداء للتحقيق في عدم كفاية الزلازل للمباني القائمة. يمكن تحقيق التحسينات في الأداء الزلزالي لهذه المباني باستخدام أنظمة التعديل التحديثي مثل الدعامة الفولاذية وجدران القص والخرسانة والغطاء الفولاذي.

في هذه الدراسة ، تم تقديم دراسة مكثفة لمعظم استخدام مخططات التعديل التحديثي ، والتي تعمل على تحسين الأداء الزلزالي للمباني القائمة ، بما في ذلك ميزاتها.

تم استخدام إجراءات التحليل القائم على الأداء باستخدام التحليل الساكن غير الخطي (pushover) لدراسة أداء المباني الحالية الخرسانية المسلحة المقواة بطوابق G + 6 بدون / وبأنظمة التعديل التحديثي المختلفة. تمت مناقشة الاستجابة الزلزالية وأداء هذه الحالات.

أظهرت نتائج التقييم والتحليلات لحالات الدراسة ضرورة التحليل ثلاثي الأبعاد ، حيث تم اقتراح نظام الرفع الفولاذي لتجاوز المفصلات البلاستيكية المنهارة ولتقليل الانزياح والانحرافات ، ومفصلات الانهيار الجانبية للخرسانة والغطاء الفولاذي والحفاظ على ليونة المبنى.

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1 Chapter 1: Introduction

1.1 General Background

Earlier most of the old constructed buildings in Israel were designed only for vertical loads and forces acting in the direction of gravity. However, the direction of forces need not be acting in the direction of gravity always. This can be illustrated by the action of a seismic wave striking the structure where the force is directly acting on the structure or due to the development of inertia forces developed within the structure due to ground movements i.e., indirectly. In both cases the structure is subjected to forces in lateral directions, for which the behavior of the structures was not known. This has led to catastrophic failure of many structures causing great loss of life and setback to the economy of the area subjected to such hazards.

Seismic hazard in the context of engineering design is generally defined as the predicted level of ground acceleration which would be exceeded with 10% probability at the site under consideration due to the occurrence of an earthquake anywhere in the region, in the next 50 years. A lot of complex scientific perception and analytical modelling is involved in seismic hazard estimation. A computational scheme involves the following steps: delineation of seismic source zones and their characterization, selection of an appropriate ground motion attenuation relation and a predictive model of seismic hazard. Although these steps are region specific, certain standardization of the approaches is highly essential so that reasonably comparable estimates of seismic hazard can be made worldwide, which are consistent across the regional boundaries. The **National Geophysical Research Institute (NGRI), Hyderabad,** India was identified as one such center responsible for estimating the seismic hazard for the Indian region. As it is well known, earthquake catalogues and data bases make the first essential input for the delineation of seismic source zones and their characterization. Thus, preparation of a homogeneous catalogue for a region under consideration is an important task. The data from historic time to recent can broadly be divided in to three temporal categories: 1) since 1964, for which modern instrumentation-based data are available 2) 1900-1963, the era of early instrumental data, and 3) pre-1900, consisting of pre- instrumental data, which is based primarily on historical and macro-seismic information. In India, the scenario is somewhat similar. The next key component of seismic hazard assessment is the creation of seismic source models, which demand translating seismic-tectonic information into a spatial approximation of earthquake localization and temporal recurrence. For this purpose, all the available data on neo-tectonics, geodynamics, morpho structures etc., need to be compiled and viewed, overlain on a seismicity map. These maps then need to be critically studied for defining aerial seismic source zones and active faults. An earthquake recurrence model is then fitted to these source zones, for defining the parameters that characterize the seismicity of the source region, which go as inputs to the algorithm for the computation of seismic hazard.

The uncertainties involved in accurate determination of material properties, element and structure capacities, the limited prediction of ground motions that the structure is going to experience and the limitations in accurate modelling of structural behavior make the seismic performance evaluation of structures a complex and difficult process.

Many Reinforced Concrete RC buildings have often collapsed during earthquakes. The 2001 Bhuj earthquake in India killed 1002 students and teachers in failure of a public school, the 2005 Kashmir earthquake in Pakistan killed about 19,000 children, most of them in collapses, and the 2008 Sichuan, earthquake in China destroyed about 6,898 schools killing thousands of people (López et al., 2008). In Israel, earthquakes occur frequently where some cities were damaged during the past hundred years. Seismicity in Israel is mainly affected by the geodynamic processes acting along the Dead Sea Transform (See figure 1). The Dead Sea Transform is a left-lateral fault between the Arabia and the Sinai tectonic plates (Al-Dabbeek and El-Kelani, 2008; Freund et al., 1968; Ginzburg et al., 1981).

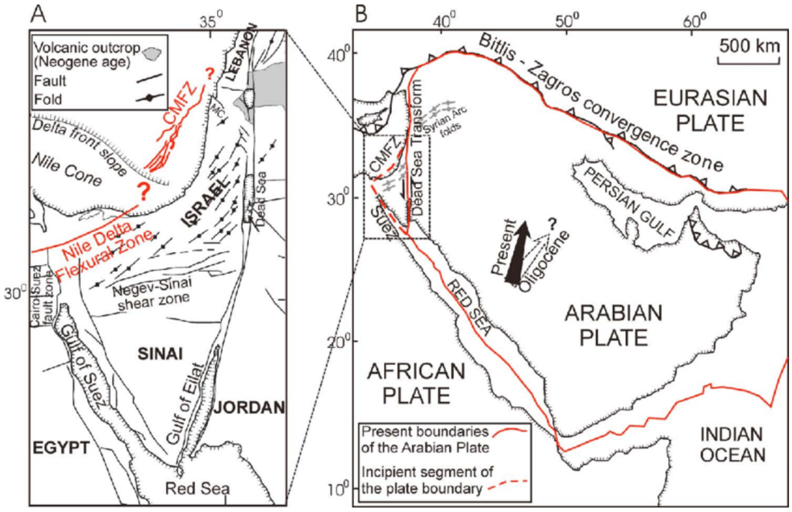


Figure 1 - Main Tectonic Elements in Israel-Sinai Vicinity

Seismic vulnerability is a measure of how susceptible a building is to damage for a given severity of the ground shaking (Borzi et al., 2008). The building vulnerability happens when there is older building design codes have been considered, poor design practices and poor code enforcement (Jalayer et al., 2010; Tesfamariam and Liu, 2010). A number of procedures have been outlined in the previous literature. The easiest and less time-consuming way, called walk-down survey, requires only superficial data collected from a brief inspection of the building. The number of stories, vertical and plan irregularities, location of the building, age of the building, its structural system and apparent material and workmanship quality are typical parameters that are used. The purpose of rapid evaluation is to identify or rank highly vulnerable buildings that deserve further investigation. The screening method is performed without any structural analysis and takes little time to complete. Ignoring the existing buildings in term of earthquake resistance can cause the following problems: 1- High risk for citizens in event of earthquakes. 2- The risk of closure of major roads or important facilities, which hinders relief efforts. 3- Expensive damage to private and public properties. 4- Legal dilemma: difficulty in specifying responsibility regarding the collapse of buildings that were not designed to resist earthquakes.

One reason behind ignoring the existing buildings is the lack of systematic procedures for evaluating such buildings and for identifying the weaknesses and risks in these buildings, which makes it difficult to adopt retrofitting policies that would improve seismic resistance of such buildings. In this master thesis, an office reinforced concrete RC building will be chosen, the seismic assessment of seismic response and performance of the RC building will be conducted using nonlinear analysis (Pushover Analysis) using ETABS software. After assessment, repair and strengthening techniques will be discussed. The results will be compared give some important remarks and conclusions regarding this building.

1.2 Concept of retrofitting reinforced concrete (RC) buildings

There are variety of structural systems used in buildings, such as, framed systems, shear-wall systems, masonry wall systems, and dual systems. This causes the retrofitting methods to vary from one system to another. There are three retrofitting domains stiffness, strength, and ductility. The increase in stiffness means increase in lateral resistance to sway drifts in buildings. More strength means the structure can bear and sustain larger loads. More ductility means that the structure can undergo more plastic deformations before failure occurs, when it is compared to other structures.

1.3 Seismic Analysis Approaches

Performance based seismic design (PBD) is a new approach to earthquake resistant design. It is more realistic than force-based design methods that are based on prescriptive and mostly empirical code formulations. (PBD) is a recent method to design buildings based on predictable and target seismic performance. Therefore, performance objectives such as immediate occupancy (IO), life-safety (LS), or collapse prevention (CP) are used to define the state of the building when exposed to earthquake loads. In one sense, performance based seismic design is a limit-state design extended to cover the complex range of performance requirements faced by earthquake engineers. There has been much researches on PBD, and many researches tried to come up with the most realistic and accurate procedures for PBD (Chopra, 2007). For details show figure 2.

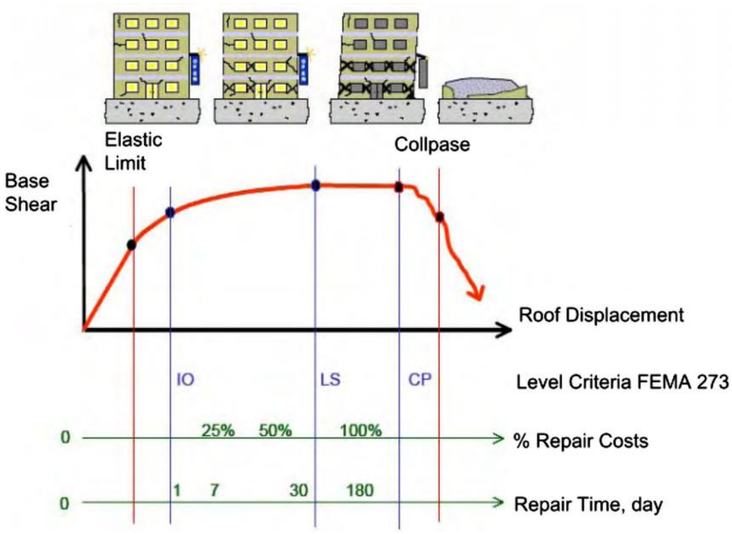


Figure 2 - PBD diagram

One common procedure is the capacity spectrum method (CSM) through pushover analysis. In this study, this method of PBD will be presented and demonstrated through a case study building to provide a tool for local engineers to assess structures against seismic behavior. Current seismic design codes in the world are generally carried out by linear static procedures (LSPs), such as equivalent lateral force (ELF) and response spectrum methods (RSA). However, the designed structures can be exposed to large inelastic deformations in strong earthquake events, which is inaccurately accounted for in the current force-based design methods (figure 3). The current seismic design codes as well as the seismic analysis and design approaches will be studied later in Chapter 2.

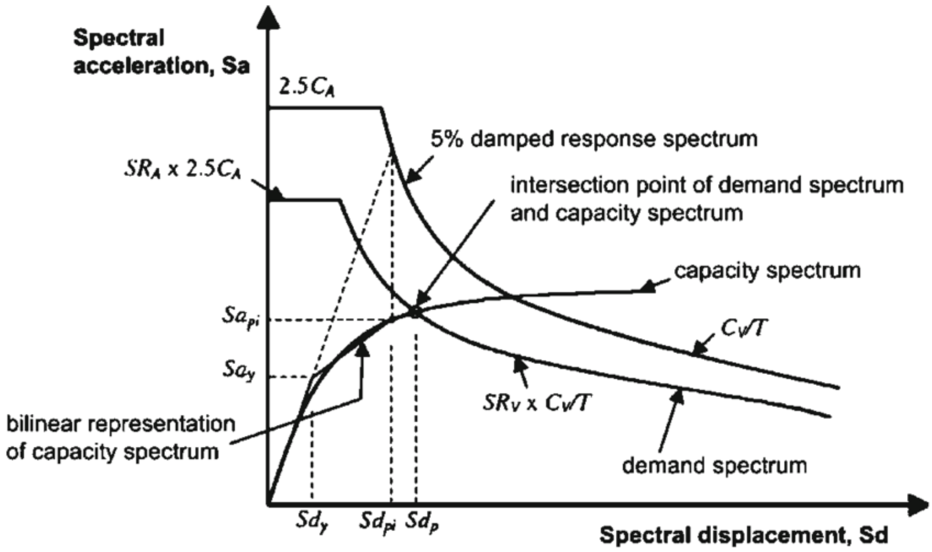


Figure 3 - Performance Point According to CSM

The most realistic design method must account for the development of plastic deformations in the structure during an earthquake event. In addition, hysteretic behavior of the structure during earthquake event must be considered, in order to predict the capacity of the structure to resist earthquake loads and not to exceed the designed limit level. The nonlinear time-history analysis method meets the previous consideration. However, it requires high accuracy in the selection of characteristics and assumptions to reach the correct results, and requires very powerful tools for the calculation-intensive nonlinear analysis. In the last two decades, the need for simple evaluation tools for existing buildings led to new methods related to performance-based approach. These include the nonlinear static analysis (pushover analysis).

The main idea in this procedure depends on estimating the capacity curve (pushover curve) and the demand response spectrum curve. The pushover curve represents the behavior of the structure during the elastic and plastic range until collapse, while the demand curve represents the magnitude of predicted earthquake force. The point of intersection between these two curves is called the performance point the pushover curve (or capacity curve) can be generated by subjecting the structure to one lateral load pattern or more depending on the natural fundamental modal shapes. Then increasing the magnitude of these loads monotonically to generate a nonlinear inelastic force-deformation relationship curve. The load vector is usually chosen to be representative of the load acting on the structure while vibrating in its first mode as a fundamental mode to be compatible with the seismic response of the building. The seismic demand curve (response spectrum curve) is a representation of the earthquake-induced response to the building, and it is presented in terms of peak acceleration-time relationship. Capacity curve (generated earlier by pushover analysis) must be converted from MDOF into an equivalent SDOF in a format representing peak acceleration and peak displacement. The resulting curve is called capacity spectrum curve. Then response spectrum is also converted into acceleration-displacement response spectra format (ADRS). Both curves are plotted as spectral acceleration with spectral displacement. The response spectrum curve must be reduced such that it accounts for reduction in stiffness and absorbed energy during earthquakes event. The performance point is determined as the intersection of the capacity spectrum and the reduced seismic demand curve.

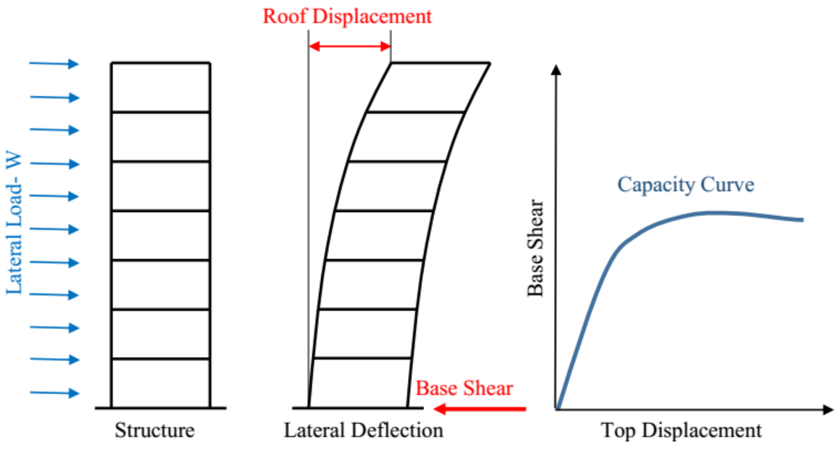


Figure 4 - Pushover Analysis based on FEMA 356.

1.4 Research Scope and Objectives

The main objective of this master thesis research project is to present a methodology for evaluating performance of existing buildings under seismic loads. Then improve the performance by means of retrofitting techniques and study the effect of these techniques on the performance of the retrofitted building. Non-linear static procedure (Pushover analysis method using ETABS program) will be used in evaluating the existing RC building before and after retrofitting until a specific performance target is achieved.

The general objectives in this study are the following:

A- Conduct a methodology for the seismic assessment of existing old RC buildings.

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

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

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

1- Selection of a representative existing building

2- ETABS software presentation and advantages respect to other soft wares, ETABS will be used for doing the nonlinear pushover analysis

3- 3-D model that simulates the existing building using the program in order to understand its behavior.

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

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

6- Identifying acceptable performance target for the selected RC building using relevant codes and standards and logical judgment.

7- Proposing retrofitting techniques and repeating pushover analysis for the retrofitted building until the performance target is achieved.

8- Comparison between different retrofitting techniques and their effect on capacity curves will be done based on their results and performance.

2 Chapter 2: Literature Review

2.1 General Background

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

Israel is a seismic zone that it is located along the Dead Sea Transform, which is an extension of ground faults separating the Arabian and African plates (Figure 5). The seismic history of the region indicates the occurrence of destructive earthquakes. The last devastating earthquake that hit the area was in 1927, which claimed the lives of dozens of residents under the rubble of their homes. (SASPARM Project, 2014).

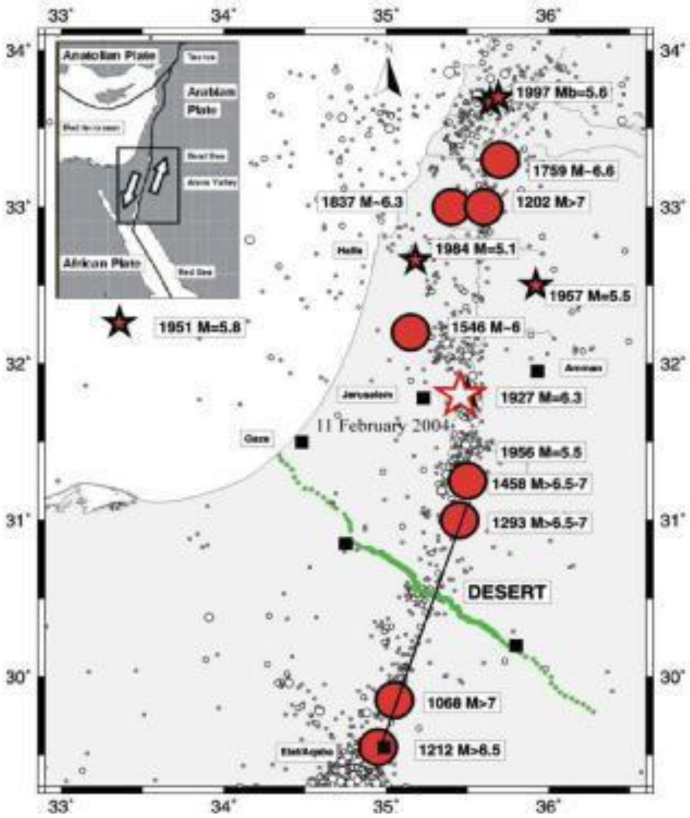


Figure 5 - Seismicity map of the Dead Sea transform region (circles represent seismic events). (SASPARM Project, 2014)

2.2 Force Based Methods

Classical seismic design codes in the world are generally based on elastic analysis methods, where earthquake is presented as static Loads. This does not agree with the real state of the earthquake, where the structures can be exposed to large inelastic deformations in strong earthquake events, and this is not well accounted for in current force-based design methods

(Chopra, 2012) presented the static procedures used by the available seismic design codes of concrete structures. The design lateral forces acting on any structure depend on vibration properties of the structure and the site classification. Based on the estimated fundamental modal behavior of the structure, formulas are specified for calculating base shear, and then lateral forces are distributed over the height of the building accordingly. Static analysis of the building for these forces provides the design forces, including shears and overturning moments for the different stories and structural elements. In these methods, the inelastic behavior of the building is incorporated as a reduction factor "R" of the base shear force.

(Wen-Cheng Liao, 2010) presented a flowchart described the Force-based design process sequence as shown in figure 6, while figure 7 shows the process of calculating the base shar of the structure, the seismic base shear force is generally reduced by a factor (R/I), where (R) represents the force reduction factor depending upon inherent ductility of the structural system, and (I) represents occupancy factor in order to increase the design base shear force for more important buildings according to the category of the building.

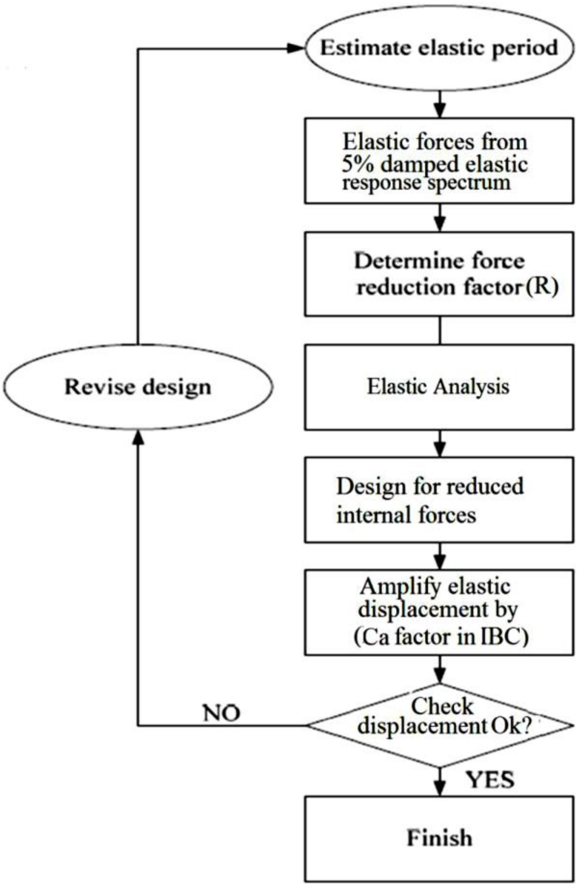


Figure 6 - Force-based design process sequence (Wen-Cheng Liao, 2010)

A design response spectrum is a graph that shows the maximum expected acceleration response of a structure to a seismic event, based on the site-specific ground motion characteristics. It is used in seismic design to ensure that the structure can withstand the expected seismic forces. To design a response spectrum, the following steps are typically taken:

1. Determine the seismic hazard level of the site based on the location and geological conditions.

2. Obtain seismic hazard maps and ground motion data for the site.

3. Develop a seismic hazard assessment

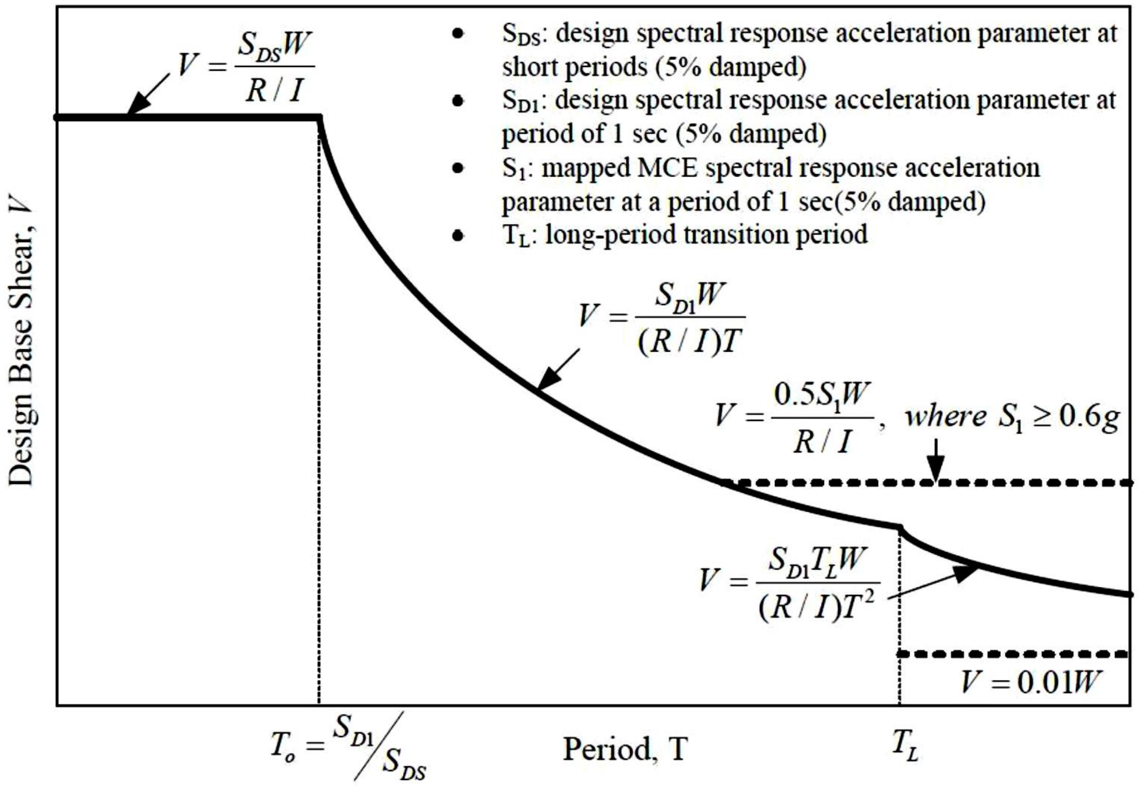


Figure 7 - 5% design response spectrum for seismic design (ASCE 7-10, 2010)

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

$\boldsymbol{F_x = C_{vx}V}$ **&**

$\boldsymbol{C_{vx}} = \frac{w_x h_x^k}{\sum_{i=0}^{n} w_i h_i^k}$

Where,

$\mathbf{F_x}$ = shear force at floor x

$\mathbf{C_{vx}}$ = vertical distribution factor

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

$\mathbf{w_i}$ **&** $\mathbf{w_x}$ = 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 (m) from the base to Level i or x

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

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

For structures having a period between 0.5 and 2.5 s, k shall be 2 or shall be determined by linear interpolation between 1 and 2. Elastic analysis is performed to determine the required member strengths. After members design for strength, a deflection amplification factor, Cd according to ASCE 7, is then used to multiply the calculated drift obtained from elastic analysis to check the specified drift limits. The process is repeated in an iterative manner until the strength and drift requirements are satisfied.

Response spectrum depends on computing the statistical peak response of a structure when subjected to a base excitation as shown in Figure 8. Each of the vibration modes are assumed to respond independently as a SDOF system. Design codes specify response spectra which determine the base acceleration applied to each mode according to its period.

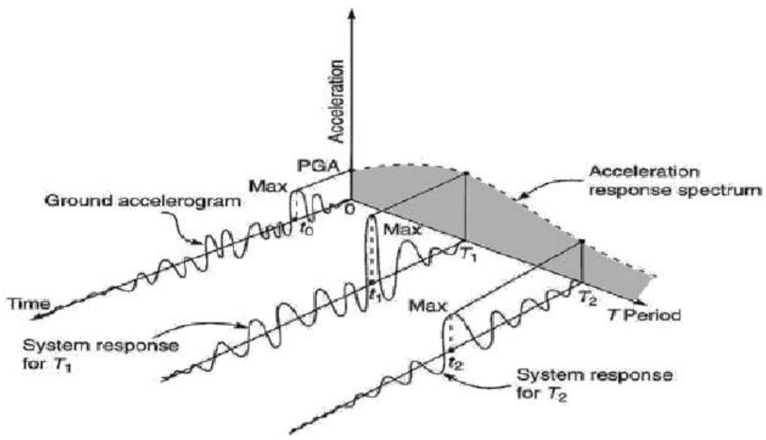


Figure 8 - Statistical maximum response of a SDOF structure subjected to a base excitation

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

2.3 Inelastic Methods

Structures endure critical inelastic distortion under a strong earthquake and dynamic qualities of the structure change with time, so examining the execution of a structure requires inelastic scientific strategies representing these dynamics.

Inelastic analytical methods comprehend the real conduct of structures by recognizing ETABS modes and the potential for dynamic breakdown. Inelastic analysis methods fundamentally incorporate inelastic time history analysis and inelastic static analysis which is otherwise called pushover analysis.

The inelastic time history analysis is the most exact technique to anticipate the force and deformation requests at different components of the structure. In any case, the utilization of inelastic time history analysis is constrained in light of the fact that dynamic reaction is exceptionally delicate to displaying and ground movement qualities. It requires appropriate demonstrating of cyclic burden disfigurement qualities considering weakening properties of exceedingly vital components. Additionally, it requires accessibility of an arrangement of delegate ground movement records that records for instabilities and contrasts in seriousness, frequency and length of time attributes. Additionally, calculation time, time required for info arrangement and interpreting voluminous output make the utilization of inelastic time history analysis impractical seismic execution assessment. Inelastic static analysis, or pushover analysis, has been the favored strategy for seismic execution assessment because of its effortlessness. Nonlinear static analysis, or pushover analysis, has been produced in the course of recent years and has turned into the favored analysis method for configuration and seismic execution assessment purposes as the methodology is generally straightforward and considers post versatile conduct. In any case, the method includes certain approximations and improvements that some measure of variety is constantly anticipated that would exist in seismic interest forecast of pushover analysis.

In spite of the fact that, in writing, pushover analysis has been appeared to catch crucial auxiliary reaction attributes under seismic activity, the exactness and the unwavering quality of weakling analysis in foreseeing worldwide and neighborhood seismic requests for the sum total of what structures have been a subject of talk and enhanced weakling systems have been proposed to conquer the specific restrictions of conventional pushover strategies.

In any case, the enhanced methodology is for the most part computationally requesting and theoretically complex that utilization of such systems is unrealistic in engineering profession and codes. As conventional pushover analysis is generally utilized for configuration and seismic execution assessment purposes, its constraints, shortcomings and the exactness of its expectations in routine application ought to be recognized by considering the components influencing the pushover forecasts. As it were, the materialness of pushover analysis in anticipating seismic requests ought to be explored for low, mid and skyscraper structures by distinguishing certain issues, for example, demonstrating nonlinear part conduct, computational plan of the method, varieties in the forecasts of different horizontal burden designs used in customary pushover analysis, proficiency of invariant parallel burden designs in speaking to higher mode impacts and precise estimation of target uprooting at which seismic interest expectation of pushover technique is performed.

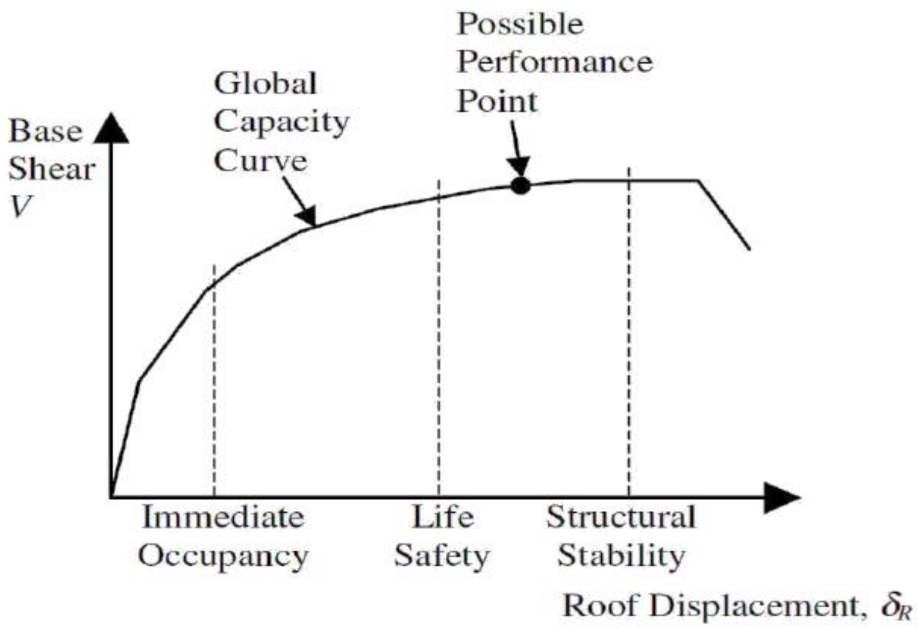


Figure 9 - Building Capacity Curve

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

2.4 Building performance levels and ranges

As mentioned above, pushover analysis is one of the methods to carry out performance-based design. Hence, it becomes imperative to understand the different performance levels and their ranges to decide the level that one wants to achieve before doing the pushover analysis. Accordingly, the terminology used in performance-based design is given below:

I. Performance Level: The intended post-earthquake condition of a building; a well-defined point on a scale measuring how much loss is caused by earthquake damage. In addition to casualties, loss may be in terms of property and operational capability.

II. Performance Range: a range or band of performance, rather than a discrete level.

III. Designations of Performance Levels and Ranges: The performance levels are grouped under two heads. One group is named as the structural performance level and ranges and the second group is named as the non-structural performance levels. Structural performance levels are identified by both a name and numerical designator i.e., S-1

IV. through S-5 while the non-structural performance levels are identified by a name and alphabetical designator i.e., N-A through N-D. They are mentioned below:

V. Structural Performance Levels & Ranges:

- S-1: Immediate Occupancy Performance Level: It relates to 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, these would generally not be required prior to re-occupancy.
- S-2: Damage Control Performance Range: It corresponds to the continuous range of damage states that entail less damage than that defined for the Life Safety level, but more than that defined for the Immediate Occupancy level. Design for Damage Control performance may be desirable to minimize repair time and operation interruption; as a partial means of protecting valuable equipment and contents; or to preserve important historic features when the cost of design for Immediate Occupancy is excessive. Acceptance criteria for this range may be obtained by interpolating between the values provided for the Immediate Occupancy (S-1) and Life Safety (S-3) levels.
- S-3: Life Safety Performance Level: It relates to 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, it is expected that the overall risk of life-threatening injury as a result of structural damage is low. It should be possible to repair the structure; however, for economic reasons this may not be practical.
- S-4: Limited Safety Performance: It corresponds to the continuous range of damage states between the Life Safety and Collapse Prevention levels. Design parameters for this range may be obtained by interpolating between the values provided for the Life Safety (S-3) and Collapse Prevention (S-5) levels.
- S-5: Collapse Prevention Performance Level: It relates to the post-earthquake damage state where the building is on the verge of experiencing 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 to more limited extent degradation in vertical-load-carrying capacity. However, all significant components of the gravity load resisting system must continue to carry their gravity load demands. Significant risk of injury due to falling hazards from structural debris may exist. The structure may not be technically practical to repair and is not safe for re-occupancy, as aftershock activity could induce collapse.
- S-6: Structural Performance not considered (covering the situation where only non-structural improvements are made)

VI. Non-structural Performance Levels:

- N-A: Operational Performance Level: It corresponds to the post-earthquake damage state of the building in which the non-structural components are able to support the building’s intended function. At this level, most non-structural systems required for normal use of the building including lighting, plumbing, etc.; are functional, although minor repair of some items may be required. This performance level requires considerations beyond those that are normally within the sole province of the structural engineer.

- N-B: Immediate Occupancy Performance Level: It corresponds to the post-earthquake damage state in which only limited non-structural damage has occurred. Basic access and life safety systems, including doors, stairways, elevators, emergency lighting, fire alarms, and suppression systems, remain operable. There could be minor window breakage and slight damage to some components. Presuming that the building is structurally safe, it is expected that occupants could safely remain in the building, although normal use may be impaired and some cleanup may be required. In general, components of mechanical and electrical systems in the building are structurally secured and should be able to function, if necessary, utility service is available. However, some components may experience misalignments or internal damage and be non- operable. Power, water, natural gas, communications lines, and other utilities required for normal building use may not be available. The risk of life-threatening injury due to non- structural damage is very low.
- N-C: Life Safety Performance Level: It corresponds to the post-earthquake damage state in which potentially significant and costly damage has occurred to non-structural components but they have not become dislodged and fallen, threatening life safety either within or outside the building. Egress routes within the building are not extensively blocked. While injuries may occur during the earthquake from the failure of non-structural components, it is expected that, overall, the risk of life-threatening injury is very low. Restoration of the non-structural components may take extensive effort.
- N-D: Hazards Reduced Performance Level: It corresponds to the post-earthquake damage state level in which extensive damage has occurred to non-structural components, but large or heavy items that pose a falling hazard to a number of people such as parapets, cladding panels, heavy plaster ceilings, or storage racks are prevented from falling. While isolated serious injury could occur from falling debris, failures that could injure large numbers of persons either inside or outside the structure should be avoided. Exits, fire suppression systems, and similar life-safety issues are not addressed in this performance level.
- N-E: Non-structural Performance Not Considered (covering the situation where only structural improvements are made.

VII. Building Performance Level:

Target Building performance is a combination of the performance of both structural and non-structural components. Three Structural Performance Levels and four Non-structural Performance Levels are used to form the four basic Building Performance Levels. They are: Collapse Prevention, Life Safety, Immediate Occupancy, and operational. Figure 10 shown below, shows these levels as discrete points on a continuous scale describing the building's expected performance, or alternatively, how much damage, economic loss, and disruption may occur.

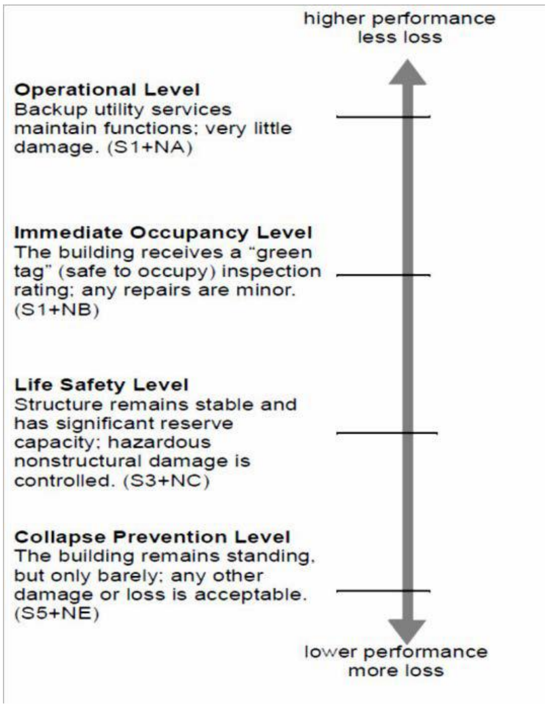


Figure 10 - Building Performance Levels (ATC, 1997a)

2.5 Structural Analysis Procedures

For seismic performance evaluation, a structural analysis of the mathematical model of the structure is required to determine force and displacement demands in various components of the structure. Several analysis methods, both linear and non-linear, are available to predict the seismic performance of the structures. FEMA 356 divided structural analysis procedures into four procedures as follows:

1. Linear Static Procedures (LSP),
2. Linear Dynamic Procedures (LDP),
3. Non-Linear Static Procedures (NLSP),
4. Non-Linear Dynamic Procedures (NLDP).

The first two methods fall under the linear or elastic analysis procedures while the latter two are non-linear analysis procedures.

In the linear procedures, the force demand on each component is obtained and compared with the available capacities by performing a linear analysis. Linear 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 on various components are determined by an elastic dynamic analysis. The dynamic analysis may be either a response spectrum analysis or an elastic time history analysis. Sufficient number of modes must be considered to have a mass participation of at least 90% for response spectrum analysis.

Any effects of higher modes are automatically included in 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 a linear analysis.

However, non-linear 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 inelastic forces to elastic. Force reduction factor, "R", is assigned considering only the type of lateral system in most codes, but it is known that this factor is a function of the period and ductility ratio of the structure as well.

Linear methods can predict elastic capacity of structure; 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. Moreover, force-based methods primarily provide life safety but they can't provide damage limitation and easy repair. The drawbacks of force-based procedures and the dependence of damage on deformation have led the researches to develop displacement-based procedures for seismic performance evaluation.

Displacement-based procedures are mainly based on inelastic deformations rather than elastic forces and use nonlinear analysis procedures considering seismic demands and available capacities explicitly. The dynamic characteristics of the structure change with time so investigating the performance of a structure requires non-linear analytical procedures accounting for these features. Non-linear analytical procedures help to understand the actual behavior of structures by identifying failure modes and the potential for progressive collapse. Non-linear analysis procedures basically include non-linear time history analysis and non-linear static analysis which is also known as pushover analysis.

The non-linear time history analysis is the most accurate method to predict the force and deformation demands of various components of the structure. However, the use of non-linear time history analysis is limited because dynamic response is very sensitive to modelling and ground motion characteristics. It requires proper modelling 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 non-linear time history analysis impractical for seismic performance evaluation.

Non-linear static analysis, or pushover 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. Inelastic static analysis procedures include Capacity Spectrum Method, Displacement Coefficient Method and the Secant Method. The theoretical background, reliability and the accuracy of non-linear static analysis procedure is discussed in detail in the following paragraphs.

Nonlinear static analysis, or pushover analysis, has been developed over the past twenty years and has become the preferred analysis procedure for design and seismic performance evaluation purposes as the procedure is relatively simple and considers post-elastic behavior. However, the procedure involves certain approximations and simplifications that some amount of variation is always expected to exist in seismic demand prediction of pushover analysis.

Although, in literature, pushover analysis has been shown to capture essential structural response characteristics under seismic action, the accuracy and the reliability of pushover analysis in predicting global and local seismic demands for all structures have been a subject of discussion and improved pushover procedures have been proposed to overcome the certain limitations of traditional pushover procedures. However, the improved procedures are mostly computationally demanding and conceptually complex that use of such procedures is impractical in engineering profession and codes.

As traditional pushover analysis is widely used for design and seismic performance evaluation purposes, its limitations, weaknesses and the accuracy of its predictions in routine application should be identified by studying the factors affecting the pushover predictions. In other words, the applicability of pushover analysis in predicting seismic demands should be investigated for low, mid and high-rise structures by identifying certain issues such as modelling nonlinear member behavior, computational scheme of the procedure, variations in the predictions of various lateral load patterns utilized in traditional pushover analysis, efficiency of invariant lateral load patterns in representing higher mode effects and accurate estimation of target displacement at which seismic demand prediction of pushover procedure is performed.

The aim of the present work is to study the behavior of an existing G+2 storied regular RCC frame structure in ETABS Ver 21 to study the development of hinges in respective members and finally to locate its performance point. To achieve the above objective, a model was created in ETABS Ver 21 for simulating the actual ground conditions. After generating the model, modal analysis was performed to obtain the data for the different mode shapes and their patterns. The fundamental mode or first mode shape details i.e., it's time period and total base shear contribution was obtained. Thereafter the hinges were generated for each member and introduced in the model and Pushover Load Case in pattern similar to the first mode shape was applied. Finally, the analysis results were checked to see the pattern of formation of hinges and their respective locations. The Acceleration versus Displacement graph was plotted to get the performance point of the structure.

2.6 FEMA 356 (ASCE 2000)

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

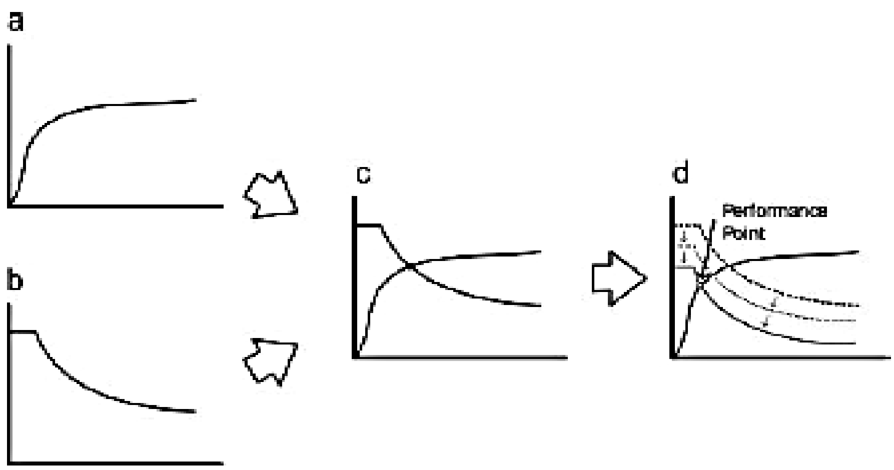


Figure 11 - Capacity Spectrum Method

a. Pushover curve

b. Demand spectrum

c. ARDS format

d. Final result

Capacity Spectrum Building performance level can be determined by target displacement using capacity spectrum method (ATC 40). The capacity spectrum method allows for a graphical comparison between the structure capacity and the seismic demand. Pushover curve represents the lateral resisting capacity and response spectrum curve represents the seismic demand. Pushover Analysis requires the development of the force-deformation curve for the critical section of beams and column by using the guideline. Such a curve is presented below.

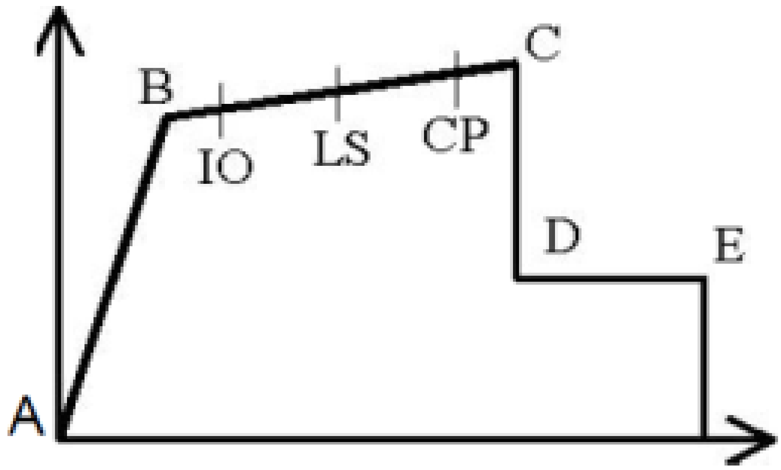


Figure 12 - Force deflection curve using Pushover

2.7 Pushover Analysis

2.7.1 Background

The pushover analysis (also named nonlinear static analysis) was introduced back in 1970's and for the last 35-40 years it has been noticed as a powerful engineering tool. Pushover analysis is able to consider the inelastic response characteristics and therefore provide information of performance of a structure in a seismic event, which the linear approach is not capable of. The main purpose of the pushover analysis is to compare the strength and deformation capacity with the demands at the corresponding performance level, by using a static nonlinear analysis algorithm. The analysis considers geometrical non-linearity and material inelasticity, as well as the internal force redistribution.

It is carried out under constant gravity loads and monotonically increased lateral forces, applied at the location of the masses in the structural model, to simulate the inertia forces. The method is able to describe the evaluation of plastic mechanism and structural damage as a function of the lateral forces since they are increased monotonically. The pushover analysis may be described as an extension of the lateral force method of linear analysis in to the nonlinear regime.

However, the method is based on many assumptions and may in some cases provide misleading results, as explained in the end of this section. Pushover analysis may be provided if there is a doubt that simple analysis provides insufficient information on the structural seismic resistance. The pushover analysis provides more relevant information and response characteristics that cannot be obtained from a Response Spectrum Analysis (RSA). Pushover analysis is also feasible for seismic analysis of existing structures and design of retrofit schemes.

Three-dimensional analytical model of a structure would be the most preferable one, but earlier only a few adequate analytical tools were available for that purpose. However, the capability of computers is growing fast and for the last few years, sophisticated finite element computer programs from Computer Structure Inc. like instance SAP2000 and ETABS Ver 21 have introduced pushover analysis of steel and concrete frame structures. In SAP2000 and ETABS, the nonlinear properties of the elements are implemented in the form of yield hinges, chosen and defined by the structural designer. Other finite element programs, for instance ANSYS and Cosmos/M, can perform pushover analysis where the nonlinear material properties are considered. However, a three-dimensional model of a typical structure would be cumbersome and with few exceptions too time consuming for a typical design process in the consulting engineering field. The basic assumption is that the response of a MDOF (multi-degree-of-freedom) structural system can be related to the response of an equivalent SDOF system. This implies that the response is controlled by a single mode and that the shape of the mode is constant throughout. It is clear that both these assumptions are not correct. However, several pilot studies have indicated that these assumptions result are in fairly good prediction of the maximum seismic response of MDOF structures as long as the response is dominated by a single mode. Several studies have shown that results of experimental tests and nonlinear dynamic analysis are similar to those obtained from the pushover analysis.

2.7.2 General

Static Nonlinear Analysis technique, also known as sequential yield analysis, or simply "pushover" analysis has gained significant popularity during the past few years. It is the one of the three analysis techniques recommended by FEMA-273/274 and a main component of the Spectrum Capacity Analysis method (ATC-40). Proper application can provide valuable insights into the expected performance of structural systems and components. Misuse can lead to an erroneous understanding of the performance characteristics. Unfortunately, many engineers are unaware of the details that have to observed in order to obtain useful results from such analysis.

In this procedure, a computer model of the structure is subjected to a predefined pattern of monotonically increasing lateral forces, to examine the non-linear behavior of structure, including the deformation and damage pattern. Hence, pushover analysis can provide significant insight into the weak links in seismic performance of a structure.

It consists of two parts. First, a target displacement for the structure is established. The target displacement is an estimate of the seismic top displacement of the building, when it is exposed to the design earthquake excitation.

Then the model is subjected to a predefined lateral force. 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.

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). Also, in force-controlled pushover procedure some numerical problems that affect the accuracy of results occur since target displacement may be associated with a very small positive or even a negative lateral stiffness because of the development of mechanisms and P-delta effects.

The pushover analysis is expected to provide information on many response characteristics that cannot be obtained from an elastic static or dynamic analysis. The following are the examples of such response characteristics:

a. The realistic force demands on potentially brittle elements, axial force demands on columns, force demands on brace connections, moment demands on beam to column connections, shear force demands in reinforced concrete beams, etc.

b. Estimates of the deformations demands for elements that have to form in-elastically in order to dissipate the energy imparted to the structure.

c. Consequences of the strength deterioration of individual elements on behavior of the structural system.
d. Identification of the critical regions in which the deformation demands are expected to be high and that have to become the focus through detailing.
e. Identification of the strength discontinuity in plan & elevation that will lead to changes in the dynamic characteristics in elastic range.
f. Estimates of the inter-story drifts that account for strength or stiffness discontinuities and that may be used to control the damages and to evaluate P-Delta effects.
g. Verification of the completeness and adequacy of load path, considering all the elements of the structural systems, all the connections, and stiff non-structural elements of significant strength, and the foundation system.

2.7.3 Limitations

Although pushover analysis has advantages over elastic analysis procedures, underlying assumptions, the accuracy of pushover predictions and limitations of current pushover procedures must be identified. The estimate of target displacement, selection of lateral load

patterns and identification of failure mechanisms due to higher modes of vibration are important issues that affect the accuracy of pushover results.

Target displacement is the global displacement expected in a design earthquake. The roof displacement at mass center of the structure is used as target displacement. The accurate estimation of target displacement associated with specific performance objective affect the accuracy of seismic demand predictions of pushover analysis.

However, in pushover analysis, generally an invariant lateral load pattern is used that the distribution of inertia forces is assumed to be constant during earthquake and the deformed configuration of structure under the action of invariant lateral load pattern is expected to be similar to that experienced in design earthquake. Thus, the capacity curve is very sensitive to the choice of lateral load distribution, selection of lateral load pattern is more critical than the accurate estimation of target displacement.

The lateral load patterns used in pushover analysis are proportional to product of story mass and displacement associated with a shape vector at the story under consideration. Commonly used lateral force patterns are uniform, elastic first mode, "code" distributions, a single concentrated horizontal force at the top of structure, triangular loading pattern, etc. These loading patterns usually favor certain deformation modes that are triggered by the load pattern and miss others that are initiated and propagated by the ground motion and inelastic dynamic response characteristics of the structure. Moreover, invariant lateral load patterns could not predict potential failure modes due to middle or upper story mechanisms caused by higher mode effects. Invariant load patterns can provide adequate predictions if the structural response is not severely affected by higher modes and the structure has only a single load yielding mechanism that can be captured by an invariant load pattern.

FEMA-273 recommends utilizing at least two fixed load patterns that form upper and lower bounds for inertia force distributions to predict likely variations on overall structural behavior and local demands. The first pattern should be uniform load distribution and the other should be "code" profile or multi-modal load pattern. The 'Code' lateral load pattern is allowed if more than 75% of the total mass participates in the fundamental load. The invariant load patterns cannot account for the redistribution of inertia forces due to progressive yielding and resulting changes in dynamic properties of the structure. Also, fixed load patterns have limited capability to predict higher mode effects in post-elastic range. These limitations have led many researchers to propose adaptive load patterns which consider the changes in inertia forces with the level of inelasticity. The underlying approach of this technique is to redistribute the lateral load shape with the extent of inelastic deformations. Although some improved predictions have been obtained from adaptive load patterns, they make pushover analysis computationally demanding and conceptually complicated. The scale of improvement has been a subject of discussion that simple invariant load patterns are widely preferred at the expense of accuracy. Whether lateral loading is invariant or adaptive, it is applied to the structure statically and a static loading cannot represent inelastic dynamic response with a large degree of accuracy.

Hence to summarize, the limitations are as follows:

a. The pushover analysis is static and cannot predict the dynamic behavior of the structure with large accuracy.

b. The pushover analysis could underestimate effects of modes that may occur in a structure subjected to severe seismic events and exaggerate others. This applies in case of higher modes i.e., in tall buildings. Hence, the pushover analysis becomes inaccurate if higher mode effects are important. However, pushover analysis procedure where effects of higher modes are considered has been available for the last few years.

c. The load pattern affects the results dramatically. Each load pattern is likely to favor certain deformation mode. Therefore, more than one load pattern should always be considered in the pushover analysis.

d. Incorporation of torsion effects due to mass, stiffness and strength irregularities could affect the results and also 3-D problems like orthogonality effects, direction of loading and semi rigid diaphragms.

The step-wise procedure to do pushover analysis through Capacity Spectrum Method as given in ATC-40 is given below:

1. Create a computer model of the structure following the modeling rules excluding the foundation.

2. Apply lateral story forces to the structure in any of the following manners:

- Simply apply a single concentrated horizontal force at the top of the structure.
- Apply lateral forces to each story in proportion to the standard code procedure without the concentrated force at the top.
- Apply lateral forces in proportion to the product of story masses and first mode shape of the elastic model of the structure.
- is generally constructed to represent the first mode response of the structure based on the assumption that the fundamental mode of vibration is the predominant response of the structure.
- Same as Level c until first yielding. For each increment beyond yielding, adjust the forces to be consistent with changing deflected shape.
- Similar to c and d above, but including the effects of higher modes of vibration in determining yielding in individual structural elements while plotting the capacity curve for the building in terms of first mode lateral forces and displacements. The higher mode effects may be determined by doing higher mode pushover analyses.

3. Calculate member forces for the required combinations of vertical and lateral load.

4. Adjust the lateral force level so that some elements are stressed to within 10% of its member strength.
5. Record the base shear versus the roof displacement.
6. Revise the model using zero stiffness for the yielding elements.
7. Apply a new increment of lateral load to the revised structure such that another element/s yields.
8. Add the increment of lateral load and the corresponding increment of roof displacement to the previous totals to give accumulated values of base shear and roof displacement.
9. Repeat steps 6, 7 and 8 until the structure reaches an ultimate limit, such as: instability from P-Δ effects; distortions considerably beyond the desired performance level; an element group reaching a lateral deformation level at which significant strength degradation begins.

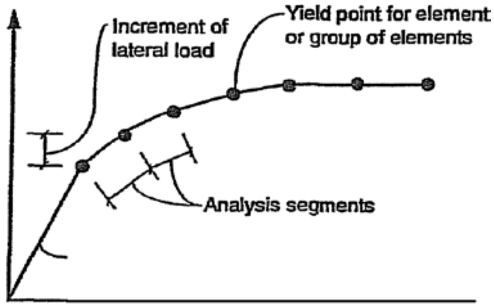


Figure 13 - Capacity Curve showing plot of Base shear vs Roof Displacement

10. Explicitly model global strength degradation. If incremental loading was stopped in step 9 as a result of reaching a lateral deformation level at which all or a significant portion of an element/s loads can no longer be resisted i.e., its strength has significantly degraded, then the stiffness of those element/s is reduced, or eliminated. A new capacity curve is created starting with step c of this step-by-step process.

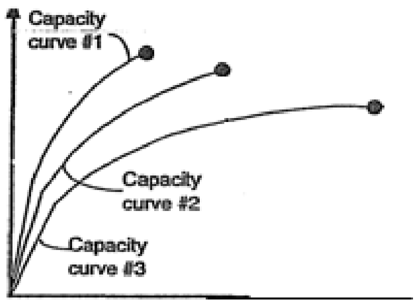


Figure 14 - Capacity Curves showing plot of Base shear vs Roof Displacement

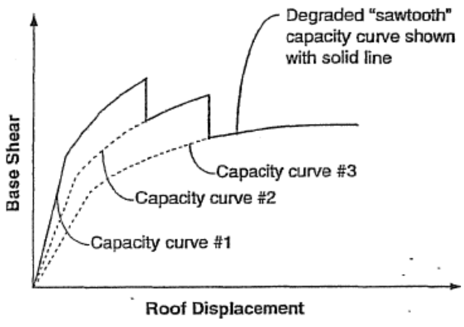


Figure 15 - Capacity curve with Global strength Degradation modelled

Once this capacity curve has been obtained, it is converted into capacity spectrum curve in Acceleration-Displacement response Spectra (ADRS) system by using the following formulae:

$$PF_1 = \left[\frac{\sum_{i=1}^{N} \frac{(w_i \phi_{i1})}{g}}{\sum_{i=1}^{N} \frac{(w_i \phi_{i1}^2)}{g}} \right] \quad \& \quad \alpha_1 = \frac{\left[\sum_{i=1}^{N} \frac{(w_i \phi_{i1})}{g} \right]^2}{\left[\sum_{i=1}^{N} \frac{w_i}{g} \right] \left[\sum_{i=1}^{N} \frac{(w_i \phi_{i1}^2)}{g} \right]}$$

$$S_a = \frac{(V/W)}{\alpha_1} \quad \& \quad S_d = \frac{\Delta_{roof}}{PF_1 \phi_{roof,1}}$$

Where:

PF1 =modal participation factor for the first natural mode.

α_1=modal mass coefficient for the first natural mode.

$\frac{w_i}{g}$ = mass assigned to level i

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

N= Level N, the level which is the uppermost in the main portion of the structure V=base shear

W=building dead weight plus likely live loads.

From IS1893 (Part I):2002, we already have the demand spectra plotted in form of (Sa/g) vs T. To make comparison, this demand spectra is also converted into ADRS system by using the following formulae:

$$S_d = \frac{1}{4\Pi^2} S_a T^2 \text{ \& the spectral acceleration} \left(\frac{S_a}{g} \right) g$$

Now that both the capacity spectrum and demand spectrum are in same ADRS system, the two graphs are plotted together. The point where the capacity spectrum meets with the demand spectrum is the Performance Point of the structure. The same can be understood from the following figure 16 shown below

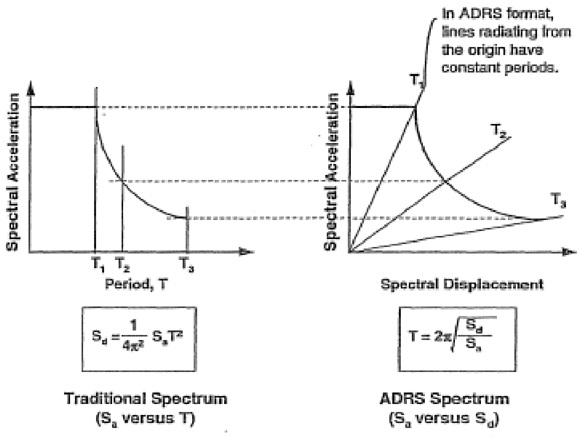


Figure 8-6. Response Spectra in Traditional and ADRS Formats

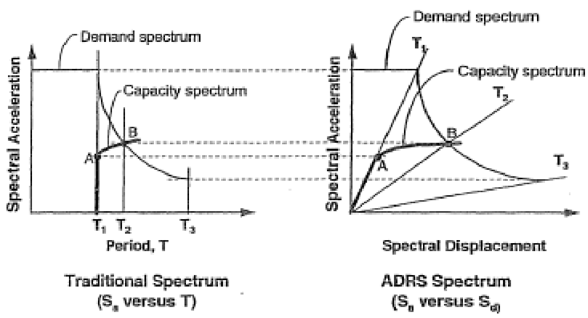


Figure 16 - Capacity Spectrum Superimposed over Response Spectra in ADRS Formats

2.7.4 Create the computational model

- Create the computational model, without pushover data, using conventional modeling techniques.

- Define properties for pushover hinges using Define > Section Properties > Hinge Properties. Hinges may be defined manually or by using one of several default specifications which are available.

- Assign the pushover hinges to selected frame objects using Assign > Frame > Hinges.

- Select Define > Load Patterns to define load patterns which will contain the loads applied during pushover analysis.

2.7.5 Define a nonlinear static load case

- Select Define > Load Cases > Add New Load Case to define a nonlinear static Load case which will apply the previously-defined load pattern. This load case may be force-controlled (pushed to a specified force level) or displacement-controlled (pushed to a specified displacement).

- Select Other Parameters > Results Saved to Multiple States such that various parameters may be plotted for each increment of applied loading.

2.7.6 Run the analysis

Select Analyze > Run Analysis to run the static-pushover analysis.

2.7.7 Review results

- To plot base shear vs. monitored displacement, select Display > Show Static Pushover Curve. Additional variables are also available for plotting.
- To plot hinge deformation vs. applied loading, select Display > Show Hinge Results. Moment as a function of plastic rotation is one such option.
- To review displacement and the step-by-step sequence of hinge formation, select Display > Show Deformed Shape.
- To review member forces on a step-by-step basis, select Display > Show Forces/Stresses > Frames/Cables.
- Select Display > Show Plot Functions to plot response at each step of the pushover analysis, including joint displacement, frame member forces, etc.

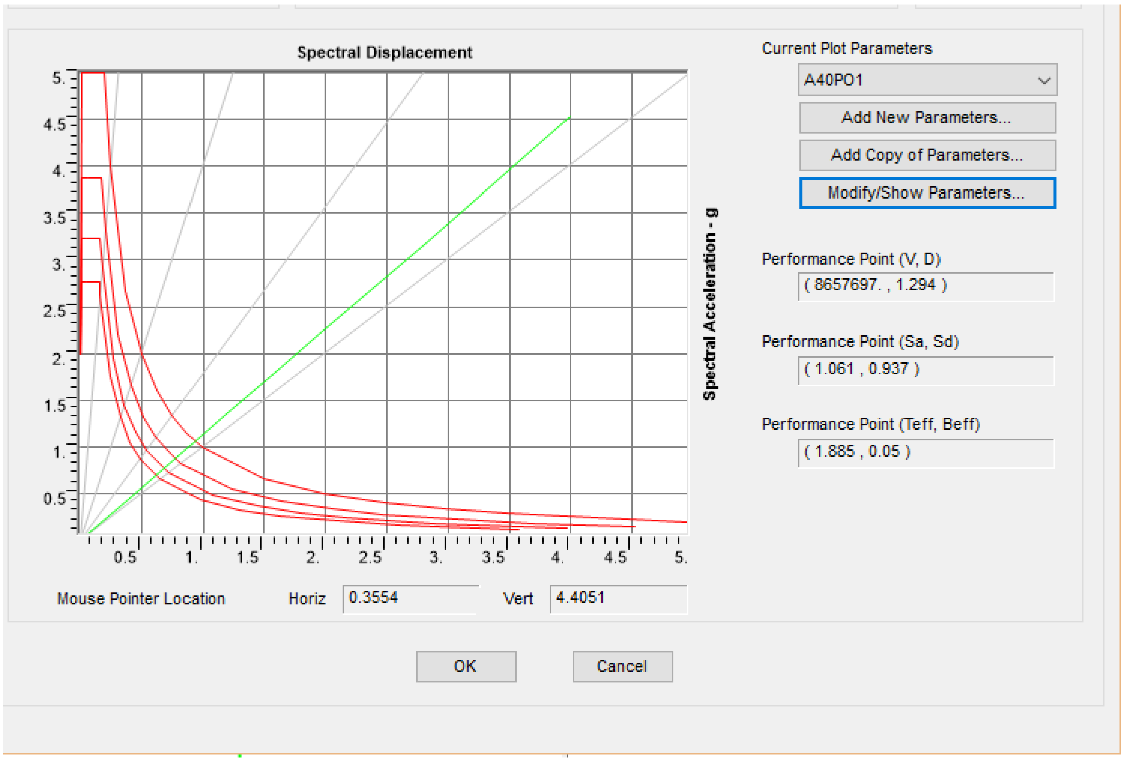


Figure 17 - sample of results

3 Chapter 3: WORK CARRIED OUT (A CASE STUDY)

3.1 Overview

The push over analysis of a structure is a static non-linear analysis under permanent vertical loads and gradually increasing lateral loads. The equivalent static lateral loads approximately represent earthquake induced forces. A plot of the total base shear versus top displacement in a structure is obtained by this analysis that would indicate any premature failure or weakness. This type of analysis enables weakness in the structure to be identified.

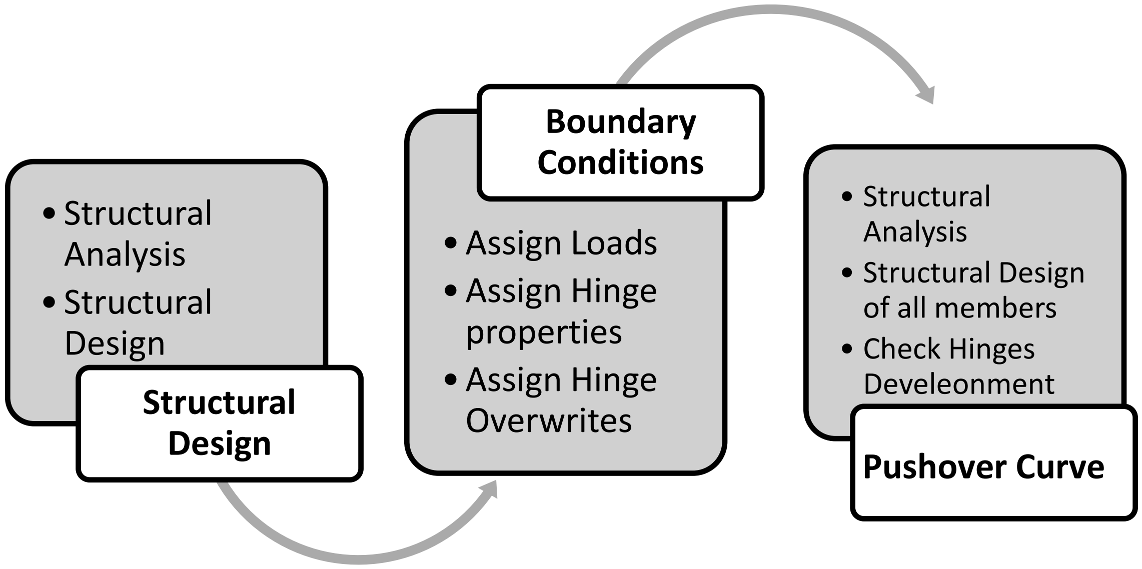


Figure 18 - Methodology Procedure

3.2 Building details

The structural details that were observed and measured on site are as mentioned below:

1. The test building is a G+6 story reinforced concrete building, located in the middle region of Israel.
2. It was constructed as a Reinforced Concrete (RC) structure.
3. It was symmetrical in plan with 7 bays of 5meters each along the x-axis, while along the y-axis 8 bays of 4.5meters.
4. The height of ground floor was 4meters and the others were 3meters, making the total height of the building as 22meters.
5. RC slab of 200mm thickness was provided.
6. The base of the foundations of the structure was located at a depth of 2.5meters below the ground level.
7. All beams including the plinth beams were 500mm500mm and all columns were 600mmX600mm in size. Only the reinforcement percentage was varied for the beams.

8. The building was constructed approximately 24 years ago.

9. As the structural drawings of the building were available, hence the same values were used in creating the model of the structure after physical verification of the same on site. Figure 19 presents the typical frame plan showing column and beam layout.

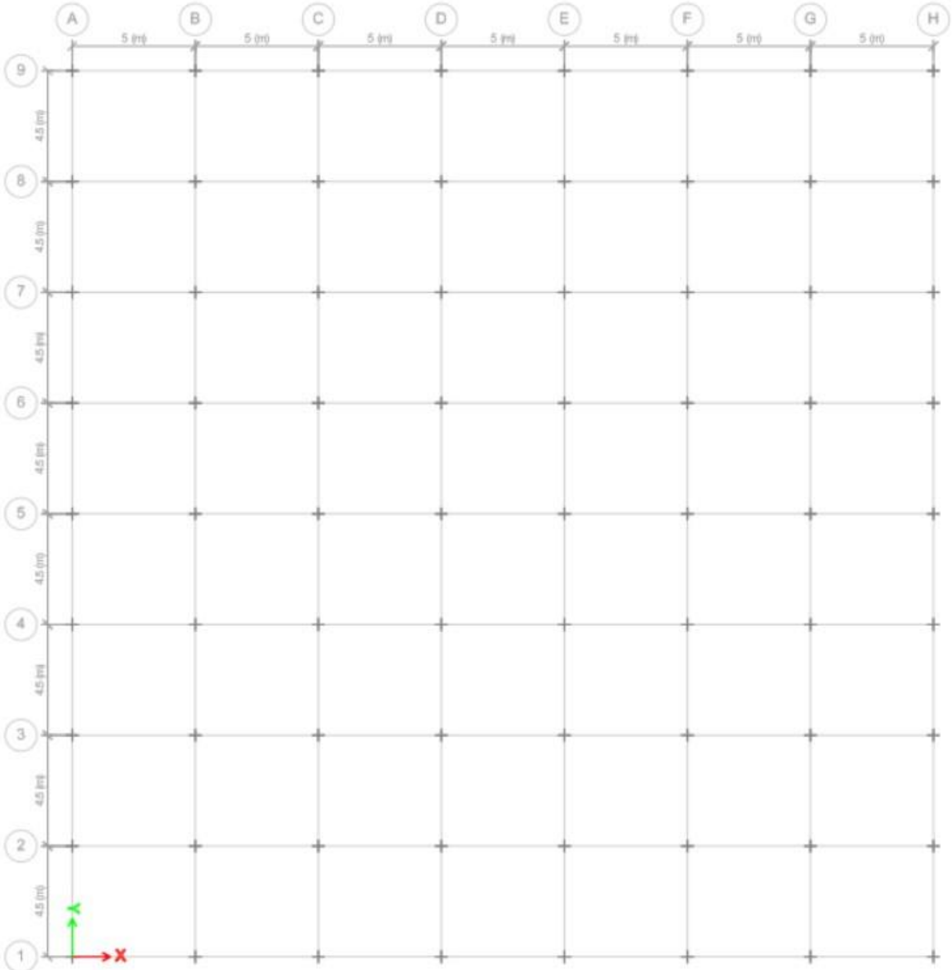


Figure 19 - typical floor column and beam layout.

3.3 LOADING

1. Floor Finish load of slab has been taken as 5 KN/m^2.

2. Live load has been taken as 4KN/m^2, no reductions in accordance with ASCE 7-10 has been considered.

3. The seismic loads are obtained as the following and checked up by using ETABs software:

✓ Based on Israeli Standard SI 413-5, $S_1 = 0.08$, $S_s = 0.30$and $T_L = 11sec$. The seismic maps and seismic coefficients for Palestine are shown in figure 20 & 21.

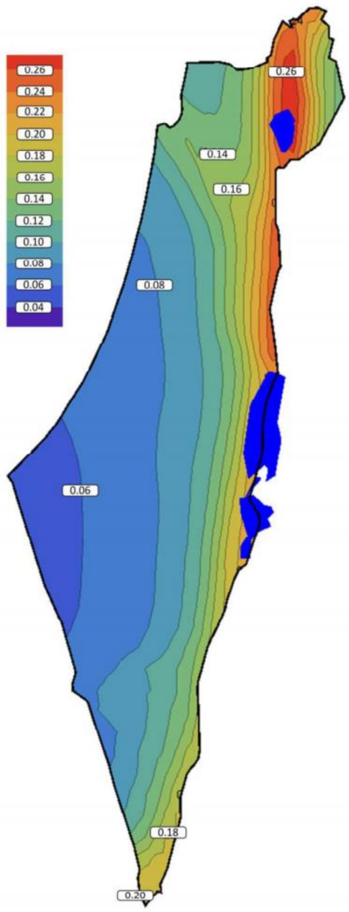
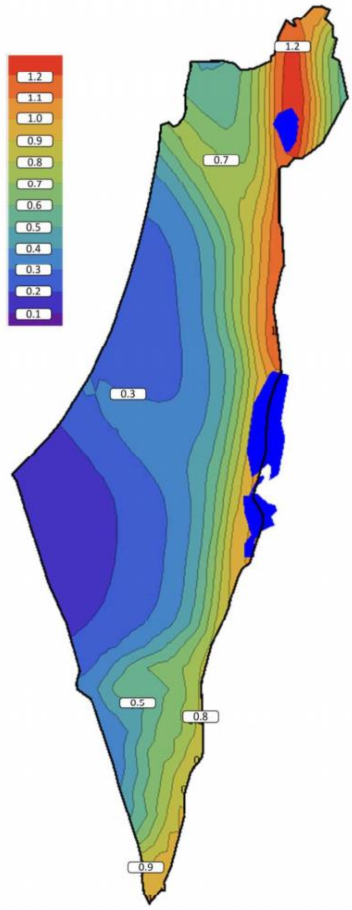
2% at 50 years	
S_1	S_S
	

Figure 20 - Seismic parameter S_1 & S_S as per SI 413-5

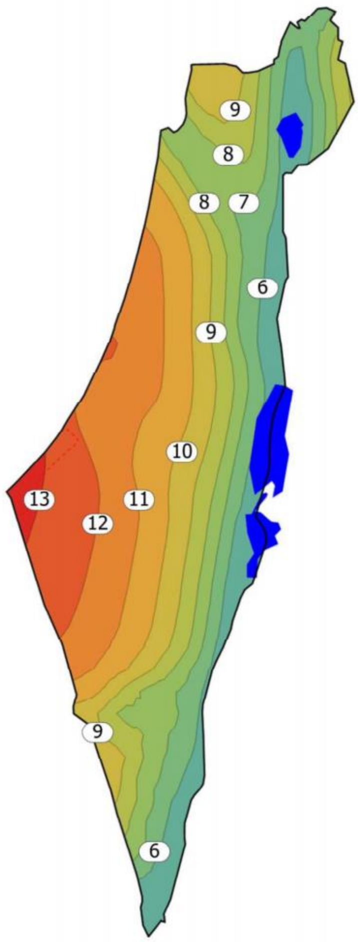
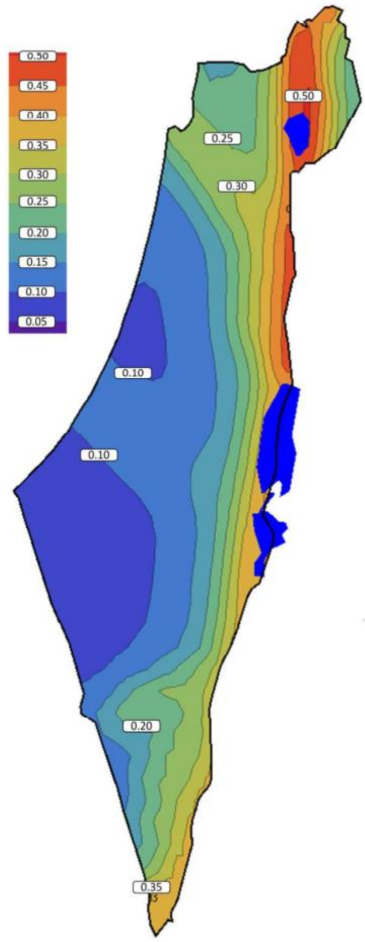
2% at 50 years	
T_L	Z
	

Figure 21 - Seismic parameter T_L & Z as per SI 413-5

- ✓ Using ASCE 7-10 -Table 20.3-1 for choosing site class; Site class= (D)
- ✓ Using ASCE 7-10 - table 11.4-1 and ASCE 7-10 - table 11.4-2, for site class "D", short-period site coefficient Fa=1.56 and long-period site coefficient Fv=2.4.
- ✓ Maximum considered earthquake spectral response accelerations adjusted for site class effects are evaluated.

$$S_{MS} = F_a S_s \; = \; 1.56(0.30g) = 0.468g$$

and,

$$S_{M1} = F_v S_1 \; = \; 2.4(0.08g) = 0.192g$$

- ✓ The 5% damped design spectral response accelerations S_{DS} at short period and S_{D1} at long period in accordance are evaluated.

$$S_{DS} = \frac{2}{3} S_{MS} \; = \; \frac{2}{3}(0.468g) = 0.312g$$

$$S_{D1} = \frac{2}{3} S_{M1} \; = \; \frac{2}{3}(0.192g) = 0.128g$$

- ✓ Occupancy importance factor, $I_e = 1.5$as evaluated from ASCE 7-10 (Table 1.5-2) and ASCE 7-10 (Table 1.5-1) (risk category IV).
- ✓ From ASCE 7-10 (Table 11.6-1) and for $S_{DS} = 0.312g$, Seismic Design Category (SDC) is C. For $S_{D1} = 0.128g$and using ASCE 7-10 (Table 11.6-2), SDC is C. Therefore, seismic design category (SDC) is "C".
- ✓ For ordinary, shear walls and using ASCE 7-10 Table 12.2-1, response modification coefficient$R = 5.0$.
- ✓ The seismic base shear V in a given direction is determined in accordance with the following equation:

$$V = C_s W$$

$$C_s = \frac{S_{DS}}{(R/I_e)} \leq \frac{S_{D1}}{T\ (R/I_e)} \geq 0.044\ S_{DS}\ I_e \geq 0.01$$

Approximate period $T_a = 0.0488(22)^{0.75} = 0.496\ sec.$

For $S_{D1} = 0.128g$and using ASCE 7-10 (Table 12.8-1); C_u=1.7

$C_u T_a = 1.70(0.496) = 0.843 sec$> 0.496 sec.

$$C_s = \frac{0.312}{5.0/1.5} = 0.09360 < \frac{0.128}{(5.0)(0.496)/1.5} = 0.07746 > 0.044(0.312)(1.5) = 0.021 \quad \text{O.K}$$

i.e., $C_s = 0.07746$

The seismic base shear *V=0.07746W*

Vertical distribution of forces:

$$F_x = C_{vx}\ V$$

Where:

$$C_{vx} = \frac{w_x\ h_x^k}{\sum_{i=1}^{n} w_i\ h^k{}_i}$$

And

K= a distribution exponent related to the building period as follows:

k = 1 for buildings with T less than or equal to 0.5 seconds

k = 2 for buildings with T more than or equal to 2.5 seconds

Interpolate between k = 1 and k = 2 for buildings with T between 0.5 and 2.5

$K = 1$

Shear forces $\boldsymbol{V_x} = \sum_{i=1}^{x} \boldsymbol{F_i}$

3.4 load combinations

The Load Combinations taken for analyzing of the structure was as per ASCE 7-10

3.5 Structural model Description

Modelling a building involves the modelling and assemblage of its various load carrying elements. The model must ideally represent the mass distribution, strength, stiffness and deformability. Modelling of the material properties and geometric details is as per details mentioned below.

3.5.1 Material Properties

The material properties used in creating the model were as follows:

1. Grade of Concrete – B250 & B300 (Figure 22),
2. Grade of Reinforcement used – F_y 420 MPa (Figure 23),
3. Poisson Ratio of Concrete – 0.2
4. Poisson Ratio of Reinforcement – 0.3
5. Density of Concrete – 25KN/m^2
6. Density of Reinforcement – 78.5KN/m^2
7. Young's Modulus of concrete (B250) – 21,538 KN/m^2
8. Young's Modulus of concrete (B300) – 23500 KN/m^2
9. Young's Modulus of reinforcement – 2.0X108 KN/m^2
10. Damping Factor – 0.05 (As per ASCE 7-10).

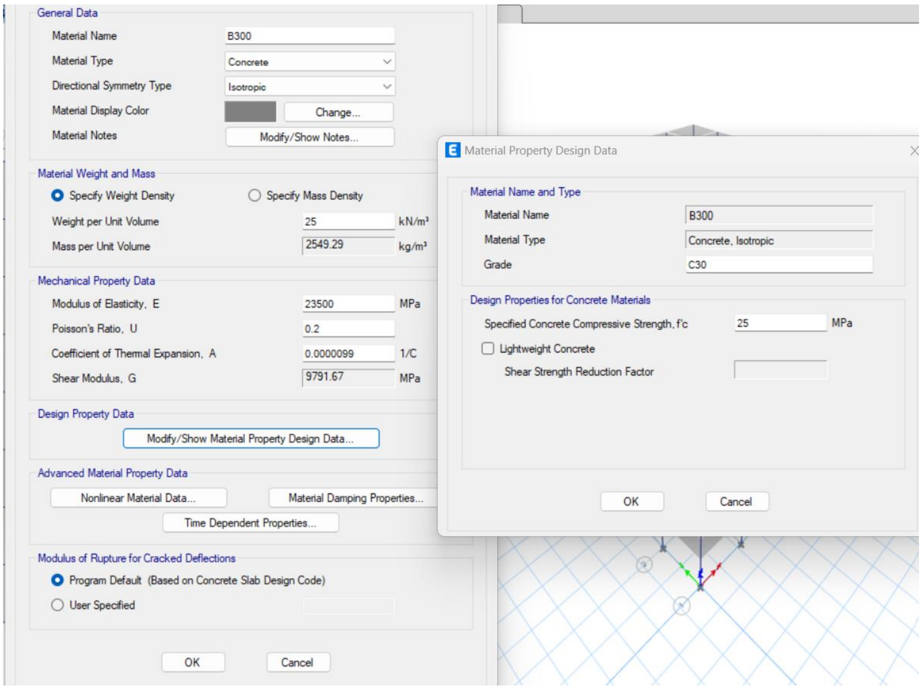


Figure 22 - Concrete Properties Definition

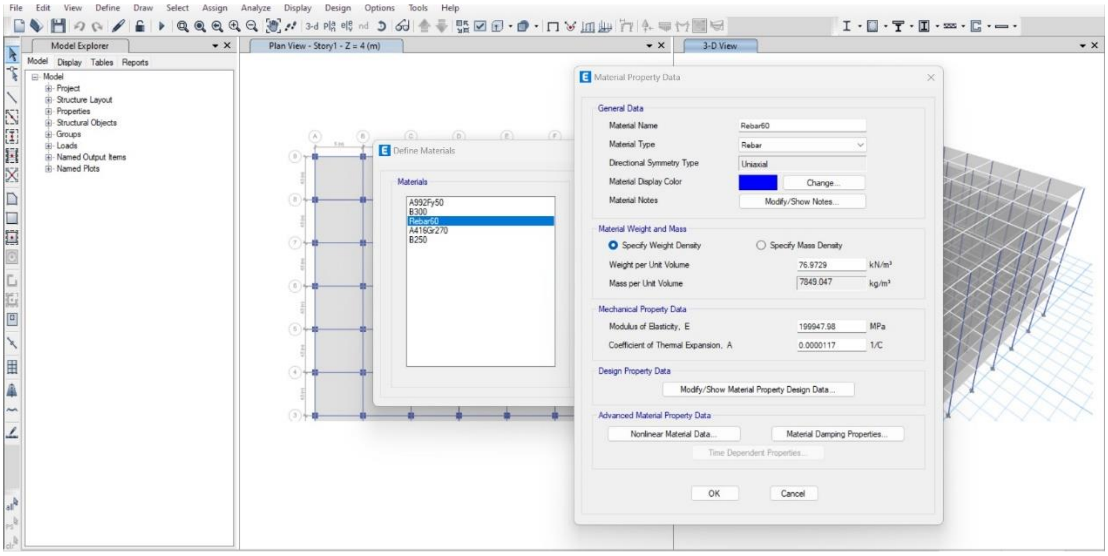


Figure 23 - Steel Properties Definition

3.5.2 Geometrical Properties

The geometrical properties measured are as follows:

1. The slab thickness – 0.2m
2. Beam cross sections on all floors – 0.5mX0.50m (see Figure 24).

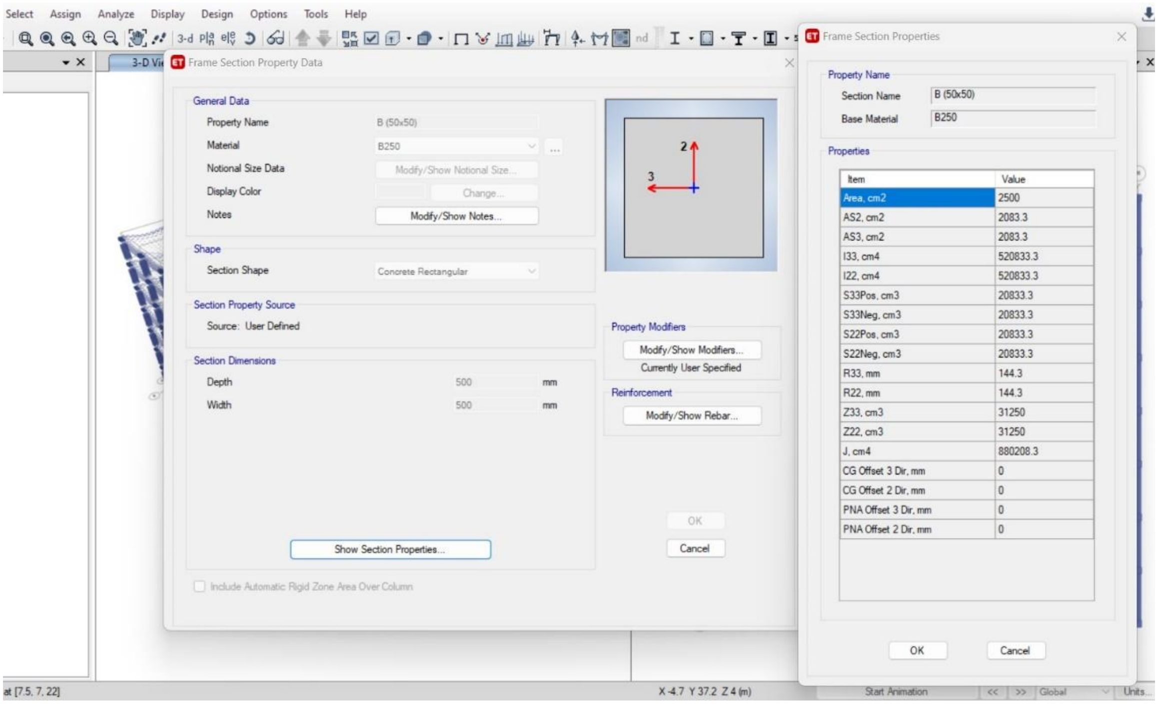


Figure 24 - Beam Section Definition

3. Column cross section on all floors – 0.6mX0.60m (see Figure 25).

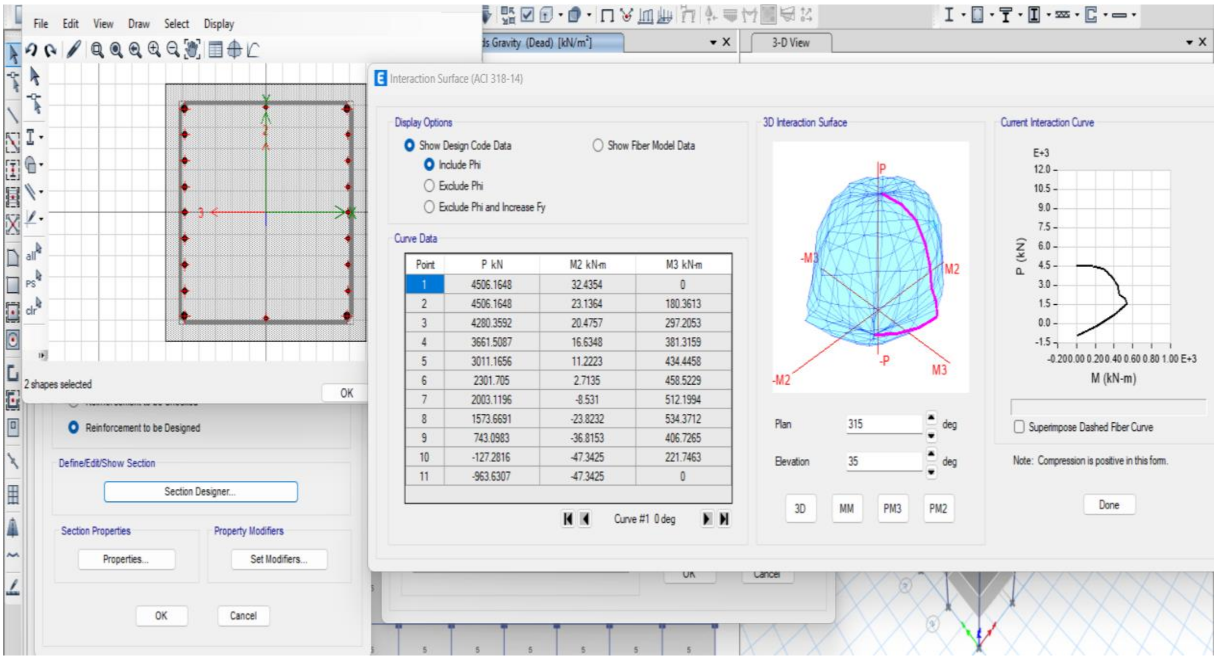


Figure 25 - Column Section Definition

4. Nos. of bays along length i.e., X-Axis – 7
5. Nos. of bays along width i.e., Y-Axis – 8
6. No. of floors Along Z-Axis – 7floors
7. Story Height – 4m for Ground, 3m for typical floors.

3.5.3 Structural Elements

The structural elements were modelled as follows:

1. Beams and columns are modelled by 3D frame elements. The beam-column joints are assumed to be rigid. Beams and columns in the present study were modelled as frame elements.
2. With the centerlines joined at nodes using commercial software ETABS Ver 21. The dead weight of the beams and columns was calculated by the program using the material densities and the geometrical dimensions of the respective members.
3. The floor slabs were modelled to act as diaphragms, which ensure integral action of all the vertical lateral load-resisting elements. The weight of the slab was taken into account by self-weight calculation by the program using the material properties and geometrical dimensions
4. The 3D model is as shown in figure26 & 27 below.

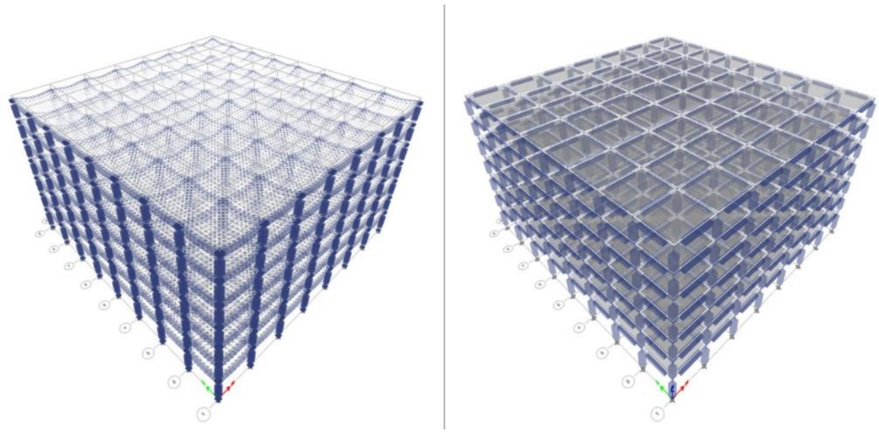


Figure 26 - 3D Model

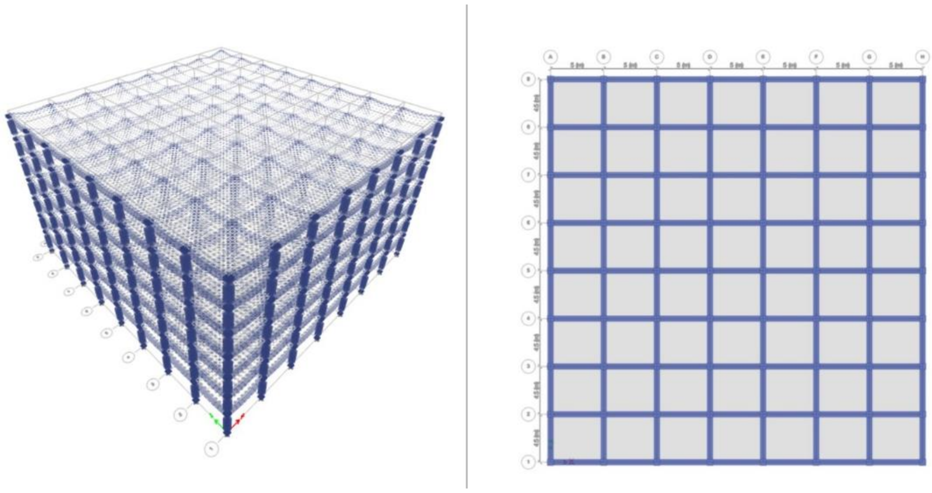


Figure 27 - 3D Model and typical structural layout

3.6 Earthquake parameters

The figures from 28 to 34 present the procedure of defining the parameters needed for the pushover analysis, some samples were shown here in figures only in one direction (x for example) but in fact it was defined for both x and y directions.

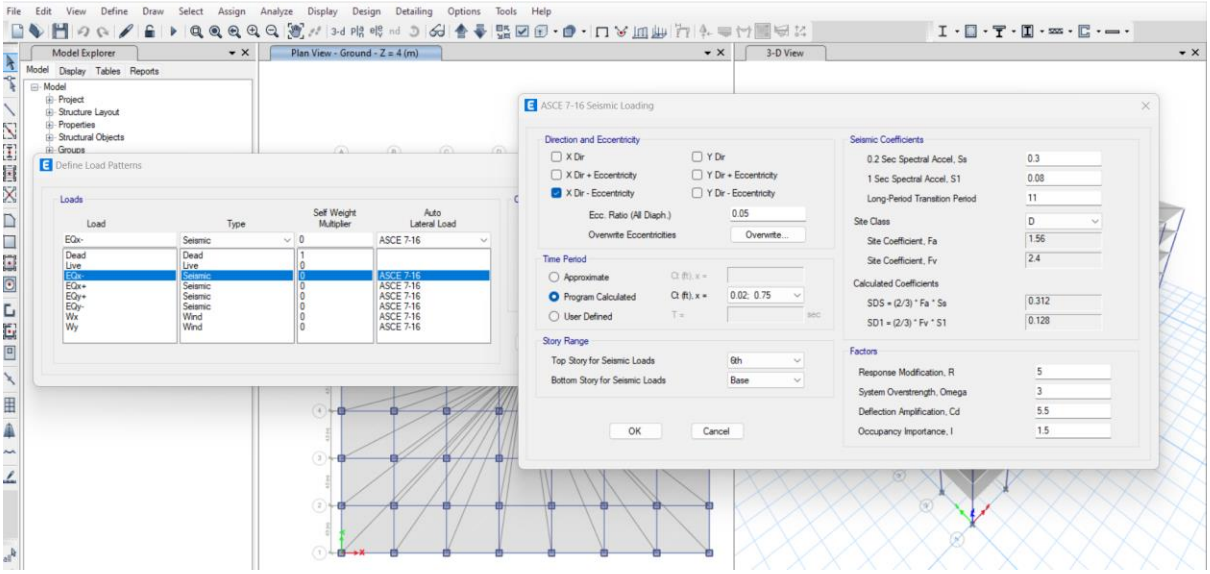


Figure 28 - Define earthquake x direction

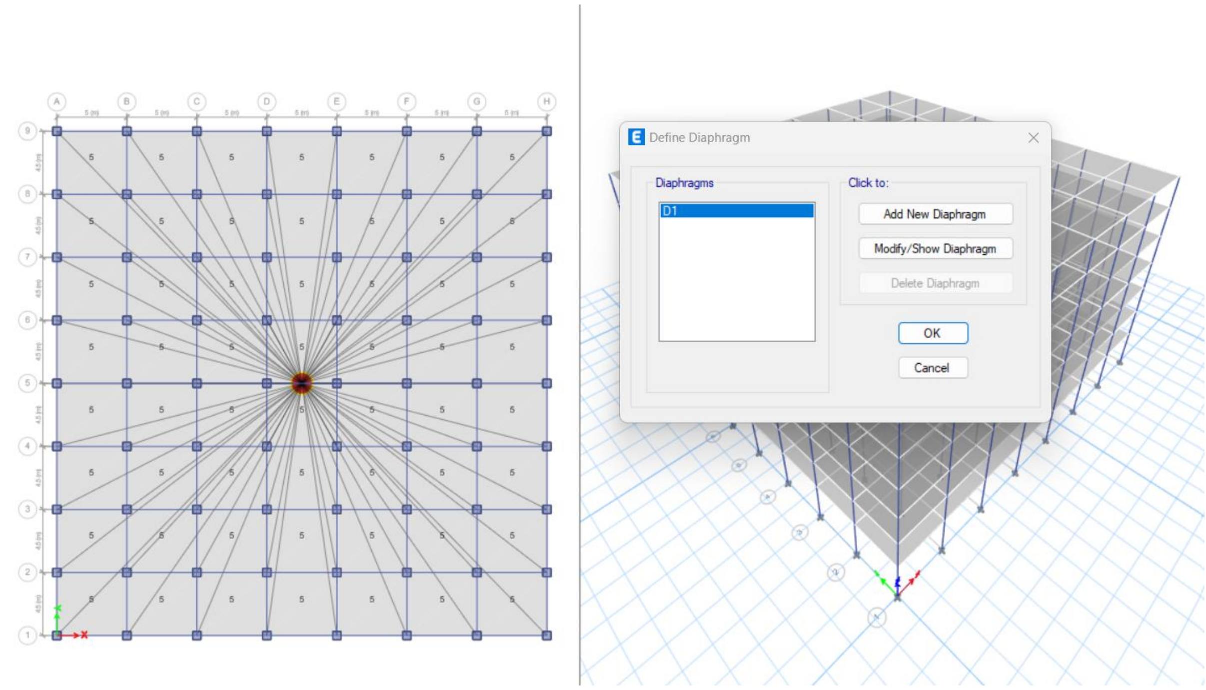


Figure 29 - Define the diaphragms

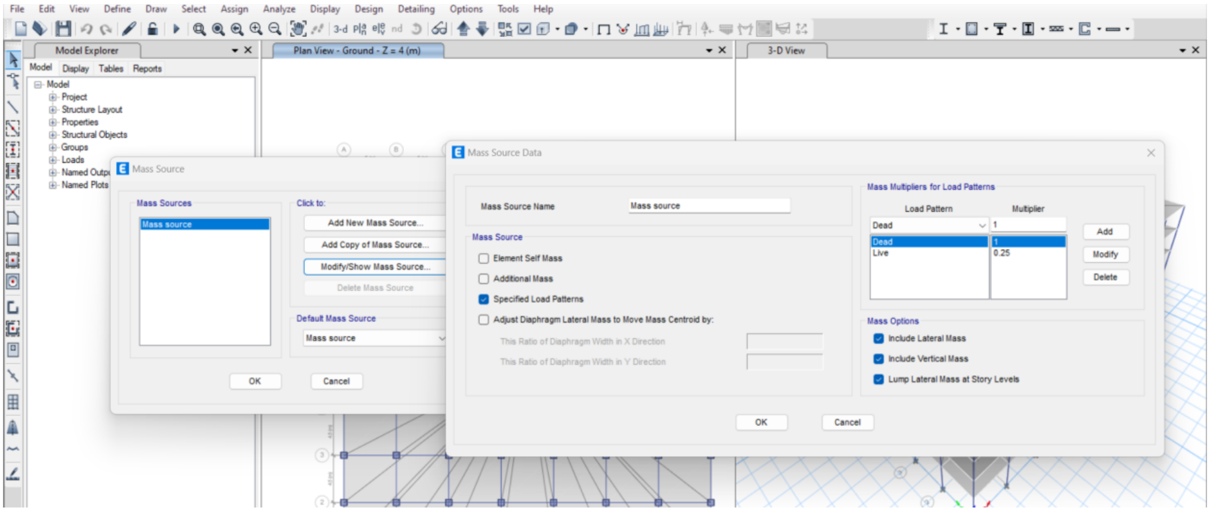


Figure 30 - Mass source (0.25 LL and 1 DL)

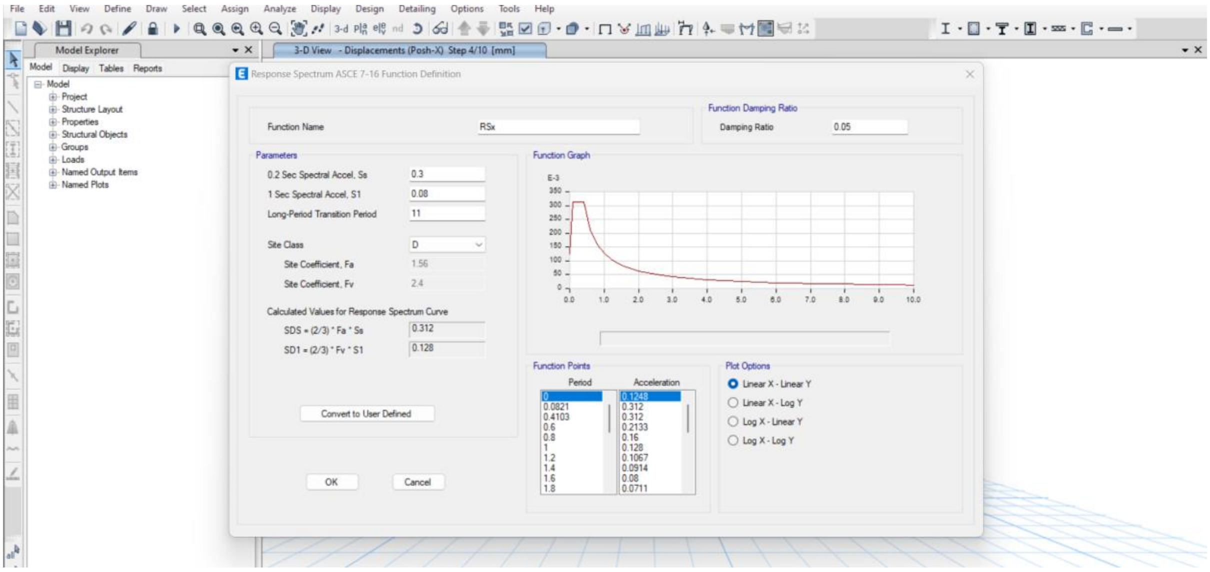


Figure 31 - Response Spectrum in X direction

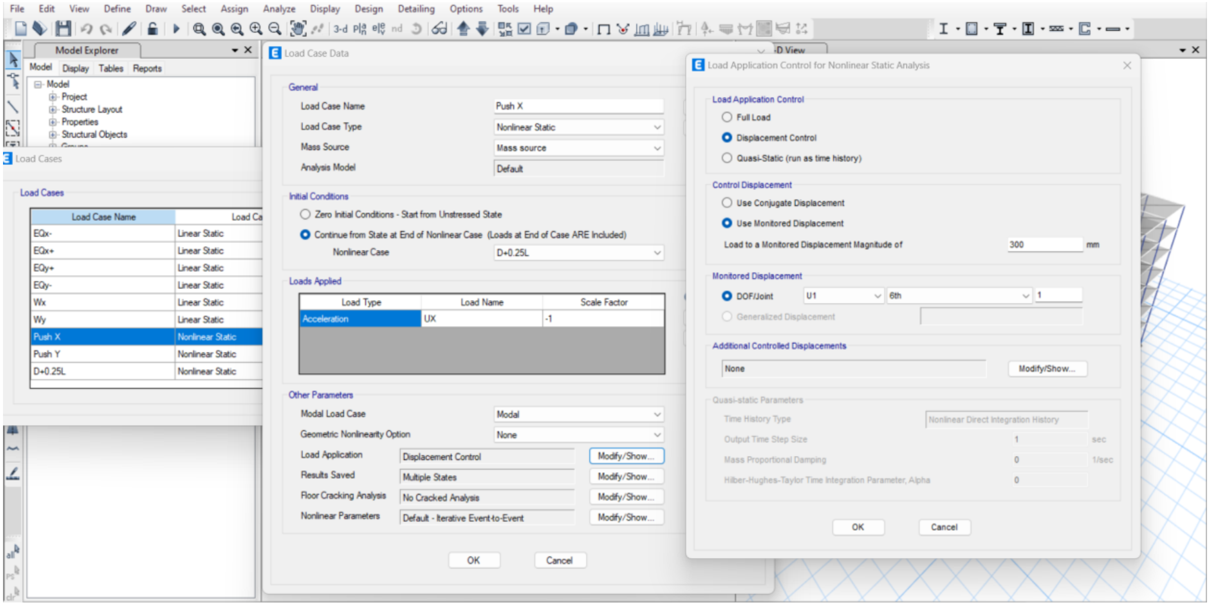


Figure 32 - Nonlinear static analysis (Pushover) – X direction

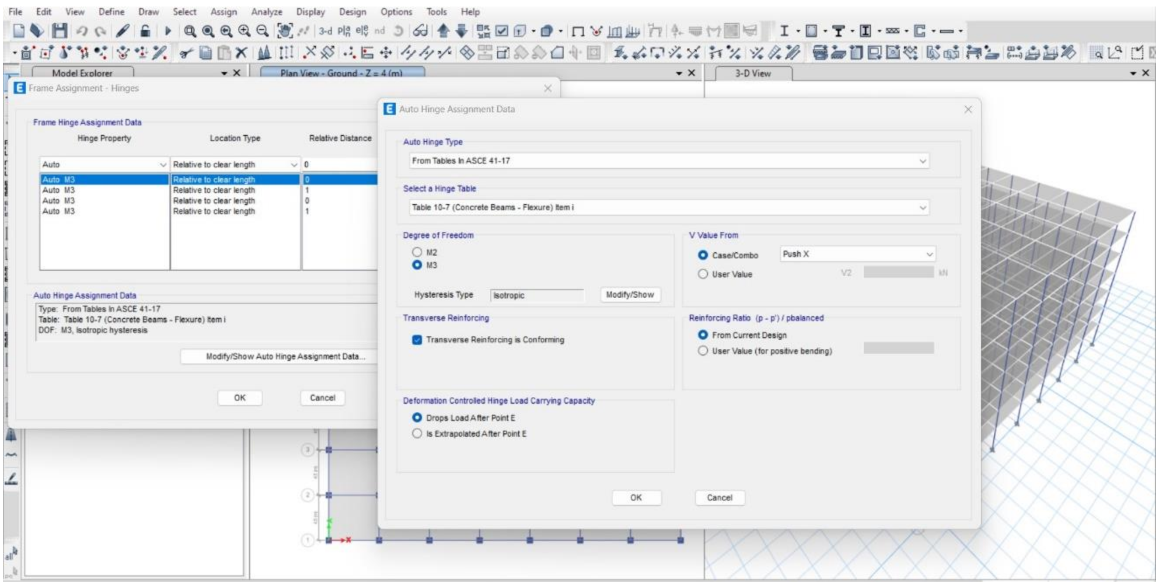


Figure 33 - Hinge Definition (x-direction) for Beam

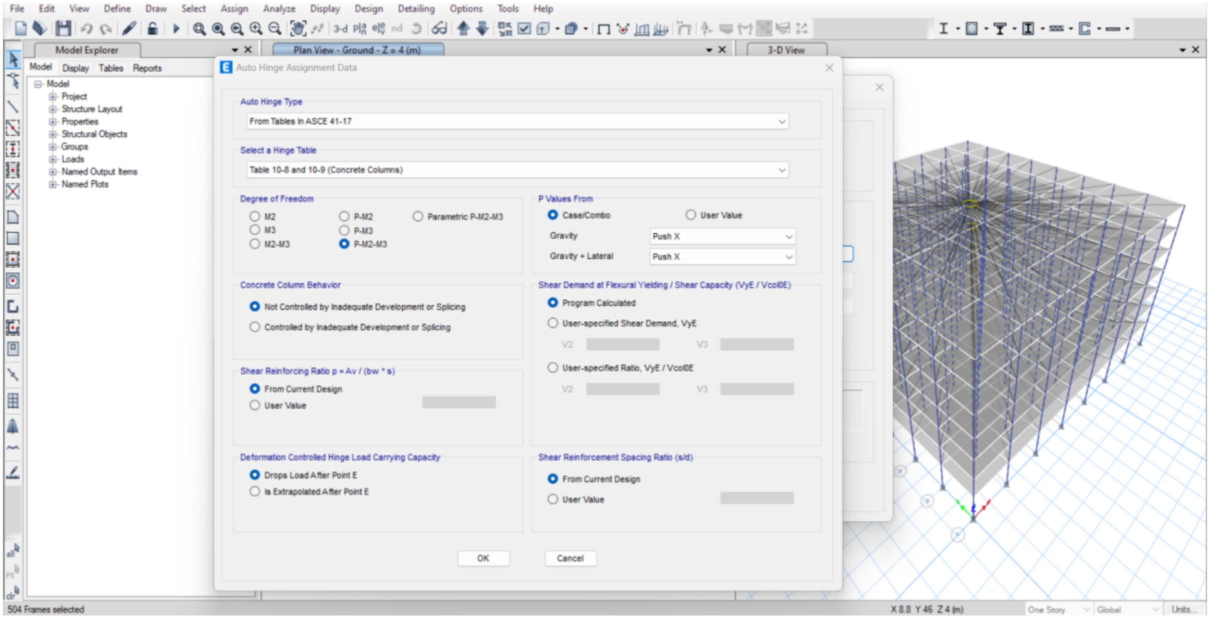


Figure 34 - Hinge Definition (x-direction) for Column

4 Chapter 4: Results and Discussion

4.1 Building story response plots due to pushover analysis

The target responses of the building story due to Pushover Analysis (PA) of the model are shown below:

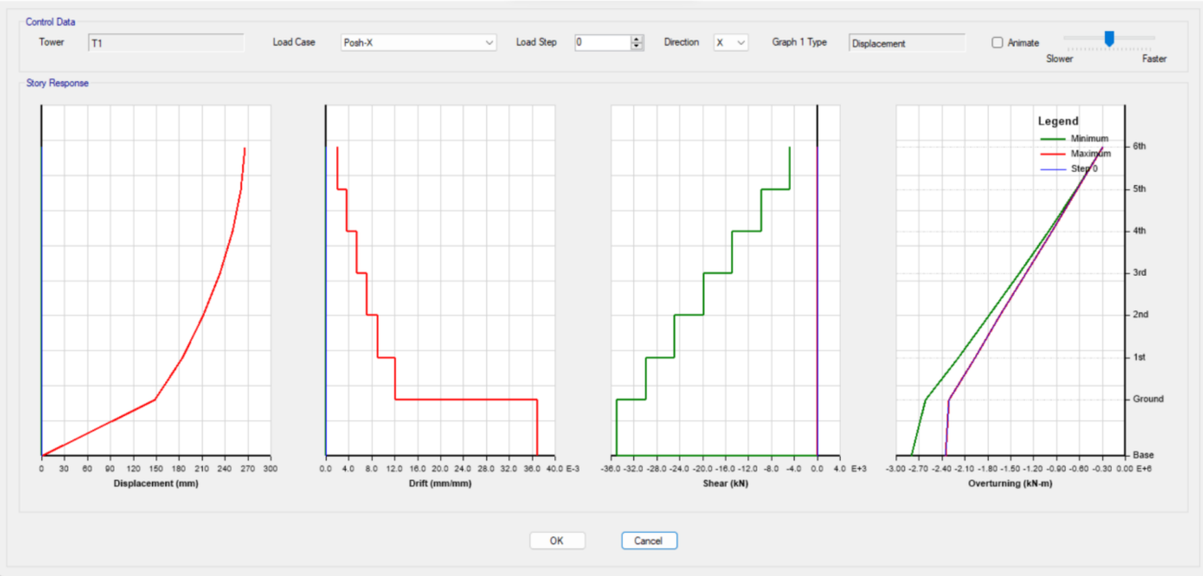


Figure 35 – Building story response due to Push X (X-direction)

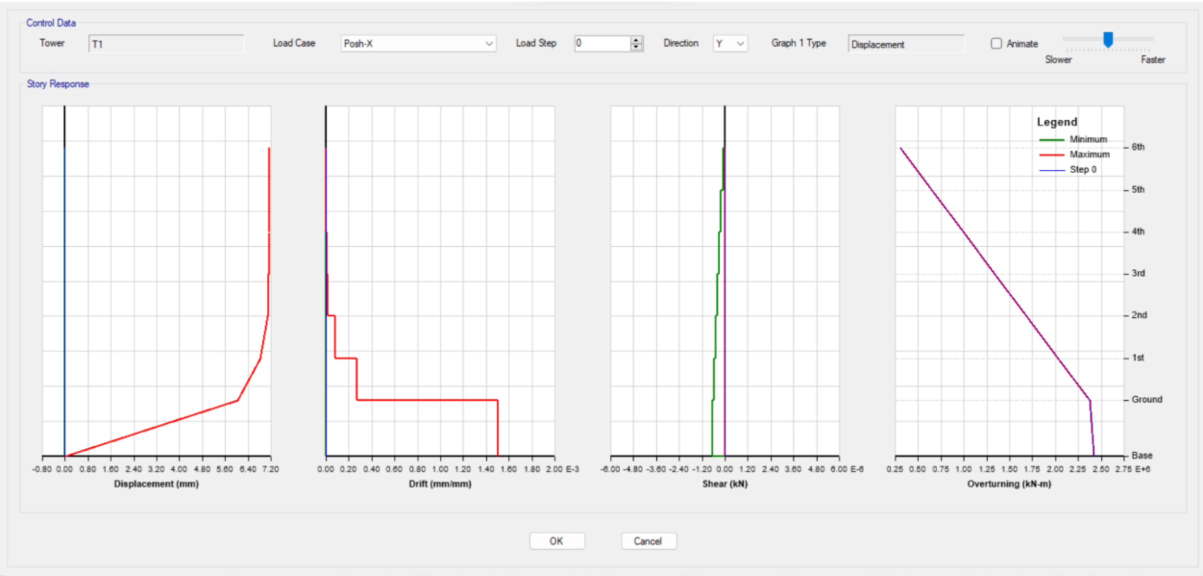


Figure 36 – Building story response due to Push X (Y-direction)

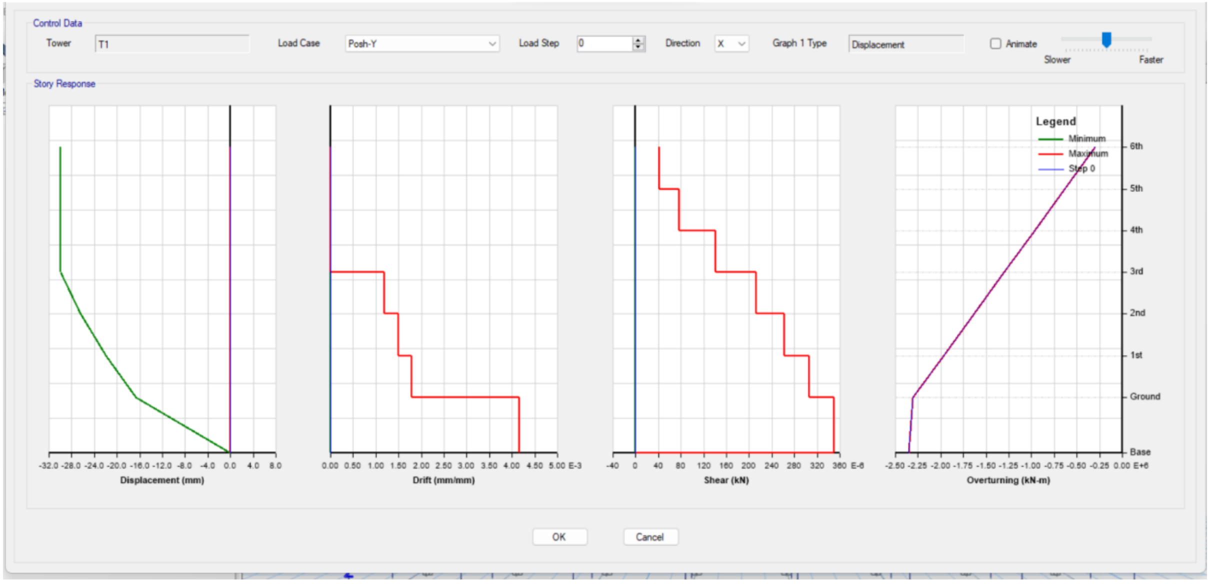


Figure 37 – Building story response due to Push Y (X-direction)

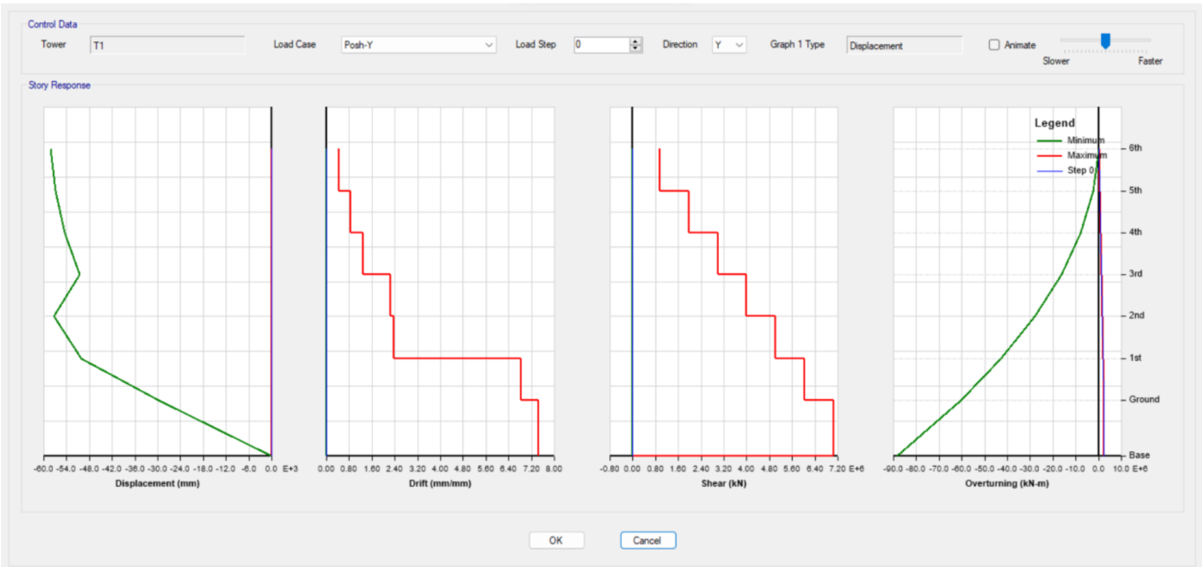
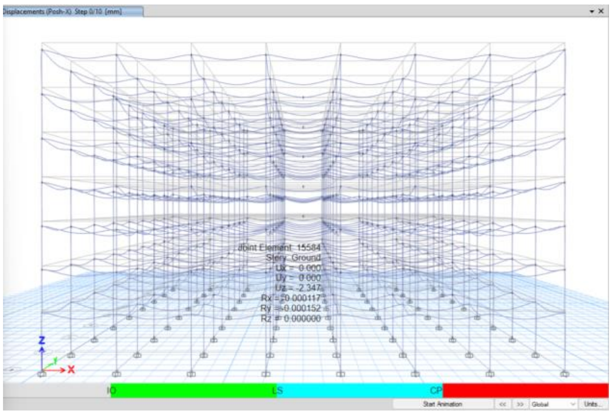


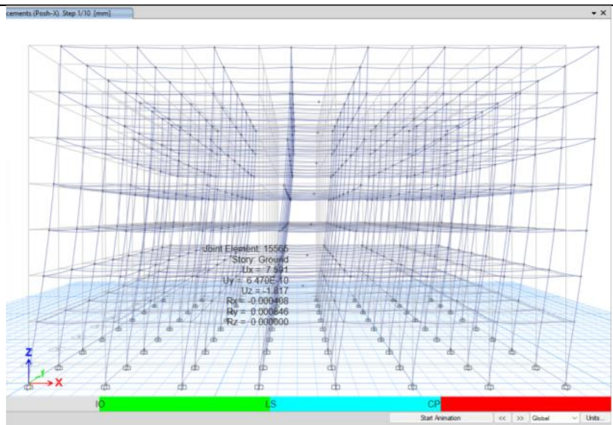
Figure 38 – Building story response due to Push Y (Y-direction)

4.2 *Deformed shape of the building*

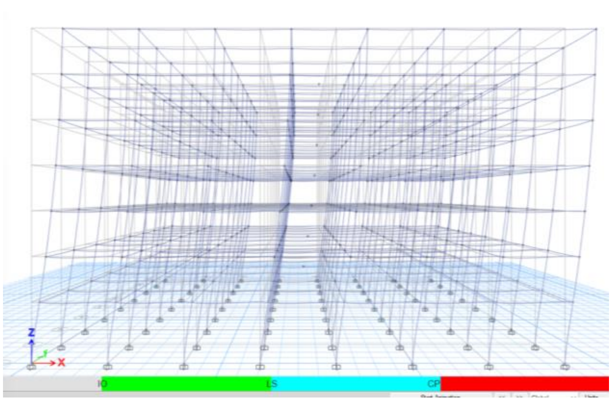
Plastic hinges formation for the building mechanisms have been obtained at different displacement levels due to pushover analysis. The hinging patterns are plotted at different levels in figures 39 to50. Plastic hinges formation starts at base columns of lower stories, then propagates to upper stories and continue with yielding of interior intermediate columns in the upper stories.



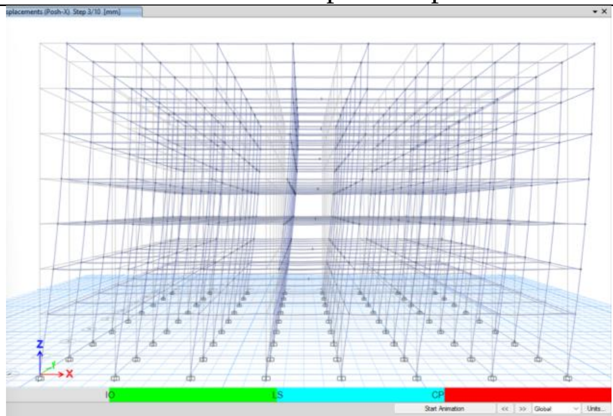
Deformed shape at step 0



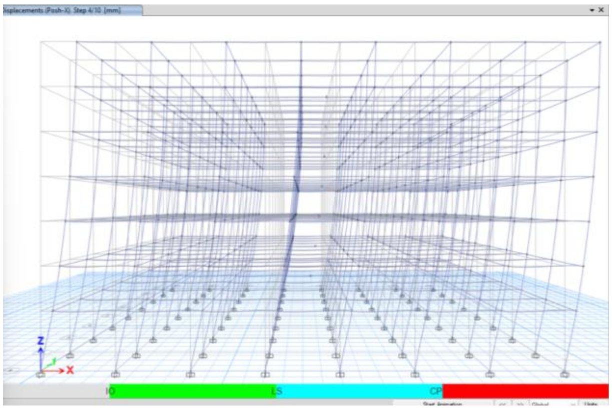
Deformed shape at step 1



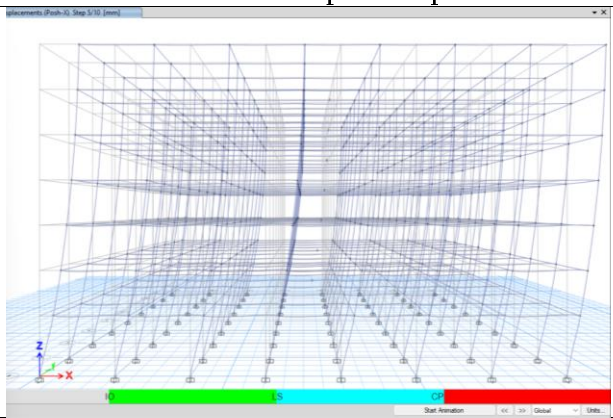
Deformed shape at step 2



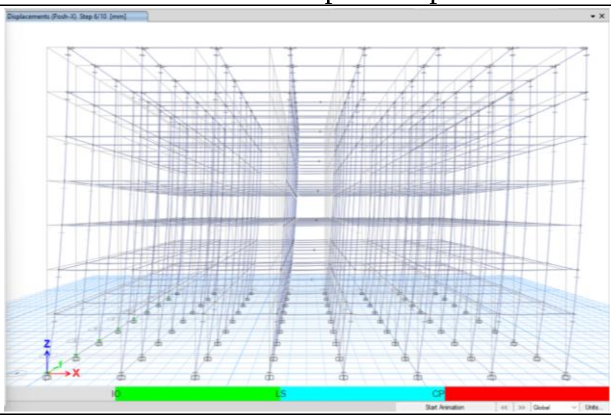
Deformed shape at step 3



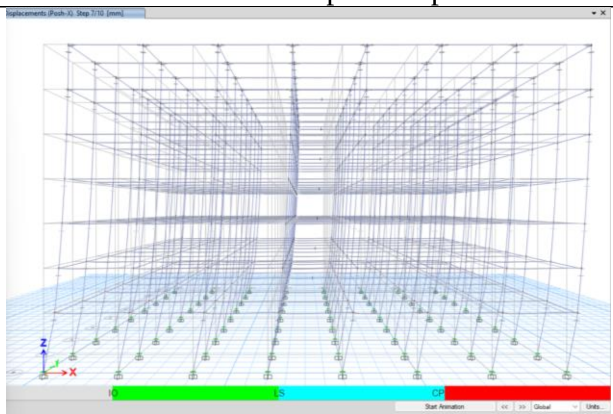
Deformed shape at step 4



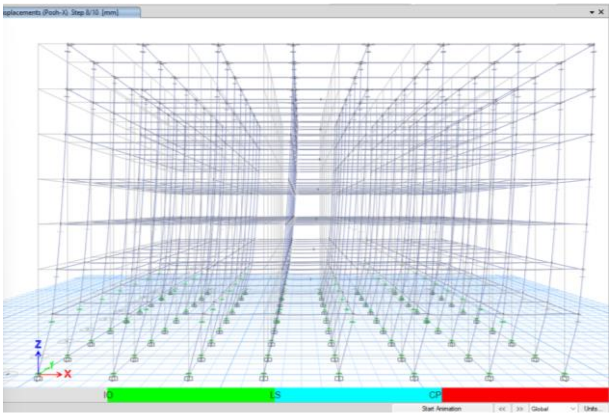
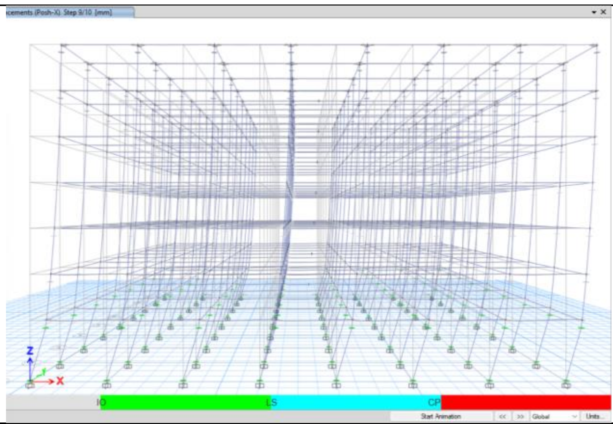
Deformed shape at step 5

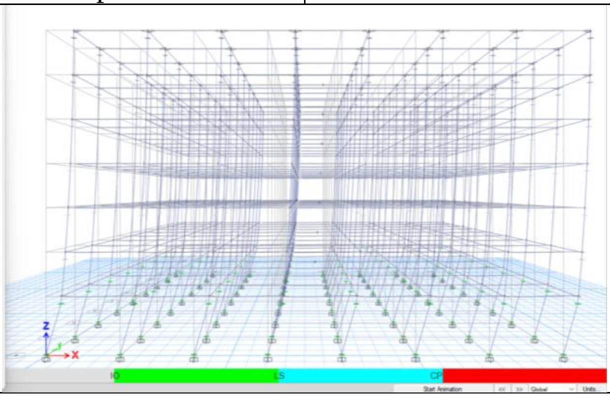


Deformed shape at step 6



Deformed shape at step 7

	
Deformed shape at step 8	Deformed shape at step 9



Deformed shape at step 10

Figure 39 – Deformed shape as per Push -X

Summary: The Building Performance Levels of Immediate Occupancy (IO) start formation at step 7

4.3 Hinge Response in Both Direction

Sample of beams and columns hinges results due to pushover analysis in x & y direction (step 5) for 3rd floor can be shown in the below figures 40 to 43.

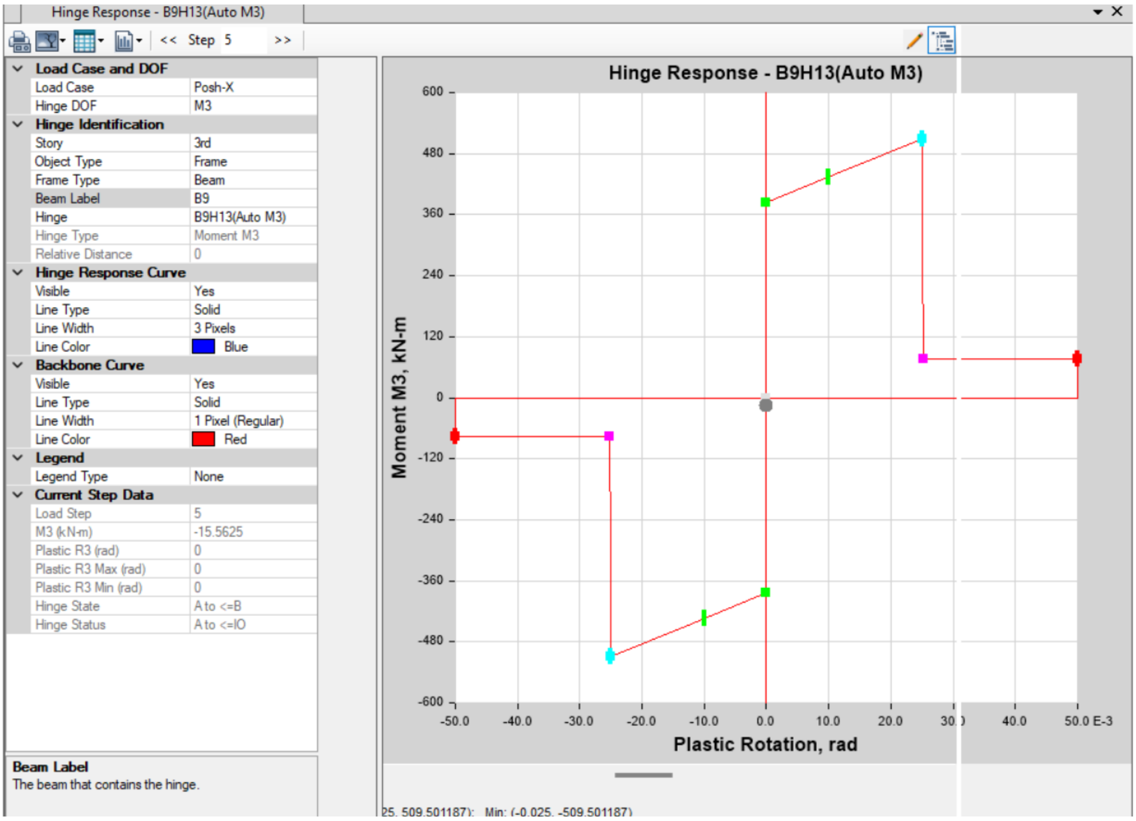


Figure 40 – Beam Hinge Result Due to PA-X

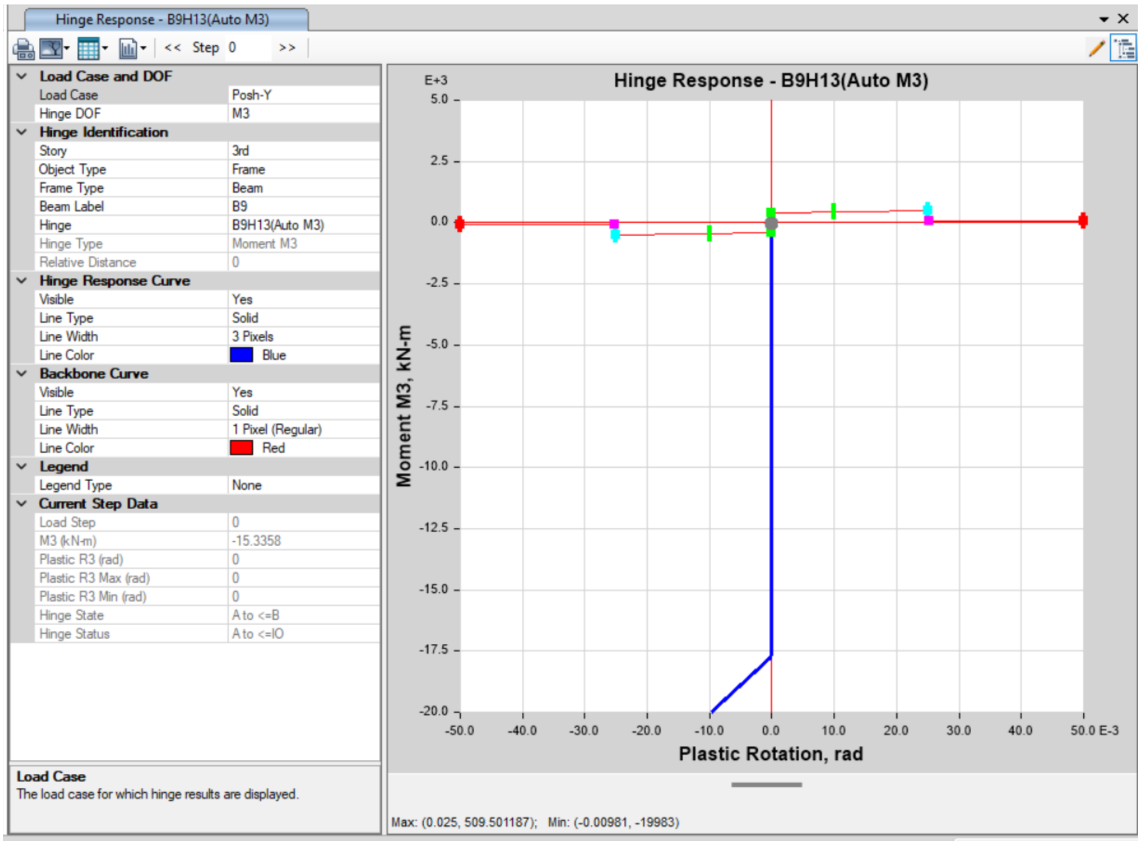


Figure 41 – Beam Hinge Result Due to PA-Y

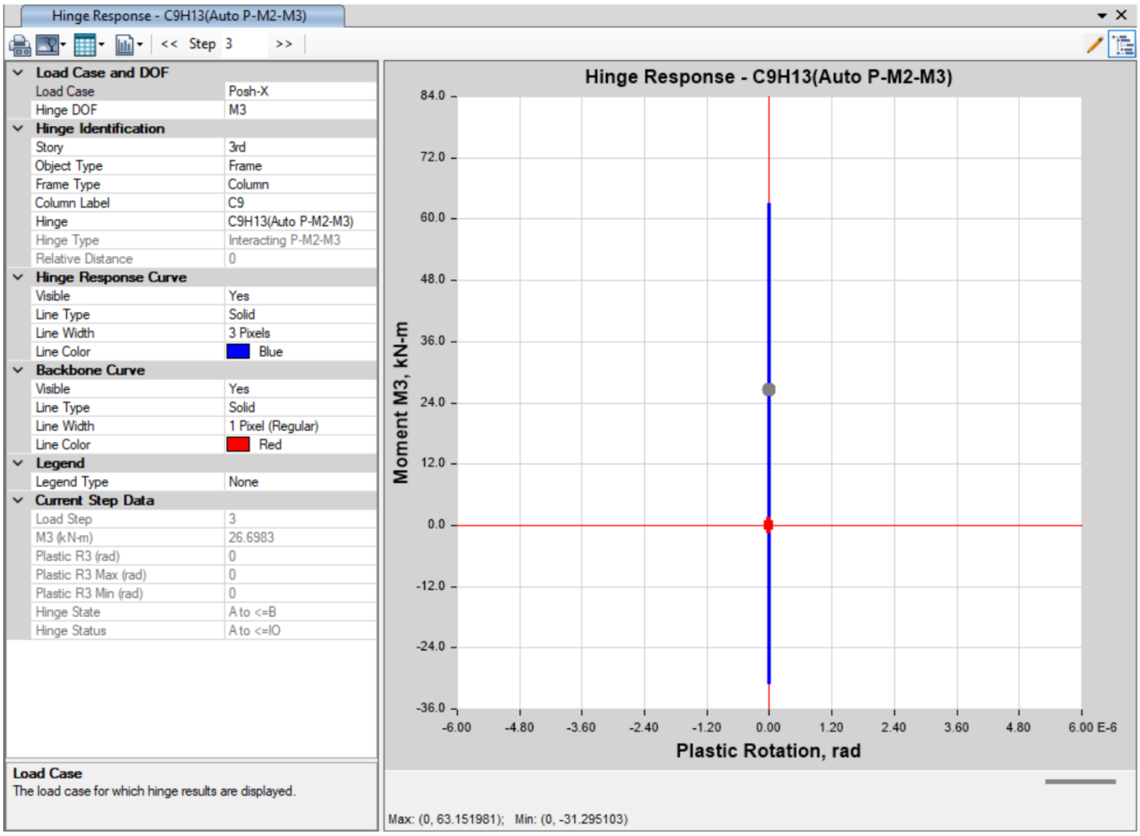


Figure 42 – Column Hinge Result Due to PA-X

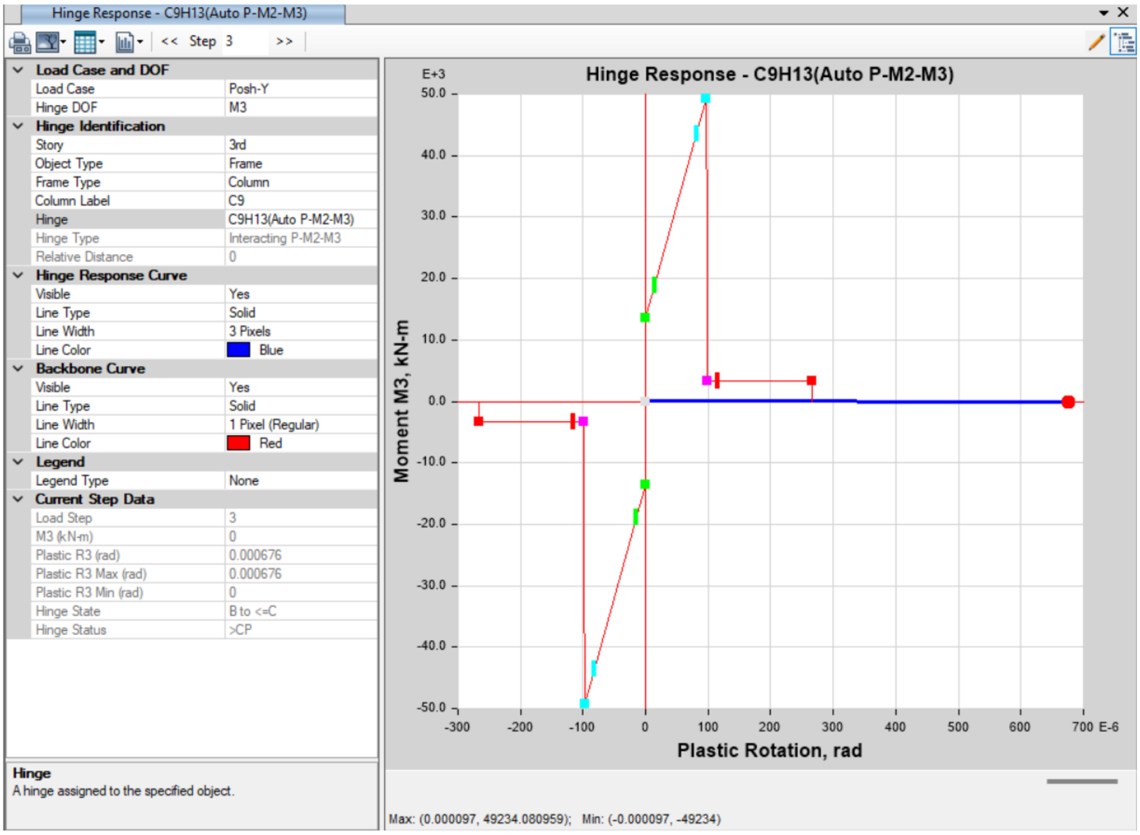


Figure 43 – Column Hinge Result Due to PA-Y

4.4 Pushover Curve

The performance point can be obtained by using superimposing demand spectrum on capacity curve into spectral coordinate. It is obvious that the demand curve tends to intersect the capacity curve at the performance point. It can be concluded that there are sufficient strength and displacement reserves at this performance point. The following show the capacity curves obtained in respective directions.

4.4.1 Pushover Curve - FEMA 440 Equivalent Linearization

This is the data for a FEMA 440 equivalent linearization pushover analysis.

Demand Spectrum Input Data

Source	ASCE 7-10 General	Ss	0.3
Site Class	D	S1	0.08
		Tl	11 sec

Effective Period and Effective Damping Parameters

Inherent Damping	0.05		
Damping Params	Default Value	Period Parameters	Default Value
A	4.2	G	0.11
B	-0.83	H	-0.018
C	10	I	0.09
D	1.6	J	0.14
E	22	K	0.77
F	0.4	L	0.05

Pushover Plot

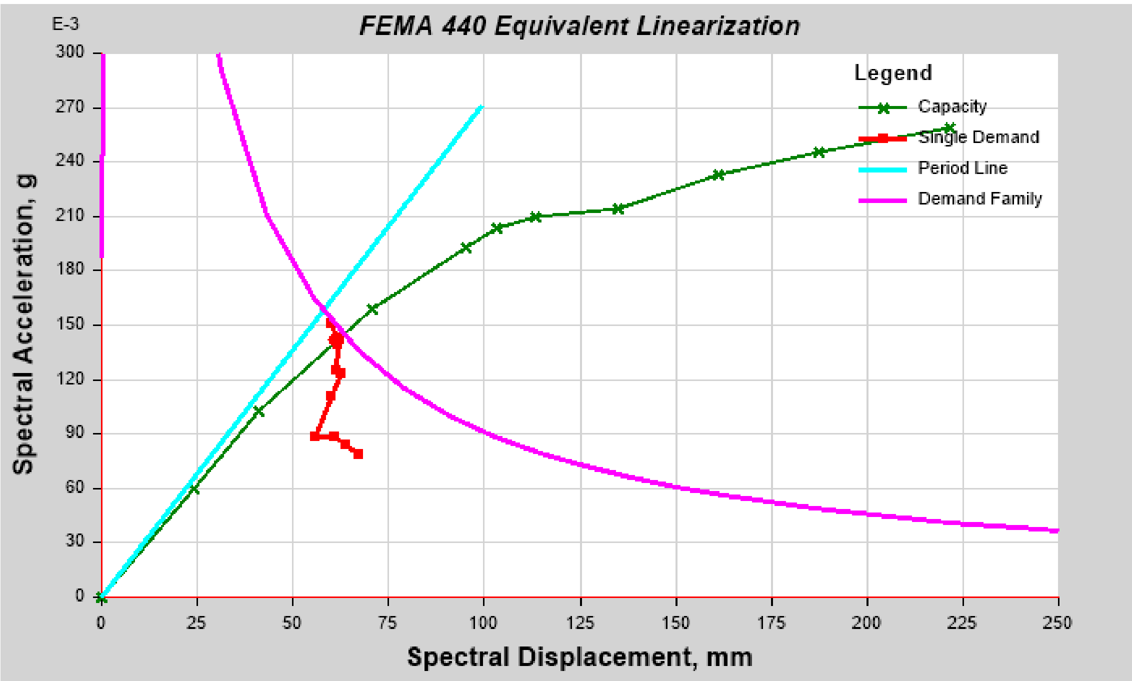


Figure 44 – FEMA 440 Capacity Curve superimposed on Demand Curve.

Performance Point

Point Found	Yes	T secant	1.315 sec
Shear	17400.2007 kN	T effective	1.345 sec
Displacement	77.511 mm	Ductility Ratio	1.499222
Sa	0.141754	Effective Damping	0.0648
Sd	61.437 mm	Modification Factor	1.045972

4.4.2 Pushover Curve - ASCE 41-13 Displacement Modification

This is the data for a ASCE 41-13 displacement modification pushover analysis.

Demand Spectrum Input Data

Damping Ratio	0.05	Source	ASCE 7-10 General
Include SSI	No	Accel Ss	1
C2 Type	Default Value	Accel S1	0.4
Cm Type	Default Value	Site Class	D
		Tl	8 sec

Pushover Plot

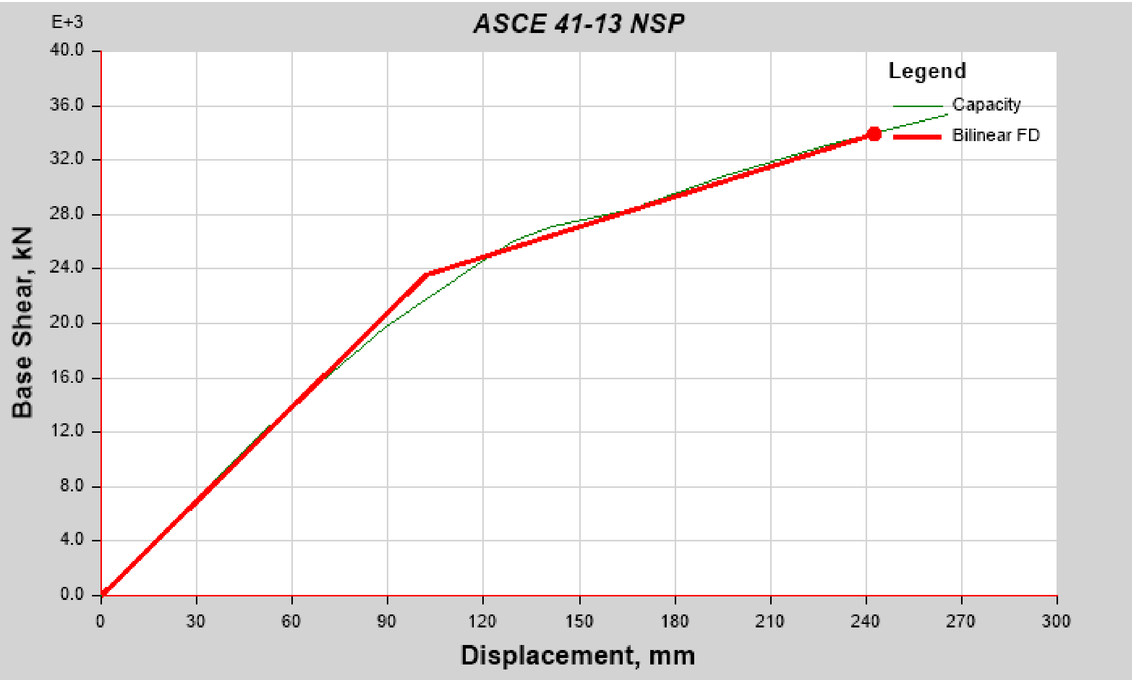


Figure 45 – ASCE 41-13 Capacity Curve superimposed on Demand Curve.

Target Displacement Results

Displacement	242.474 mm	Shear	33941.0905 kN

Calculated Parameters

C0	1.20256	Sa	0.499215 g
C1	1	Alpha	0.317829
C2	1	uStrength	2.843845
Ti	1.266 sec	Dy	101.924 mm
Te	1.279 sec	Vy	23598.501 kN
Ki	236268.223 kN/m	Weight	134432.0766 kN
Ke	231529.268 kN/m	Cm	1

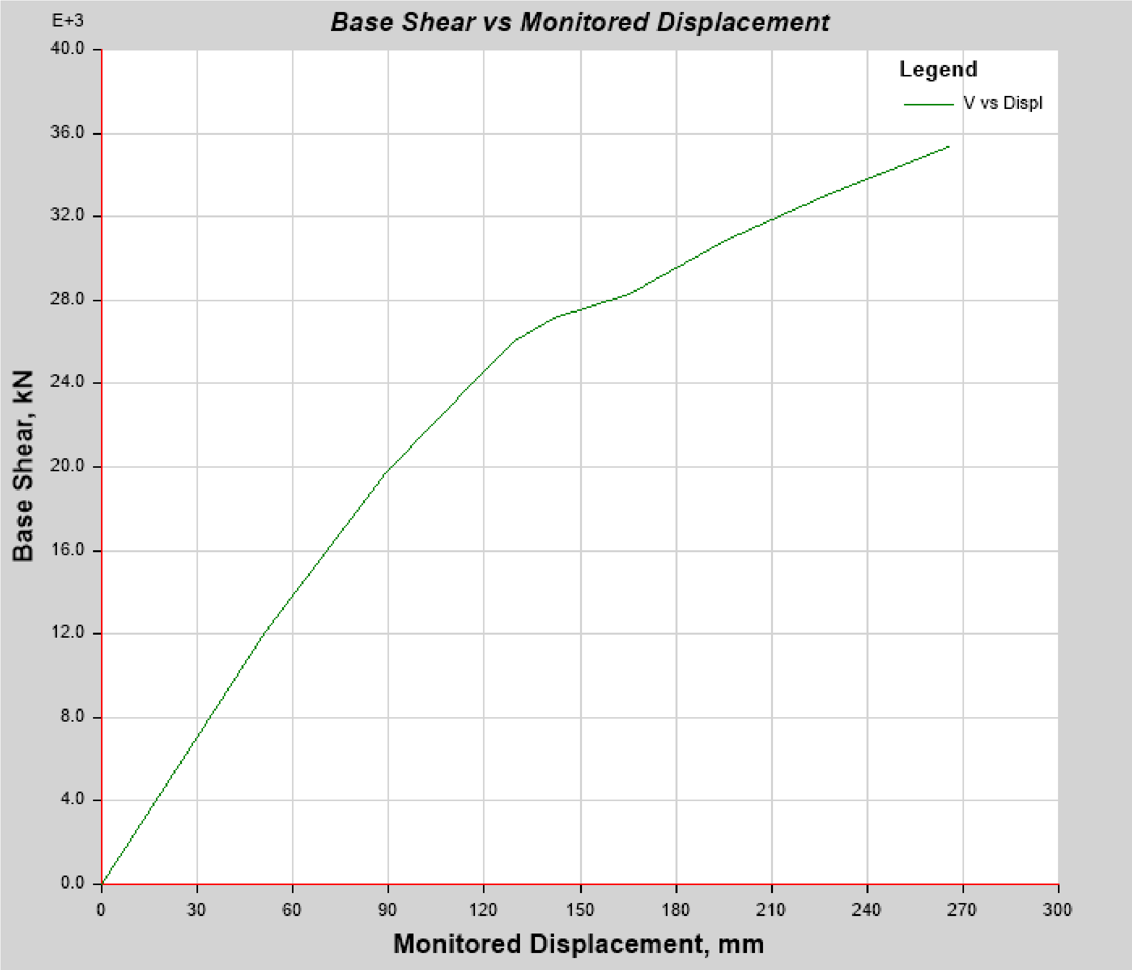


Figure 46 - Pushover Curve - Base Shear vs Monitored Displacement

4.5 Propose Structural strengthening techniques for existing building

An advanced retrofit strategy aims at establishing a holistic building strategy for the entire, remaining life cycle of the building. The ultimate option in the building strategy is to construct a new building in replacement. An advanced retrofit strategy determines whether a building reaches this condition in five, twenty-five, one hundred, or more years. Intermediate objectives of an advanced retrofit strategy can be building maintenance, partial renovation, or comprehensive renovation.

In an advanced retrofit strategy, the technical retrofit options are compared with the realizable added value and incorporated into a holistic strategy. Typology provides the information needed. The following steps are characteristic for an advanced retrofit:

4.5.1 RC jacketing

RC jacketing is one of the most commonly applied methods for the rehabilitation of concrete members. Jacketing is considered to be a global intervention method if the longitudinal reinforcement placed in the jacket passes through holes drilled in the slab and new concrete is placed in the beam–column joint figure 47. However, if the longitudinal reinforcement stops at the floor level, then RC jacketing is considered as a member intervention technique. The main advantage of the RC jacketing technique is the fact that the lateral load capacity is uniformly distributed throughout the structure of the building thereby avoiding concentrations of lateral load resistance, which occur when only a few shear walls are added. A disadvantage of the method is the presence of beams which may require most of the new longitudinal bars in the jacket to be bundled into the corners of the jacket. Because of the presence of the existing column, it is difficult to provide cross ties for the new longitudinal bars, which are not at the corners of the jacket.

Figure 47 - Reinforced concrete jacketing technique

To date, apart from qualitative guidelines provided in some Codes, no specific design rules exist for dimensioning and detailing of the jackets to reach a predefined performance target. The uncertainty with regard to bond between the jacket and the original member is another disadvantage. of the many factors influencing jacket performance, slip and shear-stress transfer at the interface between the outside jacket layer and the original member that serves as the core of the upgraded element are overriding considerations.

The effectiveness of the method has been studied by many researchers and supported by experimental work. In cases where building is in close proximity to one another, the method is modified and one-, two- or three-sided jacketing applies.

4.5.2 Addition of walls

Addition of new RC walls is one of the most common methods used for strengthening of existing structures. This method is efficient in controlling global lateral drift, thus reducing damage in frame members. During the design process, attention must be paid to the distribution of the walls in plan and elevation (to achieve a regular building configuration), transfer of inertial forces to the walls through floor diaphragms, struts and collectors, integration and connection of the wall into the existing frame buildings and transfer of loads to the foundations. Added walls are typically designed and detailed as in new structures. To this end, in the plastic hinge zone at the base they are provided with boundary elements, well-confined and detailed for flexural ductility. They are also capacity-designed in shear throughout their height and over-designed in flexure above the plastic hinge region (with respect to the flexural strength in the plastic hinge zone, not the shear strength anywhere), to ensure that inelasticity or pre-emptive failure will not take place elsewhere in the wall before plastic hinging at the base and that the new wall will remain elastic above the plastic hinge zone.

The most convenient way to introduce new shear walls is by partial or full infilling of strategically selected bays of the existing frame. If the wall takes up the full width of a bay, then it incorporates the beams and the two columns, the latter acting as its boundary elements figure 48 In case only the web of the new wall needs to be added, sometimes by shotcreting against a light formwork or a partition wall is performed. In the latter case, shotcrete is normally used for increased adhesion between the existing and the added material. An alternative to the cast-in-place infill wall technique is the addition of pre-cast panels. The pre-case infill wall system should be designed to behave monolithically, and the infill wall should be designed with sufficient shear strength to develop flexural yielding at the base of the wall.

Figure 48 - Cast-in-place infill walls

A major drawback of the addition of walls is the need for strengthening the foundations to resist the increased overturning moment and the need for integrating the wall with the rest of the structure. Foundation intervention is usually costly and quite disruptive, thus rendering the application of this technique unsuitable for buildings without an existing adequate foundation system.

4.5.3 Steel Jacketing

Confining reinforced concrete column in steel jackets is one of the effective methods to improve the earthquake resistant capacity. Steel jacking has remarkable advantages in comparison with hoops and spirals rebar’s warped around columns. Two major reasons of implanting steel jacketing can be addressed as first, having a vast amount of transverse steel which provides more confinement to the compressed concrete. Second, preventing concrete crumbling out of plane which could be considered as critical reason for deterioration of rebars and buckling of longitudinal bars in a column. Furthermore, steel jacketing is not only less interruptive, less time consuming and less expensive, but also results in minimum loss of floor area. Practically, steel jacketing (or caging) consists of steel angles at corners of RC columns and steel straps at few places along the height which provide composite action at the interface of steel and concrete element.

The concrete columns should be strengthened by steel jackets on all four sides of the column under consideration. The jacketing arrangement is fully anchored to concert columns by anchor bolts. These strong anchor bolts provide the fully composite action between concrete column and steel jacket system which fulfills the design assumptions in terms of modeling in ETABs.

The target performance level desired to meet has been selected as CP (collapse prevention) for sever earthquake. A process needs to be established in order to come up with the number of columns that requires retrofitting in terms of design efficiency. For that reason, a trial-and-error method should be followed while the building performance reached to CP level which is acceptable for an office building.

4.5.4 CFRP Retrofit Technique

Another innovative retrofit technique, according to Richard D. Iacobucci and Shamim A Sheikh, is wrapping the concrete member with carbon fiber-reinforced polymer (CFRP). This material has been used as an attractive and constructive procedure for retrofitting of existing columns. Specifically, the columns constructed prior to 1971 which have shown vulnerability in regards to inelastic conditions and may fail instantly without showing adequate warning during an intensive seismic event. Assembled fabric sheets are consisting of synthetic fibers along with resinous matrix which can be applied to any concrete sections. There are many advantages to this method. CFRP lightweight enables installation duration to be accomplished quickly with less labor expenses and service disruption of the building. This material also shows resistance to corrosion in chloride environments which can possibly leads to reduction in maintenance cost. The retrofit of vulnerable reinforcement concrete columns with CFRP will provide more ductility and increase energy dissipation capacities substantially along with vast improvement in terms of total seismic resistance. The confinement provided by CFRP jacketing technique can cover the disadvantages of insufficient steel rebars and also ramp up shear and moment capacities as the jacketing transforms column response from brittle and non-ductile action to a more ductile response. CFRP retrofit technique substantially illustrates ductility enhancement and improved seismic behavior in comparison to previously damaged columns while the level of improvements is completely correlated to the intensity of damages. As the level of defectiveness grows, more CFRP layers are required in order to achieve a performance similar to undamaged retrofitted columns. CFRP jackets will be mostly provided for columns because of maintaining the discontinuity between columns and footings or beam-column joints which strengthen the column sections and shift plastic hinge zones away from the interface to sections with lower stiffness capacities such as beams.

CFRP confining of RC columns has received considerable attention for use in feeble structures due to its unique properties, such as high strength-to-weight ratio, stiffness-to-weight ratio, as well as corrosion and fatigue resistance. CFRP increases both ultimate strength and ductility of RC members. Besides, retrofitting techniques using CFRP-confined columns are sometimes directed at increasing flexural strength, when necessary. Thus, the retrofitting method should be used carefully because increasing flexural capacity increases the forces transferred to the foundation and the connections between the structure/column.

Tyfo CFRP composite is one of the known brands in composite products for structural strengthening and retrofitting. Implementing the CFRP is generally simpler than other retrofitting methods. For any member that requires CFRP wrapping, CFRP can be applied on a thin epoxy layer which has been already coated over the surface of the member under consideration. The surface must be clean, dry and free of protrusions or cavities, which may cause voids behind the CFRP composite.

The process of the modeling in CFRP in ETABs is similar to steel jacketing method. The main difference would be the specific material that needs to be defined in program in order to mimic the CFRP Tensile strength, Tensile Modulus, Density and Poisson's Ratio. In this study, the directional material type has been selected as isotropic which implies the material is uniformity in all orientations. A trial-and-error procedure should be taken into account to identify minimum quantity of columns that requires CFRP wrapping.

4.6 Working example in Structural strengthening techniques

In this section, a case study will be taken to strengthen the existing concrete building structures by confining reinforced concrete columns with steel jackets. This will increase the load capacity of the structural elements.

The concrete columns are strengthened by 5 mm steel jackets on all four sides of the column under consideration (see Figure 49). The jacketing arrangement is fully anchored to concert columns by 25mm anchor bolts.

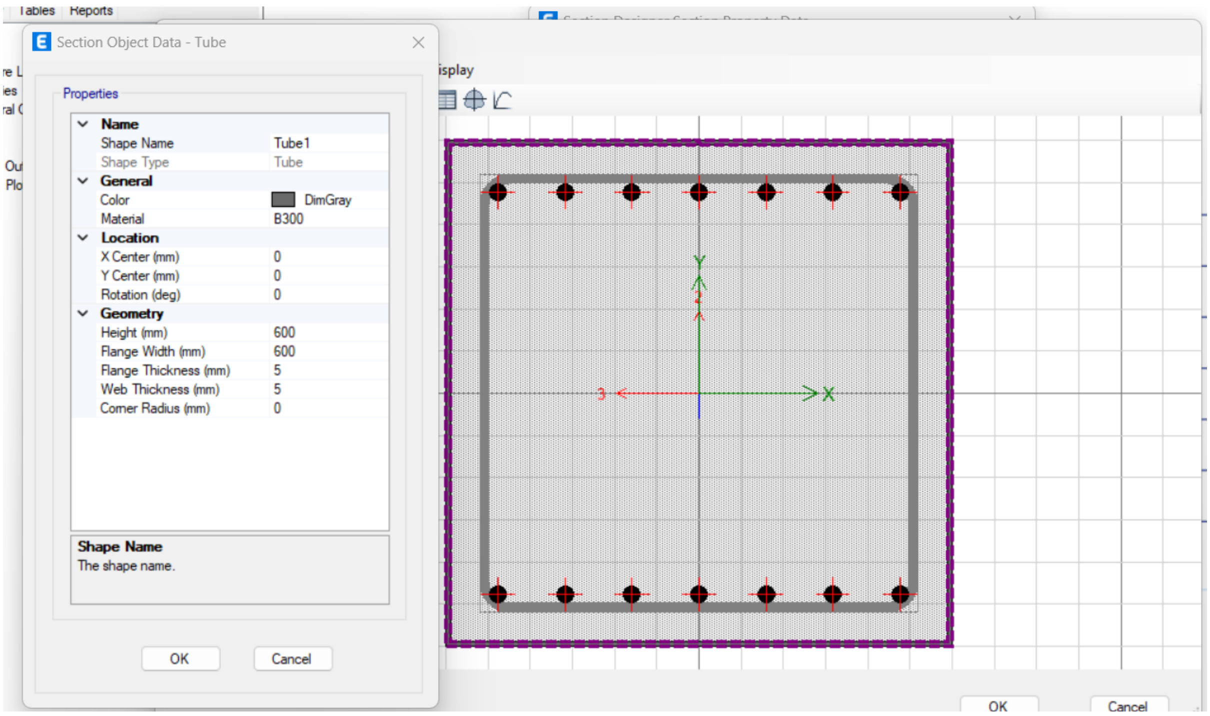


Figure 49 - Typical Steel Jacket Detail for RC Columns

Two steps have been taken to identify the actual quantity of columns that have need of retrofitting procedure. In first try, four columns on each corner received full steel jacketing for all stories as it is shown in Figure50, even after 1st trial, the building suffers from low performance in second floor at the base level Second trial, 8 more columns in the middle added to the process as it is shown in Figure 51. Once the second trial took place which made, the building performance reached to CP level which is acceptable for the building.

The graphical results of the modeling have been displayed in Figure 52&53. It is clear that all the hinges throughout the columns meet CP level of performance.

The normalized base shear-top displacement relationships obtained by pushover analysis for retrofitted structure by steel jacketing method of rehabilitation are presented in Figure 54.

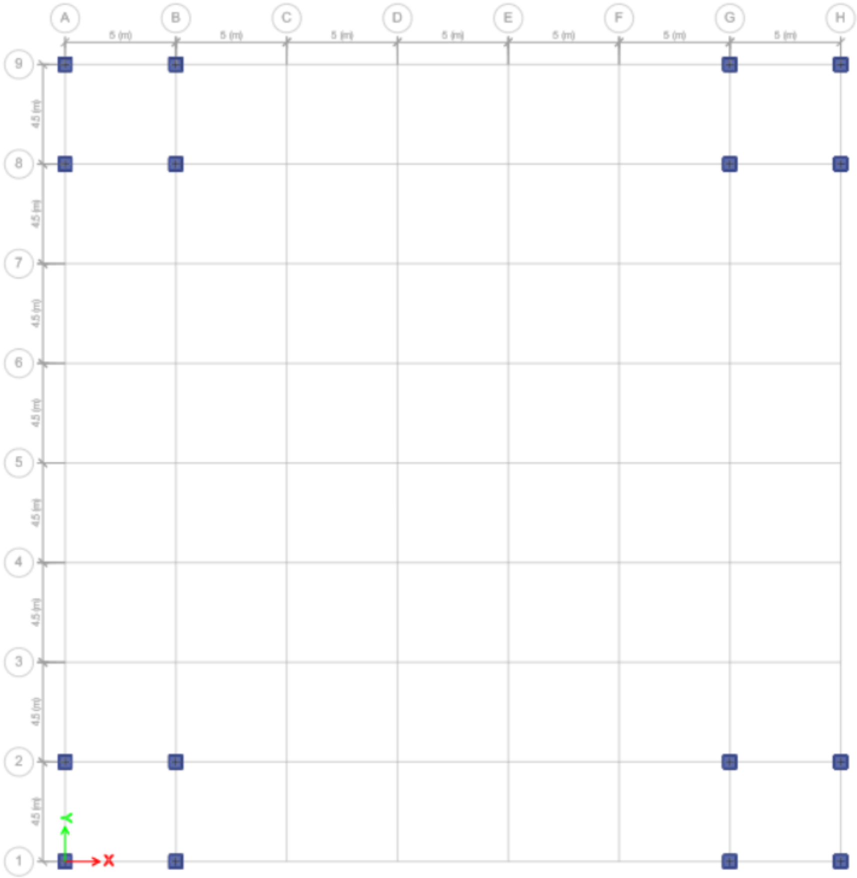


Figure 50 – 1st Trial and Error Process

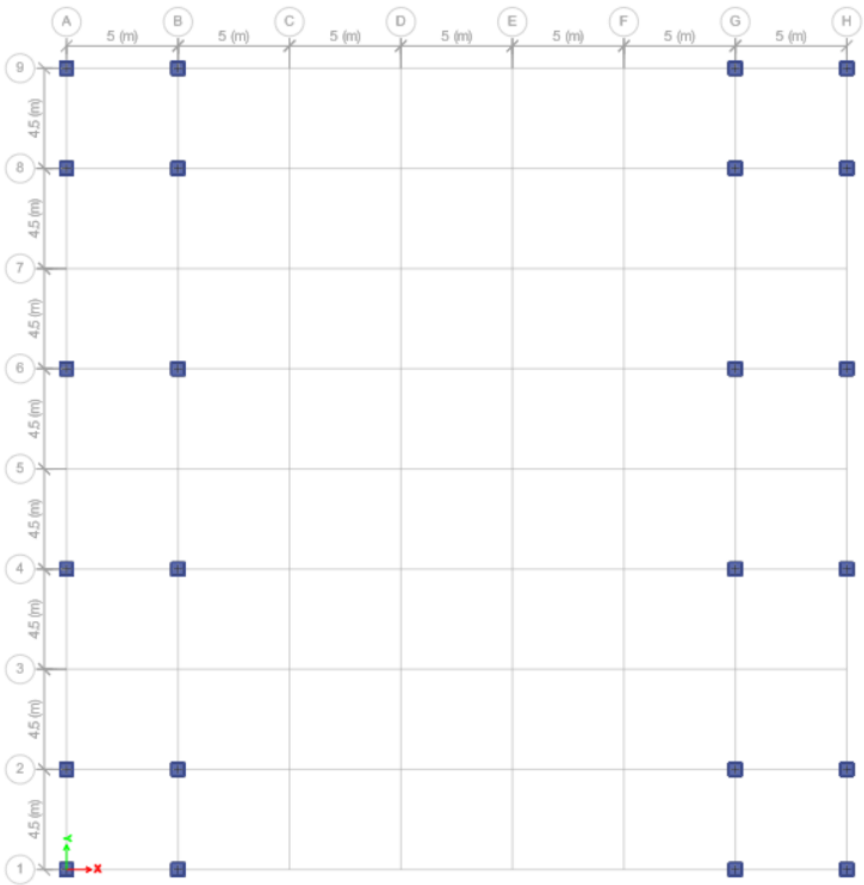


Figure 51 – 2nd Trial and Error Process

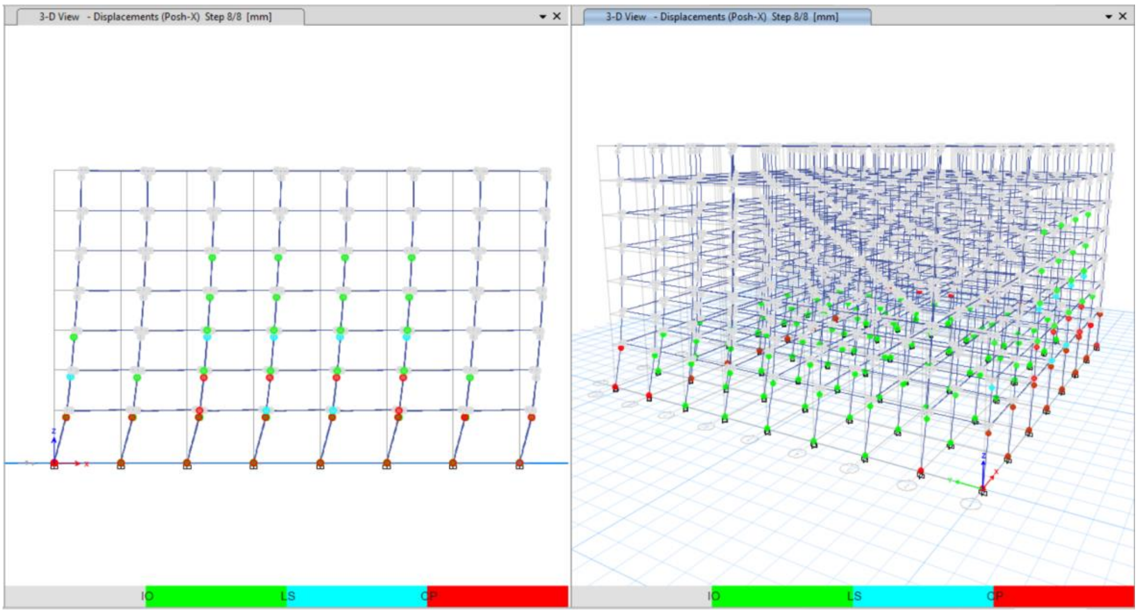


Figure 52 – 1st trial Deformed shape as per Push -X

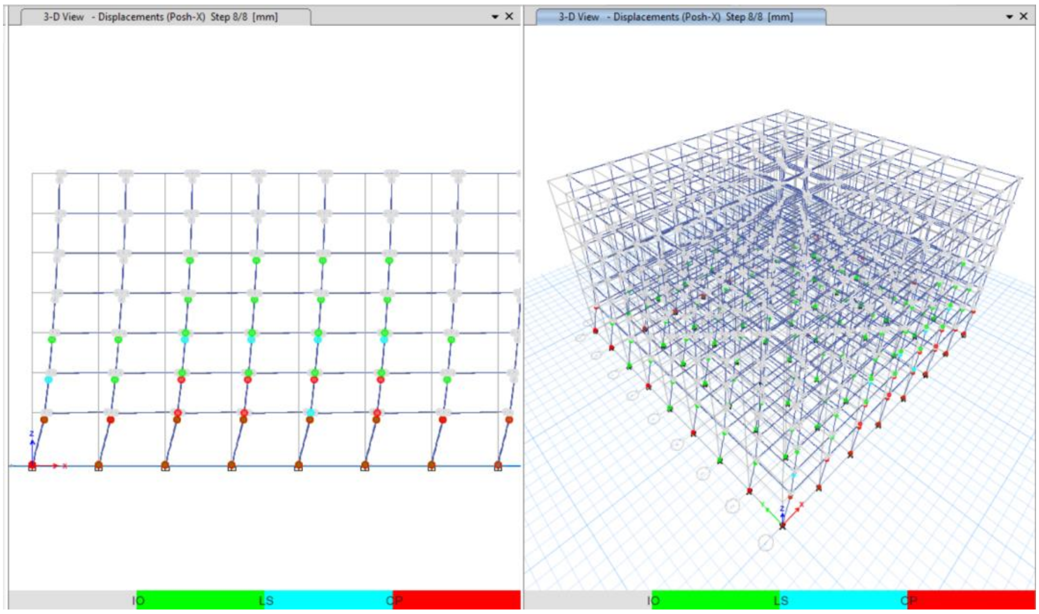


Figure 53 – 2nd trial Deformed shape as per Push -X

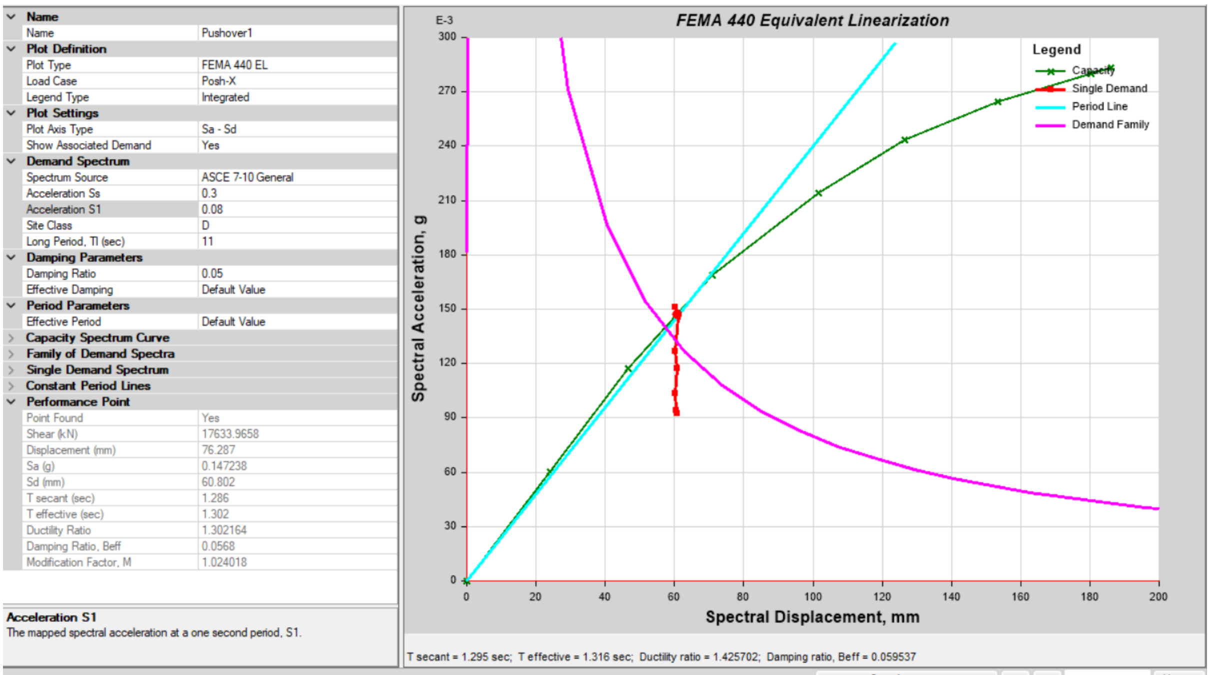


Figure 54 – FEMA 440 Pushover curve for Retrofitted Structure by Steel Jacketing

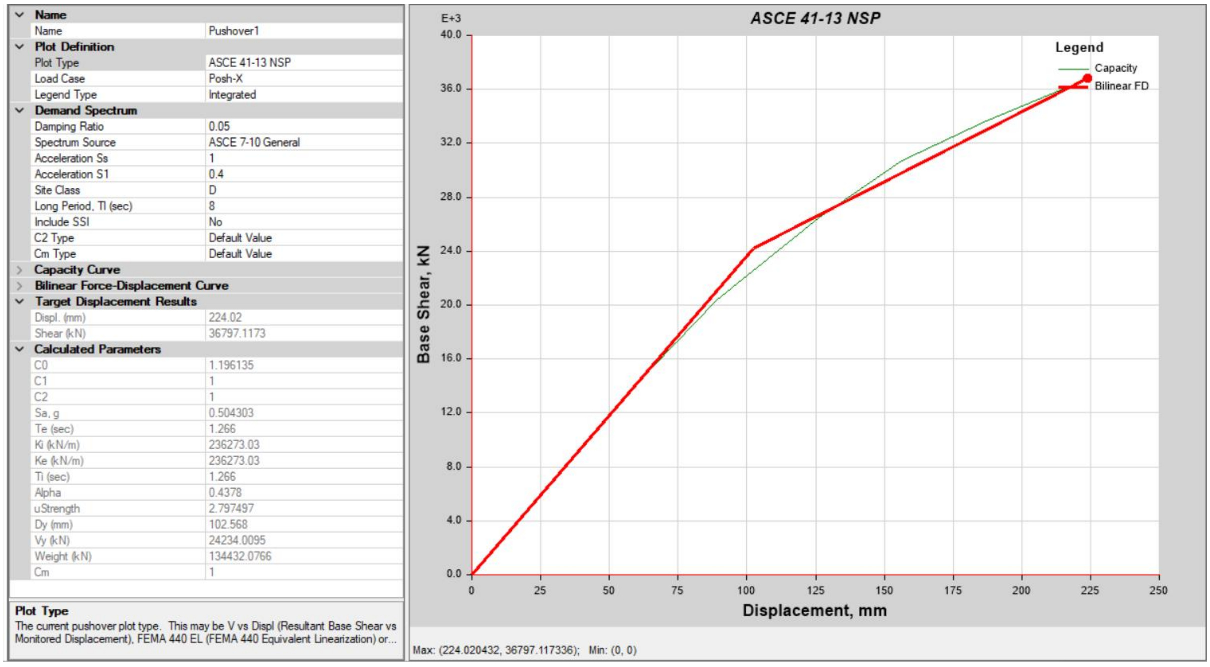


Figure 55 – ASCE 41-13 Pushover curve for Retrofitted Structure by Steel Jacketing

5 Chapter 5: Conclusions

5.1 Conclusion

- The performance of reinforced concrete building was investigated using the pushover Analysis from which the following conclusions can be drawn:
- The main output of a pushover analysis is in terms of response demand versus capacity. If the demand curve intersects the capacity envelope near the elastic range, then the structure has a good resistance. If the demand curve intersects the capacity curve with little reserve of strength and deformation capacity, then it can be concluded that the structure will behave poorly during the imposed seismic excitation and need to be retrofitted to avoid future major damage or collapse.
- The pushover analysis is a simple way to explore the nonlinear behavior of building.
- The RC bare frame which is analyzed for the static nonlinear pushover cases, 7 story frame can Carry higher base force and at lower displacement it fails

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