PUSHOVER ANALYSIS OF REINFORCED CONCRETE FRAMES DESIGNED AS PER ALBANIAN CODES OF PRACTICE

By:

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Thesis Submitted to the Faculty of Architecture and Engineering, EPOKA University, in the Fulfillment of the Requirement for the Degree of Master of Science

To my parents…

Abstract of thesis presented to the Administrative Board of EPOKA University in the Fulfillment of the Requirement for the Degree of Master of Science

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Abstract

Nonlinear static analysis has recently become one of the most commonly used analysis procedures for design and seismic performance evaluation of buildings, due to its simplicity and practicality. One of the most important reasons seismic evaluation of an existing building may be needed, is because the structure that is being evaluated has not been designed in compliance to the modern building codes, or an upgrade has been made to the existing codes of design.

In this study, a pushover analysis is performed for an existing reinforced concrete frame building, built in Albania around the year 1999. The building is designed based on the latest Albanian code for seismic design, KTP-1989. The code has not been revised since the time it was put into practice, but it is expected to be subjected to certain alterations in the near future, with the purpose of adopting the Eurocode requirements.

Eleven theoretical cases are defined, taking into consideration the minimum and maximum values of steel percentage in the reinforced concrete members, several combinations of limit values (min, max) for steel and concrete strength based on the code, different plastic hinge properties, and the implementation of steel yield strengths obtained by tensile test results of steel commonly used for reinforced concrete buildings in Albania.

The pushover analysis is carried out in Sap2000. Observations show that an increase in steel percentage, or an increase in concrete and steel strength results in significantly higher overall capacity of the structure. Also, a combination of low concrete strength with high steel strength, results in a higher overall capacity, than a high concrete-low steel combination. Furthermore, the analysis shows that the user-defined hinge model is more successful in capturing the nonlinear behavior of the structure than the default hinge model.

The pushover analysis of the frame with steel obtained from the tensile tests, indicates that, although the building is able to withstand the level of design earthquake, the examination of test results shows that there isn’t yet a standard and a control over the steel used for reinforced concrete structures.

Keywords*: Pushover analysis, Material properties, Plastic hinge properties, Albanian seismic code, Sap2000*

Abstrakti i tezes paraqitur Bordit Administrativ te Universitetit EPOKA ne permbushje te

kerkesave per Master Shkencor

ANALIZE JOLINEARE STATIKE E STRUKTURAVE BETONARME PROJEKTUAR SIPAS KODEVE SHQIPTARE NE PRAKTIKE

Nga:

Arba ÇOLLAKU

Korrik 2015

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Abstrakti

Analiza jolineare statike është bërë së fundmi një nga metodat më të përdorshme, për projektim dhe vlerësim të performancës sizmike te ndërtesave, për shkak të thjeshtësisë dhe prakticitetit të saj. Një nga arsyet më të rëndësishme pse një vlerësim sizmik i një ndërtese ekzistuese mund të nevojitet, është se struktura që po vlerësohet nuk është projektuar në përputhje me kodet moderne të ndërtimit, ose një modifikim i është bërë kodeve në fuqi.

Në këtë studim, një analizë jolineare statike ëshë kryer për një ndërtësë ekzistuese betonarme të tipit ramë, të ndërtuar në Shqipëri rreth vitit 1999. Ndërtesa është projektuar bazuar në kodin shqiptar të projektimit antisizmik, KTP-1989. Ky kod, nuk është rishikuar që prej kohës që është vendosur në përdorim, por pritet që ti nënshtrohet ndryshimeve të caktuara në të ardhmen e afërt, me qëllimin që të adoptojë kërkesat e Eurokodit.

Njëmbëdhjetë raste teorike janë përcaktuar, duke marë në konsideratë vlerat minimale dhe maksimale të përqindjes së çelikut ne elementet betonarme, kombinime të ndryshme të vlerave limit (min, max) të rezistencës së çelikut dhe betonit bazuar në kod, veti të ndryshme të çernierave plastike, dhe përfshirjen e vlerave të rezistencës së celikut marë nga provat në tërheqje të çelikut të përdorur zakonisht në ndërtesat betonarme në Shqipëri.

Analiza jolineare statike është kryer në Sap2000. Vëzhgimet tregojnë se një rritje në përqindjen e armimit ose një rritje e rezistencës së betonit apo çelikut, rezulton në një kapacitet më të lartë sizmik të strukturës. Gjithashtu një kombinim i një betoni me rezistencë të ulët me një çelik të një rezistence të larte, rezulton ne një kapacitet më të lartë në tërësi, sesa një kombinim i një betoni me rezistence të lartë dhe një çeliku me rezistencë te ulët. Më tej, analiza tregon se, modeli i çernierave i përcaktuar nga përdoruesi, është më i suksesshëm në kapjen e sjelljes jolineare të strukturës sesa modeli i çernierave "default".

Analiza jolineare statike e ramës me çelik të marë nga provat në tërheqje, tregon se, edhe pse ndërtesa është në gjendje ta përballojë tërmetin e projektimit, egzaminimi i rezultateve të testeve tregon se në Shqipëri nuk ka akoma një standart dhe kontroll mbi çelikun që përdoret në strukturat prej betoni të armuar.

Fjalë kyçe: *Analizë jolineare statike, Veti të materialeve, Veti të çernierave plastike, Kodi sizmik shqiptar, Sap2000*

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Last, but not least, I would like to heartily thank my family, for their unconditional love and support. They are the biggest motivation behind my every achievement.

Arba ÇOLLAKU

APPROVAL SHEET

I certify that an Examination Committee has met on date of viva to conduct the final examination of Arba Çollaku, student of Master of Science (MSc) in Civil Engineering, thesis entitled "**Pushover Analysis of Reinforced Concrete Frames designed as per Albanian Codes of Practice"** in accordance with *Epoka University (Higher Degree) Regulation "On second cycle study programs"*. The Committee recommends that the candidate be awarded the relevant degree. Members of the Examination Committee are as follows:

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DECLARATION

I hereby declare that the thesis is based on my original work except for quotations and citations which have been duly acknowledged. I also declare that it has not been previously or concurrently submitted for any other degree at Epoka University or other institutions.

ARBA ÇOLLAKU

Date: 08 July 2015

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Chapter 1

Introduction

1.1 General

Nowadays, several methods of analysis, both linear and nonlinear, are available for analysis of existing or new reinforced concrete buildings. Linear procedures are practical, but they have several limitations when it comes to providing information on certain response characteristics. Although an elastic analysis gives good results on the elastic capacity of structures and shows where first yielding will occur, it cannot predict failure mechanisms and redistribution of forces during progressive yielding (2) (34).

Nonlinear static analysis, also known as pushover analysis, has been studied, developed and improved over the course of the past years, and has recently become one of the most popular analysis procedures for design and seismic performance evaluation. Compared to the nonlinear time history analysis, which is considered as overly complex and unpractical for general use, pushover analysis is relatively simple and it is expected to provide valuable insight of the structure's behavior after exceeding the elastic limit (27).

This type of nonlinear static analysis, consists of a series of consecutive elastic analysis, superimposed to approximate a force-displacement curve of the overall structure. The structure is subjected to monotonically increasing lateral forces, in a predefined load pattern, which is distributed along the building height, until a target displacement is reached or the building becomes unstable. The base shear-roof displacement relationship is then plotted to get the global capacity curve. Pushover analysis there by, evaluates the seismic performance of the structure and quantifies its behavior characteristics, like strength, stiffness and deformation capacity under design ground motion (44) (50) (55).

The current codes of practice in Albania for seismic design, have been revised for the last time in 1989, which means that most of the existing reinforced concrete buildings have been designed based on this code. Consequently, a large amount of the buildings in Albania, do not comply with modern building codes and may need to go through a seismic retrofitting process to withstand future earthquakes.

Pushover analysis is a very practical method of analysis for design and seismic performance evaluation of buildings, but it is not yet a well-known and commonly used analysis method in Albania, even though most of the existing buildings need to be subjected to a seismic evaluation procedure.

1.2 Problem Statement

After the year 1944, reinforced concrete became the base material to buildings in Albania, whether they were residential, industrial, bridges, etc. (31), all of them designed based on the Albanian codes. The most recent code for seismic design in Albania is *KTP-N2-1989*, which has not gone under any revision or modification since the time it was put into practice. This means that most existing reinforced concrete buildings do not conform to modern code requirements, and may not be able to resist future earthquakes. Also, considering that not many studies have been made in the seismology field concerning Albania in particular, reinforced concrete structures in Albania pose great uncertainties in a case of seismic event.

Another issue on the other hand, are the materials used in these buildings. There are existing buildings that do not comply with the material strength requirements defined in the Albanian codes. The materials used in the new buildings are not usually tested, which means that there is no strict control over the materials used in reinforced concrete buildings in Albania.

This study aims to address the problems mentioned above, with the pushover analysis of an existing building in Albania.

1.3 Thesis Objective

The main objectives of this study are:

i. To test the limit values of steel percentage in reinforced concrete members, according to the Albanian code, with a pushover analysis of an existing building, by also pointing out the effects the steel to concrete ratio has on pushover analysis.

ii. To test the limit values of concrete and steel strength, according to the Albanian code, with a pushover analysis of an existing building.

iii. To point out the differences in pushover analysis results, because of distinctive material strengths in several combinations.

iv. To reveal the importance of using user-defined hinge properties when performing pushover analysis.

v. To incorporate real values of characteristic steel yield strength, obtained by tensile tests of steel commonly used in Albania, in a pushover analysis of an existing building.

1.4 Scope of work

The focus of this study is the nonlinear static analysis of an existing reinforced concrete building in Albania, designed based on *KTP-N2-1989*. The pushover analysis is carried out in Sap2000 to reveal the structure's overall capacity and behavior during a seismic event:

i) For different steel percentages in the reinforced concrete members, with values predefined in the current Albanian Code of seismic design

ii) For different concrete and steel strengths obtained by site tests and KTP limits, in various combinations

iii) For user-defined and default hinge properties

1.5 Thesis structure

This study is organized in six chapters, each of them described briefly as follows:

Chapter 1 serves as an introduction, providing an idea about the topic and presenting the main problems being addressed in the study. Lastly, it lists the objectives and the scope of work covered in this research.

Chapter 2 discusses the literature review and theoretical background of the topic, focusing on work done by previous researches and on modern codes guidelines concerning pushover analysis.

Chapter 3 presents the methodology used in this study. Pushover analysis is further discussed and explained, and user-defined hinge properties are presented.

Chapter 4 presents the building details and the cases taken into consideration to fulfill the objectives of this study. The procedure of modelling in Sap2000 is explained.

Chapter 5 discusses the results obtained by the pushover analysis in all the cases and the possible differences and comparisons between them.

Chapter 6 discusses the conclusions obtained by the study.

Chapter 2

Literature Review

2.1 Seismic Evaluation of Reinforced Concrete Structures

Reinforced concrete is one of the most widely used building materials in the construction industry. From the time it became a proven scientific method (around 1877), the knowledge about reinforced concrete has been rapidly developing, along with the techniques and methodologies, making it one of the most effective, practical and common building materials.

Concrete is a heterogeneous, brittle material, with good properties in compression and mostly weak properties in tension. Concrete members that do not have any type of reinforcement in them will typically fail unexpectedly once the first tension cracks form because there is nothing to prevent the cracks from advancing completely through the member (41). Reinforcing steel is used to improve the performance of concrete members under applied loads, mostly the tensile strength and ductility. Concrete and reinforcing steel are put together to improve each-other properties in an efficient, safe and economical manner (43).

Reinforced concrete is and will continue to be one of the most popular building materials for its practicality and efficiency, therefore it is very important to develop analyzing methods that will complete and expand the knowledge and understanding of the material response under a combination of different loads.

One of the biggest concerns related to structures in general, and in this case reinforced concrete structures is their performance during earthquakes. Seismic design and evaluation of reinforced concrete structures presents a great challenge for the engineering community. The risks measured in lives and money are very high, as are the uncertainties of when, where and how large future earthquakes will occur. The complexity of reinforced concrete buildings and their performance during earthquakes is also an uncertainty in its own.

One of the most important reasons seismic evaluation is developed, studied and needed is seismic retrofitting of existing buildings. Seismic retrofitting consists in upgrading the strength of an existing structure with the aim to increase it's a capacity to withstand future earthquakes. The main causes a seismic retrofitting of an existing building may be required are (3):

- The existing structure has not been designed in compliance to the current building codes
- An upgrade has been made to the seismic design code
- An upgrade has been made to the seismic zone the structure belongs to
- Deterioration of strength because of the ageing of the structure
- Modification of the existing structures affecting the building's strength in a harmful way
- Change in the use of the building increasing the floor loads

The predominant cause of earthquake damage to buildings is ground shaking, along with ground failure hazard (liquefaction, land-sliding or surface rupture).

Seismic ground shaking is defined using site soil factors and other terms developed by the Seismology Committee. To quantify the seismic hazard due to ground shaking, three earthquake hazard levels are distinguished (2):

- The Serviceability Earthquake (SE)
- The Design Earthquake (DE)
- The Maximum Earthquake (ME)

The Serviceability Earthquake (SE) is defined probabilistically as the level of ground shaking that has a 50 percent chance of being exceeded in a 50 year period. This level of earthquake ground shaking is typically about 0.5 times the level of ground shaking of the Design Earthquake. The Serviceability Earthquake has a mean return period of approximately 75 years (2).

The Design Earthquake (DE) is defined probabilistically as the level of ground shaking that has a 10% chance of being exceeded in a 50 year period. The DE has a mean return period of approximately 500 years. The Design Earthquake has the same definition as the level of ground shaking currently used as the basis for the seismic design of new buildings.

The Maximum Earthquake (ME) is defined deterministically as the maximum level of earthquake ground shaking which may ever be expected at the building site within the mown geologic framework (5% chance of being exceeded in 50 years). This level of ground shaking is typically about 1.25 to 1.5 times the level of ground shaking of the Design Earthquake.

FEMA 273 (1997) on the other hand uses two levels of earthquake shaking hazard to satisfy the basic safety objective BSO, which are:

- Basic Safety Earthquake 1 (BSE-1): has a 10% probability of exceedance in 50 years with a mean return period of approximately 474 years
- Basic Safety Earthquake 2 (BSE-2): has a 2% probability of exceedance in 50 years with a mean return period of approximately 2475 years

To address the seismic design and evaluation of reinforced concrete buildings a considerable number of general methodologies are developed. Promising new performance-based technical procedures can provide engineers with valuable insight about the actual performance of buildings during earthquakes, however it is important to emphasize, that straightforward, simple solutions that will cost-effectively produce acceptable seismic performance for all buildings do not exist (2) (21).

For many years engineers have been using unrealistic simplified static lateral force procedures to design buildings to resist seismic forces and displacements, while these traditional methods can result in an acceptable design, they mostly conceal the actual structural behavior and seismic performance of structures (2) (21) (22).

The procedure for evaluation and retrofit design proposed by ATC 40 consists of the following steps (however, according to the structure being studied some steps may be performed in a different order or de-emphasized):

- To establish seismic performance objectives, seismic performance levels and seismic demand criteria should be defined.

- Review of existing conditions of the structure, preliminary determination of deficiencies, and formulation of a valid retrofit strategy.

- Analytical methods for detailed investigations to assess seismic capacity and expected seismic performance of existing buildings and for verification of retrofit performance.

- Materials characteristics rules and assumptions for use in modeling, assignment of capacities, and assessment of acceptable performance.

2.2 Seismic Performance Levels

As stated above, one of the most important steps of a seismic evaluation procedure is establishing the seismic performance objectives. A performance objective specifies the desired seismic performance of the building. Seismic performance is described by designating the maximum allowable damage state (performance level) for an identified seismic hazard (earthquake ground motion).

The performance level of a structure is defined as a limiting damage state or condition described by the physical damage within the building, the threat to life safety of the building's occupants due to the damage, and the post-earthquake serviceability of the building (2).

A performance range on the other hand, includes a band or range of performance, rather than a discrete level. A building performance level is the combination of a structural performance level and a nonstructural performance level.

Three Structural Performance Levels (two performance ranges), and four Nonstructural Performance Levels are used to form the four basic Building Performance Levels. These performance descriptions are estimates rather than predictions, and sometimes a variation among buildings of the same Performance Level can be expected.

The three Structural Performance Levels and two Structural Performance Ranges consist of (2) (4) (21).

- Immediate Occupancy Performance Level S-1: 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 preserve 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 some minor structural repairs may be required.

- Damage Control Performance Range S-2 (extends between Life Safety and Immediate Occupancy Performance Levels): The continuous range of damage states that result in less damage than that defined for the Life Safety level, but more than that defined for the Immediate Occupancy level. 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.

- Life Safety Performance Level S-3: The post-earthquake damage state in which significant damage to the structure has occurred, but the structure has still not totally collapsed. While injuries during the earthquake may occur, the risk of life-threatening injury from structural damage is very low. It should be expected that extensive structural repairs will likely be necessary prior to reoccupation of the building, although it may not be economical to repair the damage.

- Limited Safety Performance Range (extends between Life Safety and Collapse Prevention Performance Levels) S-4: 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.

- Collapse Prevention Performance Level S-5 (or Structural Stability as mentioned in *ATC 40, 1996a*): 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. However, all significant components of the gravity load resisting system continue to carry their gravity load demands. Significant risk of injury due to falling hazards may exist both within and outside the building. Major structural repair will be necessary prior to re-occupancy, however in most cases the damage will not be technically or economically repairable.

In addition, there is the designation of Structural Performance Not Considered S-6, which represents situations where only nonstructural seismic evaluation or retrofit is performed.

The four nonstructural performance levels are discrete damage states and can be used in evaluation and retrofit procedures to define technical criteria.

- Operational performance level N-A: The post-earthquake damage state of the building in which most nonstructural systems required for normal use of the building including lighting, plumbing, etc., are functional, although minor repair of some items may be required.
- Immediate occupancy level N-B: The post-earthquake damage state in which limited nonstructural damage has occurred. Basic access and life safety systems, including doors, stairways, elevators, fire alarms etc., remain operable.
- Life safety level N-C: The post-earthquake damage state in which potentially significant and costly damage has occurred to nonstructural components but they are not threatening life safety either within or outside the building.
- Reduced Hazard level N-D: The post-earthquake damage state level in which massive damage has occurred to nonstructural components, but large or heavy items that pose a falling hazard to people such as parapets, heavy plaster ceilings, or storage racks are prevented from falling.

In addition, there is the designation of N-E, Nonstructural Performance Not Considered, to cover the situation where only structural improvements are made.

Building Performance Levels are obtained by combining Structural and Nonstructural Performance Levels. A large number of combinations is possible. Each Building Performance Level is designated with a number representing the Structural Performance Level and with a letter representing the Nonstructural Performance Level. The four most commonly used Building Performance Levels are:

- Operational Level 1-A
- Immediate Occupancy Level 1-B
- Life safety Level 3-C
- Collapse Prevention Level 5-E

2.3 Nonlinear Analysis Procedures

An analysis of the structure shall be conducted to determine the distribution of forces and deformations caused to the structure by the design ground shaking and other seismic hazards corresponding with the selected rehabilitation objectives (2) (4) (15) (21).

The seismic action effects, combined with the effects of the other permanent and variable loads in accordance with the seismic load combination, may be evaluated using several analysis methods, generally divided in two main categories: linear and nonlinear analysis.

Linear procedures, such as lateral force analysis, modal response spectrum analysis, and linear time history analysis, are practical but have a lot of limitations (all of them are stated in the design

Codes mentioned above). Although an elastic analysis gives a good indication of the elastic capacity of structures and shows where first yielding will occur, it cannot predict failure mechanisms and redistribution of forces during progressive yielding (2).

The results of the linear procedures can be inaccurate when applied to buildings with highly irregular structural systems, unless the building is capable of responding to the design earthquake in a nearly elastic manner (4) (21).

Nonlinear analysis procedures identify the modes of failure, the potential for progressive collapse in a building and are especially recommended for analysis of buildings with irregularities. Reinforced concrete structures subjected to major earthquakes generally exceed their elastic capacity, hence the use of nonlinear procedures resolves some of the uncertainties associated with elastic procedures. There are two types of nonlinear analysis:

- Nonlinear Static Analysis (NSP): Typically known as pushover analysis, suitable for buildings without significant higher-mode response.
- Nonlinear Dynamic Analysis (NDP): Nonlinear time history is a basic NDP, but it is considered a complex and impractical method for general use.

2.4 Nonlinear Static Pushover Analysis

Pushover analysis accounts for inelastic behavior of building models and provides reasonable estimates of deformation capacity while identifying critical sections likely to reach limit state during earthquakes (10).

Helmut K. et al. (1997) described the seismic design as a process that contains two steps. The first and the most important one being the conception of an effective structural system that needs to be configured in compliance with all important seismic performance objectives, ranging from serviceability considerations to life safety and collapse prevention. The second step being the building of elaborate mathematical and physical models that are needed to evaluate seismic performance of an existing system and to modify component behavior characteristics (strength, stiffness, deformation, capacity) to better suit the specified performance criteria. The nonlinear static pushover analysis is becoming a popular method for seismic performance evaluation of existing and new structures. The expectation is that the pushover analysis will provide adequate information on seismic demands imposed by the design ground motion on the structural system and its components. The purpose of this study is to summarize basic concepts on which the pushover analysis can be based, and to point out the pros and cons of using pushover analysis for a seismic performance evaluation. The purpose of the pushover analysis is to evaluate the expected performance of a structural system by estimating its strength and deformation demands in design earthquakes with the help of a nonlinear static analysis, and comparing these demands to available capacities at the performance levels of interest. This nonlinear static method of analysis provides information on many response characteristics that cannot be obtained from an elastic static or dynamic analysis. On the other hand, these benefits come at the cost of additional analysis effort, associated with incorporating all important elements, modeling their inelastic load-deformation characteristics, and executing incremental nonlinear analysis with the analytical model. A carefully performed pushover analysis will provide useful information about structural aspects that control performance during severe earthquakes. For structures that vibrate primarily in the fundamental mode, such an analysis will very likely provide good estimates of global as well as local inelastic deformation demands. It will also expose design weaknesses that may not be discovered in an elastic analysis. On the other hand, a disadvantage might be that the deformation estimates obtained from a pushover analysis may be inaccurate for structures in which higher mode effects are significant and in which the story shear force vs. story drift relationships are sensitive to the applied load pattern.

Akanshu Sharma et al. (2012) presented an experimental and numerical study carried out on a full-scale four storey reinforced concrete structure for seismic assessment by pushover analysis. For the experimental setup, only a part of the existing structure was replicated for practical reasons. The structure was designed based on non-seismic reinforcement detailing norms of Indian Standards. The experiment was carried out as a round robin exercise, in which various institutes in India participated and presented pre-test results in the form of pushover curves. The numerical results obtained by the participants are then compared with experimental results. The experiments on full-scale real life structures are the best way to study their behavior under lateral seismic loading, and on the mean time providing useful results that can be used to form a database to confirm the analysis procedures that are being used, for future studies. Tests on full-scale structures under pseudo-dynamic loads, pushover loads, and cyclic loads have been attempted in the past, but because the large costs, time and effort needed, only a few of them have been successful.

In this study, such an experiment was tried, where a 3D full scale structure with four stories and one bay along both horizontal directions, was loaded under monotonically increasing lateral pushover loads. The participants, from different academic and research institutes that were involved in the round robin exercise, used the conventional pushover method and modeled the structure using frame elements. A summary of the calculations followed by a comparison of the results in the form of base-shear vs. roof displacement curves are shown in the figure below:

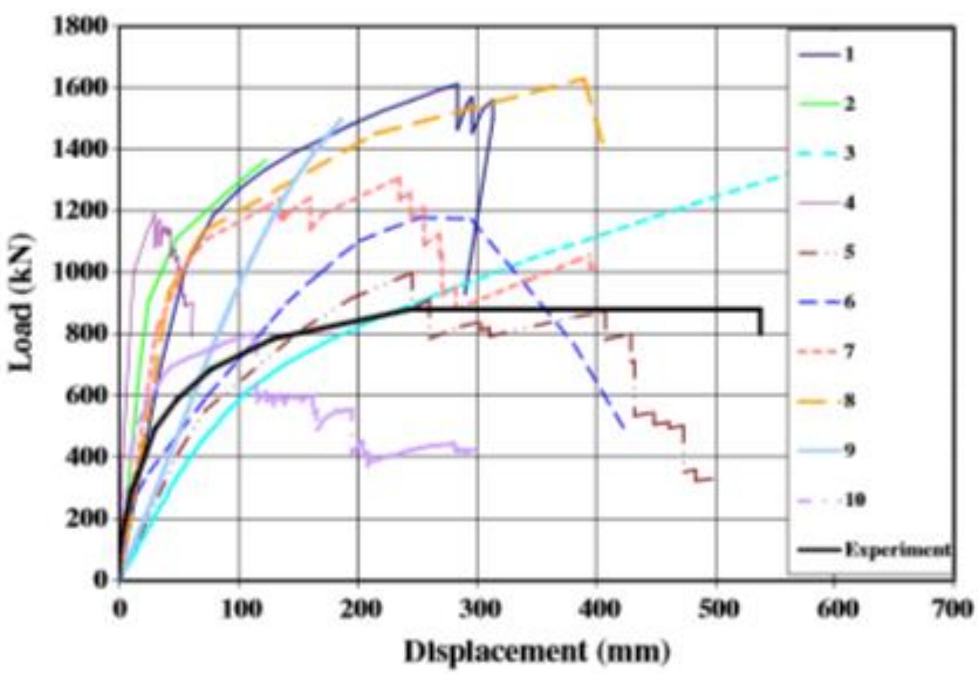


Figure 2.1: *Analysis results submitted by participants of round robin exercise* (54)

As it can be seen from the comparison of results in the figure above, participants submitted significantly different results with expected base shear values ranging from 800 KN to 1600 KN and the roof displacement values ranging from 60 mm to 600 mm. The large variation in the pre-test results points out that the result of a pushover analysis is highly sensitive to the design assumptions of the various approaches. After the experiment a pushover analysis of the structure was performed by the authors, taking into consideration the modelling parameters which were influenced by experimentally observed failure modes.

Dhileep M. et al. (2011) conducted a research with the purpose of explaining the behavior of high frequency modal responses in nonlinear seismic analysis. Pushover analysis has become a common practice the last several years due to its simplicity, but as most analysis methods it has its limitations, one of them being the uncertainties about the contribution of higher modes in pushover analysis. In typical regular structures only a few lower order modes are sufficient to evaluate the total response with reasonable accuracy. Structures that are stiff and/or irregular on the other hand require that high frequency modes and nonlinear effects are taken into account, because they significantly contribute in the seismic analysis. With the purpose of considering the response contributions of higher modes, a modal pushover analysis has been proposed by several previous studies (9) (51).

The research concludes that nonlinear response of structures with high frequency modes of vibration can be evaluated using a nonlinear static pushover analysis with an incremental force pattern given by their modal mass contribution times zero period acceleration. The higher modes with rigid content as a major contributing factor display a better accuracy in nonlinear pushover analysis of structures when compared to the damped periodic modes.

Sofyan Y. Ahmed (2013) analyzed a ten stories, five bays reinforced concrete frame subjected to seismic hazard, in the city of Mosul, Iraq. A pushover analysis is carried out on the building to check its performance under seismic effects. Lateral deformations at the calculated performance point proved that the building is capable of sustaining certain level of seismic load.

The expected seismic performance of a building can be easily calculated with the help of computer software which use nonlinear static analysis and with the Performance Based Seismic Engineering, where inelastic structural analysis is combined with seismic hazard assessment. In this study the nonlinear response of the structure taken into consideration is evaluated, under lateral static loads, equivalent to expected seismic loads, directly applied to the joints of the building frame. The results show that potential structural deficiency in the reinforced concrete frame were estimated by the nonlinear pushover procedure, when subjected to a moderate seismic loading. The analysis showed that the frame is capable of withstanding the assumed seismic force with some significant yielding at several beams. The formation of plastic hinges in the frame members can be seen in the beams only, which means that the building behaves like a strong column-weak beam mechanism. All the plastic hinges formed in the beams are located in the Collapse Prevention Performance Level, so strengthening of the beams is demanded. Maximum total drift, maximum inelastic drift, and structural stability do not exceed the limitations of the performance level, therefore the studied building is considered safe against seismic loads.

Ioannis P. Giannopoulos (2009) conducted a comparative research on the seismic assessment of a reinforced concrete building according to *FEMA 356 (2000)* and *Eurocode 8*. Nonlinear methods of analysis predict directly the amount and location of plastic yielding within a structure, which makes them a more efficient procedure than elastic analysis reduced by a ductility factor. Nonlinear static (pushover) analysis is a commonly used technique, which is becoming important in standards and guidance materials like *Eurocode 8* and *FEMA 356 (2000)*. The purpose of the paper is to compare the methods given by these two documents. A nonlinear static (pushover) analysis is carried out for an existing five-story reinforced concrete frame, which has been designed for moderate seismicity according to the past generation of Greek seismic codes, using SAP2000. The outcome of this study is to provide useful information for further development of *Eurocode 8*. The comparison demonstrates that there are differences in the results produced by the two approaches.

From the curves of hinge plastic rotation supply for *FEMA* and *EC8* it is observed that for beams the *EC8* limit states are increasing with roof displacement, while in columns they remain almost constant. Additionally the *EC8* NC limit state values for beams are less than the corresponding *FEMA* CP values, while in columns it is the opposite.

2.5 Other Nonlinear Static Analysis Methods

The structural engineering community has developed a new generation of design and rehabilitation procedures that incorporate performance based engineering concepts. The need for changes in the existing seismic design methodology implemented in codes of practice has been recognized. Appropriate approaches for a successful seismic design seem to be a combination of the nonlinear static pushover analysis and the response spectrum approach (14). Examples of such methods are the capacity spectrum method, described in *ATC 40, 1996a* and the nonlinear static procedure described in *FEMA 273 (1997), FEMA 356 (2000)*. Another example is the N2 method, N stands for nonlinear analysis and 2 for two mathematical models.

The development of the N2 method started in the mid 1980's (17) (18). The method has been gradually upgraded and improved into more mature versions, the applicability has been extended to bridges, and recently the N2 method has been formulated in the acceleration-displacement format (16).

The N2 method is a relatively simple nonlinear method for the seismic analysis of structures. It combines the pushover analysis of a multi-degree of freedom (MDOF) model with the response spectrum analysis of an equivalent single-degree of freedom (SDOF) system (20). Inelastic spectra, rather than elastic spectra with the equivalent damping and period are applied. This feature represents the major difference with respect to the capacity spectrum method. Generally the results of the N2 method are accurate, provided that the structure oscillates predominately in the first mode (19).

Applications of the method are, for the time being, restricted to the planar analysis of the structures. The inelastic demand spectra, are not appropriate for near fault ground motions, for soft soil sites, for significant stiffness and/or strength deterioration, and for systems with low strength.

2.6 Plastic hinge properties

While nonlinear analysis methods like static pushover are commonly accepted and recommended as a reliable tool by international codes for seismic assessment of buildings, accuracy of the estimate of seismic capacity strongly depends on input parameters of such analysis. Some of the basic inputs are: 1) axial force–bending moment yield interaction, 2) moment-curvature, and 3) moment-rotation characteristics accounting for appropriate nonlinearity of constitutive materials of reinforced concrete elements, and they need to be readdressed for an accurate pushover analysis (6).

2.6.1 Inelastic behavior and modelling of concrete

The pushover analysis procedure is considered as one of the most effective tools for performance evaluation of buildings with respect to objectives set in performance based earthquake engineering. One of the most essential steps to be considered while conducting pushover analysis is modelling. Appropriate model requires the determination of the nonlinear properties of each component in the structure that are represented by strength and deformation capacities (50).

Concrete under multiaxial compressive stress state exhibits significant nonlinearity, which can be successfully represented by different nonlinear models (29) (32) (52). In order to study the behavior of normal or high strength concrete, one of the most important steps is to establish appropriate analytic stress-strain models that capture the real behavior. A better the stress-strain model, accounts for a more reliable estimation of strength and deformation behavior of concrete structural members. Another important characteristic of concrete is that it exhibits different behavior in its confined and unconfined states. Apart from higher strength, confined concrete tends to show a much greater ductility when compared to unconfined concrete. Thus, it becomes important and desirable to have a stress-strain model that differentiates the behavior of confined and unconfined concrete.

The stress-strain model produced by *Hognestad (1951)* is one of the most commonly used models for regular concrete. The stress-strain curve is defined by the equation given below:

$$\sigma_c = f_c\left[\frac{2\varepsilon_c}{\varepsilon_{c0}} - \left(\frac{\varepsilon_c}{\varepsilon_{c0}}\right)^2\right] \tag{1}$$

where σ_c and ε_c are the compressive stress and strain in the principal i-direction respectively, and f_c and ε_{c0} are the peak stress and strain at peak stress respectively.

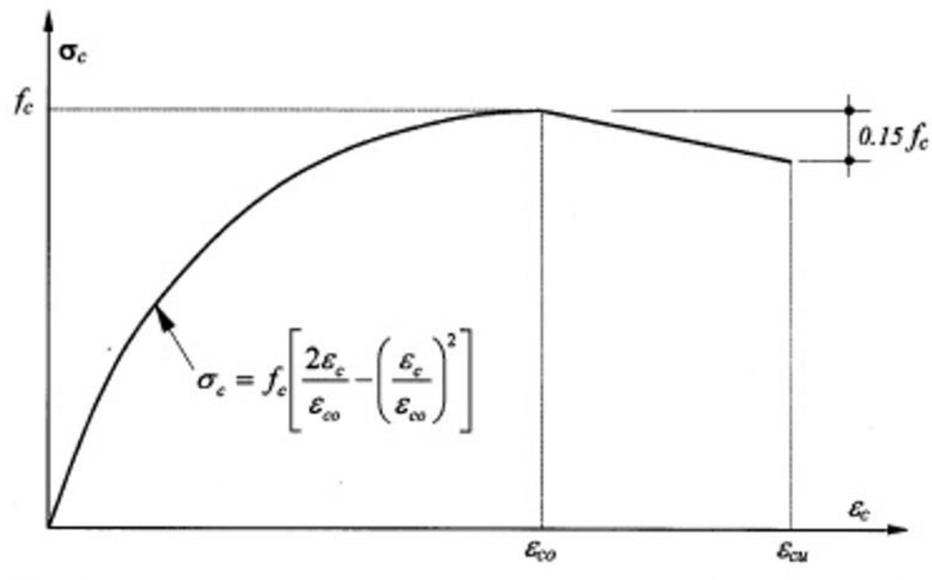


Figure 2.2: *Stress-Strain curve for ordinary concrete proposed by Hognestad (1951)*

For high performance concrete, Hognestad's equation (1) is written as follows:

$$\sigma_c = f_c\left[k\,\frac{\varepsilon_c}{\varepsilon_{c0}} - (k-1)\left(\frac{\varepsilon_c}{\varepsilon_{c0}}\right)^2\right] \tag{2}$$

where:

$$k = 2 - [(f_c - 40)/70] \qquad (60MPa \le f_c \le 94MPa) \tag{3}$$

ε_{cu} is modified according to experimental results as follows:

$$\varepsilon_{cu} = [2.2 + 0.015(f_c - 40)\,]10^{-3} \qquad (60MPa \le f_c \le 94MPa) \tag{4}$$

Kent and Park (1971) proposed a stress-strain equation for both unconfined and confined concrete. In their model they generalized *Hognestad's (1951)* equation to better describe the post-peak stress-strain behavior. In this model the ascending branch is represented by modifying the Hognestad second degree parabola by replacing $0.85f'_c$ by f'_c and ε_{c0} by 0.002.

$$f_c = f'_c\left[\frac{2\varepsilon_c}{\varepsilon_{c0}} - \left(\frac{\varepsilon_c}{\varepsilon_{c0}}\right)^2\right] \quad (5)$$

$$f_c = f'_c[1 - Z(\varepsilon_c - \varepsilon_{co})] \quad (6)$$

in which:

$$Z = \frac{0.5}{\varepsilon_{50u} - \varepsilon_{co}} \quad (7)$$

where:

ε_{50u} : the strains corresponding to the stress equal to 50% of the maximum concrete strength for unconfined concrete.

$$\varepsilon_{50u} = \frac{3+0.29f'_c}{145f'_c-1000} \qquad (f'_c in\ Mpa) \quad (8)$$

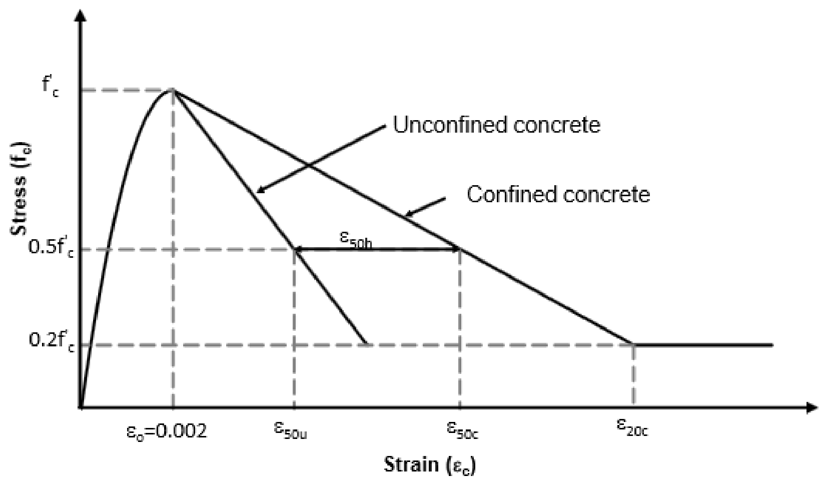


Figure 2.3a: *Proposed Stress-Strain model for confined and unconfined concrete- Kent and Park (1971) model*

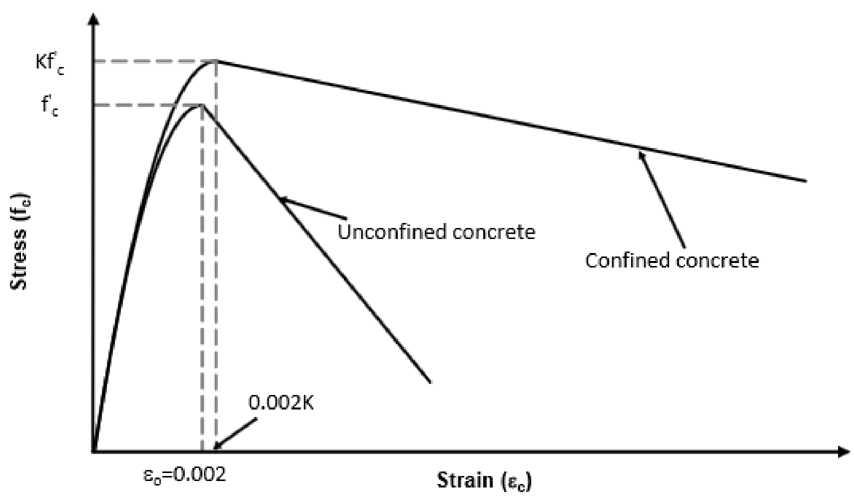


Figure 2.3b: *Stress-Strain behavior of compressed concrete confined by rectangular steel hoops-Modified Kent and Park (Scott et al. 1982) model*

Kent and Park made provisions in their stress-strain model to accommodate the behavior of confined concrete. Based on results from earlier tests, it was shown that confining the concrete with rectangular or square hoops was not very effective and that there was only a slight increase in the concrete compressive strength due to confinement. For this reason it was assumed in this model that the maximum stress reached by confined concrete remained the same as the unconfined cylinder strength, f'_c . Thus the ascending branch of the model is represented by the same second degree parabola.

Confinement only affected the slope of the post-peak branch and empirical equations were used to adjust this. The expression for the falling branch of the stress-strain relation is given by:

$$f_c = f'_c[1 - Z(\varepsilon_c - \varepsilon_{co})] \tag{9}$$

in which:

$$Z = \frac{0.5}{\varepsilon_{50h} + \varepsilon_{50u} - \varepsilon_o} \tag{10}$$

where:

$$\varepsilon_{50h} = \varepsilon_{50c} - \varepsilon_{50u} = \frac{3}{4} p'' \sqrt{\frac{b''}{s}} \tag{11}$$

ε_{50c} and ε_{50u} are the strains corresponding to the stress equal to 50% of the maximum concrete strength for confined and unconfined concrete respectively.

$$\varepsilon_{50u} = \frac{3+0.29f'_c}{145f'_c-1000} \qquad (f'_c in\ Mpa) \tag{12}$$

$\frac{b''}{s}$ is the ratio between the width of the concrete core and the center to center spacing of hoops, p" is the volumetric ratio of confining hoops to volume of concrete core measured to the outside of the perimeter hoops and is expressed as:

$$p'' = \frac{2\,(b''+d'')A_s''}{b''d''s} \tag{13}$$

where b" and d" are the width and depth of the confined core respectively, A_s'' is the cross-sectional area of the hoop bar and 's' is the center to center spacing of the hoops. It is assumed that concrete can sustain some stress at indefinitely large strains. However, the failure of the member would occur before the strains in concrete become impractically high. Hence, for this model it was assumed that the concrete can sustain a stress of 0.2 f'_c from a strain of ε_{20c} to infinite strain.

Scott et al. (1982) conducted experiments on a number of square concrete columns reinforced with either 8 or 12 longitudinal rebars and transversely reinforced with overlapping hoops. Their tests were conducted at rapid strain rates, typical of seismic loading. Unlike the *Kent and Park (1971)*, substantial strength enhancement due to the presence of good confining reinforcement details was observed. Thus simple modifications were made to the *Kent and Park (1971)* model in order to incorporate the increase in the compressive strength of confined concrete at high strain rates (Fig. 2.3b). The maximum achieved concrete stress is assumed to be Kf'_c and the strain at maximum concrete stress is $0.002\,K$, where 'K' is a factor that is defined later. The branches of the stress-strain curve for the modified Kent and Park relation for low strain rate are given as:

$$f_c = Kf_c'[\frac{2\varepsilon_c}{0.002K} - \left(\frac{\varepsilon_c}{0.002K}\right)^2] \qquad \text{(for } \varepsilon_c \leq 0.002\,K\text{)} \qquad (14)$$

$$f_c = Kf_c'[1 - Z_m(\varepsilon_c - 0.002K)] \qquad \text{(for } \varepsilon_c > 0.002\,K\text{)} \qquad (15)$$

In which:

$$Z_m = \frac{0.5}{\frac{3 + 0.29f'_c}{145f'_c - 1000} + \frac{3}{4}\rho_s\sqrt{\frac{h"}{s_h}} - 0.002K} \qquad (16)$$

where f_c' is in MPa, ρ_s the ratio of volume of rectangular steel hoops to volume of concrete core measured to the outside of the peripheral hoop, h” is the width of concrete core measured to the outside of the peripheral hoop and s_h the center to center spacing of hoop sets. In the above expressions the value of ‘K’ is obtained from the following expression:

$$K = 1 + \frac{\rho_s f_{yh}}{f_c'} \qquad (17)$$

where f_{yh} is the yield strength of the hoop reinforcement.

2.6.2 User-defined and default hinge properties

K Rama Raju et al. (2012) conducted a research on seismic performance evaluation of existing reinforced concrete buildings designed as per past codes of practice. In this paper, a typical 6-storey reinforced concrete building frame is designed for four design cases, in compliance with the Indian Standard and it is analyzed using user-defined nonlinear hinge properties or default-hinge properties, given in SAP 2000 based on the *FEMA 356 (2000)* and *ATC 40, 1996a* guidelines. An analytical procedure is developed to evaluate the yield, plastic and ultimate rotation capacities of reinforced concrete elements of the considered frame structures and these details are used to define user-defined inelastic effect of hinge for columns and for beams. The possible differences in the results of pushover analysis due to default and user-defined nonlinear properties at different performance levels of the building are studied. A significant variation is observed in base shear capacities and hinge formation mechanisms for four design cases with default and user-defined hinges at yield and ultimate. It is assumed that this occurs because the orientation and the axial load level of the columns cannot be taken into account properly by the default-hinge properties. Based on the observations in the hinging patterns, it is noticeable that the user-defined hinge model is more successful in capturing the hinging mechanism and the nonlinear behavior of the structure compared to the model with the default hinge.

Mehmet Inel et al. (2006) state in their research that modeling for a nonlinear static analysis requires the determination of the nonlinear properties of each component in the structure, quantified by strength and deformation capacities, which depend on the modeling assumptions. Pushover analysis is carried out for user-defined and default nonlinear hinge properties, based on *FEMA 356, 2000* and *ATC 40, 1996a* guidelines. The misuse of default-hinge properties might lead to unreasonable and inaccurate displacement capacities. This study presents the possible differences in the results of pushover analysis due to default and user-defined nonlinear component properties. Observations made from the analysis of different considered cases, show that plastic hinge length and transverse reinforcement spacing have no influence on the base shear capacity, and considerable effects on the displacement capacity of the frames.

The capacity curve for the default-hinge model is reasonable for structures which are built in compliance with the modern codes, but it may not be suitable for structures built according to former codes of practice. Considering that most existing buildings (especially in Albania) do not conform to requirements of modern codes, the use of default hinges needs special attention.

2.7 Reinforced Concrete Structures in Albania

In our country, Albania, reinforced concrete started to be used as a building material around the year 1920, especially for bridge construction by foreign companies. After the year 1944, reinforced concrete became the base material to our buildings, whether they were residential, industrial etc. (58).

In the tables below, statistics collected from the Institution of Statistics (*INSTAT*) in Albania (31), for buildings categorized based on the materials, are shown. Table 1 shows the statistics for buildings from the year 1960 to the year 1990 and Table 2 the statistics for buildings from the year 1991 to the year 2001.

Table 2.1: *Buildings in Albania according to the year of construction and the building materials (1)*

Year of construction	Prefabricate	Brick, Stone	Wood	Other
Before 1960	0	101 286	1667	5953
1960-1991	21 594	243 372	3094	13 368

Table 2.2: *Buildings in Albania according to the year of construction and the building materials (2)*

Year of construction	Prefabricate	Brick, Stone	Wood	Other
1991-1995	4575	43 324	743	4238
1996-2001	7776	59 811	1234	6145

It can be assumed that the buildings categorized as "Other" are mostly reinforced concrete buildings. As it can be seen from the statistics, there is a large number of existing reinforced concrete structures. The focus of this study are reinforced concrete frame structures, and considering the fact that these kinds of buildings are widely spread in Albania, the current study is even more relevant.

2.8 Albania's Seismic Condition

During the last century, several devastating earthquakes occurred in the Western Balkan region, particularly along the whole Adriatic costal area, causing a large number of casualties, substantial structural and nonstructural damage and economical loss (23).

Our country is part of the Ionian-Adriatic seismic zone, in which are included: Shkodra region, Korce-Oher-Peshkopi region, with active areas in Lushnje, Elbasan, Diber, and especially Vlore-Tepelene-Erseke area, which has been activated recently, causing consecutive ground motions since May 2006. From 6 August 2006 to 9 August 2006, 161 seismic events have been registered in this area.

Throughout the previous century, our country has been subjected to several earthquakes, with the most severe one being the one in Shkoder in 1905. 200 victims and 500 injured people were registered, along with 1500 destroyed buildings and other thousands with serious unrepairable damages. The earthquake near Shkodra, that destroyed a large area in the northwest part of Albania and southeast part of Montenegro (the area of the Scutari Lake) and was strongly felt in the Apulia region of Italy and in Croatia, had an intensity of 9 in the Rihter scale (1 to 12) according to seismologists.

Other major earthquakes with devastating outcomes were: the one in Tepelene in 1920, with an intensity of 8 (Rihter Scale) that destroyed 2500 buildings and left 15000 people homeless; the one in Peshkopi in 1924 that destroyed 80% of the houses; and the one in the area of Diber-Librazhd that destroyed 177 villages and caused a 10 km crack in the ground, with a 50 cm vertical displacement of the soil.

Considering its past seismic activity, it can be said that Albania is a country with high earthquake activity, and despite the fact that the Technical Institute of Engineering (ISTN) has done some work in this direction, there is a big void when it comes to seismic evaluation and design of buildings (13).

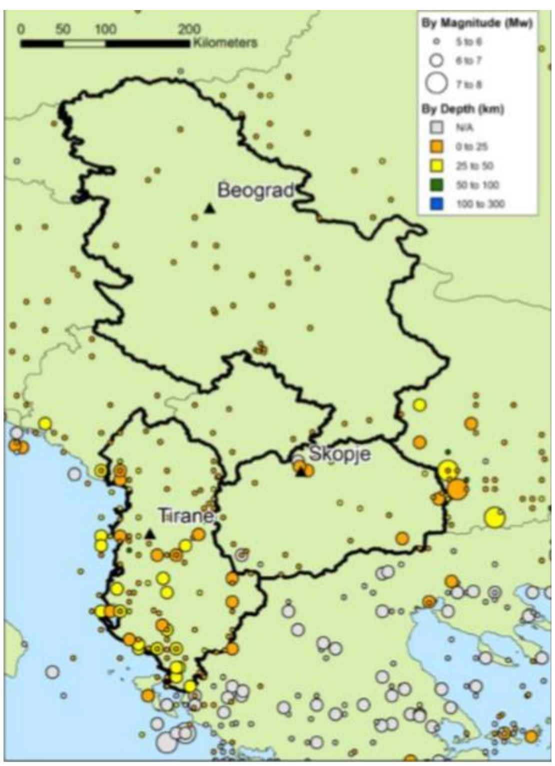


Figure 2.4: *Epicenters of historical earthquakes (510 B.C. to 2010) in the Southeast Europe region, by magnitude (events > M_w 5) and hypocentre depth* (23)

Chapter 3

Methodology

3.1 Introduction

The methodology used for the research is presented and defined in this chapter. The process followed to fulfill the objectives of this project is explained. This chapter includes a review of the research method and design relevance. Pushover analysis is further discussed and explained, and user-defined hinge properties are presented.

3.2 Pushover Analysis

The seismic evaluation and retrofit of reinforced concrete buildings presents a great challenge to the owners, architects and engineers in Albania. For many years engineers have been using and continue to use unrealistic simplified static lateral force procedures to design buildings to resist earthquake forces.

Various analysis methods, both elastic (linear) and inelastic (nonlinear), are available for the analysis of existing reinforced concrete buildings. Inelastic analysis procedures better demonstrate how buildings work by identifying modes of failure and the potential for progressive collapse. Pushover analysis, while it is a simple and effective nonlinear static method of analysis, it also is not very known and rarely used by engineers for seismic evaluation in Albania.

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 most importantly, because it considers post-elastic behavior.

Pushover analysis is first and foremost based on the assumption that the response of the structure is controlled by the first mode of vibration and mode shape, or by the first few modes of vibration, and that this shape remains constant throughout the elastic and inelastic response of the structure (55). This theory provides the basis for transforming a dynamic problem to a static problem, by converting the response of a multi degree of freedom (MDOF) structure to the response of an equivalent single degree of freedom (SDOF) system (ESDOF). This concept is illustrated in the figure below:

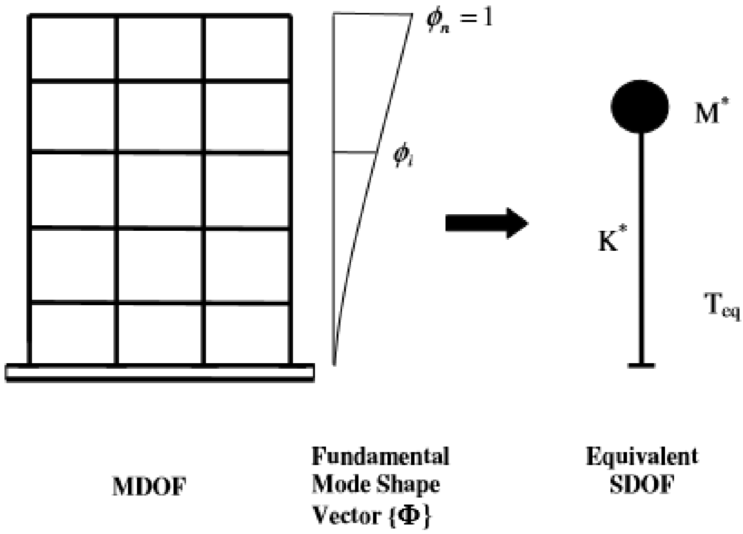


Figure 3.1: *Conceptual diagram for transformation of a MDOF system into a SDOF system* (55)

The initial period T_{eq} of the equivalent single degree of freedom system will be:

$$T_{eq} = 2\pi\sqrt{\frac{M^*}{K^*}}$$

M^* and K^* are the mass and the elastic stiffness of the equivalent multi degree of freedom system respectively.

Principally, pushover analysis is an extension of the "lateral force procedure" of static linear analysis into the nonlinear procedure. This type of analysis is carried under constant gravity loads and monotonically increasing lateral loading applied on the masses of the structural model. The loading pattern is meant to simulate inertia forces due to the horizontal component of the seismic action. While the applied lateral forces increase in the course of the analysis, the formation of plastic hinges can be observed, along with the evolution of the plastic mechanism and damage, as a function of the magnitude of the imposed lateral loads and of the resulting displacements.

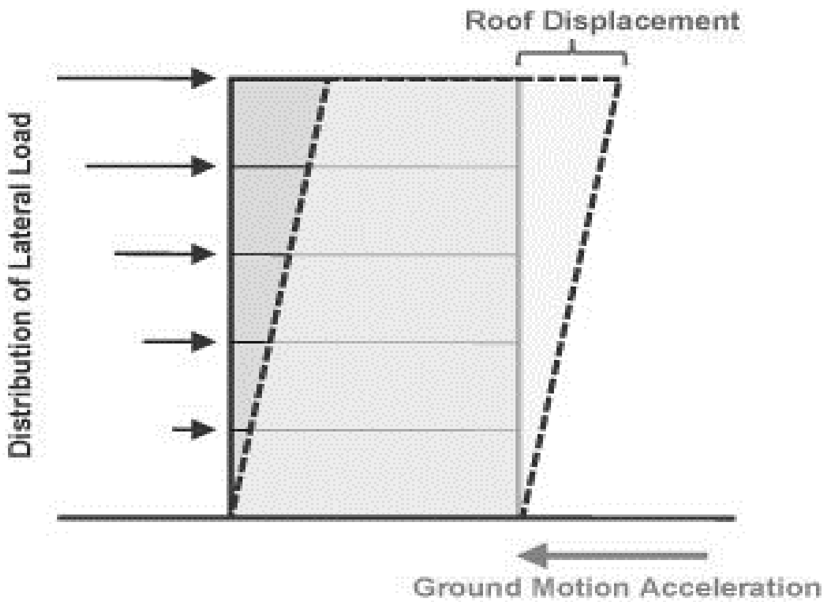


Figure 3.2: *Simple graphic demonstration of pushover analysis* (55)

Pushover analysis consists of a series of elastic analysis, superimposed to approximate a force-displacement curve of the overall structure. A two or three dimensional model is first created and gravity loads are applied initially. A predefined lateral load pattern which is distributed along the building height is then applied. The lateral forces are increased until some members yield. The structural model is modified to account for the reduced stiffness of yielded members and lateral forces are again increased until additional members yield. The process is continued until a control displacement at the top of building reaches a certain level of deformation or structure becomes unstable. The roof displacement is plotted with base shear to get the global capacity curve (44) (52).

Pushover analysis can be performed as force-controlled or deformation-controlled. When performing a force-controlled pushover analysis, the full load combination is applied as specified, which means the load is known (for example gravity loading). In a deformation controlled pushover analysis, elements, or systems exceed their elastic limit in a ductile manner. Force or stress levels for these components are less important than the amount or extent of deformation beyond the yield point (2).

Pushover analysis has been one of the most popular methods for seismic performance evaluation of structures by the major rehabilitation guidelines and codes because it is conceptually and computationally simple. Pushover analysis allows tracing the sequence of yielding and failure on member and structural level as well as the progress of overall capacity curve of the structure (5).

The purpose of pushover analysis is to evaluate the expected performance of structures by estimating the strength and deformation demands with the help of static inelastic analysis, and comparing these demands to available capacities at the performance levels of interest.

3.2.2 Lateral Load Patterns

Some of the most commonly used load patterns in a pushover analysis are (55):

1. Mode Shape distribution based on the fundamental mode or other mode shapes of interest

$$F_i = W_i \Phi_{ij}$$

where W_i is the weight of the 'i' storey, and Φ_{ij} the i^{th} element of the mode shape vector corresponding to the 'i' storey for mode j.

2. The FEMA load distribution

$$F_i = \frac{W_i h_i^k}{\sum_{l=1}^{n} W_l h_l^k} \cdot V_b$$

where k is a coefficient which can be assumed to be dependent on the fundamental period T_n of the structure. It can be set equal to 1.0 for structures that have period shorter than 0.5 seconds and equal to 2.0 for T >2.5 seconds. A linear variation between 1 and 2 can be used to obtain a simple transition between the two extreme values (4).

3. A uniform load distribution

$$F_i = W_i$$

4. A single concentrated horizontal force at the top of the structure.

5. An inverted triangular distribution

$$F_i = \frac{W_i h_i}{\sum_{l=1}^{n} W_l h_l} \cdot V_b$$

where h_i is the height of the 'i' storey, n is the total number of the storeys, and V_b is the base shear given by the following equation:

$$V_b = S_d(T_1)W$$

where $S_d(T_1)$is the acceleration ordinate of the design spectrum at the fundamental period T_1, and W is the total weight of the structure.

In order to represent all forces which are produced when the system is subjected to earthquake excitation, a pattern of increasing lateral forces needs to be applied to the mass points of the system. By incrementally applying this pattern up to the inelastic stage, progressive yielding of the structural elements can be monitored. The choice of the load pattern to capture a dynamic phenomenon through a static analysis is important and has been recognized by several studies (25), but it is not included in the scope of this study.

Considering that the study is mainly concentrated on the section and material properties, and their effects on pushover analysis, the inverted triangular load pattern is considered appropriate.

The procedure of calculating the loads applied to the joints of the frame building at each storey level is shown below.

First of all, the fundamental period of vibration is calculated, based on Eurocode 8 (*EN 1998-1:2004, Section 4.3.3.2.2*). For buildings with heights of up to 40 m the value of T_1 may be approximated by the following expression:

$$T_1 = C_t\, H^{3/4}$$

where:

C_t is 0.085 for moment resistant space steel frames, 0.075 for moment resistant space concrete frames and for eccentrically braced steel frames and 0.050 for all other structures

H is the total height of the building in meters, which is 22.4 m for the considered building

The approximate value of the fundamental period of vibration in this case is:

$$T_1 = 0.772\ s$$

To calculate the ordinate of the design spectrum at the fundamental period $S_d(T_1)$, several parameters which describe the shape of the elastic response spectrum should be defined first. The values of these parameters, depend upon the ground type. For the building analyzed in the next Chapters, the ground type based on EN 1998-1:2004 classification, is ground type C.

The base shear is calculated according to Section 4.3.3.2.2, of Eurocode 8. The total weight of the building is calculated, considering all the dead load coming from all the members, and 30% of the live load.

$$V_b = S_d(T_1)W\,\lambda = 427.04\,KN$$

λ is the correction factor, the value of which in this case is 0.85 (*EN 1998-1:2004, Equation 4.5*)

Based on all the calculations, the lateral loads, applied at each storey level, are as shown in the Table below:

Table 3.1: *Lateral Loads applied at each storey level*

Storey	Floor Weight (KN)	Floor Height (m)	Wi hi	Wt ht	Vb (KN)	Fi (KN)
1	2003.367	3.2	6410.774	314127.9	427.04	15.2
2	2003.367	6.4	12821.55	314127.9	427.04	30.4
3	2003.367	9.6	19232.32	314127.9	427.04	46
4	2003.367	12.8	25643.1	314127.9	427.04	60.83
5	2003.367	16	32053.87	314127.9	427.04	76.57
6	2003.367	19.2	38464.65	314127.9	427.04	91
7	2003.367	22.4	44875.42	314127.9	427.04	106.7

3.3 Nonlinear Hinge Properties

A hinge property is a named set of rigid-plastic properties that can be assigned to one or more frame elements. Yielding and post-yielding behavior can be modeled using distinct user-defined hinges. Currently hinges can only be introduced and assigned to frame elements, at any location along that element. Each hinge represents concentrated post-yielding behavior in one or more degrees of freedom. Uncoupled moment, torsion, axial force and shear hinges are available. There is also a coupled $P - M_2 - M_3$ hinge, which yields based on the interaction of axial force and bi-axial bending moments at the hinge location. Hinges only affect the behavior of the structure in nonlinear static and nonlinear direct-integration time history analyses.

As it is mentioned in Chapter 2, *ATC 40, 1996a* and *FEMA 356, 2000;* have developed a set of modeling parameters, acceptance criteria and procedures of pushover analysis. These documents also describe the actions followed to determine the yielding of frame members during the analysis. Two actions are used to control the inelastic behavior of the member during the pushover analysis, which are deformation-controlled (ductile action) or force-controlled (brittle action) (1).

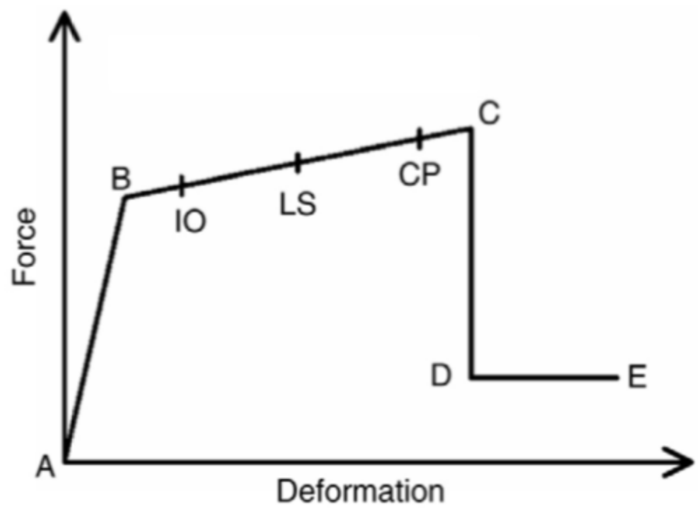


Figure 3.3: *Force-deformation (or moment-rotation) relationship of a typical plastic hinge for the deformation-controlled option, flexural failure*

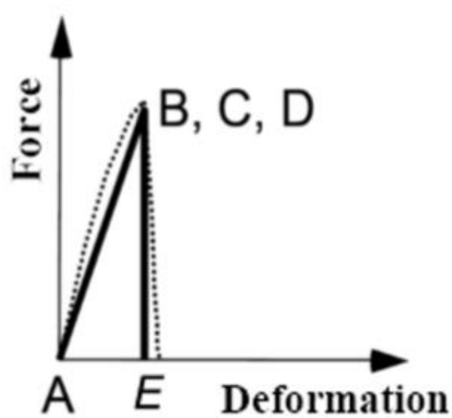


Figure 3.4: *Force-deformation relationship of a typical plastic hinge for the force-controlled option, shear failure*

In this study, the deformation-controlled option is used to define the plastic hinges. Based on the type of the structure being studied (moment-resisting frame) and the objectives set, this option was considered most suited for the modelling.

As it is shown in Figure 3.4, the plastic deformation curve, used for deformation-controlled hinges, has the same shape, whether it is in the force-displacement format, or the moment-rotation one. The defined moment-rotation curve gives the yield value and the plastic deformation following the yield for each frame element. The meaning of the five points A-B-C-D-E in the curve, is explained below:

Point A: The origin

Point B: Yielding point. No deformation occurs in the hinge up to point B, regardless of the rotation value specified for this point. Only the plastic rotation beyond point B will be exhibited by the hinge.

Point C: Ultimate capacity for pushover analysis.

Point D: Residual strength for pushover analysis.

Point E: Total failure

The three points labeled as IO (immediate occupancy), LS (Life safety), CP (collapse prevention), are used to define the acceptance criteria or performance level for the plastic hinge. These are informational measures that are reported in the analysis results and used for performance-based design. They do not have any effect on the behavior of the structure. This study defines these three points as 20%, 50% and 90% respectively of the plastic hinge deformation capacity (30).

When defining the hinge moment-rotation plastic deformation curve, the values for moment and rotation may be entered directly, or they may be entered as normalized values, by specifying the scale factors used to normalize the curve. In this study, the normalized values are entered when defining the moment-rotation curve for beams and columns user-defined plastic hinges. The scale factor (SF) used to normalize the values is the yield rotation. When default hinge properties are used, the program automatically uses the yield values for scaling.

Plastic hinge length is used to obtain ultimate rotation values from the ultimate curvatures. Several plastic hinge lengths equations have been proposed. Two of the most widely used are:

1. $L_p = 0.5H$

2. $L_p = 0.08L + 0.022f_{ye}d_{bl} \geq 0.044f_{ye}d_{bl}$

L_p: the plastic hinge length

H: the section depth

L: the critical distance from the critical section of the plastic hinge to the point of contraflexure

f_{ye}: the expected yield strength

d_{bl}: the diameter of longitudinal reinforcement

The first one is the simplest form of plastic hinge length (45). The second one is proposed by *Priestley et al. 1996*.

The plasticity that is distributed over the length of the element can be approximated by inserting many hinges, at relative locations within the element (0.05; 0.15, 0.25…0.95), each with respective deformation properties.

3.3.1 Modelling Procedure

One of the most important steps in the implementation of pushover analysis, is modelling. The nonlinear behavior of structures and elements has to be considered in the model that is going to be analyzed (25). The model requires the determination of the nonlinear properties of each component in the structure that are quantified by strength and deformation capacities (30). Previous studies and observations clearly show that the user defined plastic hinge model is better than the default hinge model (27) (30) (44) (50) (55), because it reflects better the nonlinear behavior compatible with the element properties.

The limit state design procedure of reinforced concrete elements has undergone major modifications in recent times with more emphasis toward a performance-based engineering approach. This design approach demands a thorough understanding of axial force–bending moment (P-M) yield interaction of elements, for moment-resistant reinforced concrete (RC) frames under seismic loads, in particular.

In this study, user defined hinge properties are used during the modelling process of the reinforced concrete frame, this requires the generation of axial force-bending moment yield interaction, and moment-rotation values.

Seismic design philosophy demands energy dissipation/ absorption by inelastic deformation for collapse prevention during major earthquakes, which means that sufficient ductility ensured in the design procedure is an important requirement for suitability of reinforced concrete structures to resist seismic loads. Ductility also ensures effective redistribution of moments at critical sections as the collapse load is approached. Ductility, depends mainly on the moment-curvature relationship at critical sections where plastic hinges are expected to be formed at collapse.

Pushover analysis accounts for inelastic behavior of the building models and provides a reasonable estimate of deformation capacity while identifying critical sections likely to reach limit state during earthquakes. Researchers emphasized that accuracy of results obtained from pushover analysis are strongly influenced by basic inputs like: 1) stress-strain relationship of constitutive materials; 2) axial force-bending moment (P-M) yield interaction; as well as 3) moment-rotation capacity of members (6).

3.3.2 Stress-Strain relationship for concrete and steel

Concrete under multiaxial compressive stress state exhibits significant nonlinearity, which can be successfully represented by nonlinear models (8) (28) (45). In the current study the nonlinear elastic response of concrete is characterized by parabolic stress-strain relationship as shown in the figure below.

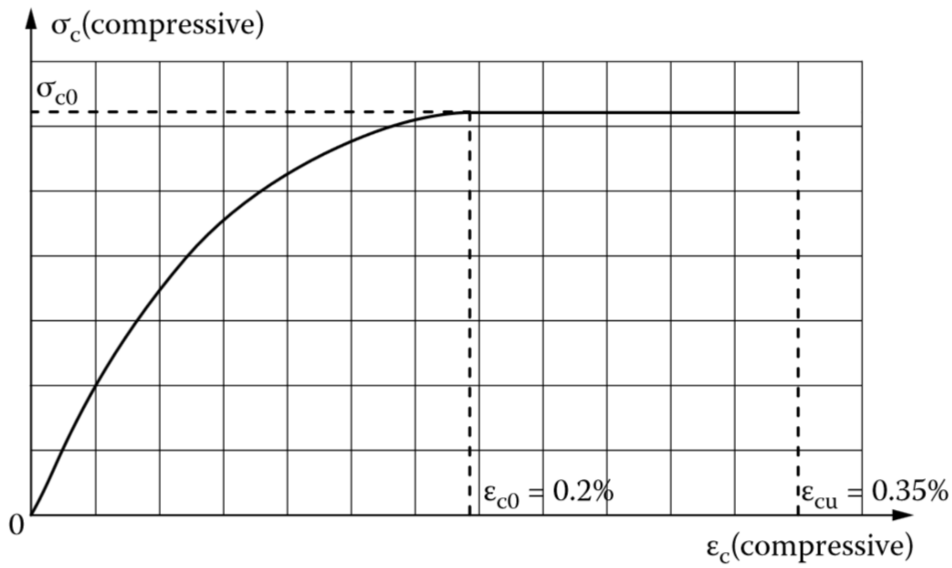


Figure 3.5: *Stress-Strain relationship for concrete* (11)

Elastic limit strain and strain at cracking are limited to 0.2% and 0.35%, respectively (11). Tensile stresses in concrete are ignored in the study. The design ultimate stress in concrete in compression is given by:

$$\sigma_{c0} = \frac{(0.83)(0.85)R_{ck}}{\gamma_c} \tag{18}$$

where：

R_{ck} compressive cube strength of concrete (N/mm2)

γ_c partial safety factor for concrete

The stress-strain relationship for concrete under compression stresses is given by:

$$\sigma_c(\varepsilon_c) = a\varepsilon_c^2 + b\varepsilon_c + c \qquad 0 < \varepsilon_c < \varepsilon_{c0} \tag{19}$$

$$\sigma_c(\varepsilon_c) = \varepsilon_{c0} \qquad\qquad \varepsilon_{c0} < \varepsilon_c < \varepsilon_{cu} \tag{20}$$

where compression stresses and strains are assumed to be positive in the analysis.

Constants a, b, and c in equation (18) are determined by imposing the following conditions:

$$\sigma_c(\varepsilon_c = 0) = 0 \qquad c = 0 \tag{21}$$

$$\sigma_c(\varepsilon_c = \varepsilon_{co}) = \sigma_{c0} \qquad =\gg \; a\varepsilon_{c0}^2 + b\varepsilon_{c0} = \sigma_{c0}$$

$$\left[\frac{d\sigma_c}{d\varepsilon_c}\right]_{\varepsilon_c=\varepsilon_{c0}} = 0 \qquad 2a\,\varepsilon_{c0} + b = 0$$

By solving the above equations, we get the constants a, b and c:

$$a = -\frac{\sigma_{c0}}{\sigma_{c0}^2};\;\; b = \frac{2\sigma_{c0}}{\sigma_{c0}};\;\; c = 0 \tag{22}$$

By substituting them in equation (20) we get:

$$\sigma_c(\varepsilon_c) = -\frac{\sigma_{c0}}{\varepsilon_{c0}^2} + \frac{2\sigma_{c0}}{\varepsilon_{c0}}\varepsilon_c \qquad 0 < \varepsilon_c < \varepsilon_{c0} \tag{23}$$

Steel is isotropic and homogeneous material exhibiting stress-strain relationship as shown in the figure below:

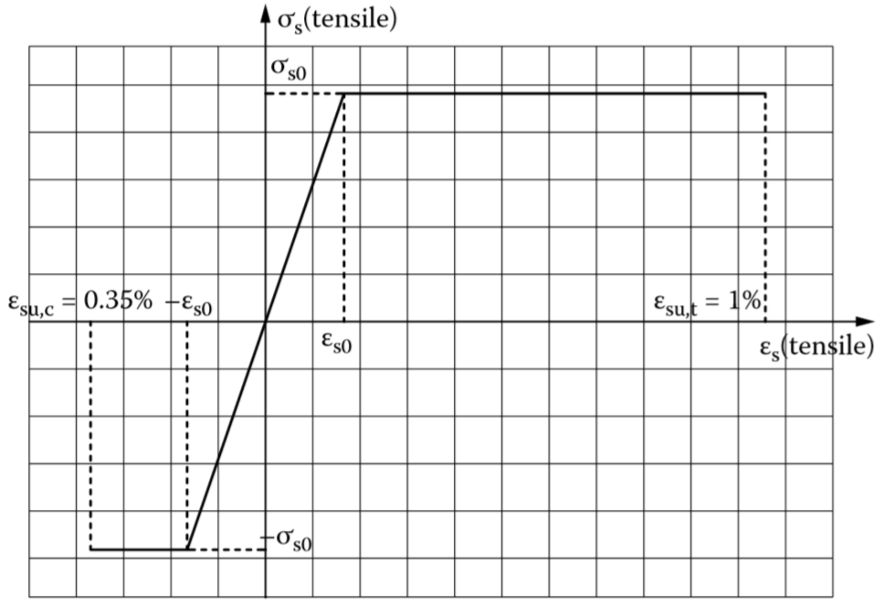


Figure 3.6: *Stress-Strain relationship for steel* (11)

While the ultimate limit strain in tension and that of compression are taken as 1% and 0.35%, respectively, elastic strain in steel in tension and compression are considered the same (11). The design ultimate stress in steel is given by:

$$\sigma_{s0} = \frac{\sigma_y}{\gamma_s} \tag{24}$$

where:

σ_y: yield strength of steel (N/mm2)

γ_s: partial safety factor for steel

The stress-strain relationship for steel is given by:

$$\sigma_s(\varepsilon_s) = E_s\varepsilon_s \qquad -\varepsilon_{s0} < \varepsilon_s < \varepsilon_{s0} \tag{25}$$

$$\sigma_s(\varepsilon_s) = \sigma_{s0} \qquad \varepsilon_{s0} < \varepsilon_s < \varepsilon_{su,t} \qquad (\varepsilon_{su,t} = \varepsilon_{su}) \tag{26}$$

$$\sigma_s(\varepsilon_s) = -\sigma_{s0} \qquad -\varepsilon_{su} < \varepsilon_s < -\varepsilon_{s0} \tag{27}$$

3.3.3 Axial force-bending moment (P-M) yield interaction

According to the detailed mathematical model of axial force-bending moment yield interaction (P-M) of reinforced concrete rectangular sections in Eurocode, there are six subdomains defining the boundary of P-M yield interaction (7).

Results obtained for the failure interaction curve of reinforced concrete rectangular sections under axial force-bending moment (P-M) yield interaction show that the boundary curve is divided into two main parts, namely, 1) tension failure with weak reinforcement resulting in yielding of steel and 2) compression failure with strong reinforcement resulting in crushing of concrete.

To examine the axial force-bending moment (P-M) yield interaction behavior a reinforced concrete beam of rectangular cross-section shown in the figure below taken as a simple example:

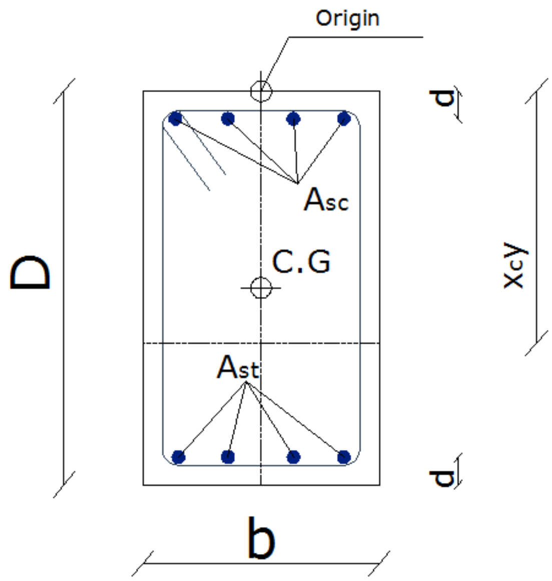


Figure 3.7: *Cross section of reinforced concrete beam*

Interaction behavior becomes critical when one of the following conditions apply: 1) reinforcement in tension steel reaches ultimate limit; 2) strain in concrete in extreme compression fiber reaches ultimate limit; or 3) maximum strain in concrete in compression reaches elastic limit under only axial compression (6).

As we previously stated, P-M limit domain consists of six subdomains which are generally described below. Only the upper boundary curves (corresponding to positive-bending moment M) will be examined since there exists a polar symmetry of the domains with respect to the center of the domain (6) (42).

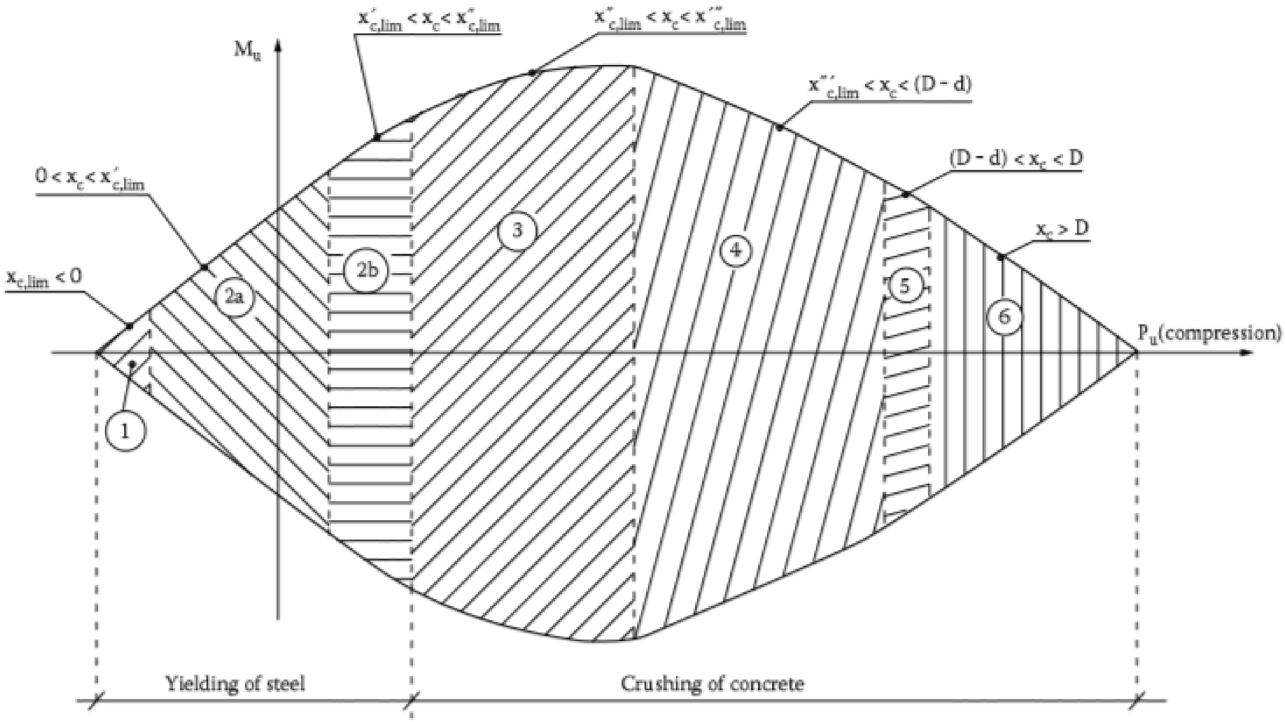


Figure 3.8: *Axial force-Bending moment (P-M) interaction curve for different subdomains* (6)

In subdomains 1 and 2, collapse is caused by yielding of steel, whereas in subdomains 3 to 6 the collapse is caused by crushing of concrete.

For each subdomain, formulas for the calculation of the axial force and bending moment are generated, based on several contributing factors like the depth of neutral axis, the area of steel and stress-strain relationships for the consecutive materials. The formulas developed by *Srinivasan Chandrasekaran, et al. 2010* are put into an Excel spreadsheet and used to calculate in every case in this study the axial force-bending moment (P-M) interaction curve, which is then used for the user-defined hinge in Sap2000.

An example of the (P-M) interaction curve for the beams, in Case 1, with all the details and characteristics presented in Chapter 4, Section 4.3.1., is shown in the figure below:

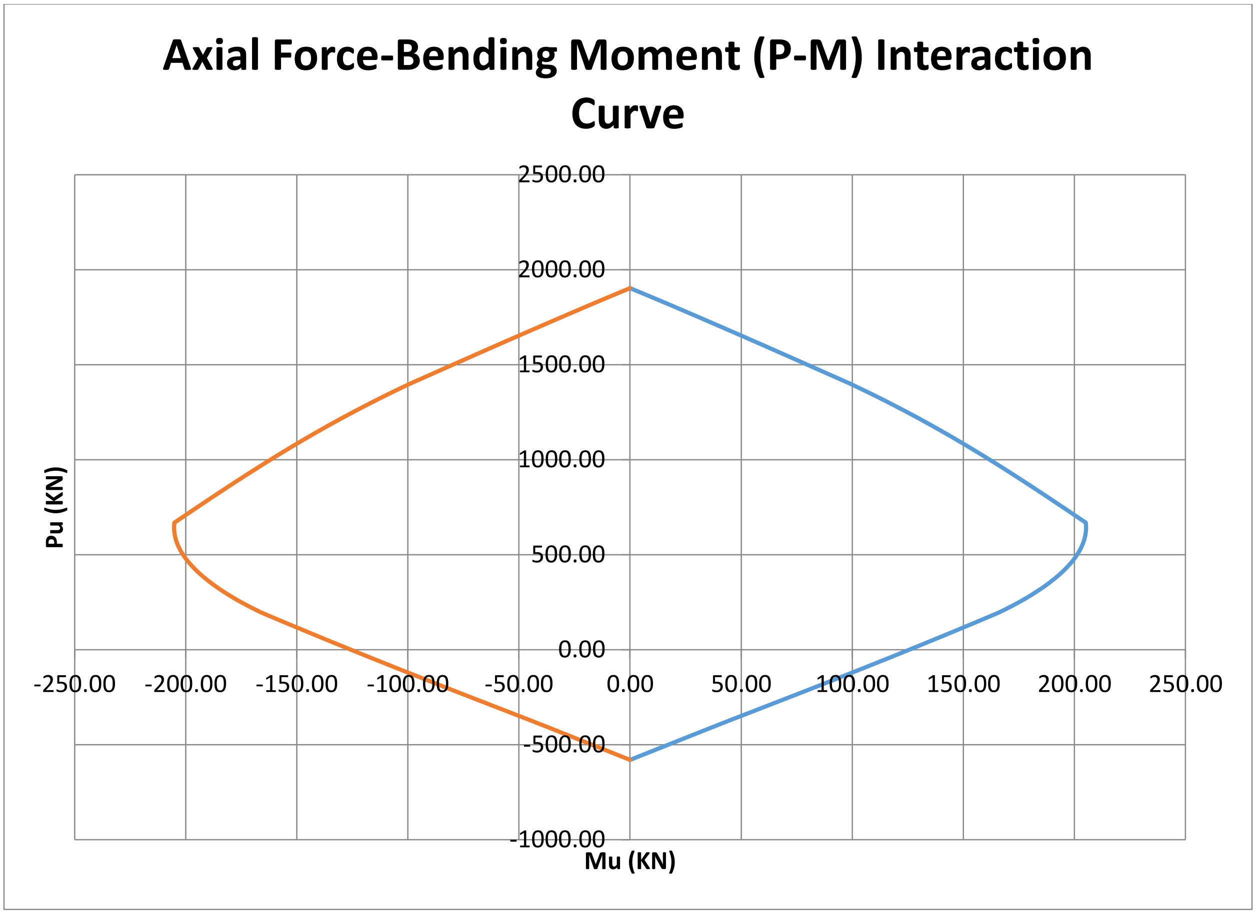


Figure 3.9: *Calculated (P-M) interaction curve for beams in Case 1*

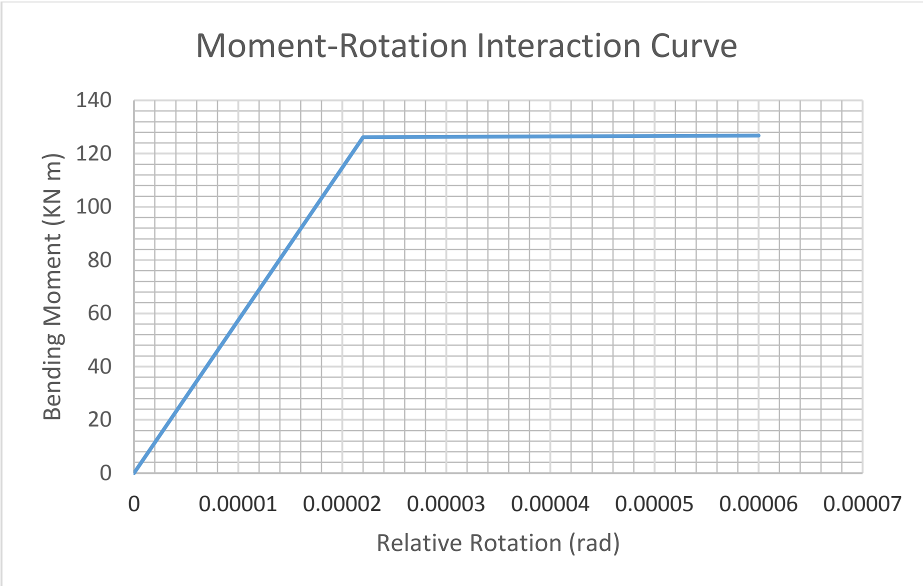
3.3.4 Moment-rotation capacity

Moment-rotation relationships of reinforced concrete sections provide an estimate of the section's ductility, which is a valuable design parameter, especially for seismic design. Moment-rotation relationships, in elastic and elastic-plastic ranges, are derived from the proposed bilinear modeling of moment-curvature relationships. Ductility, a measure of energy dissipation by inelastic deformation during major earthquakes, depends mainly on moment-curvature relationship at critical sections, where plastic hinges are expected or imposed to be formed at collapse, it also ensures effective redistribution of moments at these sections, as collapse load is approached (6) (33) (46).

Moment-plastic rotation relation for a member section consists of plastic rotation and corresponding moments as ratio of yield moment. This relation affects the behavior of a section once a hinge forms there (47).

The formulas developed by *Srinivasan Chandrasekaran, et al. 2010*, for moment-rotation relationships are put into an Excel spreadsheet, as in the axial force-bending moment case, and used to calculate in every case in this study the moment-rotation interaction curve, which is then used for the user-defined hinges in Sap2000.

An example of moment-rotation interaction curve, for the fixed beams with a span L=4m, in Case 1, is shown in the figure below:



Δθ	M
(rad)	(KN-m)
0	0
0.000022	126.24
0.000060	126.73

Figure 3.10: *Moment-Rotation interaction curve for the fixed beams in Case 1*

Chapter 4

Cases Presentation and Modelling in Sap2000

4.1 Introduction

In this chapter the cases taken into consideration to fulfill the objective of the study are presented and explained. The detailed procedure of modelling is Sap2000 is shown for the first case. For the other cases the same process is followed, with exception of the material or reinforcement properties specific for each case, which are described in the respective sections.

4.2 Reinforced concrete building description

Reinforced concrete frame buildings became widely spread structures in Albania, especially after the year 1960 (31). Several projects of reinforced concrete frame structures were reviewed for this study. A similarity in the geometry and the materials used is observed. Three of the projects are taken into consideration and one of them is used as an example for further analysis. The selected building is a residential reinforced concrete frame structure located in Tirana, Albania, built around the year 1999. It is a 5 bay, 7 storey building with C25/30 concrete and S355 steel.

The storey height is 3.2 m and the foundation is a plate. The building is designed as a moment resisting frame, based on KTP-N.2-89. A table with the dimensions of the main structural components is shown below.

Table 4.1: *Dimensions of the structural components*

Columns	Beams	Slab
60x40 cm; 40x60 cm	50x30 cm	15 cm

Considering that a 2D analysis is going to be carried out, the dead load transmitted from the slabs to the beams of the structure is calculated. The most loaded frames are the internal ones, and since the building is symmetrical in both directions, the frames along the axis B and C, are equally loaded, so one of them is chosen for further analysis.

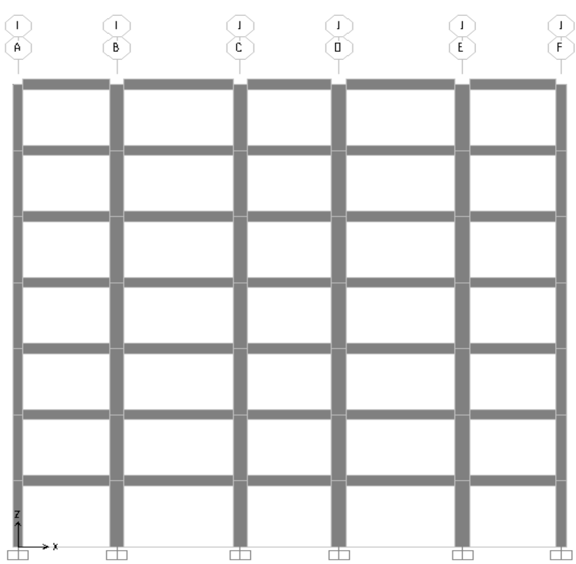


Figure 4.1: *Extrude view of the frame modelled in Sap2000*

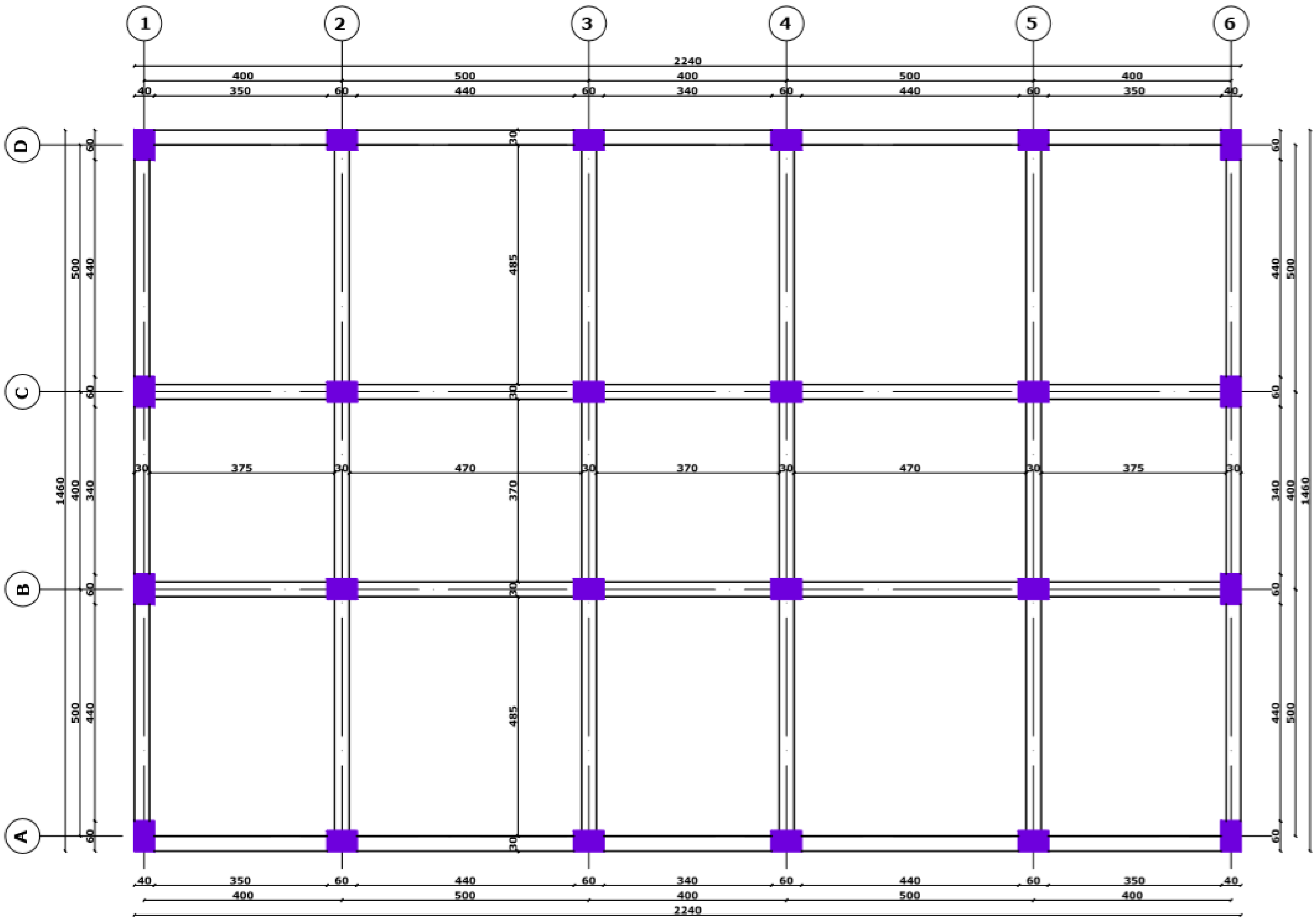


Figure 4.2: *Structural Plan of the building*

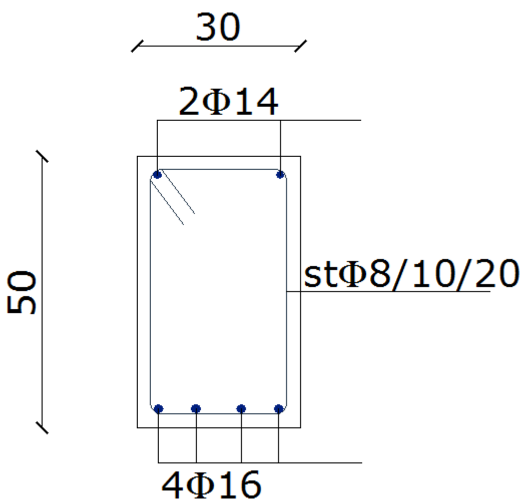


Figure 4.3: *Beam reinforcement details (span)*

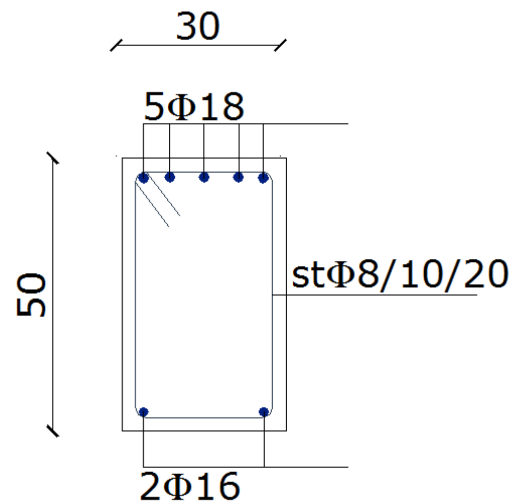


Figure 4.4: *Beam reinforcement details (support)*

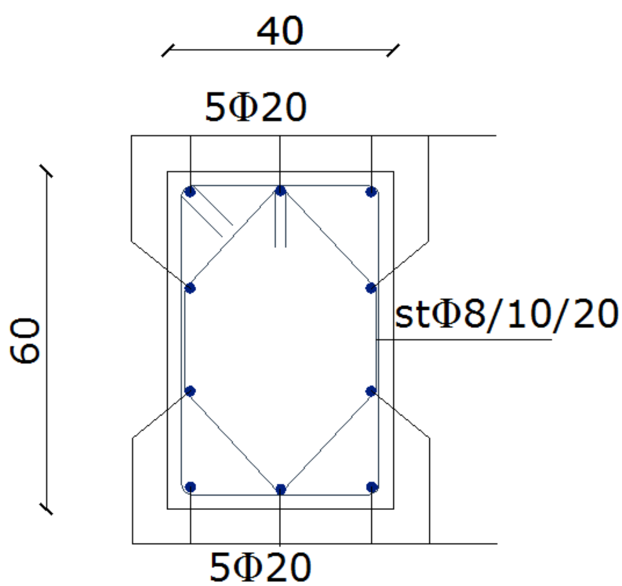


Figure 4.5: *Column reinforcement details*

4.3 Cases

Eleven cases are defined to accomplish the objective of this study. For the first six and for the last four cases only the geometry of the building is taken into consideration, whereas the materials and reinforcement are changed based on the values obtained by the Albanian codes and for the last four cases based on the steel properties obtained by test results. For the seventh case, the reinforced concrete frame is analyzed for user-defined and default hinge properties.

4.3.1 Case 1

In the earthquake-resistant design practice of structures in Albania, the Ultimate Strength Design Method, combined with additional structural provisions is used. It aims at ensuring a sufficient enough strength and stability for building structures in order to resist the moderate and severe earthquake motions without any structural damage (37). According to the Albanian Code for earthquake design, the material strength requirements for concrete and steel in reinforced concrete structures are as shown below:

Concrete: Compression Strength of 200 kgf/cm^2 (20 MPa) and higher

Steel: Characteristic Yield Strength of 3800 – 4500 kgf/cm^2 (380 – 450 MPa)

For the first case the following material characteristics are used:

Table 4.2: *Material characteristics for Case 1*

Concrete	Steel
20 MPa	415 MPa

The reinforcement for the beams and columns is calculated based on Albanian Code. The minimum percentage of reinforcement is considered in this case.

Table 4.3: *Steel Percentage for Case 1*

Min. Percentage of Steel for Reinforced Concrete Beams	Min. Percentage of Steel for Reinforced Concrete Columns
$\mu = 0.5\%$	$\mu = 0.5\% \div 2.5\%$

Based on the values shown below, the area of steel is calculated (36) (56) (57).

For beams:

$$A_{s,min} = {}^{0.5}/_{100}\ \ b\ h_0$$

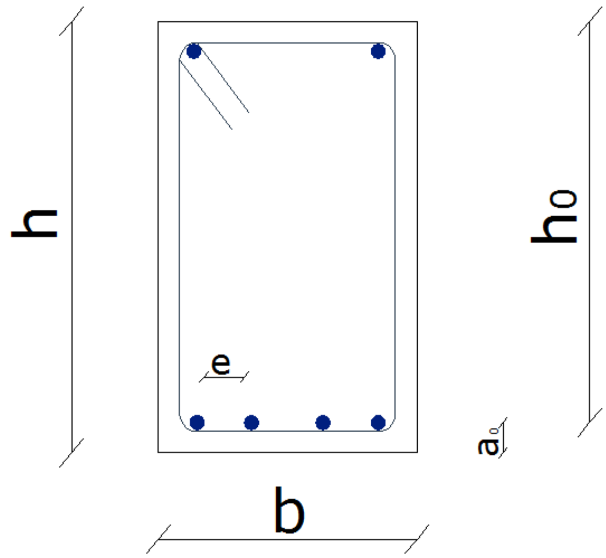


Figure 4.6: *Main characteristics of the beam cross section*

Where:

$h_0 = h - a$

h: height of the beam

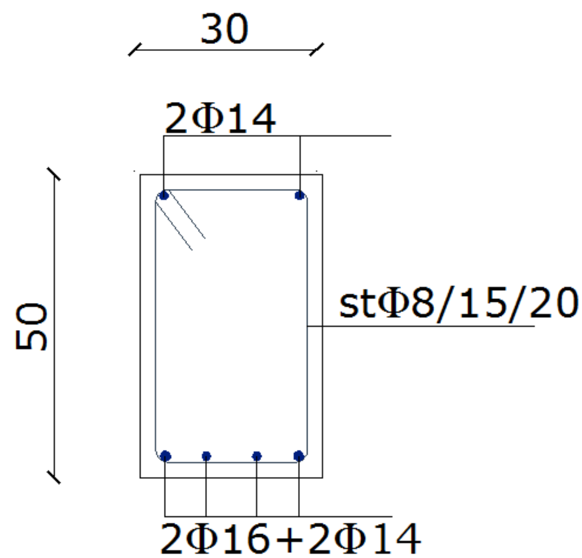
$a = 3.5$ (for reinforcement diameter $d \leq 20mm$)

$a = 4$ (for reinforcement diameter $d > 20mm$)

In this case, the minimum steel area for the beams is:

$$A_{s,min} = \frac{0.5}{100}\, 30\, (50 - 3.5) = 6.975\ cm^2$$

The beams are reinforced with $2\emptyset 16 + 2\emptyset 14$ bars with a total area of steel $7.01\ cm^2$.



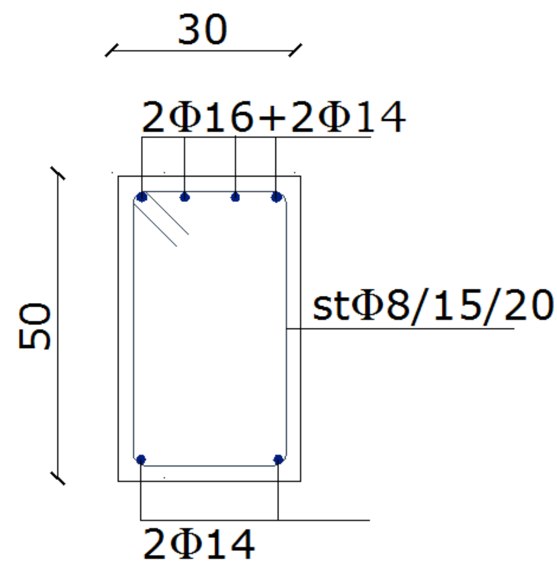


Figure 4.7: *Beam reinforcement for Case 1 (span)* **Figure 4.8**: *Beam reinforcement for Case 1 (support)*

For the columns the minimum percentage of steel varies between $0.5\% \div 2.5\%$ and it depends on the ratio shown below (38):

Table 4.4: *Steel percentage for columns*

$\frac{l_0}{i} < 17$	$17 \leq \frac{l_0}{i} \leq 35$	$35 \leq \frac{l_0}{i} \leq 83$	$\frac{l_0}{i} > 83$
$\mu = 0.5\%$	$\mu = 1\%$	$\mu = 2\%$	$\mu = 2.5\%$

Where:

l_0: is the calculated length of the element. For the fixed column $l_0 = L/3$, where L is the total length of the element.

$$l_0 = \frac{320}{3} = 106.67\ cm$$

i: is the inertia ratio of the cross section, which is calculated:

$$i = \sqrt{\frac{I_c}{A_c}} = \sqrt{\frac{bh^3}{12bh}}$$

For the 60x40 columns:

$$i = \sqrt{\frac{60 * 40^3}{12 * 60 * 40}} = 11.55$$

$${}^{l_0}/_{i} = 9.2 < 17$$

The minimum percentage of steel in this case is $\mu = 0.5\%$.

For the 40x60 columns:

$$i = \sqrt{\frac{40 * 60^3}{12 * 60 * 40}} = 17.32$$

$${}^{l_0}/_{i} = 6.16 < 17$$

The minimum percentage of steel in this case is also $\mu = 0.5\%$.

The calculated steel area based on the minimum percentage of steel is:

$$A_{s,min} = {}^{0.5}/_{100}\ \ b\ h_0 = 11.3cm^2$$

Considering that the distance between the bars should not exceed 20 cm (39) (40) (48), $10\emptyset16$ are chosen as reinforcement.

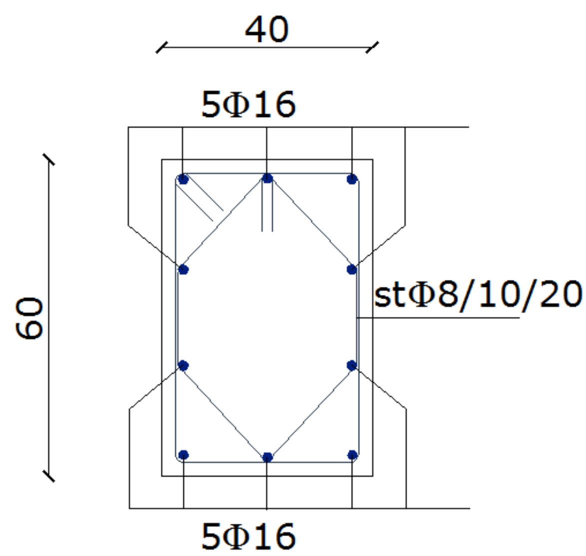


Figure 4.9: *Column reinforcement for Case 1*

The stirrup spacing for beams and columns respectively is taken as shown below, based on KTP-N.2-89 (Chapter 3, Section 3.16, Table 14 and 15) for and earthquake intensity $I > VIII$, whereas the diameter of the stirrups is 8 mm:

Table 4.5: *Stirrup spacing for beams and columns according to KTP-N.2-89*

	Beams	Columns
Non-critical zone	$a_{st} = 200mm$	$a_{st} = 200mm$
Critical zone	$a_{stz} = 150mm$	$a_{stz} = 100mm$

4.3.1.1 Modelling in Sap2000 for Case 1

In this section the procedure of performing a pushover analysis in Sap2000, for Case 1 is presented. The beams and columns are modeled using the proposed expressions for axial force-bending moment and moment-rotation presented in Chapter 3 (6) (26). The building frame is modeled in Sap2000 using the geometric and structural details mentioned in the sections above.

After setting the units to KN-m, and modelling the 2D Frame based on the geometric properties of the structure, it is necessary to mark the tip node as "Roof Top" to monitor the pushover curve at this node.

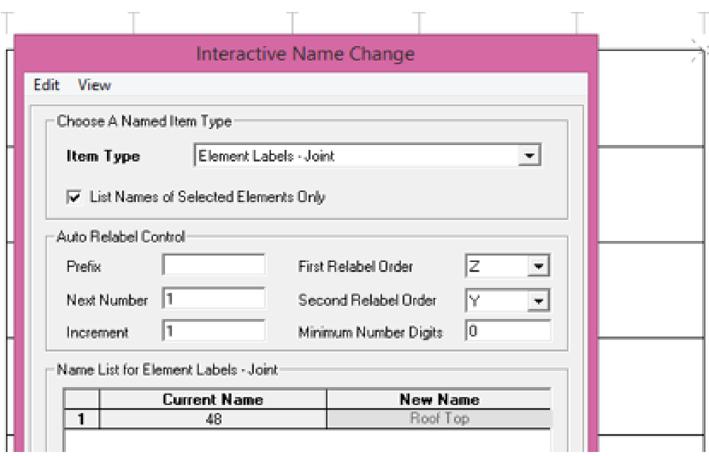


Figure 4.10: *Display of joint "Roof Top"*

The material properties for concrete and reinforcing steel and the beam and column sections are defined based on the characteristics shown in the previous section, and then assigned to the frame accordingly.

After that, the nonlinear hinges are defined.

The beam hinges are defined as deformation controlled, $P - M_3$ hinges.

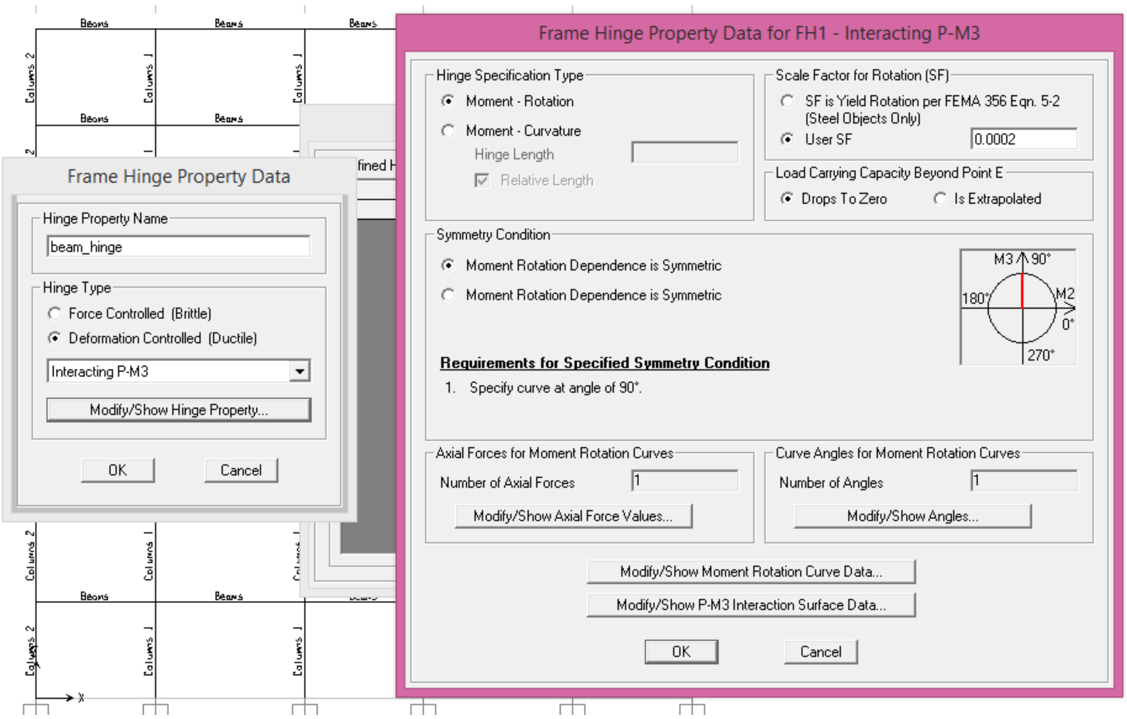


Figure 4.11: *Nonlinear hinge properties for beams, step 1*

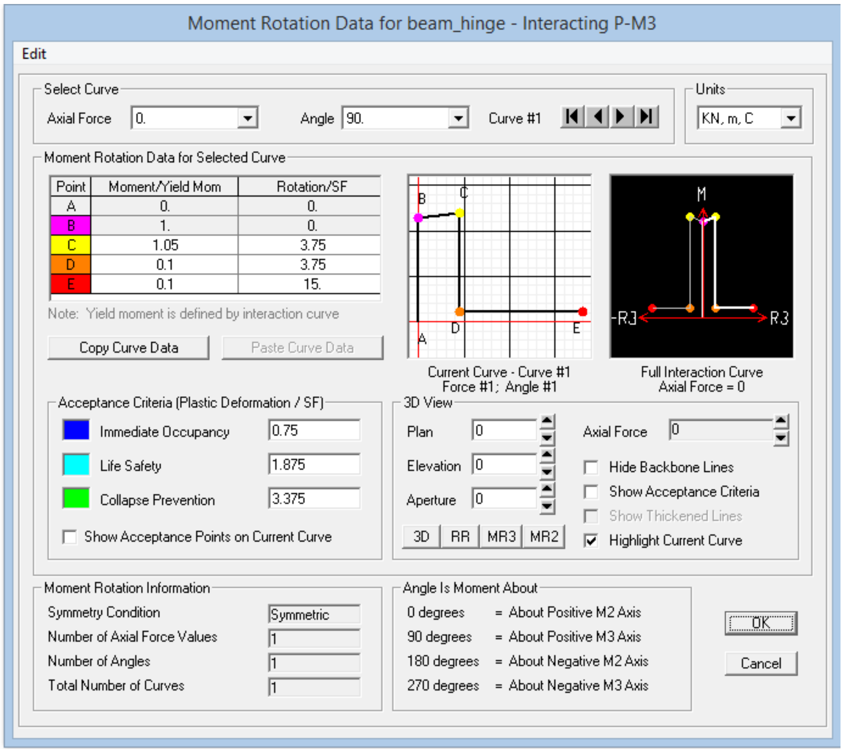


Figure 4.12: *Nonlinear hinge properties for beams, step 2*

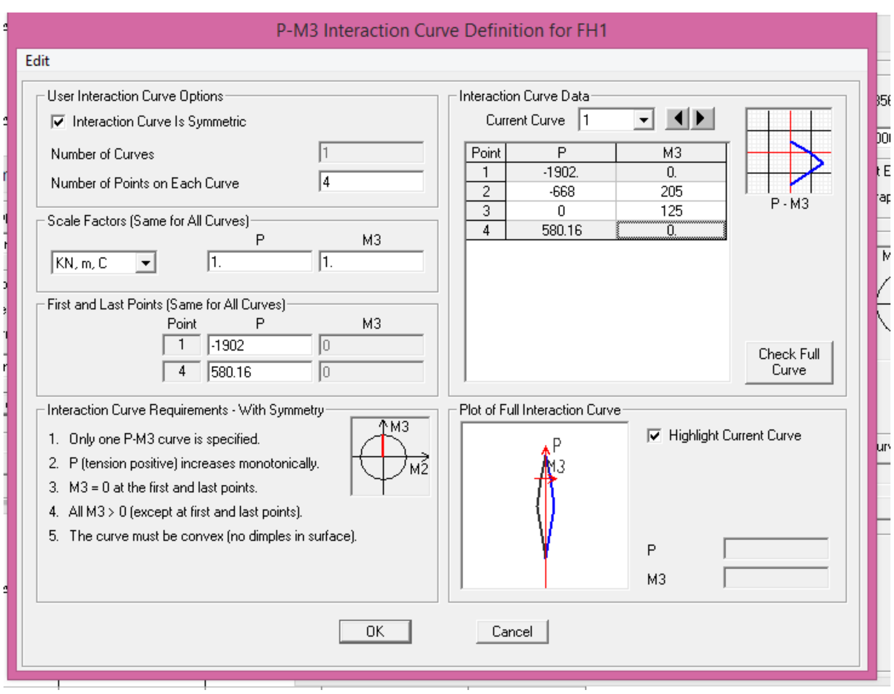


Figure 4.13: *Nonlinear hinge properties for beams, step 3*

After the $P - M_3$ interaction curve is defined, Sap2000 has an option to check full curve if it is acceptable.

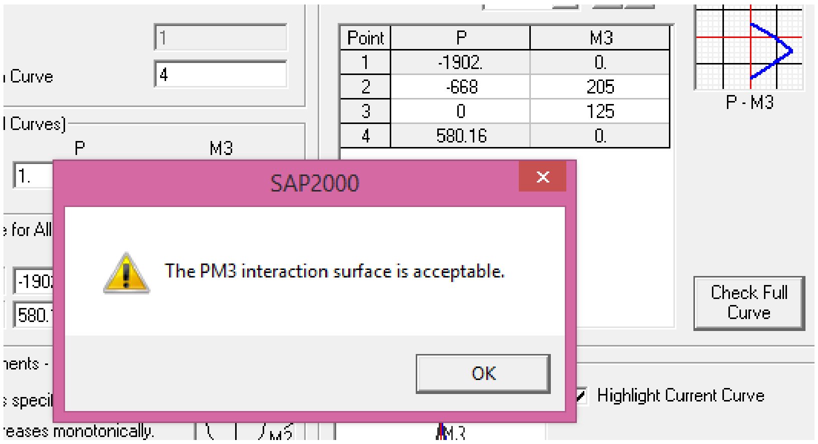


Figure 4.14: *Nonlinear hinge properties for beams, checking of the $P - M_3$ interaction surface*

The column hinges are defined as deformation controlled, $P - M_2$ hinges.

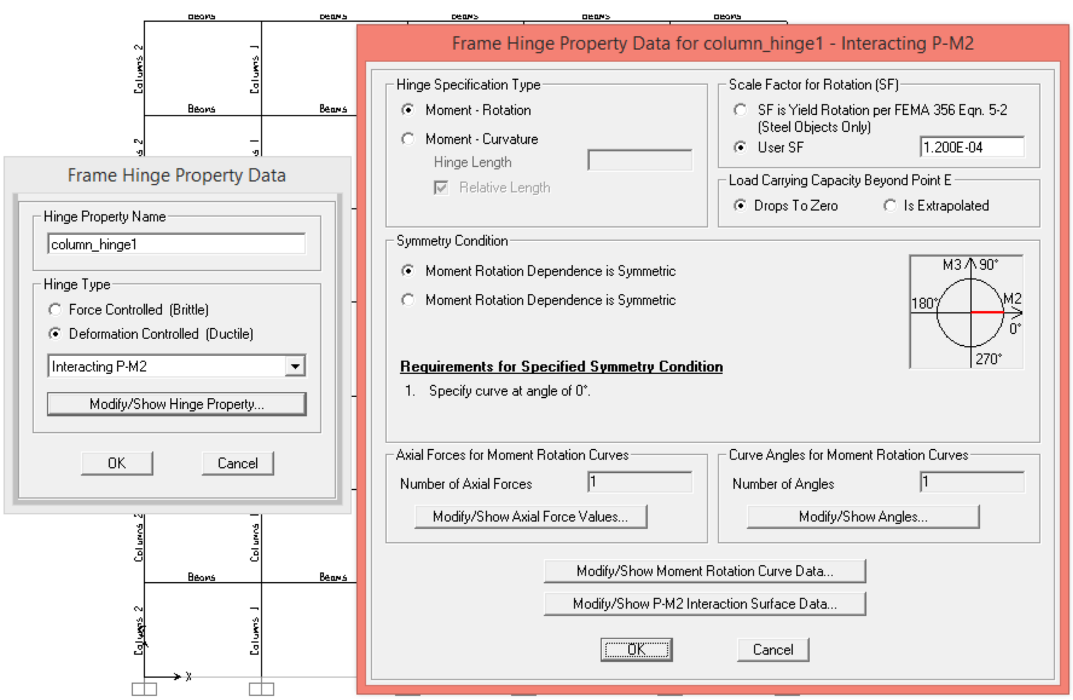


Figure 4.15: *Nonlinear hinge properties for beams, step 1*

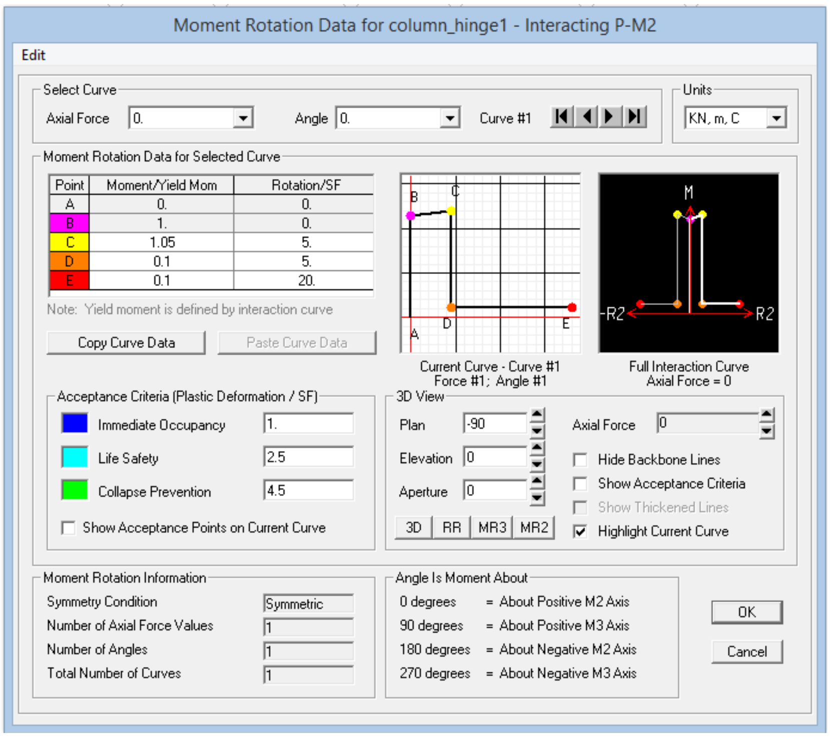


Figure 4.16: *Nonlinear hinge properties for beams, step 2*

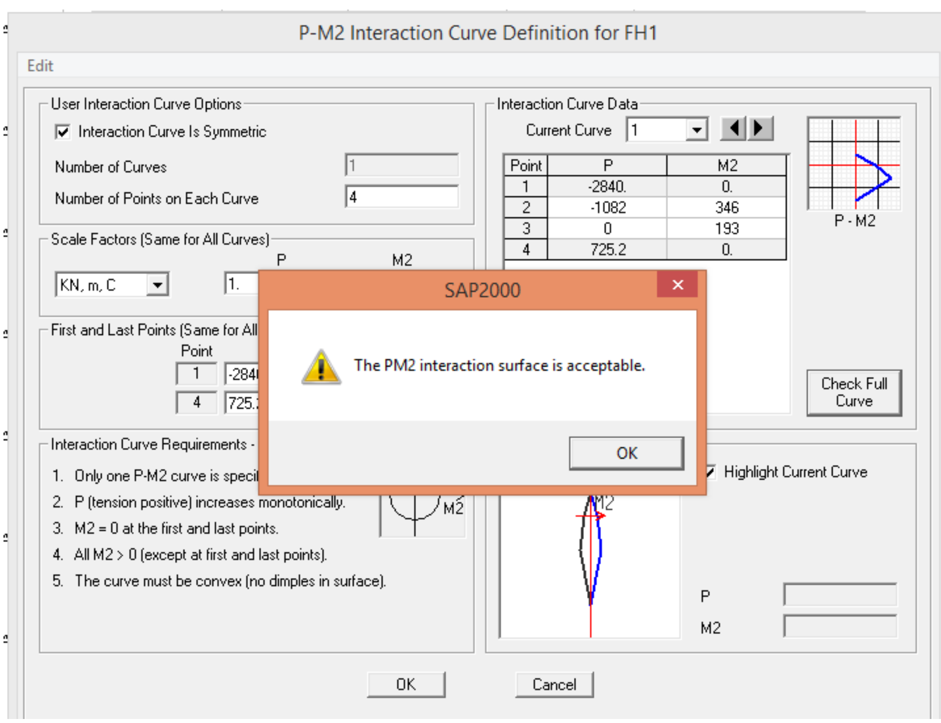


Figure 4.17: *Nonlinear hinge properties for beams, step 3*

To ensure rigidity of connections between beams and columns, end length offsets are assigned.

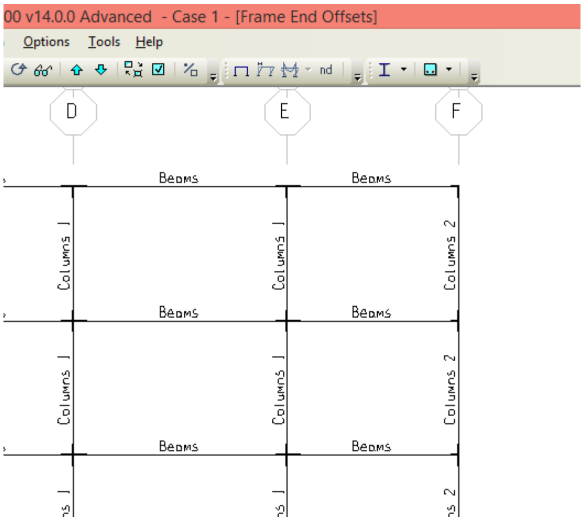


Figure 4.18: *Assigning end offsets*

The defined hinges are then assigned to the beams and columns respectively.

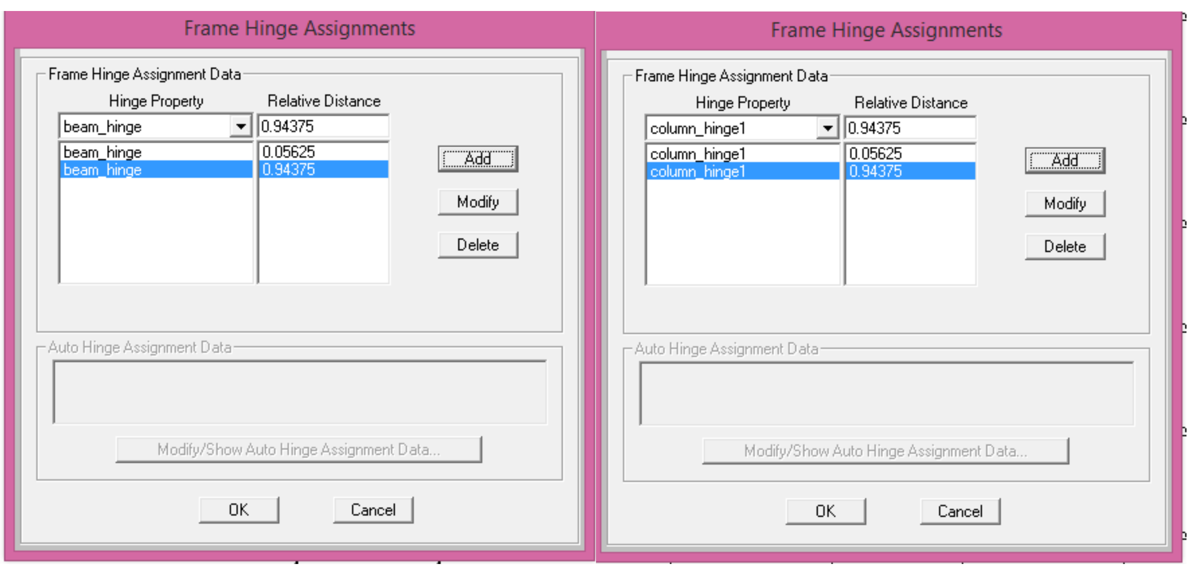


Figure 4.19: *Assigning tensile hinges to beams and compression hinges to columns*

The load patterns Dead, Dead from Slab, Live and Lateral are defined. Along with the dead load transmitted from the slabs to the beams, the dead load transmitted from the masonry walls is calculated, and assigned to the beams as a distributed load.

As stated by KTP 6-78 (*Section 4.1*), the live load value for residential buildings is 1.5 $^{KN}/_{m^2}$. This live load is also accordingly distributed to the beams from the slabs.

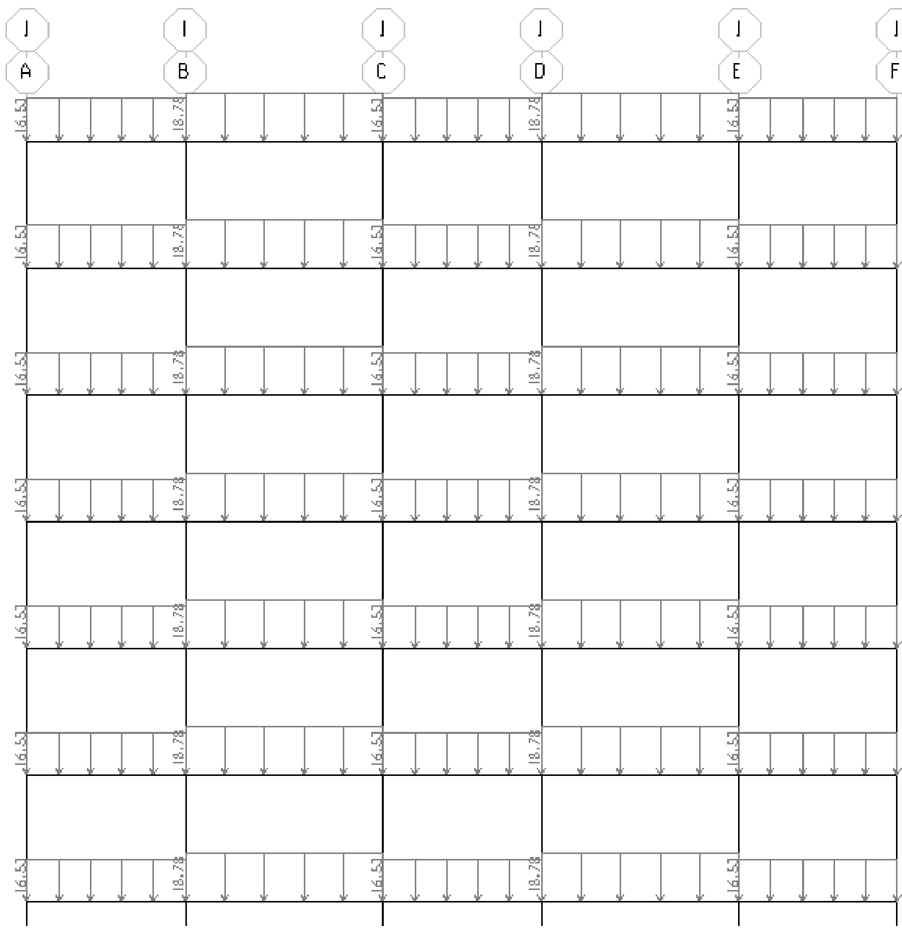


Figure 4.20: *Dead loads transmitted from slabs and walls, distributed to the beams*

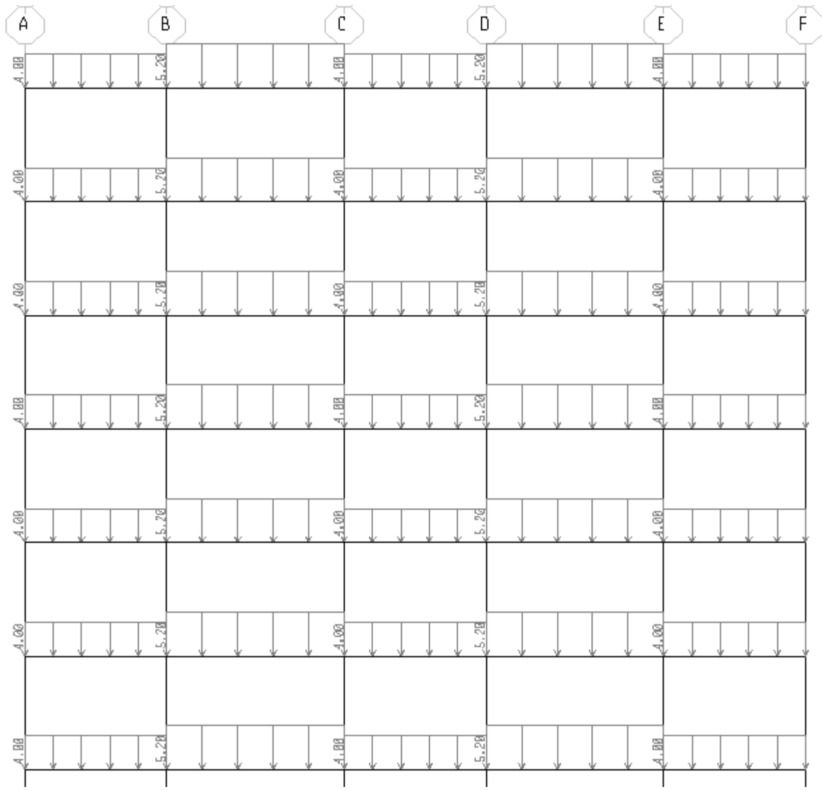


Figure 4.21: *Live load transmitted from slabs, distributed to beams*

After that, diaphragm action is assigned to the model at each floor separately.

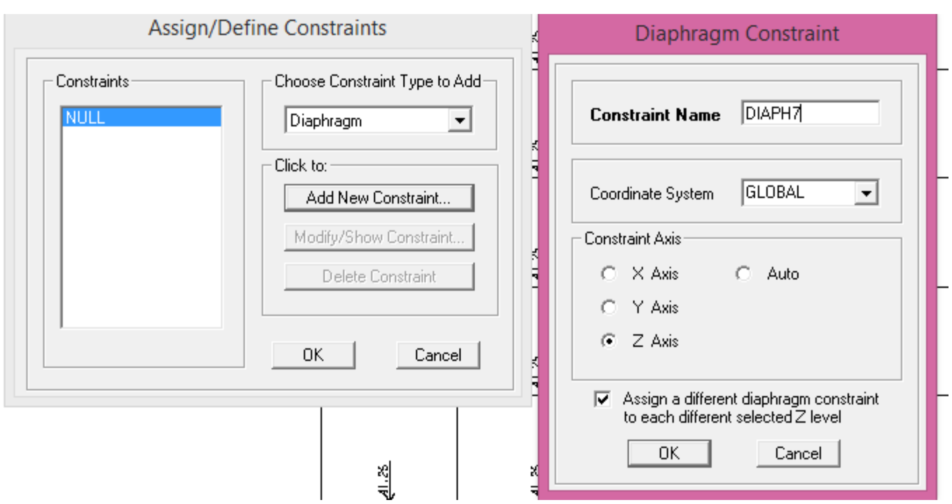


Figure 4.22: *Assigning diaphragm action*

The pushover loads, calculated in the previous Chapter are assigned at each floor level in the global X direction.

Subsequently, the analysis cases are defined. It is essential to apply the dead and live loads on the frame before the frame is pushed using lateral load. So, the "Initial Pushover" case is defined first.

The seismic inertia forces are considered as specific loads in *KTP-N-2-89*. The load combination which includes seismic forces is classified as specific combination. In case of specific combination of loads, the combination coefficient for seismic forces is equal to 1.0, whereas for dead loads this coefficient is 0.9 and for live loads 0.4 (*KTP-N-2-89, Section2.3*).

The "Pushover" analysis case is applied using displacement control, continuing from the previous case. While defining the parameters, results are saved at multiple states to trace the formation of hinges.

The modelling part is now complete, the last step is running the analysis and obtaining the results.

4.3.2 Case 2

For the second case the same material characteristics as in Case 1 are used:

Table 4.6: *Material characteristics for Case 2*

Concrete	Steel
20 MPa	415 MPa

The reinforcement for the beams and columns is calculated based on Albanian Code. In this case the maximum percentage of reinforcement is considered.

Table 4.7: *Steel Percentage for Case 2*

Max. Percentage of Steel for Reinforced Concrete Beams	Max. Percentage of Steel for Reinforced Concrete Columns
$\mu = 4\%$	$\mu = 4\%$

Based on the values shown below, the area of steel is calculated.

For beams:

$$A_{s,max} = \frac{4}{100}\ b\ h_0 = \frac{4}{100}\ 30\ (50 - 4) = 55.2\ cm^2$$

$$h_0 = h - a$$

$a = 4$ in this case, because the reinforcement diameter $d > 20mm$

The maximum rebar diameter for beams with a height between 30 cm to 90 cm, is 28mm (56) (57) (58). Considering that the minimum spacing between the rebars should be 2.5 cm, for this case the beams are reinforced with $8\emptyset 28$ bars with a total area of steel $49.26\ cm^2$.

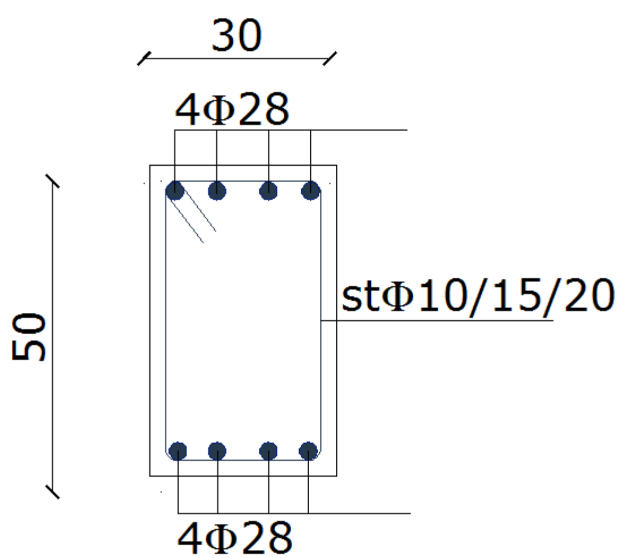


Figure 4.23: *Beam reinforcement for Case 2*

The calculated steel area for columns is:

$$A_{s,max} = \frac{4}{100}\ b\ h_0 = \frac{4}{100}\ 40\ (60 - 4) = 89.6\ cm^2$$

The columns are reinforced with $14\emptyset 28$ bars with a total area of steel $86.1\ cm^2$.

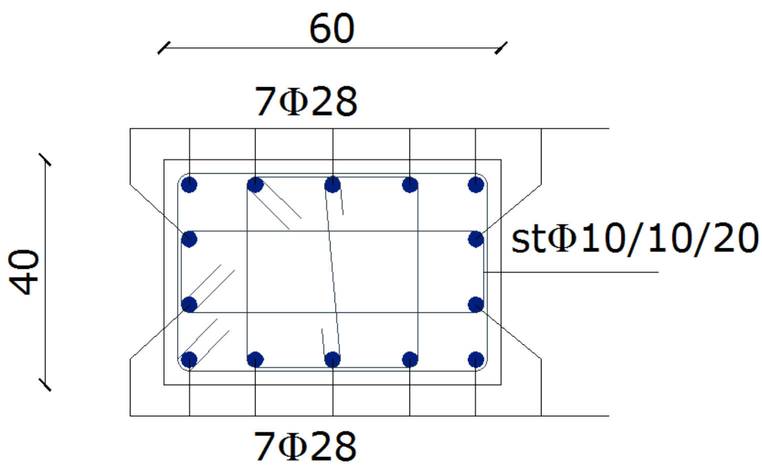


Figure 4.24: *Column reinforcement for Case 2*

The stirrup diameter for critical sections of columns in seismic regions, should not be smaller than $1/_3$ of the diameter of the longitudinal rebars. In this case stirrups with a diameter of 10 mm are used. The stirrup spacing is the same with the one used in Case 1.

4.3.2.1 Modelling is Sap2000 for Case 2

For Case 2, the same modelling steps as the ones shown in section 4.3.1.1 for Case 1 are followed. When defining the section properties, the reinforcement details shown in the section above are used. The axial force-bending moment interaction curves for beams and columns hinges are re-calculated and defined, and then assigned to the respective sections.

After the necessary modifications are made to the model, the frame is analyzed once again, to obtain results for Case 2.

4.3.3 Case 3 to 6

For the next four cases, the material properties of concrete and steel are changed to minimum and maximum values, to point out the effect these characteristic strengths have on the pushover analysis. The minimum and maximum characteristic yield strengths of steel are taken based on the Albanian Code (37) and the minimum and maximum compression strength values for concrete are taken from KTP-N.30-91. The material properties used for cases 3 to 6 are summarized in the table below:

Table 4.8: *Summarized material Properties for Cases 3, 4, 5, 6*

	Concrete Compression Strength (MPa)	Steel Characteristic Yield Strength (MPa)
Case 3	15 (Min)	450 (Max)
Case 4	30 (Max)	380 (Min)
Case 5	15 (Min)	380 (Min)
Case 6	30 (Max)	450 (Max)

4.3.3.1 Modelling in Sap2000 for Cases 3 to 6

The same modelling steps as the ones shown in previous sections for Case 1 and 2 are followed. When defining the material properties, the concrete and steel characteristic strengths shown in the table above are used. The axial force-bending moment interaction curves for beams and columns hinges are re-calculated and defined, and then assigned to the respective sections.

4.3.4 Case 7

Pushover analysis can be carried out for either user-defined nonlinear hinge properties or default hinge properties, available in Sap2000, based on *FEMA 356* or *ATC 40* guidelines. Case 7 is included in the study to compare and point out the differences in the results of pushover analysis due to default and user-defined hinge properties. The material and section properties, along with the reinforcement detailing are taken as they are in the original project.

Table 4.9: *Summarized data for Case 7a and 7b*

	Concrete	Steel	Nonlinear hinges
Case 7a	30 MPa	355 MPa	User-defined
Case 7b	30 MPa	355 MPa	Default

4.3.4.1 Modelling in Sap2000 for Case 7a

User-defined nonlinear hinge properties are applied in Case 7a. All the modelling steps are the same as the ones followed in the previous cases. The material properties used for the Sap2000 model are shown in Table 4.9. The axial force-bending moment interaction curves are calculated for all the beam and column sections.

After the respective hinges are defined and assigned, the analysis is carried out, and the pushover curve is obtained.

4.3.4.2 Modelling in Sap2000 for Case 7b

Default hinge properties are used for Case 7b, to identify the differences in the results of pushover analysis compared to Case 7a. In practical use, most often, the default hinge properties, provided by *FEMA 356* and *ATC 40* are preferred, because of the simplicity and convenience. Sap2000 has already implemented these default hinge properties. The modelling process is carried out, with all the steps and details mentioned in the previous sections. When it comes to assigning the hinges, the sections are first selected (beams and then columns), and the respective hinges are assigned to them, based on *FEMA 356* tables for concrete beams and columns. This procedure is shown in the figures below:

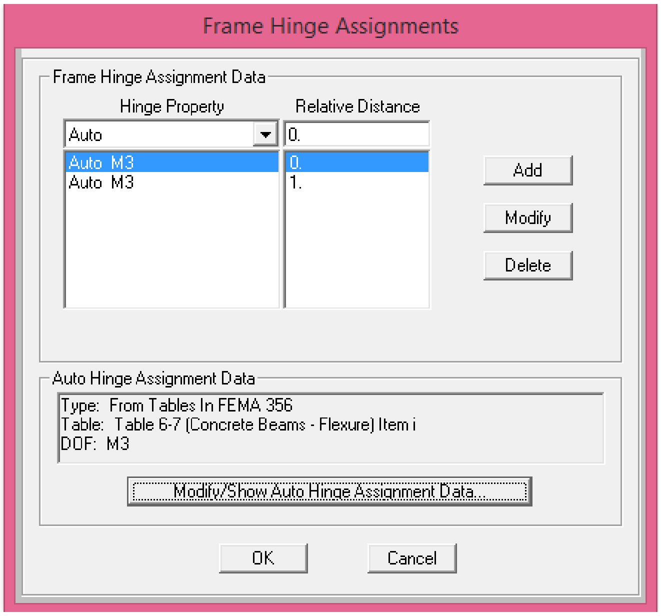


Figure 4.25: *Hinge assignments for beams, step 1*

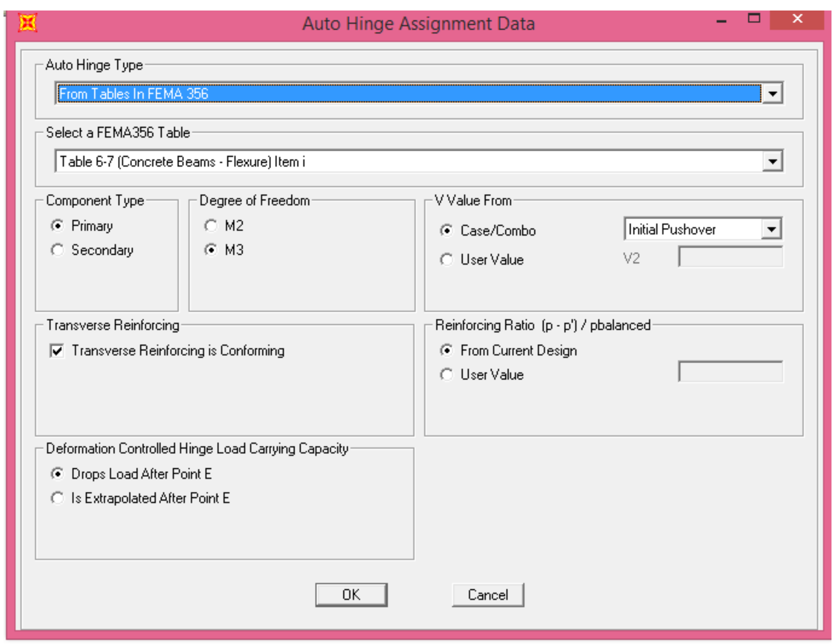


Figure 4.26: *Hinge assignments for beams, step 2*

The beam hinges are defined as M_3 hinges, and the default properties are based on Table 6-7 of *FEMA 356*, for concrete beams in flexure. The column hinges, on the other hand, are defined as $P - M_2$ hinges, as it is shown in the figures below:

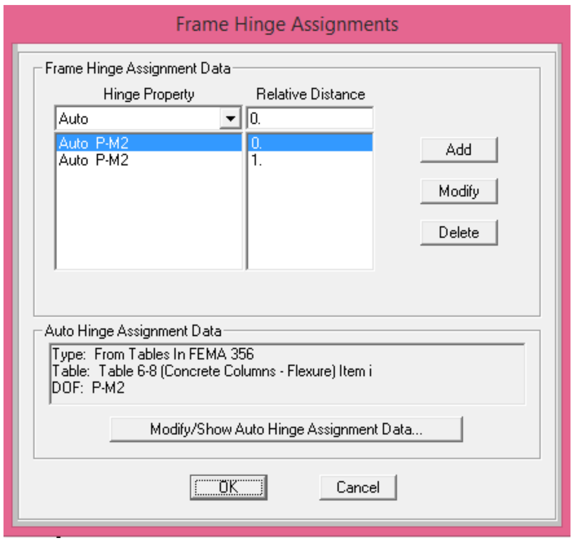


Figure 4.27: *Hinge assignments for columns, step 1*

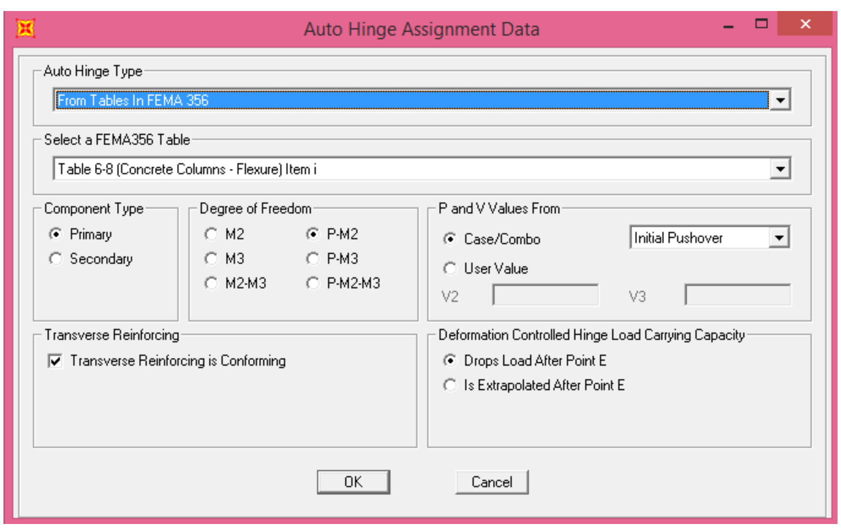


Figure 4.28: *Hinge assignments for columns, step 2*

All the other modelling characteristics remain the same with the previous case. The analysis is run, and the results for default hinge properties are obtained.

4.3.5 Cases 8 to 11

For the last four cases, the minimum and maximum compression strength values for concrete are taken from KTP-N.30-91. The minimum and maximum steel yield strengths are taken from the results of tension steel tests, on specimens of reinforcing steel used in Albania.

The reinforcement steel is subjected to the tensile test in laboratory conditions, for each specimen the test is carried out three times. The diameter of the tested specimens varies from 5 mm to 32mm, for each of them the values of the yield strength, tensile strength and relative elongation are obtained. The tensile test is carried out for a total of 1559 specimens. The chosen values to use for Case 8 to 11 are the minimum value, 470 MPa, and the mean value, 550 MPa.

The combination of material properties used for cases 8 to 11 is summarized in the table below:

Table 4.10: *Summarized material Properties for Cases 8, 9, 10, 11*

	Concrete Compression Strength (MPa)	Steel Characteristic Yield Strength (MPa)
Case 8	15 (Min)	470 (Min)
Case 9	30 (Max)	550 (Mean)
Case 10	15 (Min)	550 (Mean)
Case 11	30 (Max)	470 (Min)

4.3.5.1 Modelling is Sap2000 for Cases 8 to 11

The concrete and steel strengths shown in the table above are used when defining the materials in Sap 2000 for cases 8, 9, 10 and 11. The axial force-bending moment interaction curves for beams and columns hinges are re-calculated and defined, and then assigned to the respective sections, as it is explained in details for the user-defined hinges in the previous cases.

4.4 Cases and Modelling Summary

Table 4.11: *Cases Summary*

Cases Summary			
Cases	Concrete (MPa)	Steel (MPa)	
Case 1	20	415	Min. Steel %
Case 2	20	415	Max. Steel %
Case 3	15 (Min)	450 (Max)	
Case 4	30 (Max)	380 (Min)	
Case 5	15 (Min)	380 (Min)	
Case 6	30 (Max)	450 (Max)	
Case 7	30	355	User-defined
			Default
Case 8	15 (Min)	470 (Min)	
Case 9	30 (Max)	550 (Max)	
Case 10	15 (Min)	550 (Max)	
Case 11	30 (Max)	470 (Min)	

Table 4.12: *Modelling Assumptions Summary*

Modelling Assumptions		
Material	Stress-Strain Relationship	Concrete (According to D.M. 9 gennaio 1996)
		Steel (Bilinear elasto-plastic model)
Loading	Self Weight of Members	Weight per unit volume 24 KN/m3
	Seismic Design Load Combination	0.9 (dead load)+ 0.4 (live load)+ (seismic action)
	Mass Source	From element and additional masses
	P-delta effect	Not considered
Structural Modelling	Analysis Program	Sap 2000 V14.0.0
	Rigid offset at beam-column joints	At beam and column ends
	Slab Modelling	Rigid Diaphragm
	Axial force-Bending moment Interaction	Yes

Chapter 5

Results and Discussions

5.1 Introduction

In this chapter, analysis and results of the study will be discussed. Pushover analysis of the reinforced concrete frame building is performed in Sap2000 software. The capacity curves (base shear vs. roof displacement curves) are generated for each case, based on their respective characteristics, presented and explained in the previous chapter, along with the tables that give information about the number of hinges formed at different performance levels. In all cases lateral forces are applied monotonically with step by step nonlinear analysis (displacement controlled). The capacity curve gives an insight of the maximum base shear that the structure can resist, for each case.

5.2 Analysis Results for Case 1

After the pushover analysis for Case 1 is carried out in Sap2000, the analysis results are obtained. Table 5.1 shown below displays the number of hinges formed at each performance level, and the displacement and base shear at each step of the analysis.

Table 5.1: *Summarized pushover analysis results for Case 1*

TABLE: Pushover Curve - Pushover											
Step	Displacement	BaseForce	AtoB	BtoIO	IOtoLS	LStoCP	CPtoC	CtoD	DtoE	BeyondE	Total
	m	KN									
0	-2.04E-17	0	154	0	0	0	0	0	0	0	154
1	0.017772	379.471	152	2	0	0	0	0	0	0	154
2	0.029791	546.02	128	10	2	0	0	14	0	0	154
3	0.040333	615.67	113	11	1	5	2	22	0	0	154
4	0.124004	792.023	99	5	0	0	0	50	0	0	154
5	0.215818	906.978	91	3	0	0	0	60	0	0	154
6	0.288307	978.398	84	3	0	2	0	65	0	0	154
7	0.306582	987.928	84	0	0	0	1	69	0	0	154
8	0.387365	1009.561	84	0	0	0	0	70	0	0	154
9	0.467365	1030.668	84	0	0	0	0	70	0	0	154
10	0.547365	1054.693	84	0	0	0	0	70	0	0	154
11	0.627365	1079.553	84	0	0	0	0	70	0	0	154
12	0.707365	1104.81	84	0	0	0	0	70	0	0	154
13	0.787365	1130.068	84	0	0	0	0	70	0	0	154
14	0.867365	1155.35	84	0	0	0	0	70	0	0	154
15	0.947365	1180.677	84	0	0	0	0	70	0	0	154
16	1.027365	1206.005	84	0	0	0	0	70	0	0	154
17	1.107365	1231.333	84	0	0	0	0	70	0	0	154
18	1.187365	1256.695	84	0	0	0	0	70	0	0	154
19	1.267365	1282.057	84	0	0	0	0	70	0	0	154
20	1.347365	1307.419	84	0	0	0	0	70	0	0	154
21	1.427365	1332.782	84	0	0	0	0	70	0	0	154
22	1.507365	1358.144	84	0	0	0	0	70	0	0	154
23	1.587365	1383.506	84	0	0	0	0	70	0	0	154
24	1.6	1387.512	84	0	0	0	0	70	0	0	154

The deformed shape due to pushover analysis and the sequence of hinge formation on a step by step basis has been obtained and shown in the figures below:

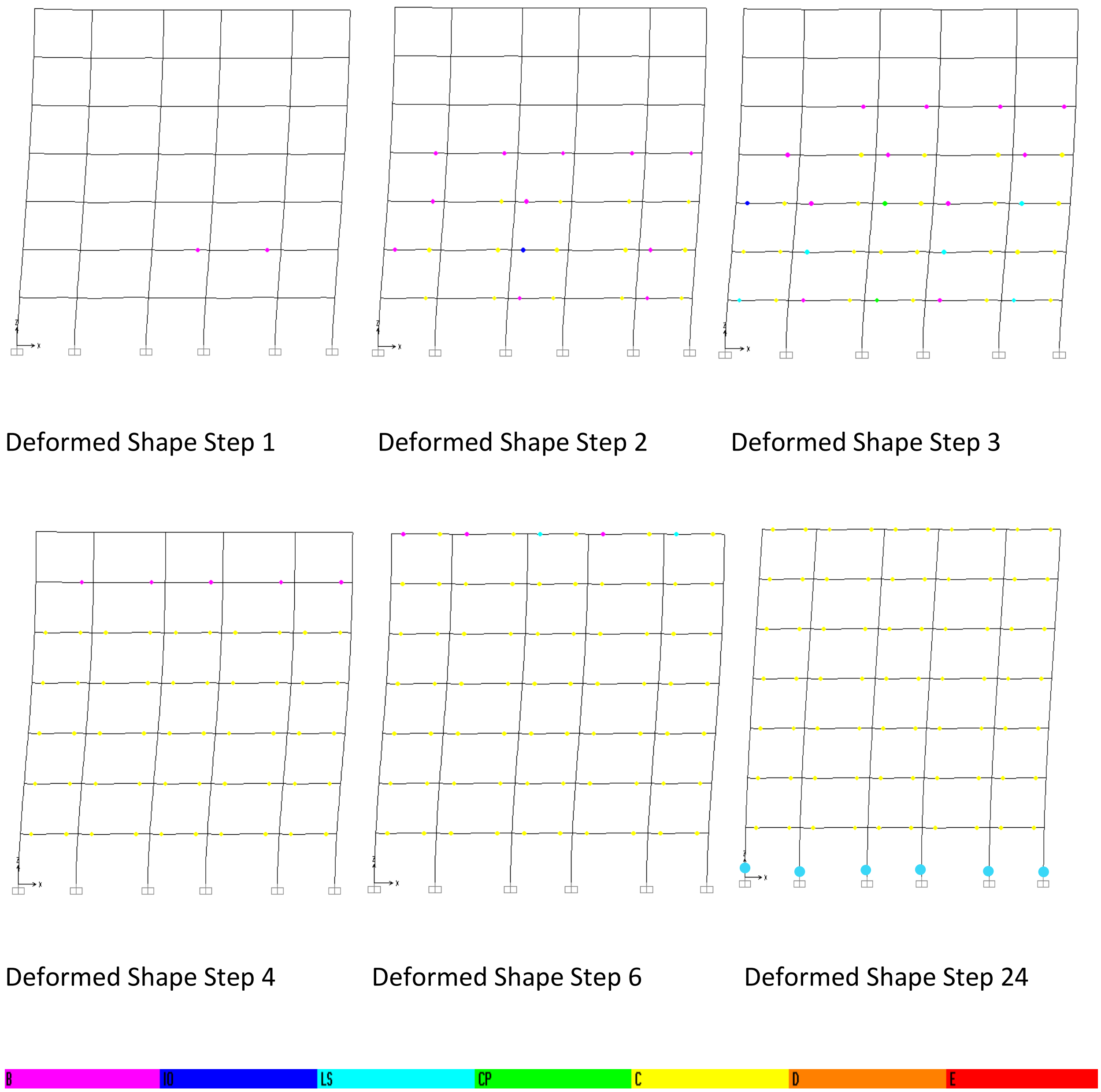


Figure 5.1: *Step by step plastic hinge formation*

As it can be seen from Table 5.1 and the hinging patterns shown above, from step 3 to the final step, a large number of hinges fall into the C to D state, where significant strength degradation and initial failure begins. Resistance to lateral loads beyond point C is usually unreliable. Considering that for Case 1, the steel to concrete ratio, is at a minimum value, based on the

Albanian Codes of practice, such results were expected. It is obvious from the analysis that the beams of the structure need strengthening. However, sequence of formation of plastic hinges (yielding) in the frame members can be clearly seen in the beams only, which shows that the building behaves like the strong column weak beam mechanism.

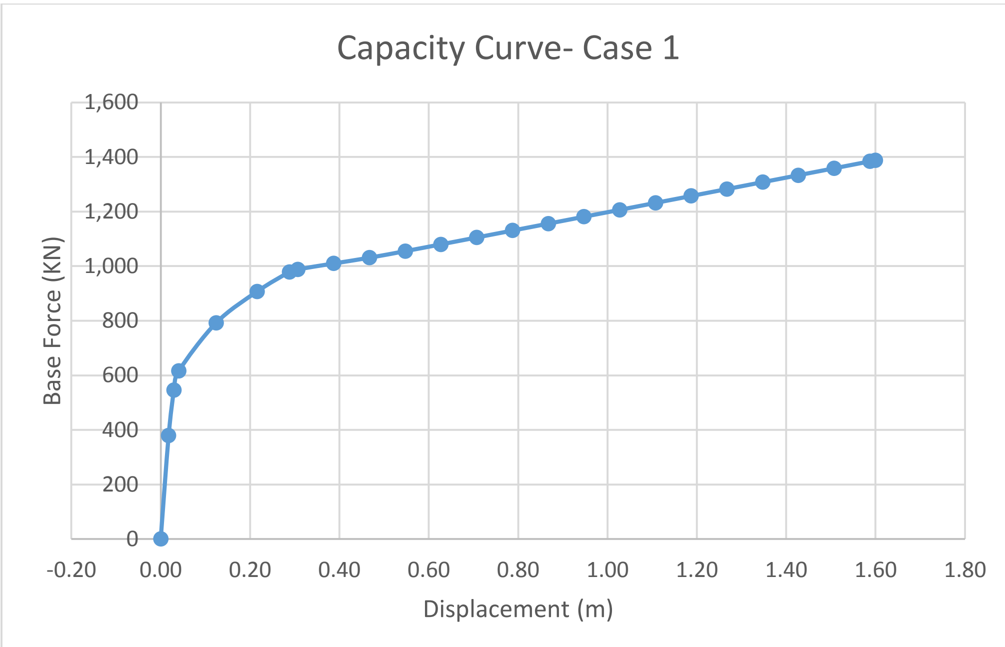


Figure 5.2: *Capacity Curve for Case 1*

Based on the results for displacement-base shear values obtained by the analysis in Sap2000 and shown in Table 5.1, the pushover curve is generated. The behavior of the structure is observed to be linear up to the value of base shear around 546 KN, and the value of displacement around 2.9 cm. The ultimate base shear at the assigned displacement of 1.6 m, is 1387.5 KN.

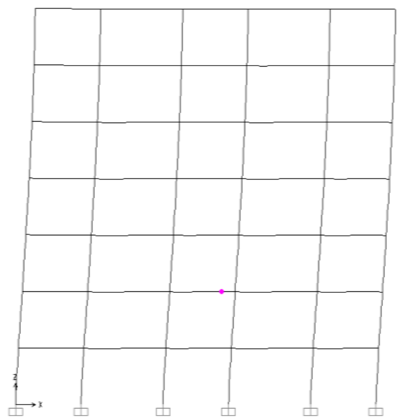
5.3 Analysis Results for Case 2

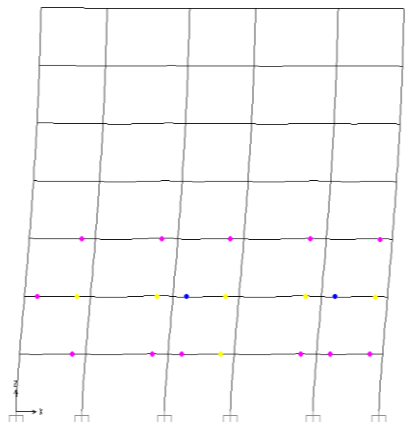
Case 2 takes into consideration the maximum percentage of steel. All the material characteristics are the same as the ones used for Case 1, only the steel to concrete ratio has been changed to reveal its effect to the pushover analysis results. Table 5.2 shows all the summarized outcomes, attained by the pushover analysis for Case 2.

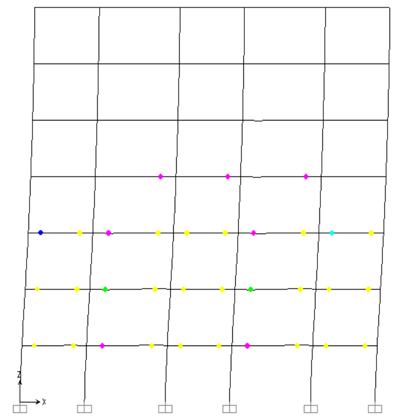
Table 5.2: *Summarized pushover analysis results for Case 2*

TABLE: Pushover Curve - Pushover											
Step	Displacement	BaseForce	AtoB	BtoIO	IOtoLS	LStoCP	CPtoC	CtoD	DtoE	BeyondE	Total
	m	KN									
0	-1.74E-17	0	154	0	0	0	0	0	0	0	154
1	0.05971	1275.831	153	1	0	0	0	0	0	0	154
2	0.07565	1562.457	134	11	2	1	0	6	0	0	154
3	0.092125	1700.949	121	7	1	1	2	22	0	0	154
4	0.176171	1946.128	110	4	0	0	0	40	0	0	154
5	0.324149	2190.917	103	1	0	0	0	50	0	0	154
6	0.40977	2311.284	97	3	0	1	0	53	0	0	154
7	0.561563	2477.539	94	0	0	0	0	60	0	0	154
8	0.641563	2561.571	94	0	0	0	0	60	0	0	154
9	0.749268	2673.193	91	2	0	0	0	61	0	0	154
10	0.813339	2731.103	84	1	0	3	0	66	0	0	154
11	0.824878	2737.335	84	0	1	0	0	69	0	0	154
12	0.835267	2740.387	84	0	0	0	0	70	0	0	154
13	0.915267	2752.779	84	0	0	0	0	70	0	0	154
14	0.995267	2764.791	84	0	0	0	0	70	0	0	154
15	1.075267	2778.236	84	0	0	0	0	70	0	0	154
16	1.155267	2792.953	84	0	0	0	0	70	0	0	154
17	1.235267	2808.664	84	0	0	0	0	70	0	0	154
18	1.315267	2825.12	84	0	0	0	0	70	0	0	154
19	1.395267	2841.58	84	0	0	0	0	70	0	0	154
20	1.475267	2858.04	84	0	0	0	0	70	0	0	154
21	1.555267	2874.501	84	0	0	0	0	70	0	0	154
22	1.6	2883.706	84	0	0	0	0	70	0	0	154

As it can be seen, not only the base shear values are different from Case 1, but also the number of the formed hinges at each stage.



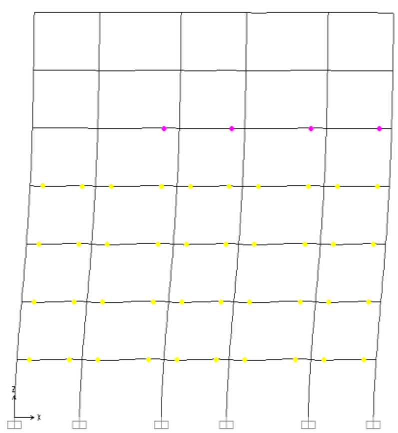


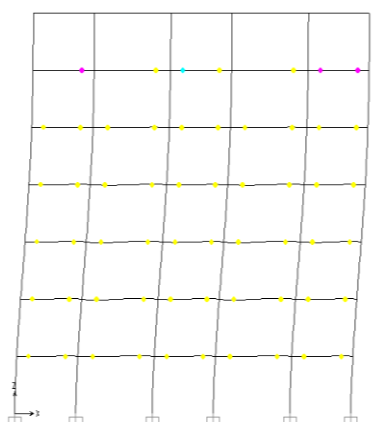


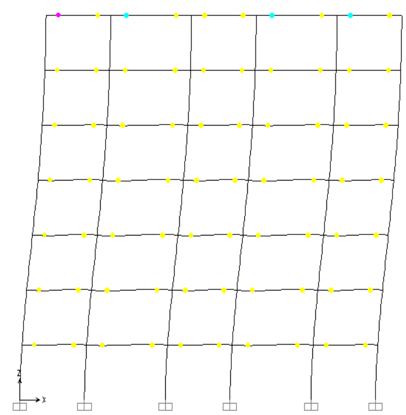
Deformed Shape Step 1

Deformed Shape Step 2

Deformed Shape Step 3



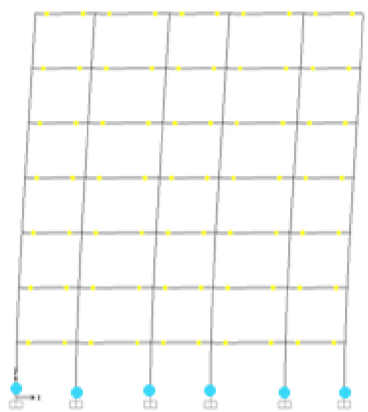




Deformed Shape Step 4

Deformed Shape Step 6

Deformed Shape Step 10



Deformed Shape Step 22

Figure 5.3: *Step by step plastic hinge formation*

The hinging patterns are shown step by step in the figures above. The behavior of the frame is linear up to the value of base shear around 1562, and a 7.5 cm displacement. The value of the base shear at the maximum deflection reaches 2883 KN.

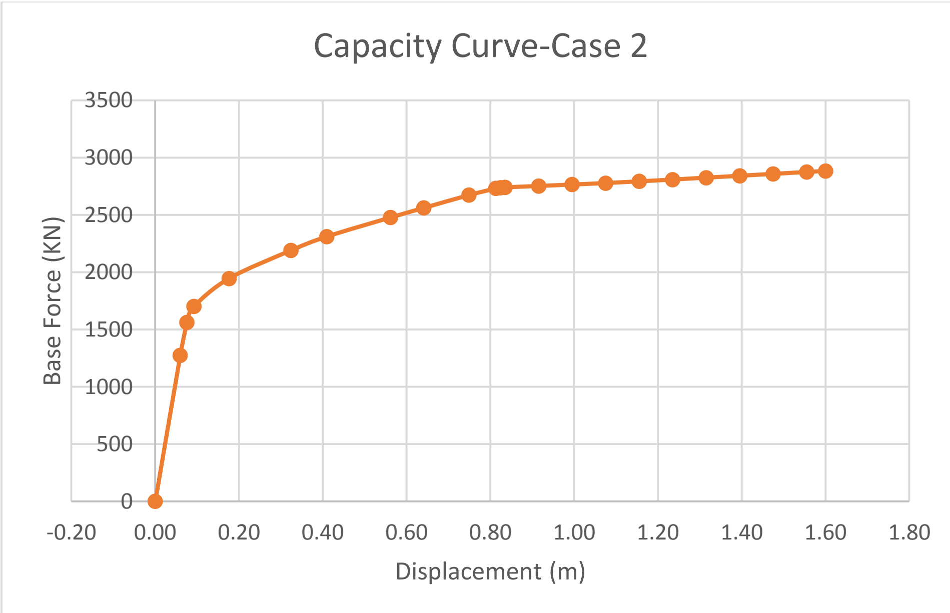


Figure 5.4: *Capacity Curve for Case 2*

5.4 Analysis Results for Case 3 to 6

For Cases 3 to 6, the material properties of concrete and steel are changed to minimum and maximum values. The minimum and maximum characteristic yield strengths of steel are taken based on the Albanian Code (37) and the minimum and maximum compression strength values for concrete are taken from KTP-N.30-91. The aim is not only to observe the effects concrete and steel properties have on the pushover analysis, but also to test in a way the limit values presented by the Albanian Codes of practice.

For each case, the analysis is carried out with all the details stated in the previous Chapter, and the obtained results are summarized in the Capacity Curves.

The sequence of formation of the plastic hinges can be distinctly seen in the beams only, for all 4 cases.

The hinging pattern for Cases 3 to 6 is similar, there are only a few differences in the number of the hinges in distinctive stages that can be seen more clearly when comparing Case 5 and Case 6 to each other.

The capacity curves for each case are shown in the Figure below. There is a 16 % increase in the maximum base shear when comparing the case with the minimum values of concrete and steel strength (Case 5), to the one with maximum characteristic strength values (Case 6).

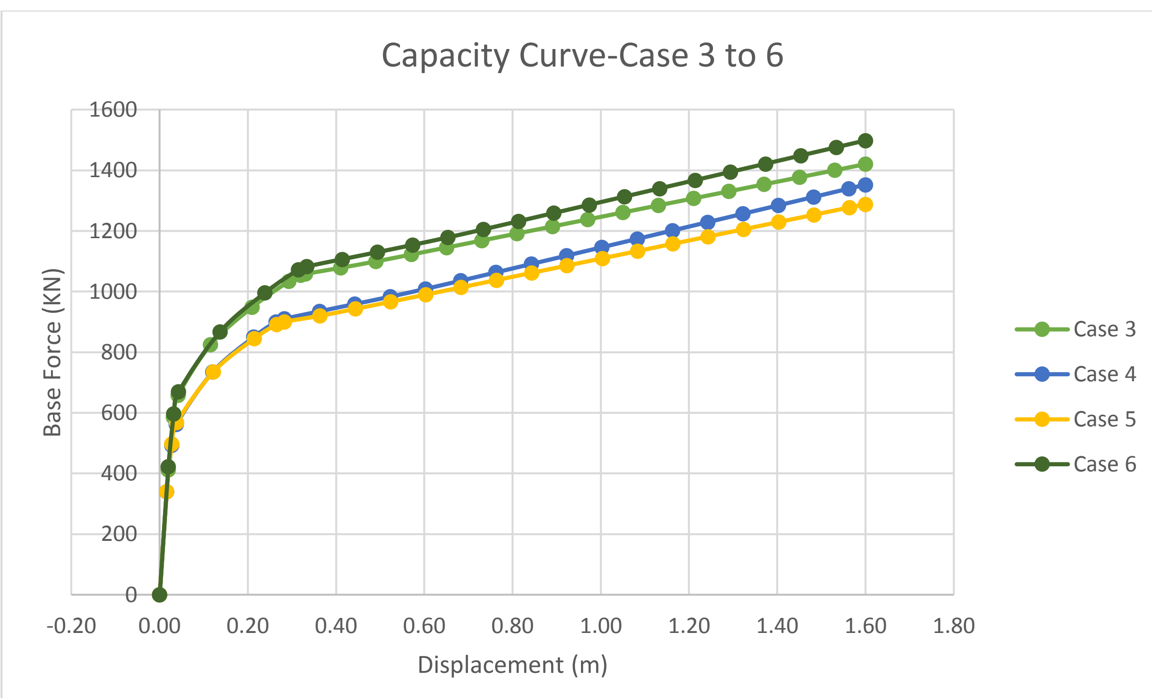


Figure 5.5: *Capacity Curve for Case 3 to 6*

5.5 Analysis Results for Case 7

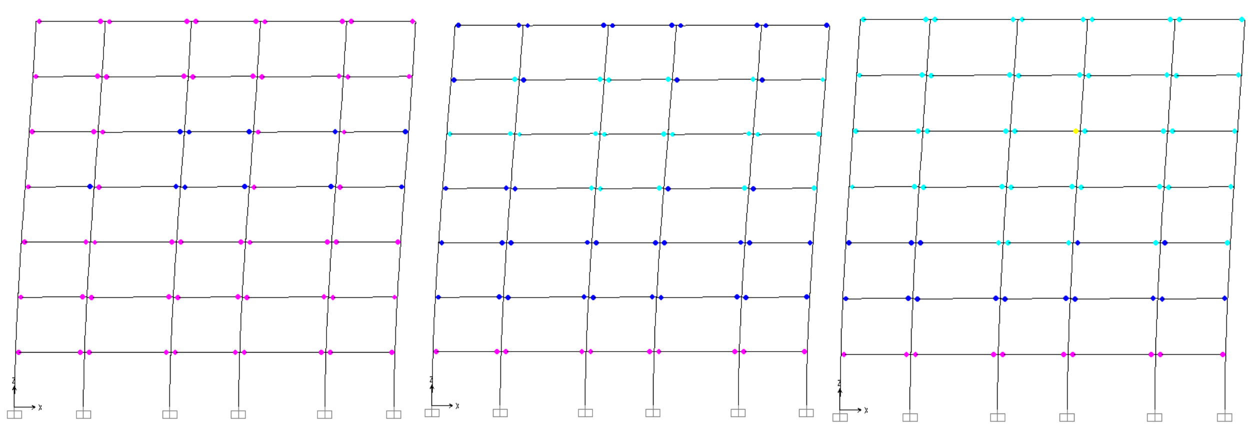
For Case 7, user-defined and default hinge properties were used. Sap2000 has implemented default hinge properties from *FEMA 356* guidelines. The material and section properties, along with the reinforcement detailing are taken as they are in the original project, only the hinge properties are changed.

Table 5.3: *Summarized pushover analysis results for Case 7a*

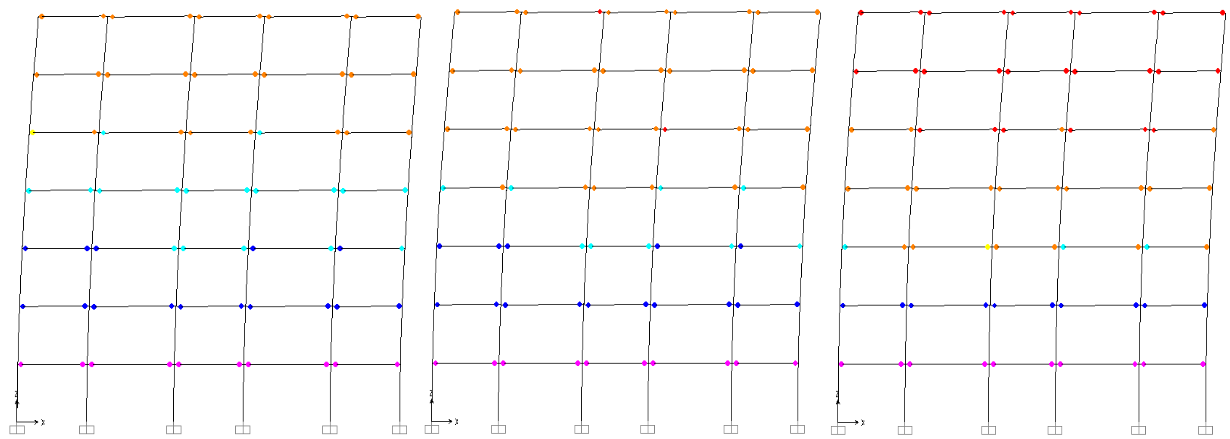
TABLE: Pushover Curve - Pushover											
Step	Displacement	BaseForce	AtoB	BtoIO	IOtoLS	LStoCP	CPtoC	CtoD	DtoE	BeyondE	Total
	m	KN									
0	-1.74E-17	0	154	0	0	0	0	0	0	0	154
1	0.014943	319.285	151	3	0	0	0	0	0	0	154
2	0.026075	469.112	129	10	1	0	0	14	0	0	154
3	0.03644	537.778	115	9	3	5	0	22	0	0	154
4	0.118238	706.157	99	1	0	0	0	54	0	0	154
5	0.202501	809.924	90	4	0	0	0	60	0	0	154
6	0.252646	858.769	84	3	0	2	0	65	0	0	154
7	0.270546	868.247	84	0	0	0	0	70	0	0	154
8	0.350546	892.868	84	0	0	0	0	70	0	0	154
9	0.430546	916.763	84	0	0	0	0	70	0	0	154
10	0.510546	940.601	84	0	0	0	0	70	0	0	154
11	0.590546	966.309	84	0	0	0	0	70	0	0	154
12	0.670546	993.439	84	0	0	0	0	70	0	0	154
13	0.750546	1021.171	84	0	0	0	0	70	0	0	154
14	0.830546	1048.99	84	0	0	0	0	70	0	0	154
15	0.910546	1076.81	84	0	0	0	0	70	0	0	154
16	0.990546	1104.631	84	0	0	0	0	70	0	0	154
17	1.070546	1132.451	84	0	0	0	0	70	0	0	154
18	1.150546	1160.272	84	0	0	0	0	70	0	0	154
19	1.230546	1188.149	84	0	0	0	0	70	0	0	154
20	1.310546	1216.026	84	0	0	0	0	70	0	0	154
21	1.390546	1243.902	84	0	0	0	0	70	0	0	154
22	1.470546	1271.779	84	0	0	0	0	70	0	0	154
23	1.550546	1299.684	84	0	0	0	0	70	0	0	154
24	1.6	1316.934	84	0	0	0	0	70	0	0	154

For Case 7a, user-defined hinge properties are used, so the hinge formation process is similar to the previous cases. The behavior of the structure is linear up to the base shear value of 469 KN, with a 2.6 cm displacement. The maximum base shear at the assigned displacement is approximately 1317 KN. The hinging pattern shows that the formed hinges can be seen in the beams only.

Case 7b on the other hand, considers default hinge properties, calculated based on the tables for reinforced concrete beams and columns in *FEMA 356*. It should be noted that, although using default hinge properties is convenient and simplifies the analysis, a lot of approximations and assumptions are made by the program when using default hinges. The hinging pattern for Case 7b is shown below.



Deformed Shape Step 6 Deformed Shape Step 8 Deformed Shape Step 9



Deformed Shape Step 34 Deformed Shape Step 46 Deformed Shape Step 85

Figure 5.6: *Step by step plastic hinge formation*

When compared to Case 7a, with user-defined hinge properties, the hinging pattern in Case 7b is quite different. A large number of hinges fall into the D to E state, at which the residual resistance of the building, allows the frame elements to sustain gravity loads only. Significant degradation and failure of the beams of the upper stories is possible.

For Case 7b, the structure behaves linearly up to a base shear of 577 KN. The maximum base shear reaches 707.4 KN at step 13, and the ultimate base shear is 264 KN.

The capacity curves for both cases are shown in the graph below.

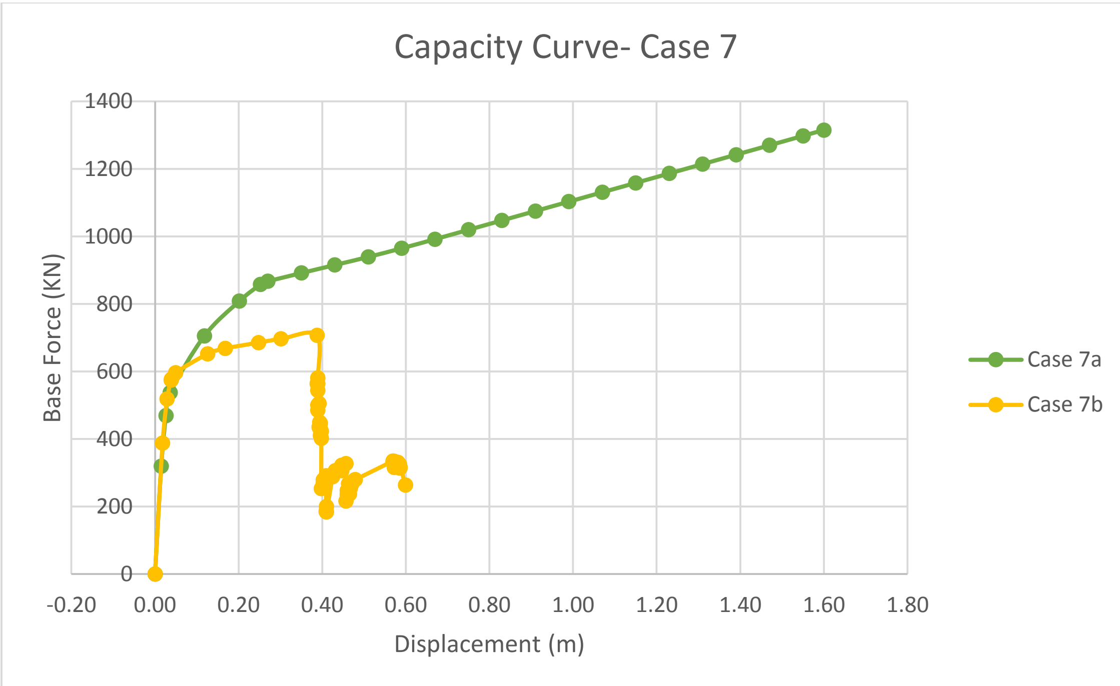


Figure 5.7: *Capacity Curve for Case 7a and 7b*

5.6 Analysis Results for Case 8 to 11

For the four last cases in this study, the values of concrete material properties are taken from the Albanian Codes of practice, whereas the characteristic strength values for steel are taken from the results of real tension tests on steel that is commonly used in Albania for reinforced concrete structures. The aim is to analyze the behavior of a structure reinforced with steel that is usually used nowadays in Albania, under seismic action. The analysis is carried out using user-defined hinges for each case.

The hinging patterns are very similar for the last four cases. The formation of hinges can be observed only in the beams of the frame, confirming again that the structure behaves like a strong column-weak beam mechanism.

Visible changes can be seen in the capacity curves when compared to each other, as it is shown below. When comparing Case 8, with the minimum values of concrete and steel strength, with Case 9, a 17% increase in the maximum base shear is observed.

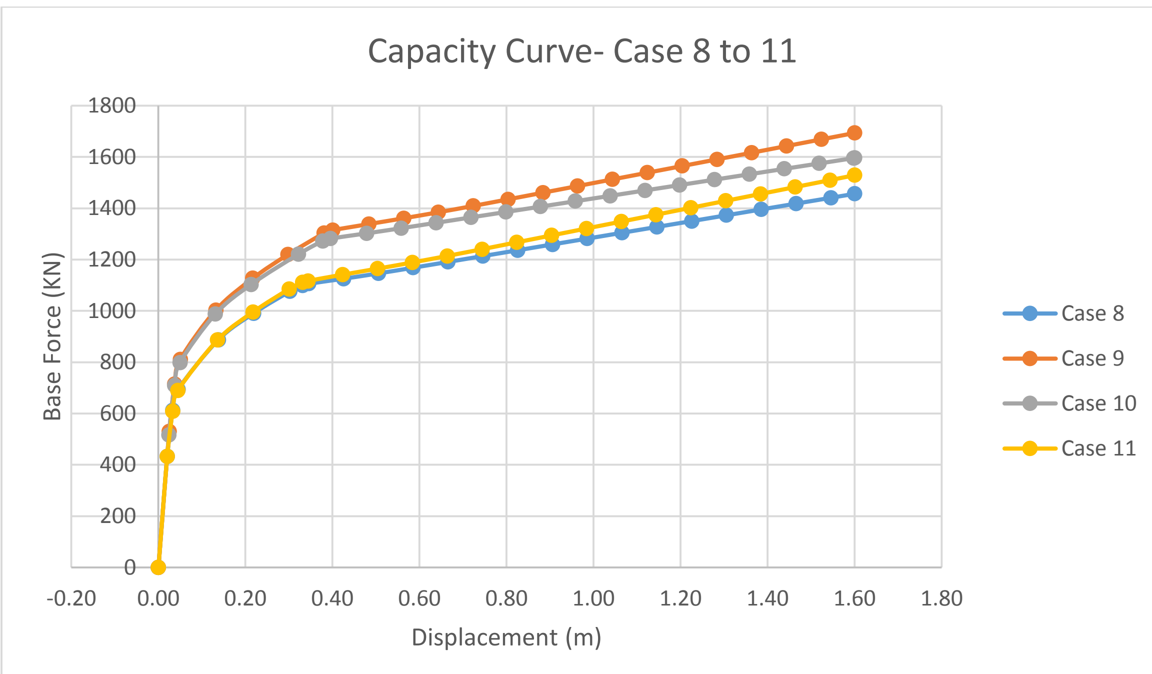


Figure 5.8: *Capacity Curve for Case 8 to 11*

Chapter 6

Conclusions

6.1 Concluding Remarks

All of the cases analyzed in this study, are taken into consideration to fulfill certain objectives and goals. The eleven cases, presented in Chapter 5 are classified into four groups with similar characteristics, with the purpose of comparing and demonstrating the differences in the results of the analysis for each case. All the comments and conclusions for each group are explained in the following sections.

6.2 Reinforcement Percentage Effects on Pushover Analysis

The latest Albanian Code of practice for earthquake resistant design is *KTP-N2-1989*. It aims at ensuring a sufficient enough strength and stability for structures in order to resist the moderate and severe earthquake motions without any critical structural damage. The minimum and maximum percentages of steel for reinforced concrete members in Case 1 and 2, are taken as per the Albanian Code, with the aim of testing the Code’s minimum and maximum values and examining the effects the steel to concrete ratio has on pushover analysis of reinforced concrete structures.

Based on these values, the steel area is calculated in both cases for the beams and columns of the frame, and the rebars are placed in the members with consideration of the regulations for the diameters, minimum and maximum distances for the longitudinal bars, along with the regulations for the diameter and spacing of the transverse reinforcement.

The structure is then modelled in Sap2000 for each case. Considering that user defined hinges are expected to better demonstrate the nonlinear behavior of the structure, deformation controlled $P - M_3$ hinges are defined for beam sections, and $P - M_2$ hinges for column sections.

A comparison between the axial force-bending moment interaction curves for beam and column sections in Case 1 and 2 is shown in the graphs below:

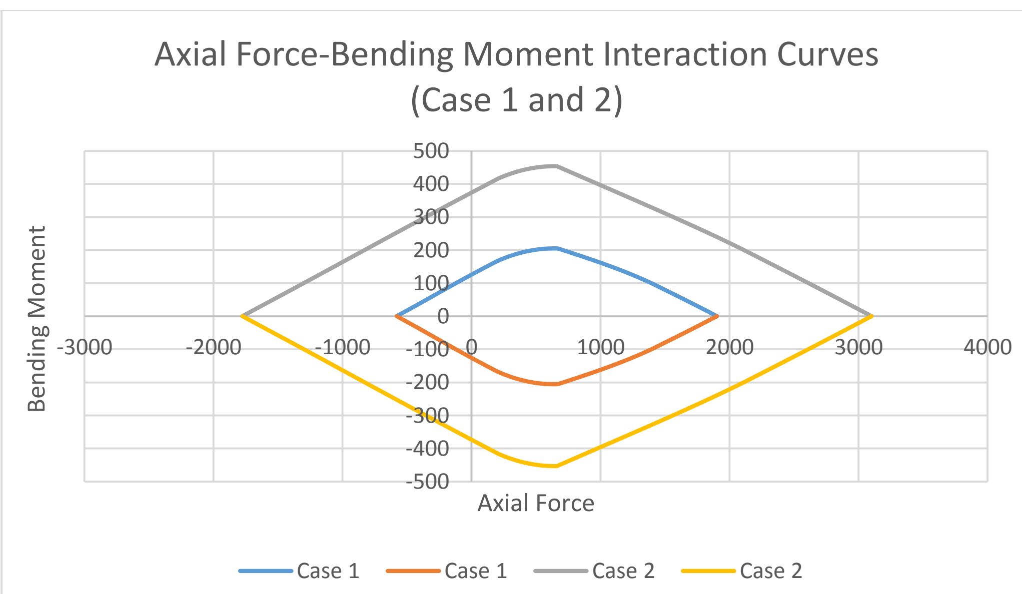


Figure 6.1: *A comparison between $P - M_3$ interaction curves of beam sections for Case 1 and 2*

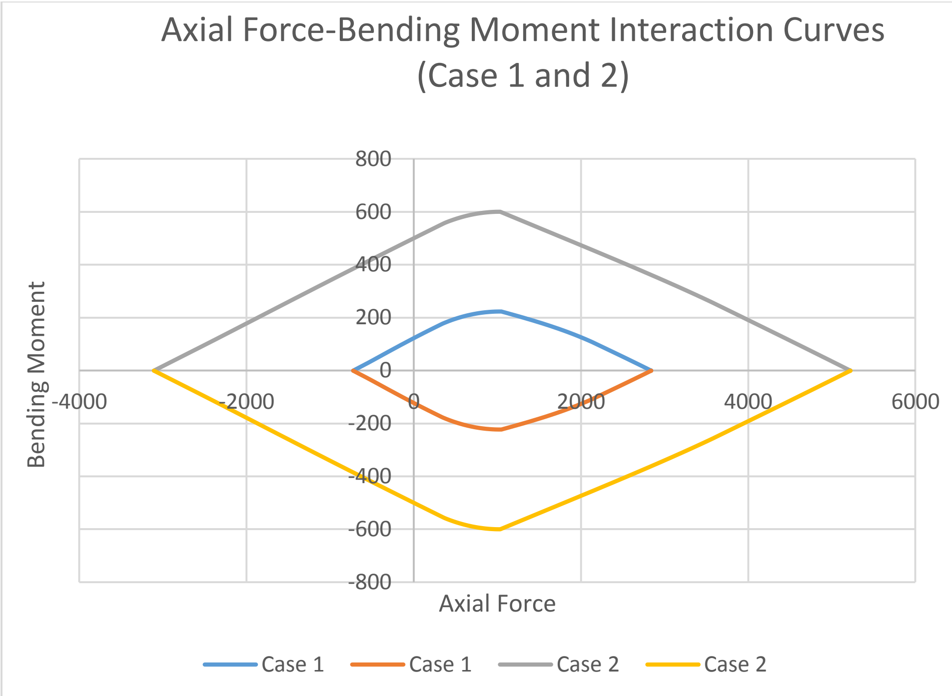


Figure 6.2: *A comparison between* $P - M_2$ *interaction curves of column sections (60x40) for Case 1 and 2*

A clear difference can be observed when comparing the axial force-bending moment interaction curves for the two first cases. The sections with a higher percentage of steel display bigger values of axial force and bending moment in every subdomain of the curve. For example, when comparing the interaction curves for the beam sections, the value of the bending moment at the end of Subdomain 2b (yielding of steel) for Case 2, is 2.5 times bigger than the one for Case 1.

After the user-defined hinges are calculated and assigned to the respective sections, the pushover analysis is carried out. The capacity curves for both cases are shown in the graph below:

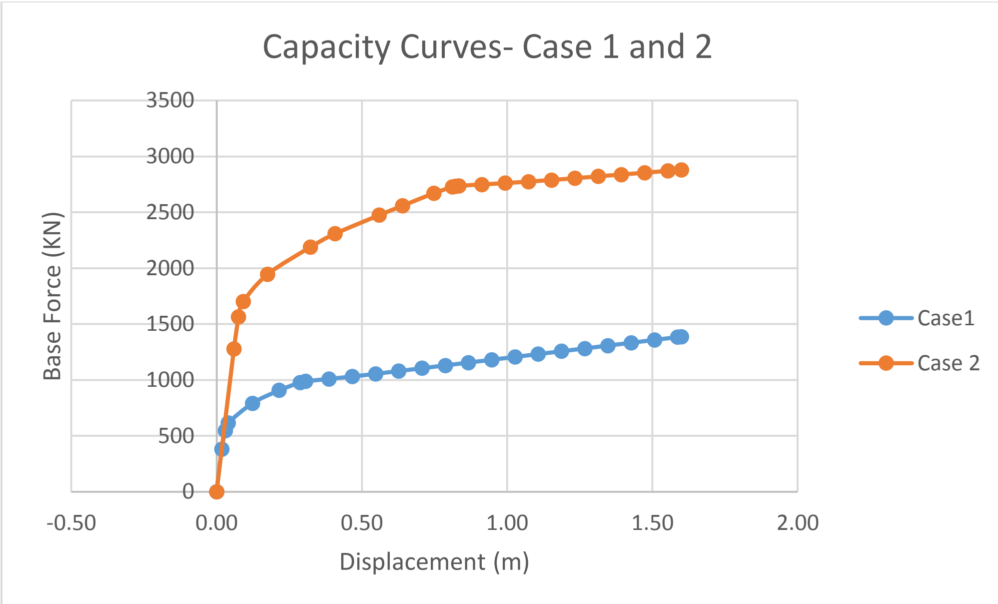


Figure 6.3: *A comparison between the pushover curves for Case 1 and 2*

It can be clearly seen from the pushover curves, that the overall capacity in Case 2 is bigger than in Case 1. The maximum base shear at the assigned displacement of 1.6 meter reaches 2883 KN in Case 2, whereas the maximum base shear in Case 1 is 1387.5 KN.

The hinging pattern is similar in both cases, and it can be seen only in the beams of the reinforced concrete frame, which means that the structure behaves like a strong column-weak beam mechanism. For Case 1, a considerable number of hinges fall at the end of the collapse prevention performance level, which means that potential degradation of stiffness and strength has occurred to the structure.

The pushover analysis for Case 1, which takes into account the minimum percentage of steel in beam and column sections, reveals that, although the frame building is able to resist the applied lateral loads, substantial damage occurs to the structure, which in most cases is not technically or economically repairable. Considering this, it can be said that the minimum percentage of steel for reinforced concrete structures, as it is predefined in the Albanian Codes of practice, is mostly a guideline, rather than a value that can be used for design.

6.3 Material Properties Effects on Pushover Analysis

Case 3 to 6 are included in the study, to incorporate the limit values for concrete and steel strength requirements as per the Albanian *Earthquake-Resistant Design Regulations, 1989* and KTP-N.30-91, in the pushover analysis of the reinforced concrete frame.

User-defined hinges are calculated and used in all four cases, to better indicate the effect that the material properties have on the results of pushover analysis. The obtained capacity curves, are shown in the graph below, to compare the outcomes of the nonlinear static analysis for these four cases.

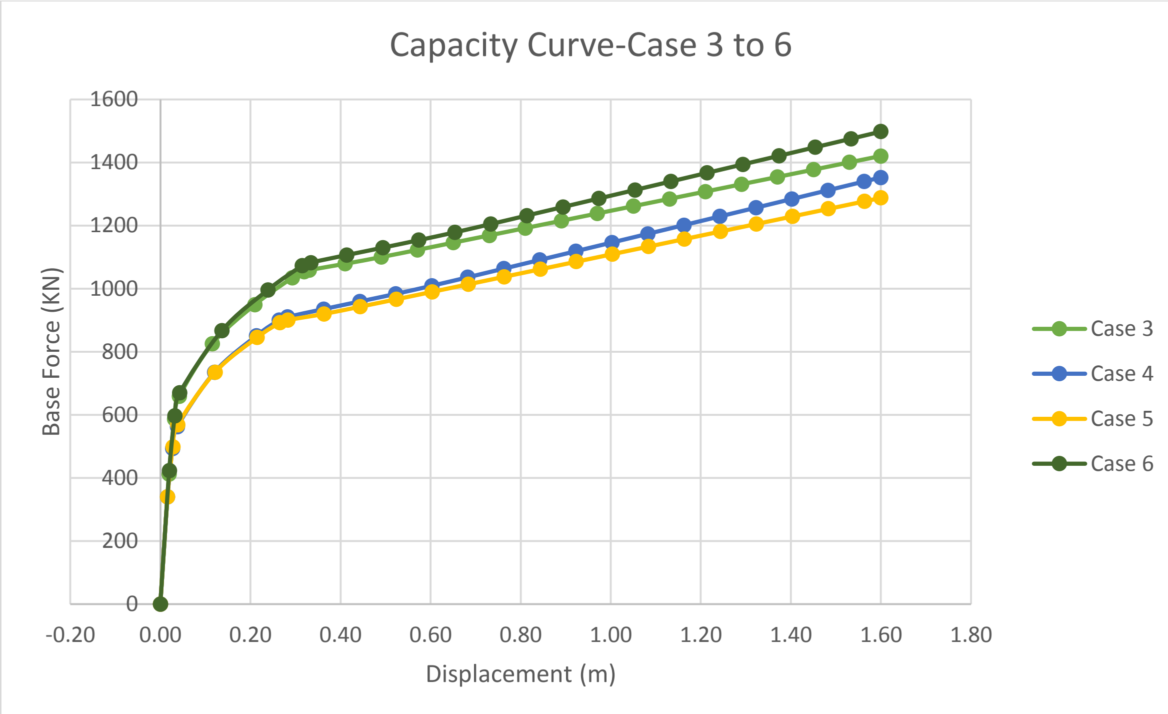


Figure 6.4: *A comparison between the capacity curves for Case 3 to 6*

As it can be seen from the graph, Case 5, in which a combination of low strength concrete and steel is used, demonstrates the lowest overall capacity of the structure, with a maximum base shear of 1288 KN. For Case 6, in which a combination of high strength concrete and steel is used, the maximum base shear at the assigned displacement, is 16 % bigger than the one in Case 5, reaching 1498 KN.

The pushover analysis, also shows that, a combination of low concrete strength with high steel strength (Case 3), results in a higher overall capacity, than a high concrete-low steel combination (Case 4).

The hinging patterns are similar for all four cases, with some small differences in the number of hinges formed in distinct steps, which are more obvious when comparing Case 5 to Case 6.

6.4 User-defined and default hinge properties

To carry out a nonlinear static analysis, either user-defined or default hinge properties, based on several seismic design guidelines, can be used. For 10 out of 11 cases in this study, user-defined hinge properties are used, in order to demonstrate a more thorough understanding of how the section and material properties effect the results of pushover analysis. An analysis of the reinforced concrete frame, with user-defined and then with default hinge properties is performed in Case 7, with the purpose of revealing the possible differences in the outcomes of the pushover analysis.

The graph below shows the capacity curves, for Case 7a in which user-defined hinge properties are used, and Case 7b designed with default hinge properties based on *FEMA 356* tables.

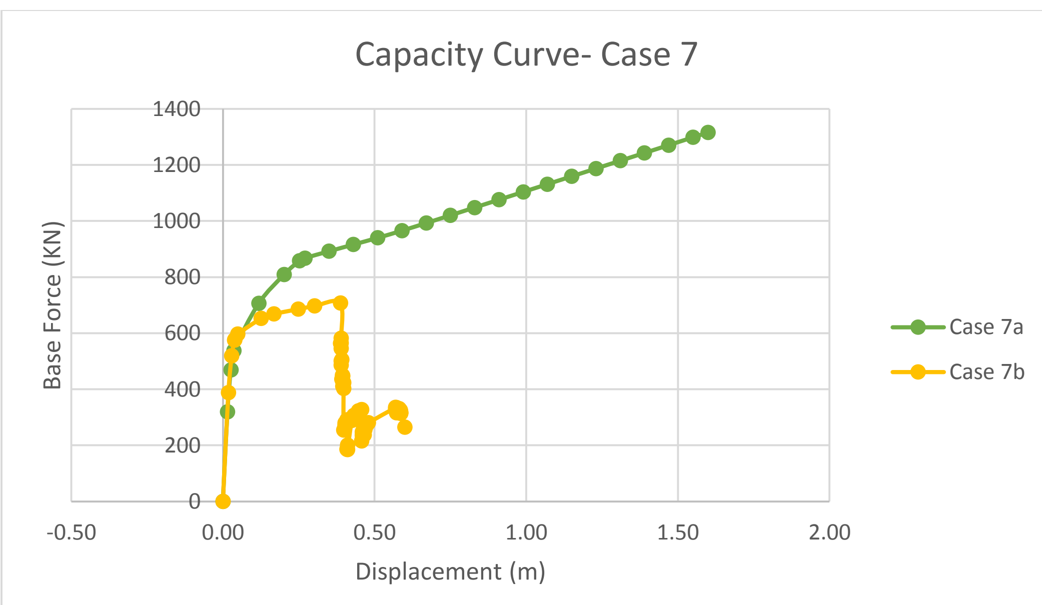


Figure 6.5: *A comparison between the pushover curves for Case 7a and 7b*

A significant variation is observed, in the base shear capacity, and in the plastic hinge formation mechanism at yield and ultimate stages. This may be due to the fact that, the default hinge model does not take into consideration the columns weak or strong axis orientation, and their axial load level. This model assumes the same deformation capacity for all columns, whereas for the user-defined case all the factors mentioned above are taken into account.

Consequently, it can be said that a pushover analysis carried out with user-defined hinge properties, is more successful in capturing the nonlinear behavior of the structure.

6.5 Pushover analysis with steel commonly used in reinforced concrete structures in Albania

The characteristic strength values, obtained from the tensile test of steel specimens from steel commonly used in reinforced concrete structures in Albania, are incorporated in the pushover analysis of the frame for Case 8 to 11.

It should be noted that, based on the examination of the tensile test results, the yield strength of 16 specimens is not acceptable for the use of reinforcing steel according to BS EN 10080:2005, whereas 75 specimens have a yield strength higher than 600 MPa, which exceeds the maximum permissible yield strength value in Eurocode 2. A considerable amount of the specimens tested fail to fulfill the material ductility requirements (15), and a further analysis of the test results shows a range of variation of the characteristic strengths values. Subsequently, it can be said that there is not yet an adequate standard for the reinforcing steel used nowadays in Albania.

The minimum and the mean value of the steel characteristic yield strength obtained by the tensile test results, in combination with minimum and maximum values of concrete strength are used for Case 8 to 11. The capacity curves for each case are shown below:

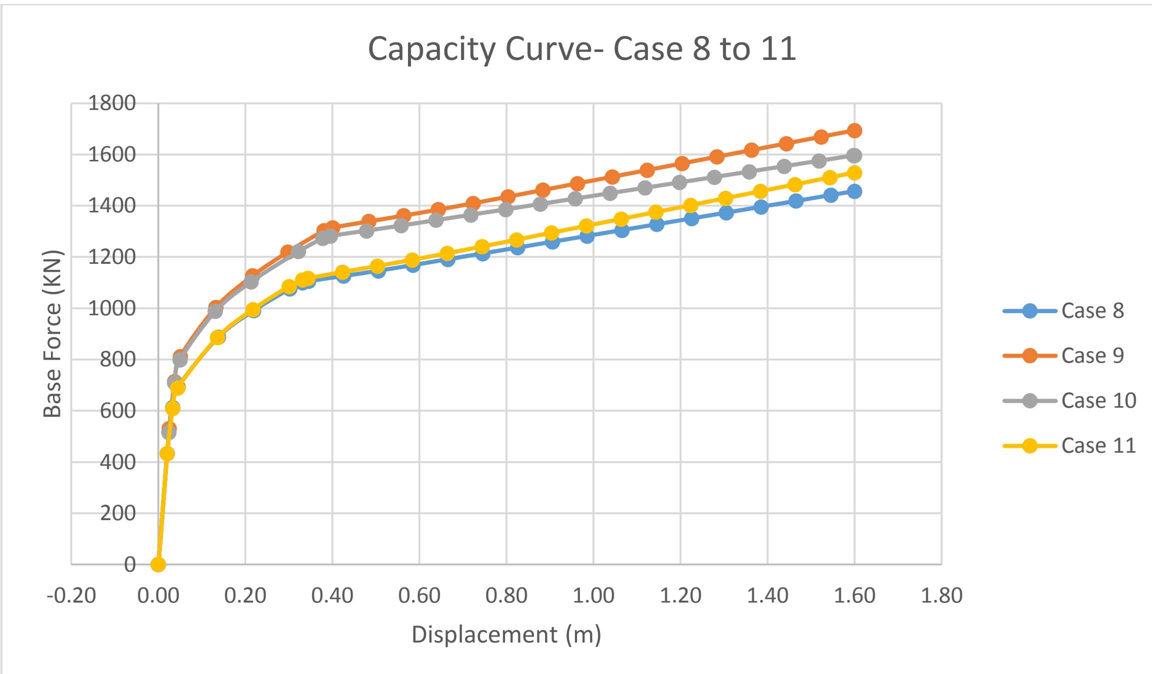


Figure 6.6: *A comparison between the capacity curves of Case 8 to 11*

The results are similar to the ones in Case 3 to 6. The maximum base shear in Case 9 is 17 % bigger than the one in Case 10. The Cases in which the reinforcing steel has higher yield strength, display a higher overall capacity.

6.6 Conclusion Summary

- The reinforcing steel to concrete ratio, effects the behavior of reinforced concrete members. For smaller values of this ratio, reinforcement yields plastically before the concrete is crushed in compression, while for larger values, it may initiate crushing of concrete prior to the yielding of reinforcement.

- Analyzing of the frame building for Case 1, which considers the minimum percentage of steel, reveals that although the frame building is able to resist the level of the design earthquake, substantial damage occurs to the structure, which in most cases is not technically or economically repairable.

- Material properties effect the pushover analysis of the building. Higher concrete and steel strength result in higher maximum base shear.

- A combination of low concrete strength with high steel strength, results in a higher overall capacity, than a high concrete-low steel combination.

- The use of default hinge properties may be reasonable for structures built in compliance with the modern codes of design, but may not be appropriate for others. Considering that most buildings in Albania are built based on old codes of practice, and usually do not conform to the requirements of modern codes, using a default hinge model may not be suitable.

- Based on the observations, the user-defined hinge model is more successful in capturing the nonlinear behavior of the structure.

- Although the steel used nowadays in Albania, has higher values of characteristic yield strength, test results show that a considerable amount does not fulfill the material ductility requirements. It can be said that, there isn't yet a standard and a control over the steel used for reinforced concrete structures.

- Overall, the frame building is able to withstand the calculated lateral loads in all the cases, with some potential degradation of stiffness and strength in the beams, for the cases with minimum percentage of steel or minimum values of concrete and steel strength.

- The structure behaves like a strong column-weak beam mechanism in all cases, conforming to the modern philosophy of design.

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