

**PERFORMANCE ANALYSIS OF A RETROFITTED EXISTING
REINFORCED CONCRETE STRUCTURE CONSIDERING
SOIL-STRUCTURE INTERACTION**

**M.Sc. Thesis by
Francesc JANÉ FIGUEROLA**

Department : Civil engineering

Programme : Structural engineering

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**M.Sc. Thesis by
Francesc Jané Figuerola
(990090203)**

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**Supervisors (Chairman) : Prof. Dr. Engin ORAKDÖĞEN (İTÜ)
Y. Doç. Dr. Ercan YÜKSEL (İTÜ)
Members of the Examining Committee : Doç. Dr. Kutlu DARILMAZ (İTÜ)
Doç. Dr. Oğuz Cem ÇELİK (İTÜ)
Y. Doç. Dr. Mecit CELİK (İTÜ)**

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**GÜÇLENDİRİLEN MEVCUT BİR BETONARME YAPI SİSTEMİNİN
ZEMİN-YAPI ETKİLEŞİMİ DİKKATE ALINARAK
PERFORMANS ANALİZİ**

**YÜKSEK LİSANS TEZİ
Francesc Jané Figuerola
(990090203)**

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Tezin Savunulduğu Tarih : 18 Mayıs 2010**

**Tez Danışmanı : Prof. Dr. Engin ORAKDÖĞEN (İTÜ)
Y. Doç. Dr. Ercan YÜKSEL (İTÜ)
Diğer Jüri Üyeleri : Doç. Dr. Kutlu DARILMAZ (İTÜ)
Doç. Dr. Oğuz Cem ÇELİK (İTÜ)
Y. Doç. Dr. Mecit ÇELİK (İTÜ)**

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FOREWORD

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Istanbul, April 2010

Francesc JANÉ FIGUEROLA

TABLE OF CONTENTS

	<u>Page</u>
ABBREVIATIONS	ix
LIST OF TABLES	xi
LIST OF FIGURES	xiii
SUMMARY	xv
ÖZET.....	xvii
1. INTRODUCTION	1
1.1. Purpose of the Thesis	2
1.2. Background	2
1.3. Hypothesis	3
2. METHODOLOGY	5
2.1. Objectives.....	5
2.2. Static and Modal Analyses	5
2.3. Non-Linear Analyses.....	6
2.3.1. Displacement controlled analysis.....	6
2.3.2. Load controlled analysis	7
2.4. Performance Based Design-Performance Determination.....	7
2.4.1. ATC40 procedure.....	10
2.4.2. FEMA 440 Ppocedure.....	14
3. CASE STUDY: REINFORCED CONCRETE BUILDING.....	17
3.1. Objectives.....	17
3.2. Description	17
3.3. Existing Data of the Building.....	18
3.3.1. Slab loads	18
3.3.2. Infill wall loads	19
3.4. Unknown Data of the Building.....	19
3.4.1. Material properties	19
3.4.1.1. Existing elements	19
3.4.1.2. New elements	20
4. MATHEMATICAL MODEL	23
4.1. Modelling the 3D Structure.....	23
4.2. Definition of Loads.	23
4.2.1. Slab loads	23
4.2.2. Infill wall loads	25
4.2.3. Frames self weight	26
4.2.3.1. Load coming from the lower part of the beam.....	26
4.2.4. Statically equivalent earthquake loads	27
4.2.5. Storey masses	28
4.3. Checkings and Final Determinations.....	29
4.3.1. Modal analysis	29
4.3.2. Static analysis.....	30
4.3.3. Determination of the reinforcements	30
4.3.3.1. Columns	31

4.3.3.2. End span of beams.....	31
4.3.4. Preparation for the pushover analysis.....	32
4.3.4.1. Plastic hinges for the pushover analysis.....	33
4.3.4.2. Location of the plastic hinges.....	33
4.3.4.3. Definition of hinges.....	35
4.3.4.3.1. Hinges of Columns.....	35
4.3.4.3.2. Hinges of Beams	35
4.4. Assumptions Made in Modeling.....	35
5. RETROFITTED CASES	37
5.1. Column Jacketing	37
5.1.1. Definition of jacketed columns	37
5.1.2. Plastic hinge model	38
5.1.3. Analysed models	38
5.1.3.1. First floor column jacketed	39
5.1.3.2. First and second floor columns jacketed	39
5.1.3.3. All columns jacketed.....	40
5.2. Additional Shear Walls.....	40
5.2.1. Idealization of structure.....	40
5.2.1.1. Shear walls dimensions and reinforcements	41
5.2.1.2. Plastic hinge models.....	43
5.2.2. Models analysed	44
5.2.2.1. 4x2 shear wall case.....	44
5.2.2.2. 4x4 shear wall case.....	46
6. CONSIDERING THE SOIL-STRUCTURE INTERACTION.....	47
6.1. Soil Condition.....	47
6.2. Modelling the Tensionless Winkler Soil	47
6.2.1. Special case of shear wall models.....	49
7. DEFINITION OF THE DEMAND SPECTRUM	51
8. RESULTS AND COMPARISONS	53
8.1. Capacity Curves of the Existing and Retrofitted Structures.....	53
8.1.1. Existing building	53
8.1.2. 1 st -2 nd floor jacketed	54
8.1.3. All columns jacketed.....	55
8.1.4. 4x2 shear wall case.....	56
8.1.5. 4x4 shear wall case.....	57
8.1.6. Observations from capacity curves	58
8.2. Capacity Curves Depending on Soil Types	58
8.2.1. Fixed foundation case.....	59
8.2.2. Z2 type soil.....	60
8.2.3. Observations by soil types.....	61
8.3. Natural Periods of The Structure Depending on Soil Types	61
8.4. Performance Point Calculation.....	62
8.5. Uplift Comparison	64
8.6. Hinge Configuration at the Performance Points.....	65
9. CONCLUSION	71
REFERENCES	73
APPENDIX	75
CURRICULUM VITAE	101

ABBREVIATIONS

ADRS	: Acceleration-Displacement Response Spectra
App	: Appendix
ATC	: Applied Technology Council
BRC	: Base Reaction Capacity
CDD	: Capacity Demand Diagram
CS	: Capacity Spectra
DC	: Displacement Capacity
DRS	: Displacement Response Spectrum
DS	: Demand Spectra
E_i	: Earthquake load in “x” or “y” direction
ERS	: Elastic Response Spectrum
FEMA	: Federal Emergency Management Agency
G	: Dead loads
IR	: Infinite Rigid members
MADRS	: Modified Acceleration-Displacement Response Spectra
ME	: Maximum earthquake
PB	: Performance Based Design
PE	: Probability of Exceedance
PGA	: Peak Ground Acceleration
PGV	: Peak Ground Velocity
PSHC	: Probabilistically Seismic Hazard Coefficients
Q	: Live loads
RC	: Reinforced Concrete
RSA	: Response Spectral Acceleration
SDF	: Single Degree-of-Freedom
STR	: Structure
TEC	: Turkish Earthquake Code
TS	: Turkish Standards

LIST OF TABLES

	<u>Page</u>
Table 4. 1: Uniformly distributed loads transferred to the beams.....	25
Table 4. 2: Uniform distributed loads (KN/m) coming from infill walls.....	26
Table 4. 3: Loads from the lower part of the beam.	27
Table 4. 4: Statically equivalent earthquake loads.	28
Table 6. 1: Mechanics properties of soil types.	47
Table 6. 2: Characterisitcs of the equivalent frame members.	48
Table 8. 1: Natural periods.	62
Table 8. 2: Performance points.	63
Table 8. 3: Hinge configuration of the full jacketed case over direction “x”.	66
Table 8. 4: Hinge configuration of the full jacketed case over direction “y”.	67
Table 8. 5: Hinge configuration of 4x4 Shear wall case over direction “x”	68
Table 8. 6: Hinge configuration of 4x4 Shear wall case over direction “y”	68
Table 8. 7: Hinge configuration of 4x2 Shear wall case over direction “x”	69
Table 8. 8: Hinge configuration of 4x2 Shear wall case over direction “y”	69
Table 8. 9: Performance classification of different cases.....	70
Table D. 1: Properties of the foundation elements.....	96

LIST OF FIGURES

	<u>Page</u>
Figure 1. 1: Tectonic deistribution in Middle East.	2
Figure 2. 1: Displacement control drawn explanation	7
Figure 2. 2: Load control drawn explanation.	7
Figure 2. 3: Seismic Zoning. Velocity contours of exceedance 10% in 50 years.....	9
Figure 2. 4: ERS.....	9
Figure 2. 5: DS definition from ATC 40.....	11
Figure 2. 6: Transformation of DS to Spectral doamain.....	11
Figure 2. 7: Determination of candidate point.	12
Figure 2. 8: CS bilinearization.	13
Figure 2. 9: New candidate point with MADRS.....	13
Figure 2. 10: Performance point with FEMA 440.	15
Figure 4. 1: Loads distribution diagram due to slabs.....	24
Figure 4. 2: Typical infill wall.	25
Figure 4. 3: Mathematical model for modal analysis.....	29
Figure 4. 4: The first mode shape of the building.	30
Figure 4. 5: Sections of the actual building.	32
Figure 4. 6: Moment distribution for a lateral load.....	34
Figure 4. 7: Hinge assignation.	34
Figure 4. 8: 3D mathematical model.....	36
Figure 5. 1: Jacketed columns sections.	38
Figure 5. 2: 3D mathematical model of first floor column jacketed case.....	39
Figure 5. 3: 3D mathematical model of first and second floor columns jacketed case.....	39
Figure 5. 4: 3D mathematical model of all columns jacketed case.....	40
Figure 5. 5: Idealization of a Shear Wall.	41
Figure 5. 6: Sections of shear walls.	43
Figure 5. 7: Hinge SAP 2000's interface.....	44
Figure 5. 8: 3D mathematical model of 4x2 shear wall case.	45
Figure 5. 9: Typical plan of 4x2 shear wall case.	45
Figure 5. 10: 3D mathematical model of 4x4 shear wall case.	46
Figure 5. 11: Typical plan of 4x4 shear wall case.	46
Figure 6. 1: 3D mathematical model with mat foundation of the 4x4 shear wall case.....	49
Figure 8. 1: Capacity curves of the existing building over "x" direction.....	53
Figure 8. 2: Capacity curves of the existing building over "y" direction.....	54
Figure 8. 3: Capacity curves for the jacketing of columns of 1st-2nd floor over "x"direction.	54
Figure 8. 4: Capacity curves for the jacketing of columns of 1st-2nd floor over "y"direction.	55
Figure 8. 5: Capacity curves of all columns jacketed case over "x" direction.....	55
Figure 8. 6: Capacity curves of all columns jacketed case over "y" direction.....	56
Figure 8. 7: Capacity curves of 4x2 Shear wall case over "x" direction.....	56

Figure 8. 8: Capacity curves of 4x2 Shear wall case over "y" direction.....	57
Figure 8. 9: Capacity curves of 4x4 Shear wall case over "x" direction.	57
Figure 8. 10: Capacity curves of 4x4 Shear wall case over "y" direction.....	58
Figure 8. 11: Capacity curves of fixed foundation case over “x” direction.	59
Figure 8. 12: Capacity curves of fixed foundation case over "y" direction.	59
Figure 8. 13: Capacity curves of Z2 soil case over "x" direction.....	60
Figure 8. 14: Capacity curves of Z2 soil condition over "y" direction.	60
Figure 8. 15: Uplift comparison.	64
Figure 8. 16: Criteria for performance of the building.....	65
Figure 8. 17: Plastic Hinge configuration of the actual building under Z1 soil condition over "x" direction.....	66
Figure 8. 18: Hinge configuration of all columns jacketed case over "y".....	67
Figure A. 1: Typical plan of the first storey.	78
Figure A. 2: Typical plan of upper stories	79
Figure A. 3: Plan of the skeleton of the mathematical model.	80
Figure C. 1: Equivalent section capacity of T20/60 at the end of span for positives moments.....	84
Figure C. 2: Equivalent section capacity of T20/50 at the end of span for positives moments.....	85
Figure C. 3: Equivalent section capacity of T20/40 at the end of span for positives moments.....	86
Figure C. 4: Equivalent section capacity of T20/60 at the end of span for negative moments.....	87
Figure C. 5: Equivalent section capacity of T20/50 at the end of span for negative moments.....	88
Figure C. 6: Equivalent section capacity of T20/40 at the end of span for negatives moments.....	88
Figure C. 7: Sections of the actual building.....	89
Figure C. 8: Jacketed columns sections.	90
Figure C. 9: Sections of shear walls.....	91
Figure C. 10: Shear wall capacities and their bilinearization.	92
Figure D. 1: Normal mesh assignment.....	93
Figure D. 2: Mesh assignment for 4x4 shear wall case.....	93
Figure D. 3: Mesh assignment for 4x2 shear wall case.....	94
Figure D. 4: Meshed foundation.	94
Figure D. 5: Effective area representation.	95
Figure E. 1: Deformed shape of jacketed columns of the first floor case with fixed soil condition case.....	97
Figure E. 2: Deformed shape of jacketed columns of 1 st -2 nd case over "x" direction, on fixed case soil condition.....	98
Figure E. 3: Deformed shape of all columns jacketed case over "x" direction with fixed soil condition.....	98
Figure E. 4: Deformed shape of 4x2 shear wall case over "y" direction with fixed soil condition.....	99
Figure E. 5: Deformed shape of 4x2 shear wall case over "y" direction with Z2 soil condition.....	99
Figure E. 6: Deformed shape of 4x4 shear wall case over "x" direction with fixed soil condition.....	100
Figure E. 7: Deformed shape of 4x4 shear wall case over "x" direction with Z3 soil condition.....	100

PERFORMANCE ANALYSIS OF A RETROFITTED EXISTING REINFORCED CONCRETE STRUCTURE CONSIDERING SOIL-STRUCTURE INTERACTION

SUMMARY

In this thesis, performance evaluations of a 3D existing reinforced concrete structure and its strengthening by columns jacketing and shear-walls addition are presented. The performance evaluations are made according to Ch. 6 of FEMA440. The column jacketing is applied on three different ways. They are jacketing of 1st floor columns; 1st and 2nd floor columns and all columns of the structure. Shear-walls are added and represented as column elements.

In addition, the soil-structure interaction is included in the nonlinear static analysis of the building to realize how the modelization of the soil can affect to the response. A mat foundation is considered and different kinds of soils that are presented in TEC2007 are represented through it. The mat foundation is represented as a thick shell element. Tensionless elastic-plastic Winkler soil is used for soil idealization, which is represented by equivalent frame elements subjected only to axial forces and equivalent axial stiffness is defined for these elements. Performance evaluations and comparisons for the actual existing building and its strengthened cases are presented in detail for fixed soil condition case and for different soil conditions represented by the modelization of the foundation.

The nonlinear pushover analyses and performance evaluations are obtained by SAP2000 computer package.

Finally, damage evaluation according to the criteria presented in FEMA356 is performed to compare the different retrofitting techniques and to try to find out which is the most suitable in an engineering context.

GÜÇLENDİRİLEN MEVCUT BİR BETONARME YAPI SİSTEMİNİN ZEMİN-YAPI ETKİLEŞİMİ DİKKATE ALINARAK DEPREM PERFORMANSININ BELİRLENMESİ

ÖZET

Bu çalışmada; mevcut bir betonarme yapı sisteminin, kolon manto laması ve ilave betonarme perde kullanılarak güçlendirilmiş durumlarındaki performans değerlendirmesi yapılmıştır. Performans değerlendirmesi FEMA440 Bölüm 6 ya göre yapılmıştır. Kolon mantolaması üç farklı şekilde uygulanmıştır. Bunlar sırasıyla birinci katta mantolama, ilk iki katta mantolama ve tüm katlarda mantolama şeklinde sıralanabilir. Perde duvarlar eşdeğer kolon elemanlar olarak tanımlanmıştır.

Sınır şartlarının genel davranışa olan etkisinin belirlenmesi için, yapılan doğrusal olmayan hesaplarda zemin-yapı etkileşimi dikkate alınmıştır. Türk Deprem Yönetmeliğinde (2007) tanımlanan farklı zemin türleri dikkate alınmıştır. Güçlendirmenin perde ile yapıldığı durumda, radye temel kalın levha sonlu eleman olarak modellenmiştir. Çekme almayan elastik-plastik Winkler tipi zemin kullanılmış olup, sadece eksenel kuvvet taşıyan iki ucu mafsallı çubuk elemanlar ile zemin modellemesi yapılmıştır.

Mevcut yapı sisteminin ve farklı yöntemler ile güçlendirilmiş durumların performans değerlendirmeleri; alttan ankastre hal ve farklı özellikteki zemine mesnetlenme durumları için, ayrı ayrı gerçekleştirilmiştir.

Doğrusal olmayan itme analizleri ve performans değerlendirmeleri SAP2000 paket programı kullanılarak gerçekleştirilmiştir.

Uygulanan farklı güçlendirme yöntemlerinin birbiriyle karşılaştırılması için, FEMA356 da tanımlanmış koşullara göre hasar değerlendirmeleri yapılmıştır.

.

1. INTRODUCTION

The last two major earthquakes, Haiti (January 2010) and Chile (February 2010), of magnitude 7.0 and 8.8 respectively on the Richter scale, have reminded the world how destructive are earthquakes. More than 200 000 deaths, 250 000 wounded and 1M homeless caused by the earthquake of Haiti. More than 800 000 deaths and 2M affected due to the earthquake of Chile. Even if the magnitude was greater in the earthquake of Chile, personal and materials damages aren't comparable with the ones of Haiti. That reminds world how important is to consider earthquakes when a building is designed.

Istanbul is located on one of the active earthquake zones of the world. Several earthquakes stroke Turkey in the last century with a magnitude equal or greater to 7.0 in the Richter scale.

Nowadays, Istanbul is a huge megalopolis where 12 million people live. Its growth was uncontrolled on those last forty years due to the need to satisfy the necessities of new dwellings and it is why, probably lots of buildings aren't prepared to suffer an earthquake.

The main object of this thesis is to analyze a building that represents this situation. It is a four storey reinforced concrete building where the ground floor is used as a bank office and the upper ones for medical education purposes.

The Performance Based design (PB) method is used in this thesis for the analysis of the building and its retrofitted cases. Different procedures of this method are presented as Fajfar's method [1], the equivalent displacement method (FEMA273, [2]) and the capacity spectrum method (ATC40 [3]). PB takes in account the soil-structure interaction in the demand spectra definition, but not in the capacity curve of the structure. The foundation is also modelled to include the soil-structure interaction in the capacity curve of the structure.

Performance evaluations and soil modelizations are run with SAP2000 [4] computer package.



Figure 1. 1: Tectonic distribution in Middle East. [5]

1.1 Purpose of the Thesis

The main objectives of this thesis are to do an accurate retrofitting analysis of a sample building and to compare some retrofitting techniques. This would allow us to check which one is the most suitable for this kind of buildings.

Another objective is to determine the interaction between soil and structure. Different soil types have been used in these analyses.

1.2 Background

Guidelines and Commentary for Seismic Rehabilitation of Buildings – the ATC 40 [3] Project by the Applied Technology Council (ATC), and NEHRP Guidelines for the Seismic Rehabilitation of Buildings – FEMA 273 [2] and 356 [6] by the Federal Emergency Agency (FEMA) have been developed to include and improve the concept of performance criteria of a building, which determines the safety level of a building. In order to examine the results further on, the ATC 55 [7] and FEMA 440 [8] have been developed.

In Turkey, the Turkish Earthquake Code (TEC) which was revised in 2007 (TEC2007) [9], was developed for the assessment and rehabilitation of structures. Numerical studies comparing FEMA 356 and TEC using non-linear analysis method shows that both codes results almost similar damage levels on the basis of structural elements [10].

Besides, about the soil-structure interaction effect, Orakdöğen et al. [11], already studied the soil-structure interaction on a performance evaluation of a strengthened building by modelling different kind of foundations as, mat foundations, continuous footings types and single footing types. They conclude that foundation and soil should be included to the mathematical model when a PB is used and that the Turkish earthquake code should be revised.

1.3 Hypothesis

The following hypotheses are pronounced:

- Existing building cannot resist the code specified earthquake loads.
- Retrofitting techniques are effective enough to increase the performance of the existing building against earthquakes.
- The foundation system and soil should be considered in the model.
- The representation of different kind of soils will create differences on the capacity curves.

2. METHODOLOGY

2.1 Objectives

The main focus of this chapter is to develop a presentation of which methods will be used to analyse a building and make some comparison with those ones which could be considered and why not.

Actually, four methods will be considered in the following order: static, modal, nonlinear analysis and Performance Based design. The first one will be used to check the strength of frames. The second one allows finding out the vibration periods of the building. Nonlinear pushover analysis will be used to get the capacity curve of the structure and PB is used to find out performance points. In addition, two ways to calculate the performance point are presented.

2.2 Static and Modal Analyses

For those analyses, the main data comes from the geometry of the frames and the characteristics of materials.

Young Modulus or Elastic Modulus (E) is the determinant material characteristic.

Inertia Moments (I) comes from the geometry of frames.

Strength (K) is depending on those two frame characteristics: $K=f(E, I)$. With the determination of the strength, it is possible to get any effort or displacement that affects any frame. This is the static analyse, which will be run to get internal forces over axes of frame members. Then, determination of the reinforcements of the frame members will be able to be done.

Modal analysis is also determined by strength, but not only by it, also by the mass (M). From a relationship of both it is possible to find out the vibrational period (T) of the structure. In a simply way to show it, there is the relationship for a structure of an only free degree of freedom:

$$[\omega] = \sqrt{\frac{[K]}{[M]}} = 2 \cdot \pi \cdot f \quad (2.1)$$

$$T = \frac{1}{f} \quad (2.2)$$

2.3 Nonlinear Analyses

There are two types of nonlinearity:

The first one is due to material nonlinearity, which can be caused by the yielding, cracking, crushing, sliding and more or simply caused by an inelastic behaviour.

The second one is due to geometric nonlinearity, which can be caused by a change in the shape of the structure. The P-Δ and large displacements are also include.

The nonlinear analysis is able to represent this behaviour applying equal increments of strength or displacement and, calculating the corresponding stiffness and reactions at each frame for each step. Final solution is result of the sum of the different solution of each step.

The nonlinear analysis makes a more accurate approximation to the real behaviour of the structure than a linear one.

There is two ways to run a non-linear analysis: by displacement control or by load control.

2.3.1 Displacement controlled analysis

The nonlinear analysis is controlled by equal increments of displacement that requires unequal increments of load (Figure 2.1). The increments of load can be negative. In this way the nonlinear behaviour of a frame or a structure can be represented.

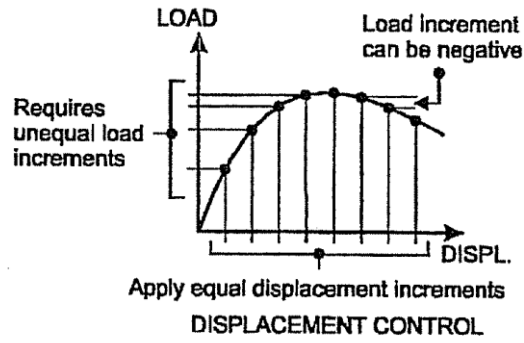


Figure 2. 1: Displacement control drawn explanation. [12]

2.3.2 Load controlled analysis

The nonlinear analysis is controlled by equal increments of load that requires unequal increments of displacement. If the load exceeds the strength, there is no solution (Figure 2.2). This kind of control is suitable for those analyses where it is known that the structure will not yield or crash.

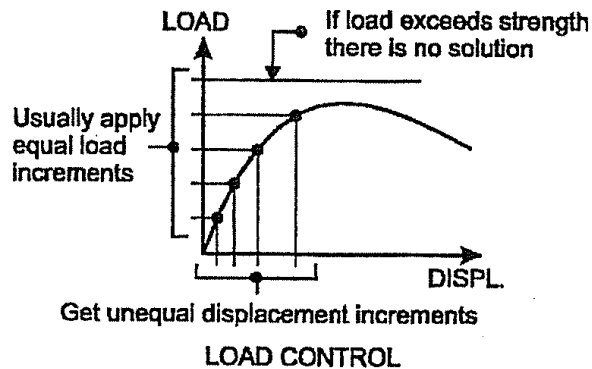


Figure 2. 2: Load control drawn explanation. [12]

2.4 Performance Based Design-Performance Determination

The Performance-Based design (PB) is a method that allows engineers to have an idea of which is the state of the building under a specified situation. That allows to make a critic of the building taking into account the safety. Its development was around 1990.

The need to improve the performance of buildings against earthquakes, have implied many researchers working on it. Those codes, and therefore, those studies have concerned different studies fields as [13]:

1. Engineering Seismology and Geology (Seismic Activity Modelling);
2. Engineering Seismology (Seismic Hazard Modelling);
3. Soil Dynamics;
4. Structural Dynamics;
5. Mechanics of Material;

The main output from the combination of Engineering Seismology and Geology and Soil Dynamics field are the Probabilistically-defined Seismic Hazard Coefficients (PSHC) which may take the form of Peak Ground Acceleration (PGA), Peak Ground Velocity (PGV) or the Response Spectral Acceleration (RSA) at some key structural periods of 0.3 and 0.1 s typically.

Those PSHC are used in two ways:

They can be plotted as Hazard Maps. Those are used in codifications.

Or, they are used to determine the elastic response spectrum (ERS), which defines the peak responses of single-degree-of-freedom (SDF) linear elastic systems possessing a range of different natural periods as it has already been said.

Engineers are used to visualize seismic hazards through those PSHC, with the two ways mentioned above on the codes, and not as a seismic event, which would represent better the physical event. Adopted approach, based on PSHCs, considers seismic hazards to be a function of the geographical location and the site conditions. This is reinforced by seismic contours maps and codified seismic zoning maps from earthquake codes, which are uniformly based on defining the PSHCs associated with a given annual probability of exceedance (PE) over a given exposure interval.

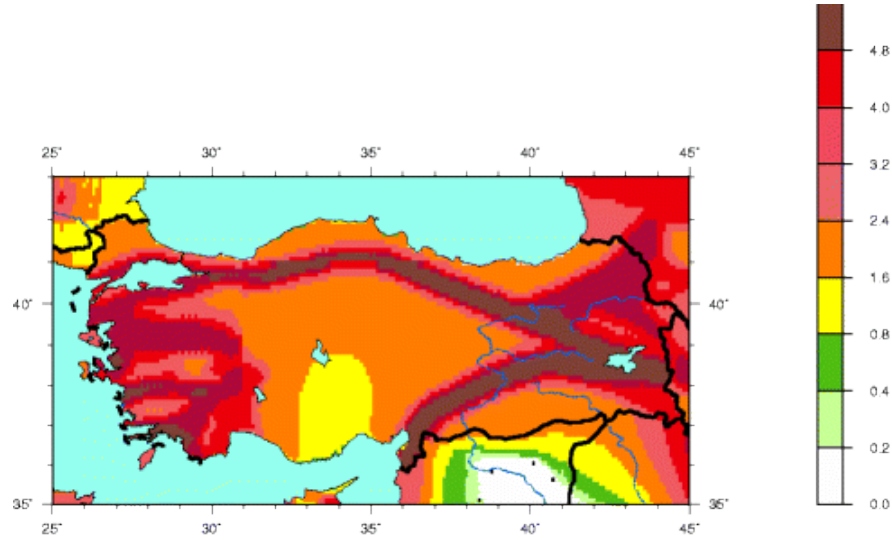


Figure 2. 3: Seismic Zoning. Velocity contours of exceedance 10% in 50 years. [14]

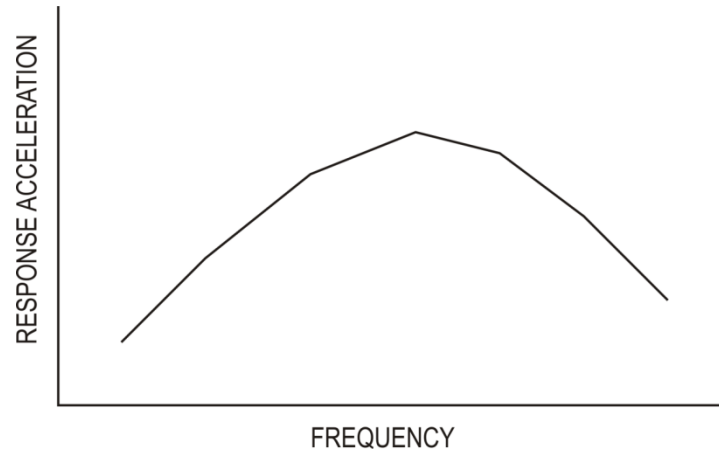


Figure 2. 4: ERS

With the introduction of PB, which means, the introduction of Displacement Response Spectrum (DRS) and Capacity Demand Diagram (CDD), the representation of ERS changes. Some models have already been developed, but it will take time to be fully accepted in codifications.

Combining DRS and CDD methods it is possible to find out the performance point. Several different literature explains the way to reach this goal like in Fajfar's method [1], the equivalent displacement method [2] and the capacity spectrum method [3]. The procedure is different for each of them even if all of them are iterative methods. Some comparisons have been done by the same Fajfar and Zamfirescu (Zamfirescu and Fajfar, 2001) concerning structural frames. In the paper of Fajfar [1] the detailed procedure is presented, as well as in FEMA 273 [2] and in

ATC 40 [3]. SAP2000 uses the improved procedure of FEMA273 presented in FEMA440 and ATC40 procedures. Despite procedures are different, they have similar formulae or even related. FEMA uses coefficients. Those ones represent concepts that are present in Fajfar's method under another name. Both methods calculate the inelastic displacement demand taking into account the elastic one. FEMA does it with 4 coefficients while Fajfar's method does the same with other corresponding tools to those coefficients. ATC is the one that is not so similar. The determination of seismic demand is determined from the equivalent elastic spectra. The inelastic behaviour of the structure is taken in account through the equivalents damping and period. More explanations can be found in [1].

The aim of the method is to reach the performance point for the posterior damage analysis in this point. This point comes from the intersection between the demand and the capacity curves. To be able to plot both curves in a same graph, curves are plotted under a pseudo-acceleration and pseudo-displacement domain. It is called Acceleration-Displacement Response Spectra (ADRS) after Mahaney, 1993. The capacity curve which is originally defined under Base shear-displacement domain has to be transformed into an ADRS. After the determination of the pseudo-target displacement, the transformation has to be done again to the base reaction-displacement domain. Damage analysis is done in this domain. In all the procedures, the period of the first mode has to be considered.

SAP2000 uses the procedures by ATC40 or FEMA440.

2.4.1 ATC40 procedure

Below the different steps for the ATC40 method are explained. The details can be found in ATC 40-8.2.2.1.2. The following steps are involved:

- 1- Development of the demand spectrum by calculating the appropriated 5% damping response spectra as a function of seismic coefficients, C_a and C_v . Soil condition and the Maximum Earthquake (ME) defines coefficients C_a and C_v . Tabulations have been provided in ATC40 to facilitate the determination. Transform the demand spectrum to the Spectral Acceleration- Spectral Displacement Domain (Figures 2.5 and 2.6).

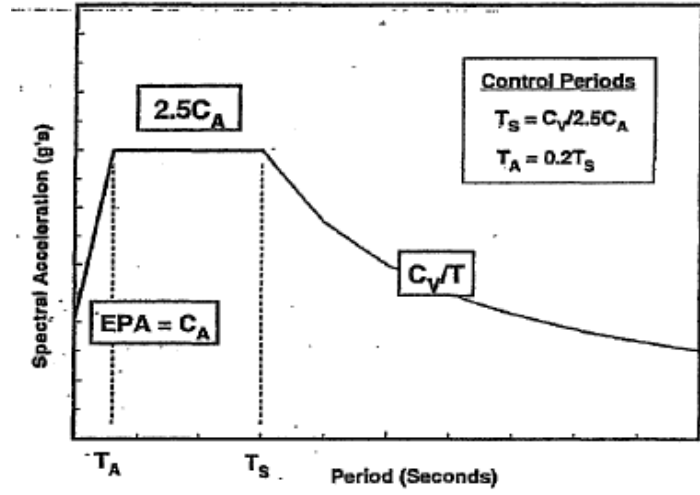


Figure 2. 5: DS definition from ATC 40 [3].

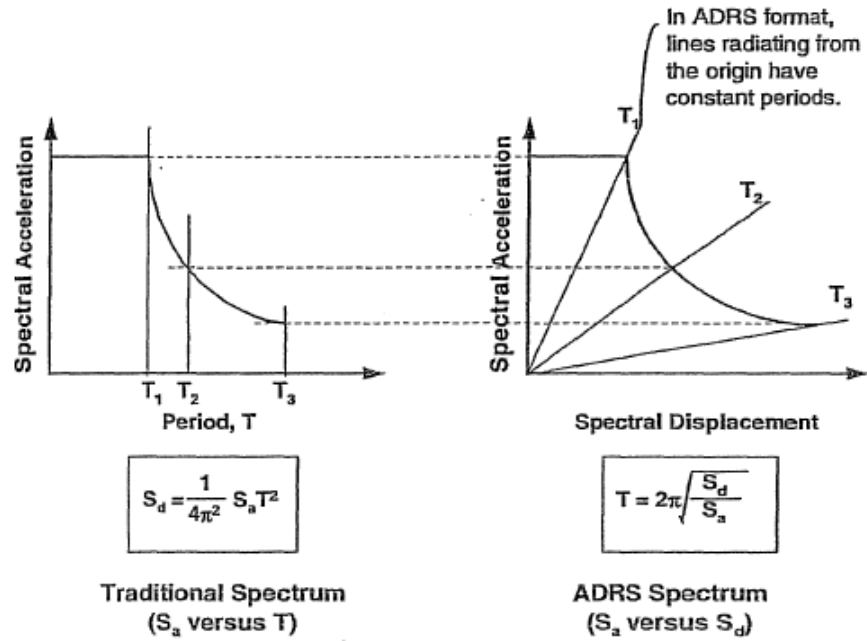


Figure 2. 6: Transformation of DS to Spectral domain [3].

- 2- Transform the capacity curve to the Spectral Acceleration- Spectral Displacement domain using the following formulae, the details can be found in ATC40-p 8-9 and 8-12.:

$$S_{ai} = V_i/W / \alpha_1 \quad (2.3)$$

$$S_{di} = \Delta_{roof} / (PF_1 \cdot \phi_{1,roof}) \quad (2.4)$$

Where:

$\alpha_1 \equiv$ Effective mass coefficient for the first natural period

$PF_1 \equiv$ Participation factor for the first natural period

$W \equiv$ Total dead load

$\phi_{1,roof} \equiv$ Top displacement at the roof level for the first mode

- 3- Choose the first candidate point (a_{pi} , d_{pi}). A good one is the one that comes from the drop of the intersection point between the elastic slope of the capacity spectra and the demand spectra into the capacity spectra. Other points from the capacity spectra are also suitable. If the candidate point matches the ultimate point of the capacity spectra, it would mean that the performance point would have been found and the process could stop (Figure 2.7).

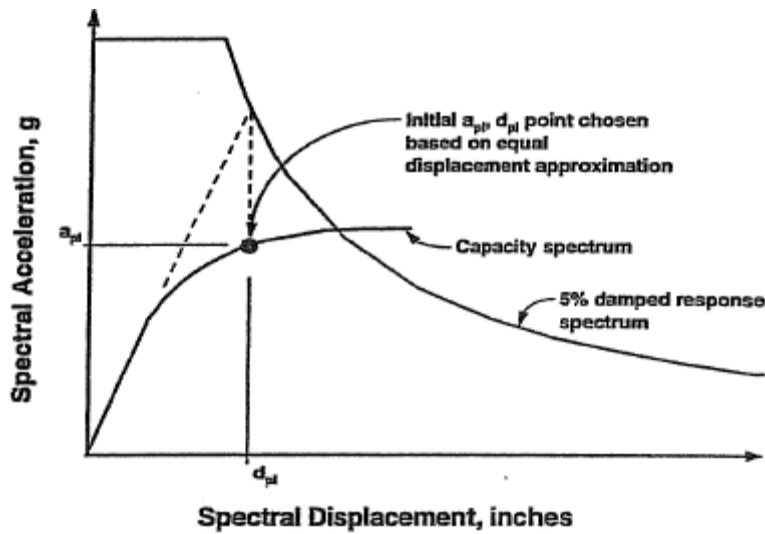


Figure 2. 7: Determination of candidate point [3].

- 4- Develop a bilinear representation of the capacity spectrum where the first stretch is the initial stiffness, or in other words, the elastic slope. The second one starts at the candidate point and has to intersect the elastic slope taking into account that the discriminated areas have to be more or less equals (Figure2.8).

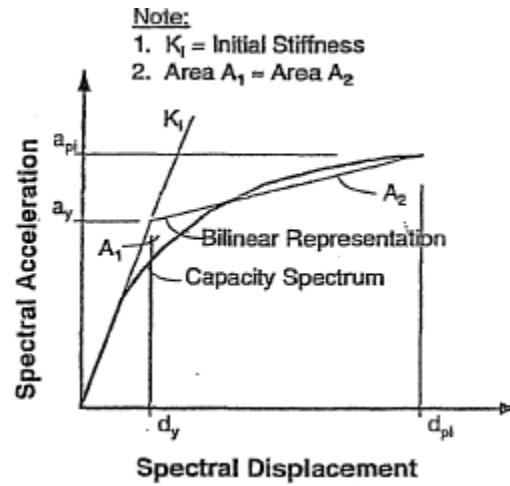


Figure 2. 8: CS bilinearization [3].

- 5- Calculate two spectral reduction factors which depends on the type of building, the shaking time, and the equivalent damping representation of the hysteretic damping associated with the bilinear approximation of the capacity spectra. Modify the DS by those spectral reduction factors and repeat the same procedure as in the third step. If the new candidate matches the previous one, or it is acceptable within the tolerance criteria, it means that the performance point has been reached. If not, continue with the process until the performance point is found out. At p 8-17 of ATC40 it is possible to find the formulae of the reduction factors (Figure 2.9).

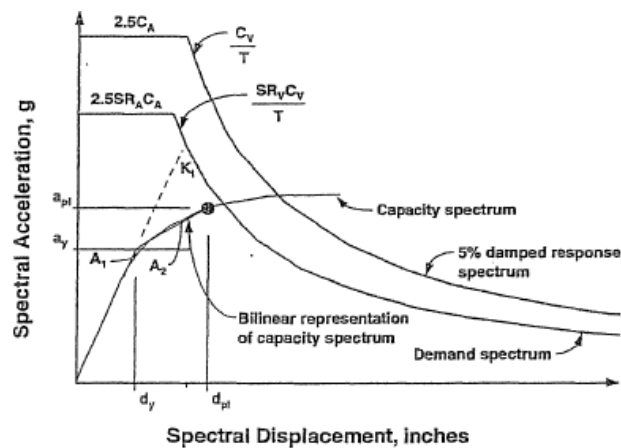


Figure 2. 9: New candidate point with MADRS [3].

The target point is a point of the capacity curve of the building. It represents the maximum expected displacement and it is supposed to be the point that defines the

behaviour of the building against some specific earthquakes conditions. That means that it is possible to have several performance points. Everything depends on the soil condition, the situation of the building and the earthquake conditions, as it has already been said.

Subsequently, the structure has to be pushed to the displacement defined by the performance point, in the way to be able to do a damage analysis.

The damage analysis used in this thesis is the one based on the plastic hinge hypothesis which is explained in FEMA356.

2.4.2 FEMA 440 procedure

The procedure used by SAP2000 is the one presented by FEMA440 as Procedure C or Equivalent Linearization (MADRS Locus of Possible Performance Point). This approach, uses the modified acceleration-response spectrum for multiple assumed solutions (a_{pi} , d_{pi}) and the corresponding ductilities to generate a locus of possible performance points. The actual performance point is located at the intersection of this locus and the capacity spectrum (Figure 2.10). Here the different steps are presented:

- 1- Select a spectral representation of the ground motion of interest with an initial damping, β_i (normally 5%). This may be a design spectrum from ATC-40 or FEMA 356, a site-specific deterministic spectrum, or an equal hazard probabilistic spectrum.
- 2- Modify the selected spectrum, as appropriate, for soil-structure interaction (SSI) in accordance with the procedures in Chapter 9 of FEMA440. This involves both potential reduction in spectral ordinates for kinematic interaction and a modification in the system damping from the initial value, β_i to β_0 , to account for foundation damping. If foundation damping is ignored, β_0 is equal to β_i .
- 3- Convert the selected spectrum, modified for SSI when appropriate, to an acceleration-displacement response spectrum format in accordance with the guidance in ATC-40. This spectrum is the initial ADRS demand.
- 4- Develop a CS in the Spectral Domain, following the procedure described in ATC40.
- 5- Choose the first candidate point as it is done in ATC40 procedure.

- 6- Develop a bilinear representation of the capacity curve as it has been done in ATC40 procedure.
- 7- For the bilinear representation developed in Step 6, calculate the values of post-elastic stiffness, α , and ductility, μ as it is explained in p. 6-7 of FEMA440.
- 8- Using the calculated values for post-elastic stiffness, α , and ductility, μ , from Step 7, calculate the corresponding effective damping, β_{eff} , (see Section 6.2.1, FEMA 440). Similarly calculate the corresponding effective period, T_{eff} , (see Section 6.2.2, FEMA 440).
- 9- Using the effective damping determined from Step 8, adjust the initial ADRS to β_{eff} (see Section 6.3, FEMA 440).
- 10- Multiply the acceleration ordinates of the ADRS for β_{eff} by the modification factor, M , determined using the calculated effective period, T_{eff} , in accordance with Section 6.2.3 of FEMA440 to generate the modified acceleration-displacement response spectrum (MADRS).
- 11- A possible performance point is generated by the intersection of the radial secant period, T_{sec} , with the MADRS.
- 12- Increase or decrease the assumed performance point and repeat the process to generate a series of possible performance points.
- 13- The actual performance point is defined by the intersection of the locus of points from Step 12 and the capacity spectrum.

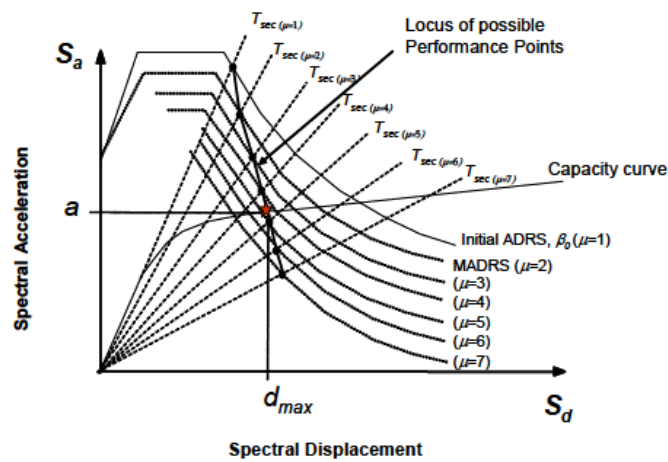


Figure 2. 10: Performance point with FEMA 440 [8].

3. CASE STUDY: REINFORCED CONCRETE BUILDING

3.1 Objectives

The purpose of this chapter is to present the considered reinforced concrete building and all the known data and those necessary that are unknown.

3.2 Description

The case study building is a reinforced concrete building located in Çapa Campus, of Istanbul University, which has 4 stories with a total of 16.30 m height and it is composed of orthogonal frames. Their disposition is almost symmetric. Planar dimensions are $21.5 \times 15.5 \text{ m} = 333.25 \text{ m}^2$ with 6 bays in X direction and 3 in Y direction.

Storey heights are 3.40 m for the first storey, 4.50 m for the second and the third stories and 3.90 m for the fourth storey.

Columns of the first storey have section width and depth as 30/55 cm and 30/45 cm for external and internal columns, respectively. Columns of the upper stories have section width and depth as 25/40 cm and 25/30 cm for the external and internal columns, respectively.

Beams have all a T-section shape. All have a flange width of 1 m and a thickness of 15 cm. Beams aligned in “x” direction have section base and height of 20/50 and 20/40 for external and internal beams respectively. Beams aligned in “y” direction have section base and height of 20/60.

The plans of the building can be founded in App. A.

3.3 Existing Data of the Building

In the rehabilitation project of the building, gravity load data is existing. There is no data about reinforcement bars of the beams and columns. Finally, it is known that the fundamental period is around: $T_0 \approx 1.2$ s. This period is greater than the usual ones for those type of buildings. It is due to several reasons. Firstly, period was calculated without taking into account the infill walls. Stiffnesses are smaller and storey heights are greater than the ordinary buildings.

Gravity loads are in the following:

3.3.1 Slab loads

Dead load:

Self weight of the slab and the one for the covers and plasters of itself is considered. Their sum makes the total dead load of the slabs.

In this way we have:

The thickness of the slab: $h_f = 0.15$ m.

The density of the concrete: $\delta_c = 25$ KN/m³.

Self weight of the slab: $(h_f * \delta_c) \dots\dots\dots 3.678$ KN/m².

Cladding and plaster weight: $\dots\dots\dots 1.470$ KN/m².

Total dead weight of slabs $\dots\dots\dots g = 5.15$ KN/m²

Live load:

The assumed value as the live load of the floor, it is a very common used value in Turkey.

Floor live load $\dots\dots\dots q = 1.962$ KN/m²

For the roof, half of the live load is applied.

3.3.2 Infill wall loads

Gravity loads corresponding to infill walls are given:

External infill walls..... $q_{we}=4.12 \text{ KN/m}^2$

Internal infill walls..... $q_{wi}=2.45 \text{ KN/m}^2$

3.4 Unknown Data of the Building

The reinforcements, their disposition and also materials properties are unknown data.

Reinforcements will be determine by a try and error process using M-KAPA computer program [15]. The reinforcements will determine the capacity of the members which is necessary to be able to run a nonlinear pushover analysis.

About the material properties, some assumptions are done. According to the common way to construct in those years, it is possible to imagine which materials were used.

Assumptions done are the following:

3.4.1 Material properties

3.4.1.1 Existing elements

Information on the mechanical properties of assumed construction materials are presented below.

Concrete:

density: $\delta_c=24.5 \text{ KN/m}^3$

Quality: C12

Characteristic compression stress: $f_{ck}=12 \text{ MPA}=12000 \text{ KN/m}^2$

Work compression stress: $f_{ck}=12 \text{ MPA} \cdot 0.85 =10200 \text{ KN/m}^2$

Using a security coefficient of $\gamma_s=1.5$ (concrete) there is:

$$f_{cd} = \frac{f_{ck}}{\gamma_s} \quad (3.1)$$

$$f_{cd} = \frac{0.85 \cdot 12000}{1.5} = 6800 \frac{\text{KN}}{\text{m}^2}$$

Elastic modulus: From TS500 [16]:

$$E(\text{MPa}) = 3250 * \sqrt{f_c} + 14000 \quad (3.2)$$

$$E=252.58 \cdot 10^5 \text{ KN/m}^2$$

Steel: Used for the reinforcing bars.

Quality: S220

$$\text{Yield stress:} \quad f_{sy}=220 \text{ MPA}=220000 \text{ KN/m}^2$$

$$\text{Tensile stress:} \quad f_{su}=275 \text{ MPA}=275000 \text{ KN/m}^2$$

$$\text{Elastic modulus: } E=2 \cdot 10^8 \text{ KN/m}^2$$

A security coefficient will be used for design: $\gamma_s=1.15$

So the following values are calculated:

$$f_{yd} = \frac{f_{yk}}{\gamma_s} \quad (3.3)$$

$$f_{yd} = \frac{220000}{1.15} = 191304.34 \frac{\text{KN}}{\text{m}^2}$$

$$f_{ud} = \frac{f_{su}}{\gamma_s} \quad (3.4)$$

$$f_{ud} = \frac{275000}{1.15} = 239130.43 \frac{\text{KN}}{\text{m}^2}$$

3.4.1.2 New elements

Jackets for columns, shear walls and the mat foundation will be considered as new elements. The material properties for those elements are better than the existing. Due to the fact that they are new, they have to be treated like design elements. Subsequently, a design coefficient will be used:

$$\gamma_s=1.5 \text{ (concrete) or } 1.15 \text{ (steel)}$$

$$\text{Concrete: density: } \delta_c=24.525 \text{ KN/m}^3$$

Quality: C40

$$\text{Characteristic compression stress:} \quad f_{ck}=40 \text{ MPA}=40000 \text{ KN/m}^2$$

So values are calculated:

$$f_{cd} = \frac{f_{ck}}{\gamma_s} = \frac{0.85 \cdot 40000}{1.5} = 22666.66 \frac{\text{KN}}{\text{m}^2}$$

$$\text{From Eq. 3.2: Elastic modulus: } E=345.55 \cdot 10^5 \text{ KN/m}^2$$

Steel: Used for the reinforcing bars.

Quality: S420

Yield stress: $f_{sy}=420 \text{ MPA}=420000 \text{ KN/m}^2$

Tensile stress: $f_{su}=550 \text{ MPA}=550000 \text{ KN/m}^2$

Elastic modulus: $E=2 \cdot 10^8 \text{ KN/m}^2$

A security coefficient will be used for design: $\gamma_s=1.15$

So values are calculated:

$$f_{yd} = \frac{f_{yk}}{\gamma_s} = \frac{420000}{1.15} = 365217.39 \frac{\text{KN}}{\text{m}^2}$$

$$f_{ud} = \frac{f_{su}}{\gamma_s} = \frac{550000}{1.15} = 478260.87 \frac{\text{KN}}{\text{m}^2}$$

4. MATHEMATICAL MODEL

In this chapter, the 3D mathematical model of the building is presented. It includes the distribution of loads, the reinforcement configuration as well as some checks that will confirm the correctness of the mathematical model.

Firstly the existing building is modelled for its analysis. The retrofitted cases will be modelled in base of the existing building model.

4.1 Modelling the 3D Structure

To achieve the 3D model, the structure skeleton has to be drawn. Material and section has been defined. To the different frame members, appropriated section type have been assigned. Finally, loads have been assigned to the structure.

The mathematical model would be ready to run a static analysis. To be able to run a modal and non-linear analysis, masses have to be assigned at each storey.

In App. A the SAP2000 frame can be found. And in App. C2, sections of the existing building with their reinforcement configuration can be found too.

4.2 Definition of Loads

Loads are known from section 3.3, but the distribution is unknown. The calculation is going to be presented here, keeping in mind that edge beams just care one slab.

4.2.1 Slab loads

The tributary areas of the edge beams surrounding a slab [17] are shown in figure 4.1:

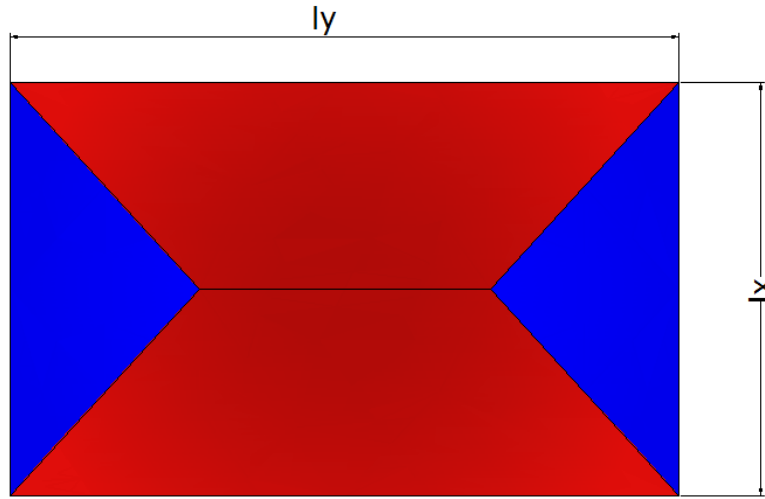


Figure 4. 1: Loads distribution diagram due to slabs.

Red trapezoids and blue triangles are the shapes of the load distribution on beams due to slabs. To simplify the introduction of those load distributions to the beams, a uniform distributed load will be used. A transformation of those non-uniform distributions to uniform ones has been done. To reach this purpose, following formulas have been used:

- For beams in the short direction (blue side), the equivalent distributed load is:

$$P_{ex} = \frac{P \cdot l_x}{3} \quad (4.1)$$

- For beams in large direction (red side), the equivalent distributed load is:

$$P_{ey} = \frac{P \cdot l_x}{3} * \left[\frac{3}{2} - \frac{1}{2 * \left(\frac{l_y}{l_x} \right)^2} \right] \quad (4.2)$$

Using those formulas, and considering both, dead and live slab's load, results are found out and presented in Table 4.1:

Table 4. 1: Uniformly distributed loads transferred to the beams.

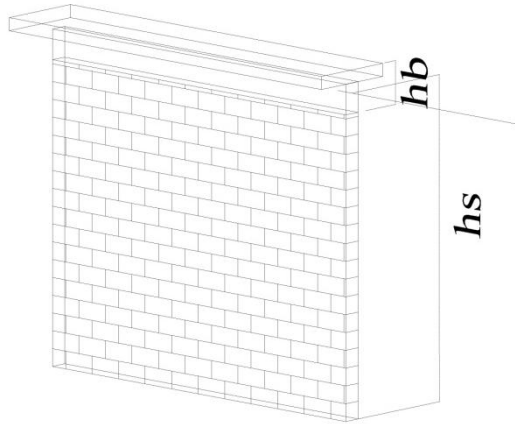
Slab between axis						
A-B/C-D	1-2		2-3/3-4		4-5/5-6/6-7	
$l_y=5.48/5.53\text{m}$	$l_x=3.38\text{m}$		$l_x=3.25\text{m}$		$l_x=3.30\text{m}$	
Load KN/m	P_{ex}	P_{ey}	P_{ex}	P_{ey}	P_{ex}	P_{ey}
Dead	5.80	7.60	5.58	7.42	5.65	7.46
Live	2.20	2.89	2.13	2.82	2.16	2.85

Slab between axis						
B-C	1-2		2-3/3-4		4-5/5-6/6-7	
$l_y=3.5\text{m}$	$l_x=3.38\text{m}$		$l_x=3.25\text{m}$		$l_x=3.30\text{m}$	
Load KN/m	P_{ex}	P_{ey}	P_{ex}	P_{ey}	P_{ex}	P_{ey}
Dead	5.80	5.98	5.58	5.98	5.65	5.98
Live	2.20	2.28	2.13	2.28	2.16	2.28

Those tables are simplified ones where similar values and shear walls with similar dimensions have been assumed as equal. Detailed tables can be found in App. B.1.

4.2.2 Infill wall loads

The procedure to determine dead loads of walls is the following:

**Figure 4. 2:** Typical infill wall.

$$g_w = h_w * q_w \quad (4.3)$$

Where:

$$h_w = h_s - n * h_b \quad (4.4)$$

h_s : height of the storey.

h_b : height of the beams between storey

n : takes value of 1 if the wall is one of the first, second, or third floor

takes value of 0.5 if the wall is one of the ground floor.

Table 4.2 presents results for loads coming from infill wall loads:

Table 4. 2: Uniform distributed loads (KN/m) coming from infill walls.

Floor	External Walls		Internal Walls	
	Beam of the slab			
	T20/60	T20/50	T20/60	T20/40
Ground floor $h_s=3.4\text{m}$	12.77	12.98	7.60	7.85
First floor $h_s=4.5\text{m}$	16.48	16.07	9.56	10.05
Second floor $h_s=4.5\text{m}$	16.48	13.60	9.56	10.05
Third floor $h_s=3.9\text{m}$	14.00	13.60	8.09	8.58

4.2.3 Frames self weight

No specific load is defined to represent the self weight of columns. Defining the density of the concrete the program considers it.

For beams, the upper part corresponding to the flange is already taken as slabs loads. The lower part of the beam is added as an additional gravity load.

4.2.3.1 Load coming from the lower part of the beam

As it is explained in the previous section, the weight of the beam has to be assumed in two different parts. The upper part of the beam, the flange of the T-section, is already borne in mind at the definition of loads coming from slabs. The lower part of the beam has to be calculated and added to those loads as a dead load. The calculation procedure is given below:

$$g_b = d * b * \delta c \quad (4.5)$$

Where:

$$d = h - h_s \quad (4.6)$$

g_b : dead load of the lower part of the beam

d : height of the lower part of the beam

h_s : thickness of the flange = 0.15m

h : height of the T-beam

b : width of base of the beam

Results given in the following Table 4.3 are obtained by using Eq. (4.5):

Table 4.3: Loads from the lower part of the beam.

Beam	T20/60	T20/50	T20/40
g_b (KN/m)	2.21	1.72	1.23

4.2.4 Statically equivalent earthquake loads

Equivalent static loads will be applied to the 3D model to define the reinforcements of beams of the original structure. They will be applied in “X” and “Y” directions. Their determination is the next and Table 4.4 recollect results:

$$F_i = \frac{W_i * h_i}{\sum W_i * h_i} * F_{BASE} \quad (4.7)$$

$$F_{BASE} = A(t) * \sum W_i \quad (4.8)$$

Considering age of the structure, it is assumed that $A(t)=0.08$.

$$W_i = g_i * A_i + n * q * A_i \quad (4.9)$$

Where: W_i : Storey weight

g_i : Total dead load of the floor (KN/m²)

q : Live load (KN/m²)

n : Reduction factor

A_i : Area of each floor (m²)

$n=0.4$

$A_i = 15.5 \text{ m} \cdot 21.5 \text{ m} = 333.25 \text{ m}^2$

$$g_i = g + V_{Tb} * \delta_c \quad (4.10)$$

V_{Tb} : Total volume of the lower part of beams for each floor

The details of calculation can be founded in App. B2.

Results are presented:

$$W_1=W_2=W_3=W_4=2331 \text{ KN}$$

$$F_{BASE}=0.08*2331*4=746 \text{ KN}$$

Table 4.4: Statically equivalent earthquake loads (KN).

Floor	h_i (m)	F_i (KN)
1	3.4m	6.463
2	7.9m	15.01
3	12.4m	23.665
4	16.3m	30.983

Those forces will be assigned at their corresponding storey level.

4.2.5 Storey masses

As it has been discussed in Section 4.1, masses of each storey are needed to find out the vibration period of the structure. From the previous section, it is known that the weight of each floor is almost equal to: $W_i=2331 \text{ KN}$.

SAP2000 needs masses to be introduced in Force-sec²-length units; they have to be divided by the gravity acceleration. That means that $W_i/9.81$ will be introduced. This value is $237.61 \text{ KN}\cdot\text{sec}^2/\text{m}$.

Four masses will be considered. One for each floor. In this way the mathematical model will be simplified to how it is shown in the Figure 4.3:

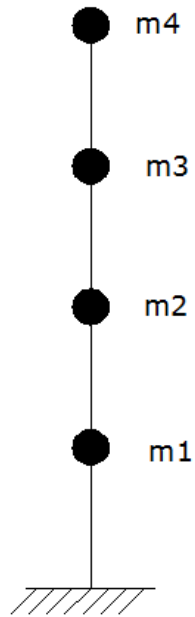


Figure 4. 3: Mathematical model for modal analysis.

4.3 Checkings and Final Determinations

After the modelling and the assigning of loads and masses, the mathematical model is ready to be run and checked.

Through a modal analysis, vibration periods have been calculated.

Through static analysis, internal forces diagrams are obtained. Depending on the internal forces, reinforcement calculations were performed.

4.3.1 Modal analysis

After the definition of masses and their assignments to the storey levels, the model is ready to run a modal analysis (Figure 4.4). The first vibration period has a value of $T_0 \approx 1.16$ s. It is close to the one known from an existing study of the building ($T_0 \approx 1.20$ s). It is possible to affirm that the 3D mathematical model is correct.

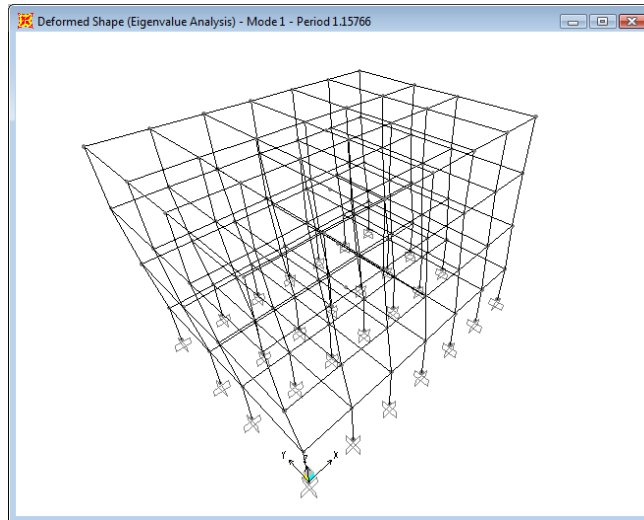


Figure 4. 4: The first mode shape of the building.

4.3.2 Static analysis

The worst load combination is the following: $G+Q+E_i$.

Its results will be used to get the reinforcement configuration of the beams.

4.3.3 Determination of the reinforcements

The capacity curve of the structure is defined through the known mechanical properties of the reinforced concrete building. Quantities of reinforcements are determinant to define the capacity curve of the structure. Results of the superposition case $G+Q+E_i$ are used to establish the reinforcement configuration of the frames. The determination of the reinforcements for beams can be done using M-KAPA program by a try and error process. Reinforcement configuration for columns is assumed.

M-KAPA program allows to determine reinforcements for T-beams and also rectangular ones, in middle span or at the end. Definition of materials and also positions of reinforcements as well as the whole reinforcement area are needed. The capacity curve of the beam has to match the solicitation imposed by the load combination.

The load combination determines that the critical moments are focused at the end of the span of beams. Therefore, the determination of the reinforcements has been focused just on this part of beams.

4.3.3.1 Columns

For the determination of reinforcements of the columns, the following assumption is done in base to Turkish construction practice:

$$A_s/A_c = 1\% \text{ or } 0.8\%$$

Reinforcement configuration of columns is given below:

$$25/30 \rightarrow A_s = 7.5 \text{ cm}^2 \rightarrow 4\emptyset 16 = 8 \text{ cm}^2$$

$$25/40 \rightarrow A_s = 10 \text{ cm}^2 \rightarrow 6\emptyset 16 = 12 \text{ cm}^2$$

$$30/45 \rightarrow A_s = 13.5 \text{ cm}^2 \rightarrow 4\emptyset 20 = 12.56 \text{ cm}^2$$

$$30/55 \rightarrow A_s = 16.8 \text{ cm}^2 \rightarrow 8\emptyset 16 = 16 \text{ cm}^2$$

Sections draws can be found at Figure 4.5.

4.3.3.2 End span of beams

At the end of the span, moments can be positive or negative.

Design moments comes from the load combination $G+Q+E_i$.

As a result of the load combination, the design moments for the upper part of the beam are introduced:

$$T20/60 \rightarrow M_d = -157 \text{ KN}\cdot\text{m}$$

$$T20/50 \rightarrow M_d = -113 \text{ KN}\cdot\text{m}$$

$$T20/40 \rightarrow M_d = -88 \text{ KN}\cdot\text{m}$$

For the lower part of the beam:

$$T20/60 \rightarrow M_d = 106 \text{ KN}\cdot\text{m}$$

$$T20/50 \rightarrow M_d = 107 \text{ KN}\cdot\text{m}$$

$$T20/40 \rightarrow M_d = 75 \text{ KN}\cdot\text{m}$$

Data of this study can be found in App. C1.

Results of this study and draws of sections can be found in Figure 4.5.

Hereafter, a summary of the study solution:

$$T20/60 \rightarrow 2\emptyset 12 + 5\emptyset 20 \text{ in the upper side; } 4\emptyset 20 \text{ in the lower side;}$$

$M_{0-} = -178 \text{ KN}\cdot\text{m}$ and $M_{0+} = 129 \text{ KN}\cdot\text{m}$

T20/50 $\rightarrow$ 2 $\emptyset$ 12 + 4 $\emptyset$ 20 in the upper side; 3 $\emptyset$ 20 in the lower side;

$M_{0-} = 122 \text{ KN}\cdot\text{m}$ and $M_{0+} = 109 \text{ KN}\cdot\text{m}$

T20/40 $\rightarrow$ 2 $\emptyset$ 12 + 4 $\emptyset$ 20 in the upper side; 4 $\emptyset$ 20 in the lower side;

$M_{0-} = -95 \text{ KN}\cdot\text{m}$ and $M_{0+} = 82 \text{ KN}\cdot\text{m}$

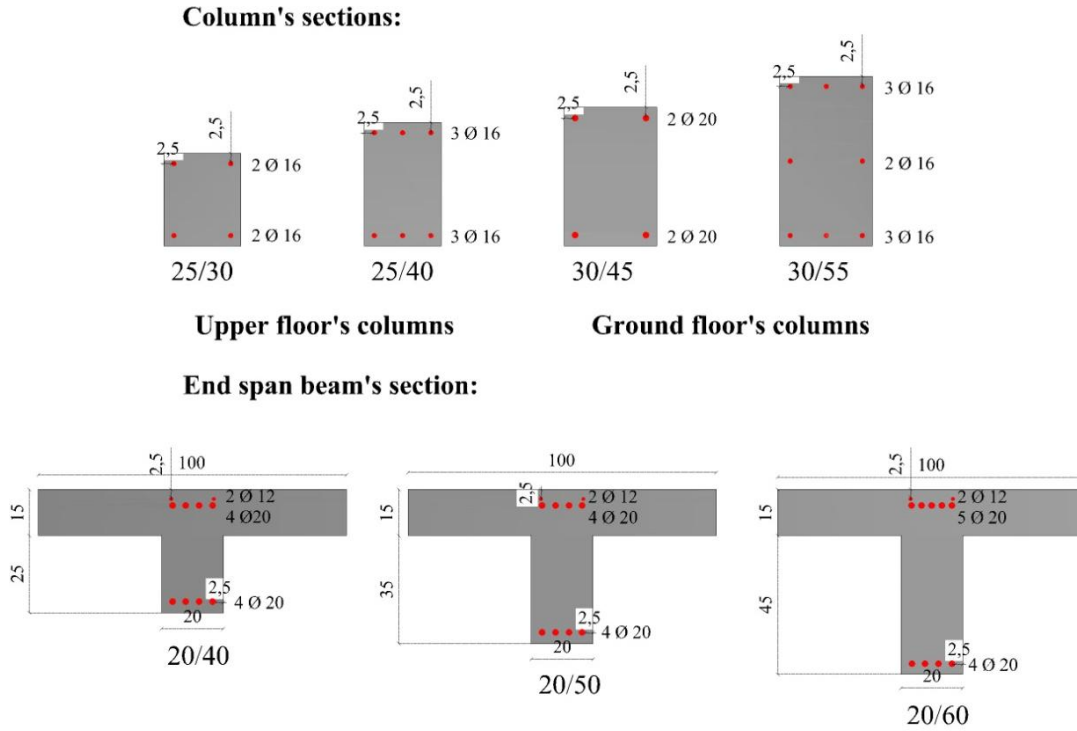


Figure 4. 5: Sections of the actual building.

4.3.4 Preparation for the pushover analysis

Two different pushover analyses are performed to reach the goal. The first one is a pushover case of all the static loads associated to gravity loads (Dead and Live), which is load controlled. The structure should not yield running gravity loads, it is why a load control is used.

Second one can be a pushover analysis of static earthquakes loads in direction “x” (Push x) or in direction “y” (Push y). Those are controlled by displacement because it is supposed that the structure will suffer non-linear behaviour. Joint between beams K406 and K445 will be the joint control. This joint is located on the roof on a corner. It allows to control displacement in both direction and it is supposed to have the highest displacement.

The aim of the pushover analysis is to find out the capacity curve of the structure. For this purpose it is necessary to define and indicate where frames of the structure abandon elastic behaviour for plastic one. This definition is done by the definition of hinges.

The cracking of the stiffness has also to be included. That will increase fundamental period of the structure but it will allow to get higher displacements. Following the turkish earthquake code:

Effective stiffnesses in the analysis have to be 60% of the real one for columns and a 40% for the beams.

4.3.4.1 Plastic hinges for the pushover analysis

The nonlinearities for various structural elements such as RC beams, columns and shear walls are modelled considering the concept of hinge defined by FEMA 356 and included in SAP2000. Those are assigned to potential hinge points. The control of hinges is made by their rotation or their curvature.

4.3.4.2 Location of the plastic hinges

The location of the hinge is imposed by the load demand. From gravity loads, it results an axial effort to the columns and a bending moment at the middle span of the beam. Even so, bending moments are more significant, and for beams, moments from earthquakes loads are bigger than from other kinds of loads. Consequently lateral equivalent static seismic loads are the critical factor.

Under a lateral load, moments diagrams are distributed as it is shown in Figure 4.6:

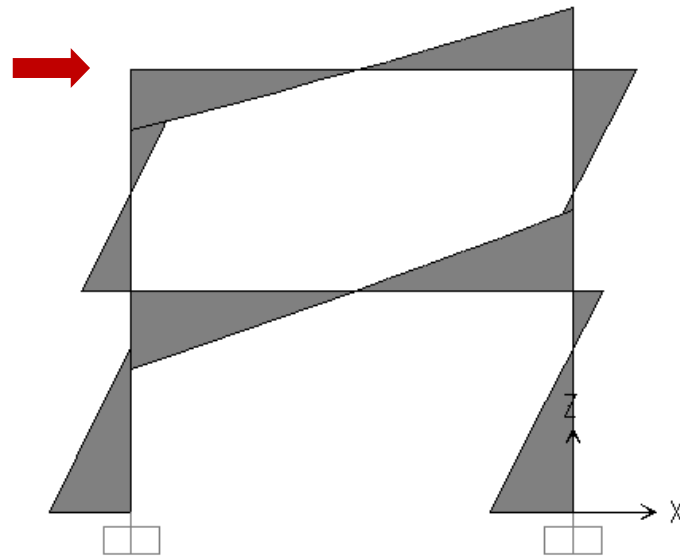


Figure 4. 6: Moment distribution for a lateral load.

For an earthquake load, the sign of the moment changes, that means that both extremities of the frame suffer the same moment. Those extremities are the places where moment is higher and they are favourable to become plastic. That means that they are meaningful points for the plastic hinge location.

The assignation would be as it is presented in Figure 4.7, in base at the previous figure:

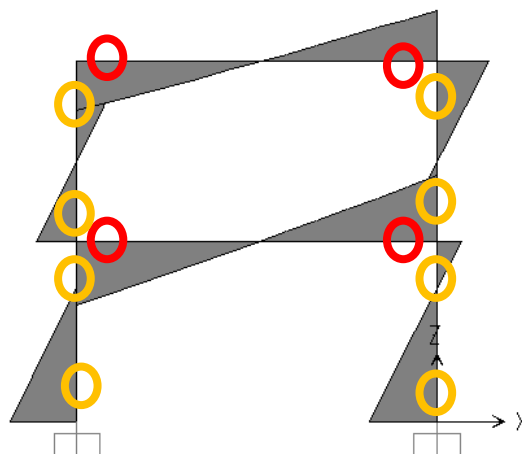


Figure 4. 7: Hinge assignation.

4.3.4.3 Definition of hinges

SAP2000 is able to define them automatically basing it on the reinforcement model, the material characteristics, and on the provisions of FEMA 356.

4.3.4.3.1 Hinges of columns

From FEMA 356, Table 6-8, a PMM hinge will be defined. A PMM hinge represents the relationship between Axial force (P)-Moment around weak axis (M_2)-Moment around strong axis (M_3). A 3D shape represents this configuration. It includes even positive as negative efforts. Program calculates it from results reached with the load combination $G+0.3Q+E_i$. (from Turkish Earthquake Code)

4.3.4.3.2 Hinges of beams

From FEMA 356, Table 6-7, a M_3 hinge will be defined. A M_3 hinge represents the relationship between Moment around strong axis (M_3) and curvature or rotation of the plastic hinge. A curve represents this configuration. It includes both signs of efforts. Program calculates it from results reached with the load combination $G+Q+E_i$. (from Turkish Earthquake Code)

Because the plastic hinges will be applied just at the extremities of the beams, section doesn't need to be a T cross section. The reason is that for negative moments, T beams behave like rectangular beams. So the beam's section for the non-linear analysis will change to a rectangular section. This will be the core of the T section with the corresponding reinforcements.

In this thesis, the capacity limit of beams has been considered, even for neighbouring beams of shear walls. Some studies consider those beams as infinitely rigid beams.

4.4 Assumptions Made in Modelling

Assumptions than have been made during the modelling of the 3D mathematical model are described below:

- a) Joints are defined as rigid.
- b) Diaphragm constraints are defined at each storey levels representing the effect of slabs.
- c) Plastic hinges are placed at column and beam end points.

The mathematical model is the one presented below:

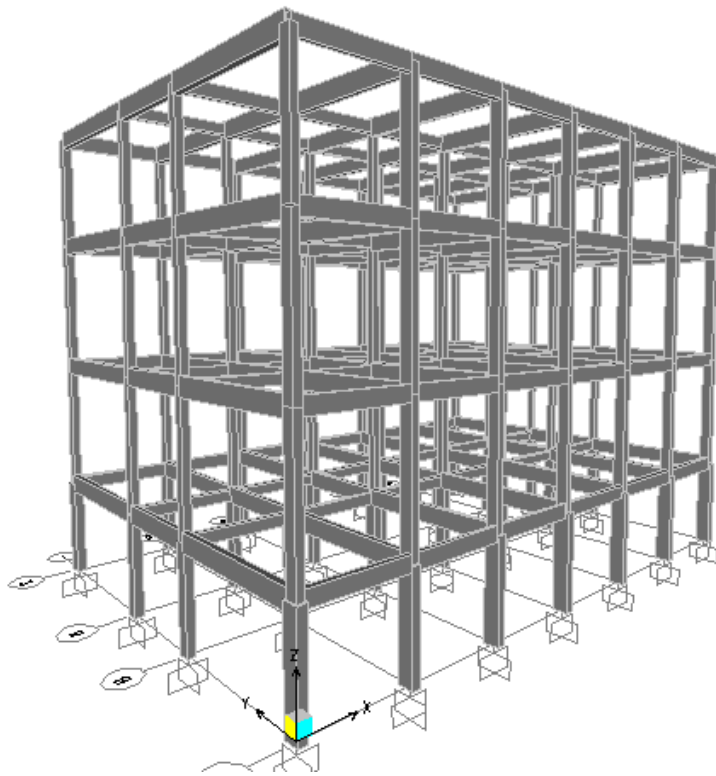


Figure 4. 8: 3D mathematical model

5. RETROFITTED CASES

To retrofit a structure means to improve the existing response with additional techniques such as column jacketing and shear-walls addition. Those techniques would use materials of higher quality. Concrete C40 and steel S420 have been used. The objective is to reach a better behaviour of the building against earthquakes. The performance is obtained by SAP2000 program. The following sections explain how they have been modelled.

5.1 Column Jacketing

Jacketing a column is increment the dimensions of the actual columns adding more concrete and more reinforcements of higher quality to reach higher capacities.

5.1.1 Definition of jacketed columns

By jacketing columns, all columns will have the same external dimensions. The existing columns are not removed, but in this case it is considered that in comparison with the new columns, the existing part will not be taken in account. A jacketed column can be represented by a box section with a void core representing the old column. The obtained jacketed columns are the following:

Columns from upper floors:

25/30 → BOX 50/65 type 1

30/45 → BOX 50/65 type 2

Columns from ground floor:

25/40 → BOX 50/75 type 1

30/55 → BOX 50/75 type 2

To determine the quantities of reinforcement the following assumption has been done: $A_s/A_c = 1\% \text{ or } 1.5\%$.

Reinforcements of the columns are given:

Any BOX 50/65 $\rightarrow A_s = 48.75 \text{ cm}^2 \rightarrow 24\phi 16 = 48 \text{ cm}^2$

Any BOX 50/75 $\rightarrow A_s = 56.25 \text{ cm}^2 \rightarrow 28\phi 16 = 56 \text{ cm}^2$

Section details can be found Figure 5.1. They have been modelled using the section designer feature of the program.

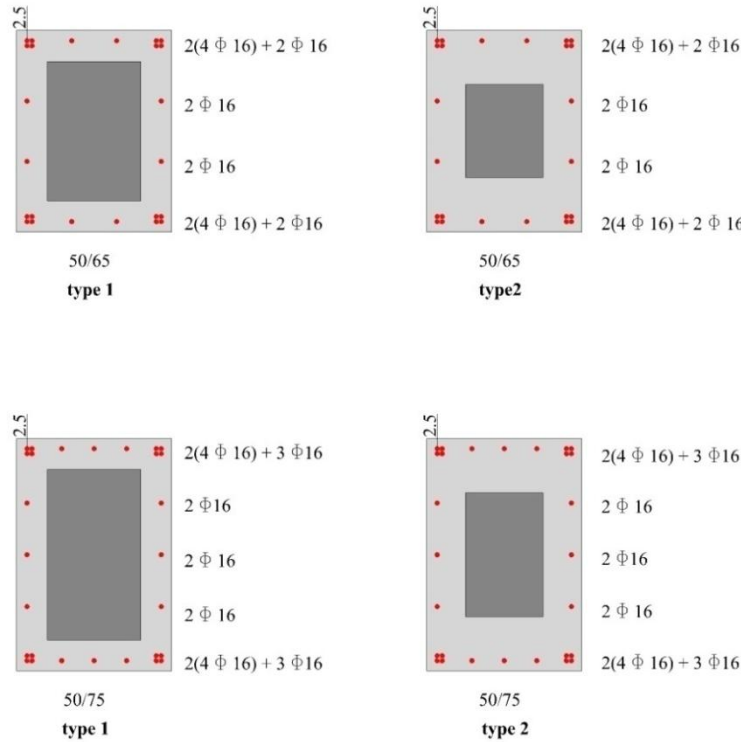


Figure 5. 1: Jacketed columns sections.

5.1.2 Plastic hinge model

Hinges have been defined using a fiber model. SAP2000 creates hinges automatically when the section is defined by the user from the material configuration and the reinforcement disposition. Their location would be the same as a normal column.

5.1.3 Analysed models

Three models are analysed. The objective is to realize how the jacketing distribution influences to the result. If the model would be suitable as a solution, hinges in those columns would appear at the lowest part of the jacketed columns at the ground level. A good solution would be represented by a configuration that fails by hinges in beams and not in columns.

5.1.3.1 First floor column jacketed

In this case columns of the first floor have been jacketed (Figure 5.2). The expected result is that this solution is not suitable.

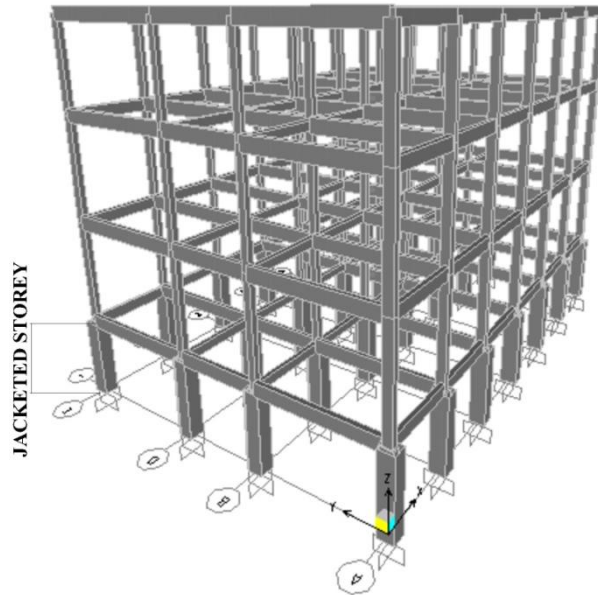


Figure 5. 2: 3D mathematical model of first floor column jacketed case.

5.1.3.2 First and second floor columns jacketed

This case represents the jacketing of columns at 1st and 2nd floors (Figure 5.3). It was expected to find an improvement in comparison with the previous case but it is also obtained that it is not a suitable solution.

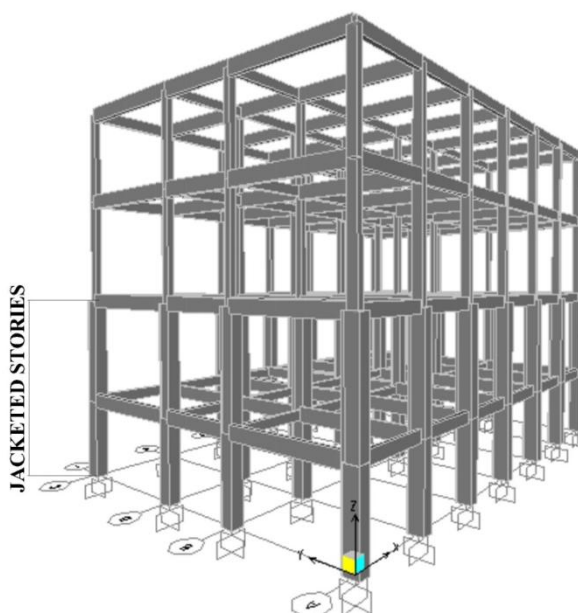


Figure 5. 3: 3D mathematical model of first and second floor columns jacketed case.

5.1.3.3 All columns jacketed

All the columns are jacketed in this model (Figure 5.4). It is expected to have a cracking by two possible ways:

- By cracking of beams. That would be the ideal solution.
- By cracking of jacketed columns. In the lower part of them. It would be a good solution too because it is the behaviour that it is expected to be.

If any of these cases appears, that would mean that it is a correct solution. A big improvement in comparison with the previous cases is also awaited.

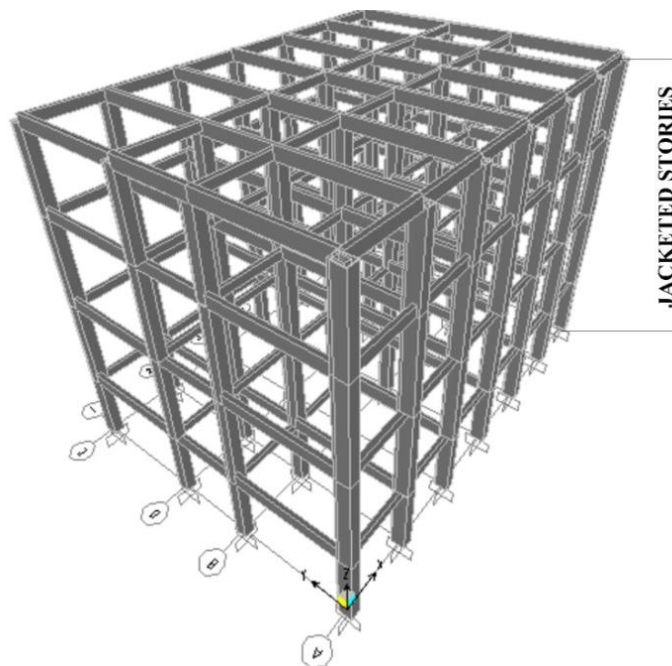


Figure 5. 4: 3D mathematical model of all columns jacketed case.

5.2 Additional Shear Walls

This is the second technique that is used. The main function of a shear wall is to support all the lateral loads as earthquake's and wind's. The shear walls considered in this thesis are made of concrete C40 and steel S420. Those are different materials if we compare them with the existing ones, but it is because they are new elements of the buildings and consequently, they are made with new materials.

5.2.1 Idealization of structure

In SAP2000 the shear walls have been modelled with the section designer tool. The awaited behaviour is to improve the general performance of the building and at the

moment of the collapse, the crack in shear walls should be focused at the feet of the shear walls. The representation of the shear wall will be done as a column frame that is independent of axial efforts. The connecting beams are considered as Infinite Rigid members (IR) (Figure 5.5). To reach this purpose, the inertia modulus is increased by a very high number: 10^6 will be the one used. Even so, those beams are fictitious, to make it sure that they don't interact with the structure more than just connectors, also Cross section (axial) Area, Shear area in 2 direction, Shear area in 3 direction and Torsion constant are increased by a big number.

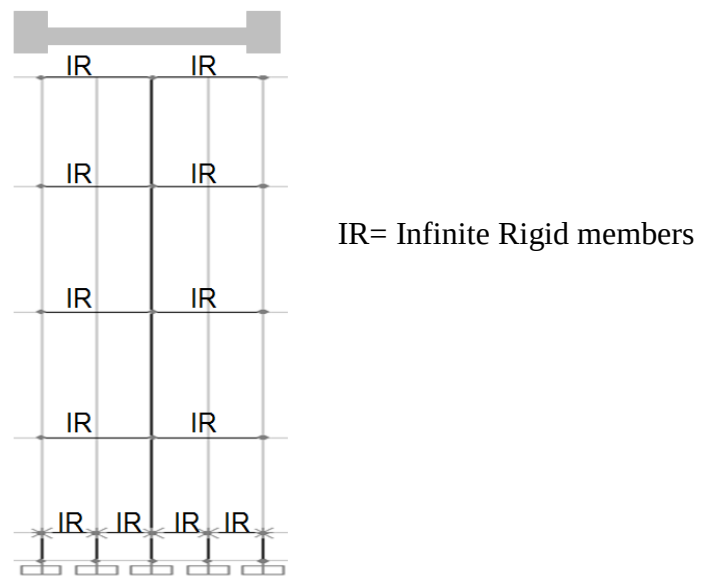


Figure 5. 5: Idealization of a Shear Wall.

5.2.1.1 Shear walls dimensions and reinforcements

To represent the shear wall the existing columns will be used. Their size will be increase in 15 cm in all directions and a thick element of 25 cm will connect them. Previous columns disappear.

Three different shear walls are modelled, they are defined by the axis where they are confined. Headings will have the following dimension:

Between axis B-C: 60/75

Between axis 2-3 and 5-6: 60/85

To determine the reinforcements that should be defined, two assumptions have been done:

For the headings of the shear walls:

$$A_s/A_c = 2\%$$

60/75 $\rightarrow A_s = 90 \text{ cm}^2 \rightarrow 6 * 4\emptyset 16 = 48 \text{ cm}^2$; used in all the corners

14 $\emptyset 20 = 43.96 \text{ cm}^2$; used between the corners

60/85 $\rightarrow A_s = 102 \text{ cm}^2 \rightarrow 6 * 4\emptyset 16 = 48 \text{ cm}^2$; used in all the corners

18 $\emptyset 20 = 56.52 \text{ cm}^2$; used between the corners.

For the connecting element:

Two layers of reinforcement will be used. Covered each by 2.5 cm of concrete.

Rebars will be $\emptyset 16$ and they will distance 15 cm between them. Drawings can be found in Figure 5.6.

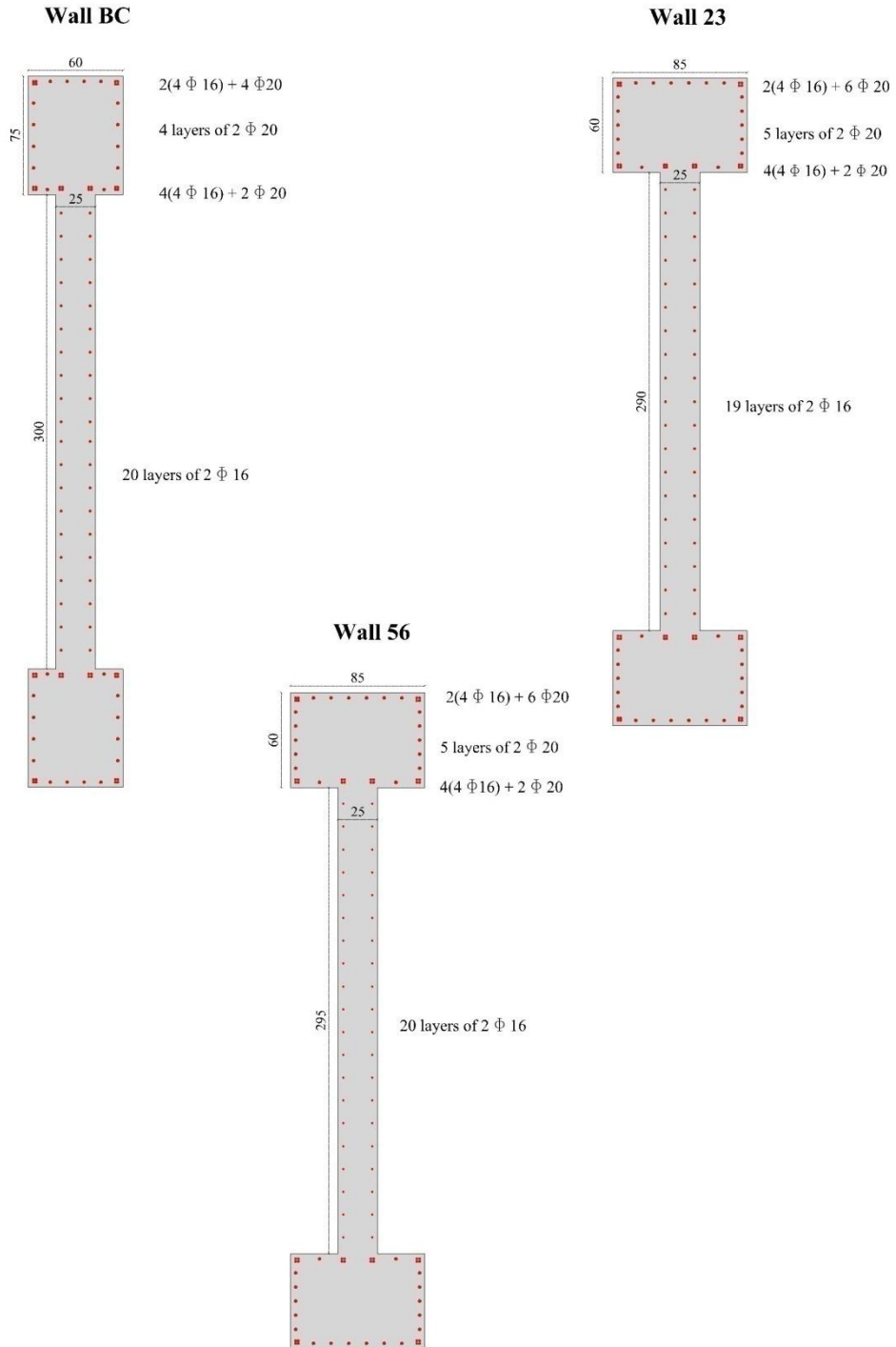


Figure 5. 6: Sections of shear walls.

5.2.1.2 Plastic hinge models

The hinge is defined manually as a M_3 hinge. Capacities curves of the shear walls are obtained from the section designer tool. The graphs representing those capacities curves can be found in App. C4. The hinge will be defined by a bilinear representation. That means that just three points are needed: the origin point (0;0),

the yield point ($M_f; \chi_f$) and the ultimate point ($M; \chi$). Due to the bilinear representation of the capacity by a bilinear curve where the second line is horizontal, M is equal to M_f . Moments are introduced to the program following the relationship M/M_f , which means that it takes 1 as a value. Curvature is introduced as the real curvatures the point supports.

Hinges are assigned to all the representative shear wall's frames, but probably it would be necessary just at the two lowest levels.

The acceptance criterion is taken from FEMA 346 Table 6-18.

The interface of the program is shown in Figure 5.7:

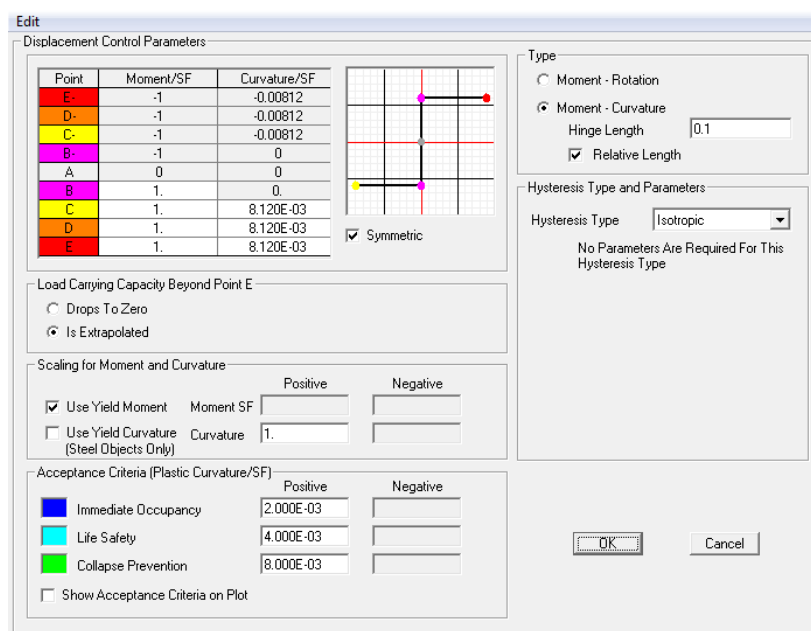


Figure 5. 7: Hinge SAP 2000's interface.

5.2.2 Models analysed

Two models with shear walls are analysed in this thesis.

5.2.2.1 4x2 shear wall case

This case is defined with four shear walls in “x” direction, between axis 2-3 and 5-6, and two in “y”, between axis B-C. All of them positioned at the external walls. With this configuration, the “x” direction is the strong one, and the first mode shape will affect the “y” direction. The reason is that there is more shear walls in “x” direction

than in “y” direction, and as it is known, a shear wall increases the rigidity and the strength of the direction at which it is aligned.

The 3D mathematical model and the disposition of shear walls are present by Figures 5.8 and 5.9.

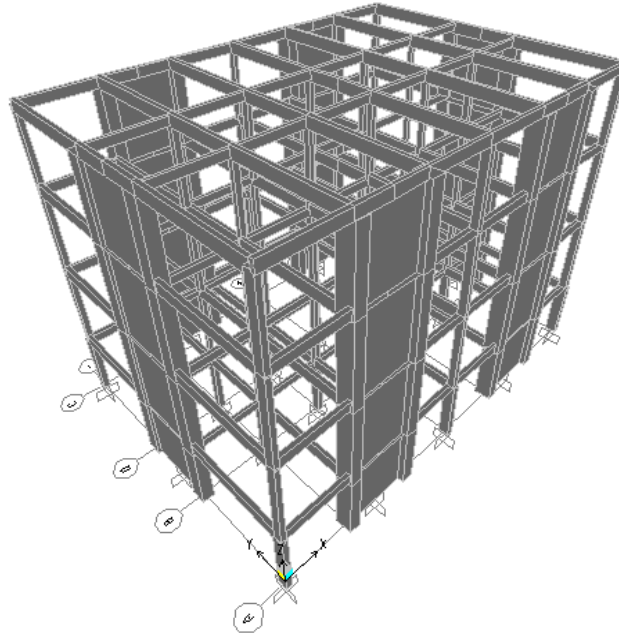


Figure 5. 8: 3D mathematical model of 4x2 shear wall case.

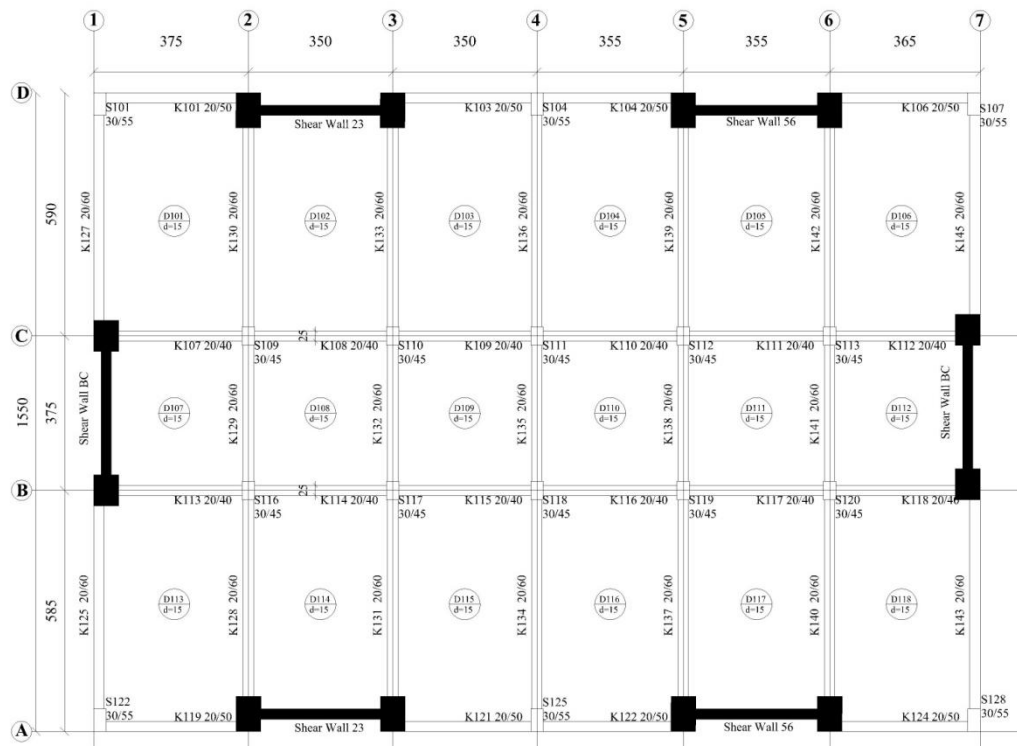


Figure 5. 9: Typical plan of 4x2 shear wall case.

5.2.2.2 4x4 shear wall case

This case is defined with four shear walls in “x” direction, between axis 2-3 and 5-6, and four in “y”, between axis B-C. It is the same model as 4x2 case but with the difference that the new shear walls are internal. They are localised, as it has already been said, between axes B-C and on axis 3 and axis 5.

The 3D mathematical model and the disposition of shear walls are presented by Figures 5.10 and 5.11.

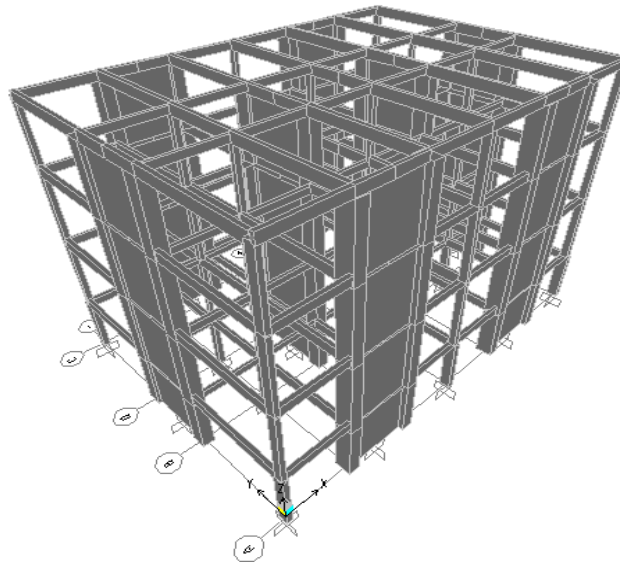


Figure 5. 10: 3D mathematical model of 4x4 shear wall case.

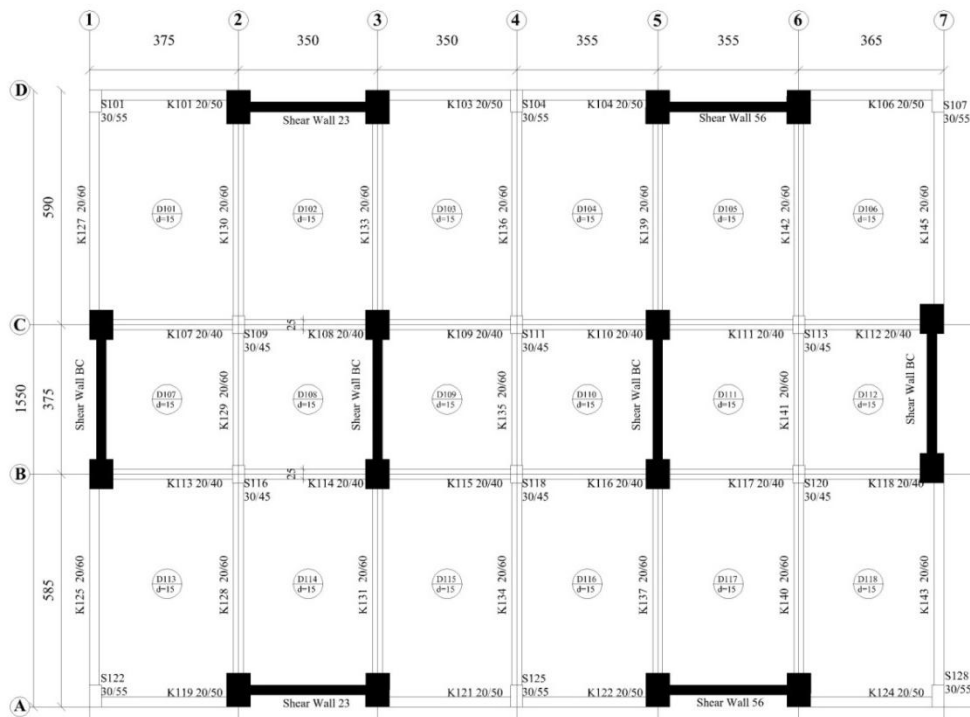


Figure 5. 11: Typical plan of 4x4 shear wall case.

6. CONSIDERING THE SOIL-STRUCTURE INTERACTION

The aim of this section is to outline the interaction between soil and structure. In general, structures are analysed being base fixed. This case is an idealized case. The soil is already considered in the definition of the demand spectrum by reducing it with the definition of kinematic soil-structure interaction and foundation damping according to Ch. 8 of FEMA440. With the modelization of the soil, it is also considered in the capacity of the structure.

6.1 Soil Condition

The different types of soil analyzed in this thesis come from TEC2007 under the name of Z1, Z2, Z3, and Z4.

The next table presents the mechanical properties of soils.

Table 6. 1: Mechanics properties of soil types.

	Z1	Z2	Z3	Z4
K_s (KN/m ³)	200 000	100 000	30 000	16 000
q_u (KN/m ²)	900	600	375	210
C_a	0.40	0.40	0.44	0.36
C_v	0.40	0.56	0.64	0.96
v_s (m/s)	1900	1140	562.5	272.5

Values for the shear wave velocity come from FEMA440 Table 5.3 as intermediate values.

6.2 Modelling the Tensionless Winkler Soil

The soil is modelled under a mat foundation as a tensionless elastic-plastic Winkler soil model. The mat foundation is represented by a thick shell element of 40 cm. With two rebar layers covered by 5 cm of concrete each. The considered materials are concrete C40 and steel S420.

The area element is meshed trying to obtain more or less the same area for each element keeping in mind that is important that the nodes of the columns have to be exactly placed on a element's node. It is a 24x16 rectangular mesh.

Horizontal movements through “x” and “y” directions are restricted. By this way the rotation over “z” axis is also restricted. The foundation will behave as an elastic member.

The elastic-plastic Winkler soil behaviour is presented by equivalent frame members subjected only to axial efforts. On those elements just compression and tension deformations are considered. Tension limits are taken as zero when compression ones are calculated from the ultimate soil bearing capacity q_u . The used material is C12 for the reason that the Winkler soil elements must exist already.

It is also necessary to define the cross section of those frames (Table 6.2), for this purpose, the following formulae is used:

$$EF = k_s * A_e * \text{Unit lenght}$$

$E \rightarrow$ Elastic modulus of C12

$F \rightarrow$ Cross section

$k_s \rightarrow$ soil modulus

$A_e \rightarrow$ Effective influence area of the frame members

Unit lenght is taken as 1m

$$C_{lim} = q_u * A_e$$

$C_{lim} \rightarrow$ compression limit

$q_u \rightarrow$ bearing capacity

Those frames have to be positioned at each corner of the mesh elements.

Table 6. 2: Characteristics of the equivalent frame members.

Cross section (m ²)	Z1	Z2	Z3	Z4
Edge points	0.00328604	0.00164302	0.00049291	0.00026288
Middle points	0.00657209	0.00328604	0.00098581	0.00052577
Corner point	0.00164302	0.00082151	0.00024645	0.00013144
C_{lim} (KN)	747.0	498.0	311.2	174.3

Calculation details can be found at App. D.

6.2.1 Special case of shear wall models

With the modelization of the mat foundation by thick shell members, the shear wall can lose the property of being rigid. To keep its rigid some frames have to be modelled on the shell element as the infinite rigid members (IR) representing the connecting beams of the shear walls.

A 3D mathematical model with the representation of mat foundation is presented in Figure 6.1.

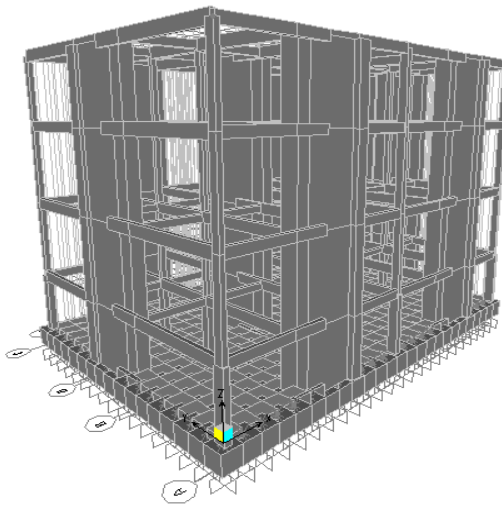


Figure 6. 1: 3D mathematical model with mat foundation of the 4x4 shear wall case.

7. DEFINITION OF THE DEMAND SPECTRUM

The definition of the spectrum demand is made by using seismic coefficients C_a and C_v . Those are determined by the type of soil. For fixed foundation case they are taken as the ones defined for Z1 soil type.

As it has already notified, the soil-structure interaction is included in the demand spectra by reducing it with the consideration of the kinematic interaction and foundation damping, according to Ch. 8 of FEMA 440. The demand spectra is reduced by multiplying it with a coefficient calculated by an empirical formula. The damping ratio of the complete structural system is taken as $\beta_0=\beta_i=0.05$.

According to FEMA 440, the effective damping and the effective period depends on various coefficients that are determined from non-linear behaviour models of single degree of freedom systems and ductility ratio μ . In this thesis those coefficients come from a model representing the 5% stiffness degradation. Those coefficients can be found on Tables 6-1 and 6-2 of FEMA 440, for effective damping and effective period respectively.

8. RESULTS AND COMPARISONS

In this section, the obtained results are presented, capacity curves of each model and for each kind of soil as well as their natural period. Over them comparisons are made and analysed. And finally the performances points are introduced. Those results represents the behaviour of the structures after pushing them with a non-linear analysis over “x” and “y” direction.

8.1 Capacity Curves of the Existing and Retrofitted Structures

Pushover curves for each kind of soil condition are represented together depending on case, from the existing actual building to the retrofitted cases.

8.1.1 Existing building

Over “x” direction:

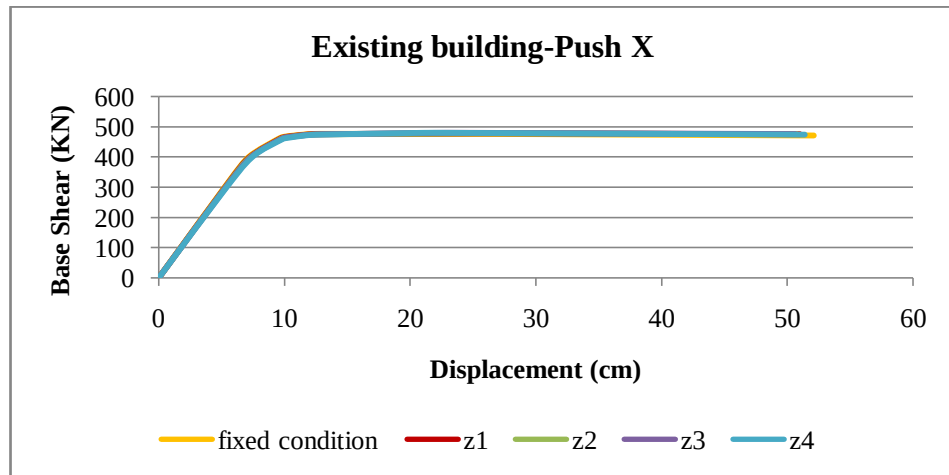


Figure 8 . 1: Capacity curves of the existing building over "x" direction.

Over “y” direction:

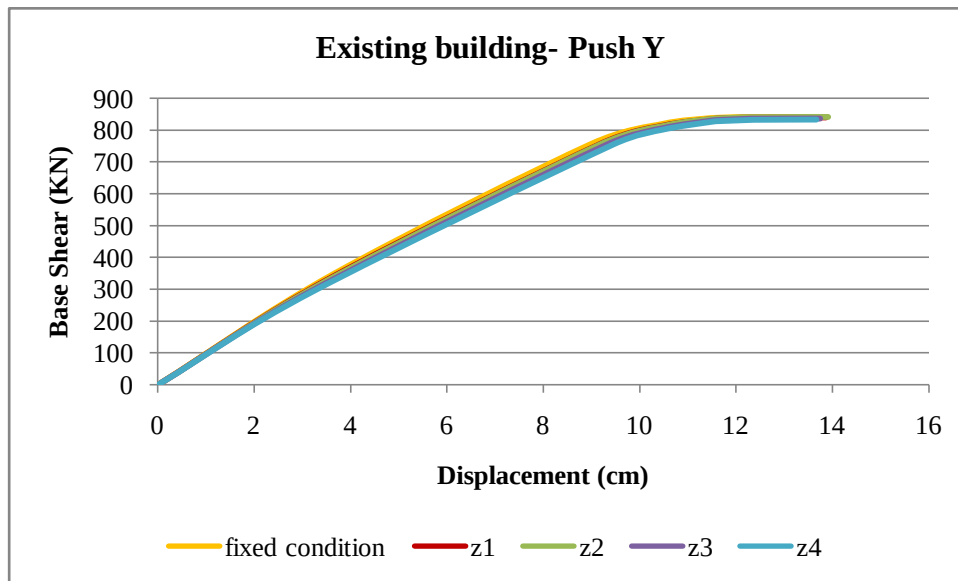


Figure 8. 2: Capacity curves for the existing building over "y" direction.

8.1.2 1st-2nd floor jacketed

Over “x” direction:

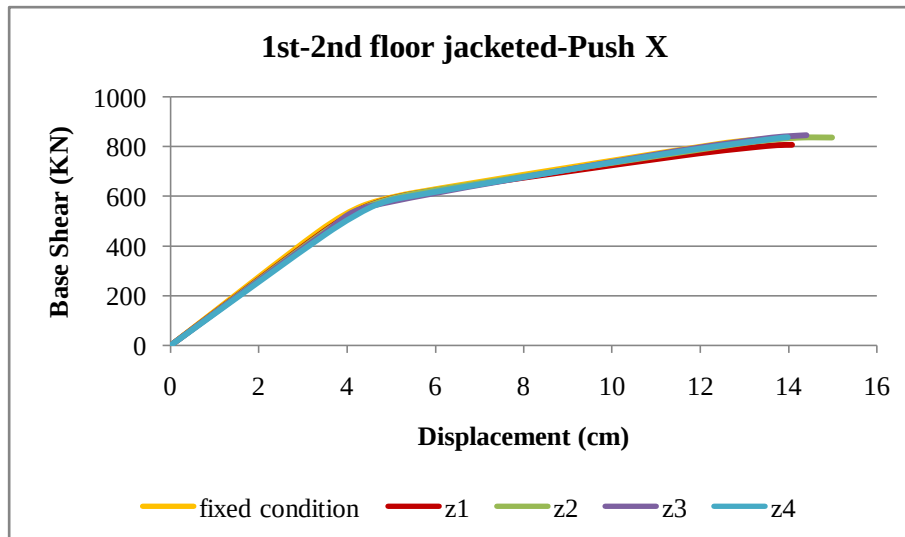


Figure 8. 3: Capacity curves for the jacketing of columns of 1st-2nd floor over "x" direction.

Over “y” direction:

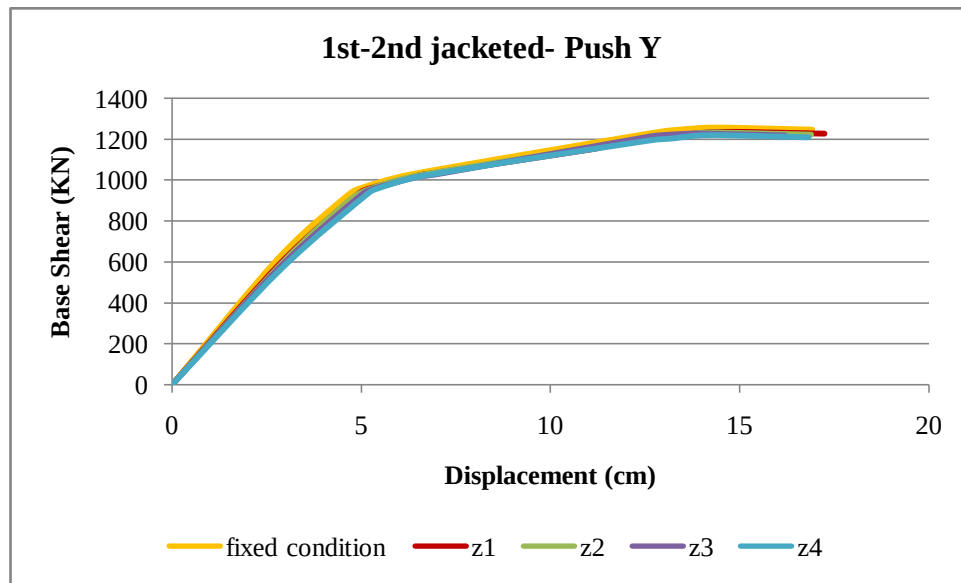


Figure 8. 4: Capacity curves for the jacketing of columns of 1st-2nd floor over "y" direction.

8.1.3 All columns jacketed

Over “x” direction:

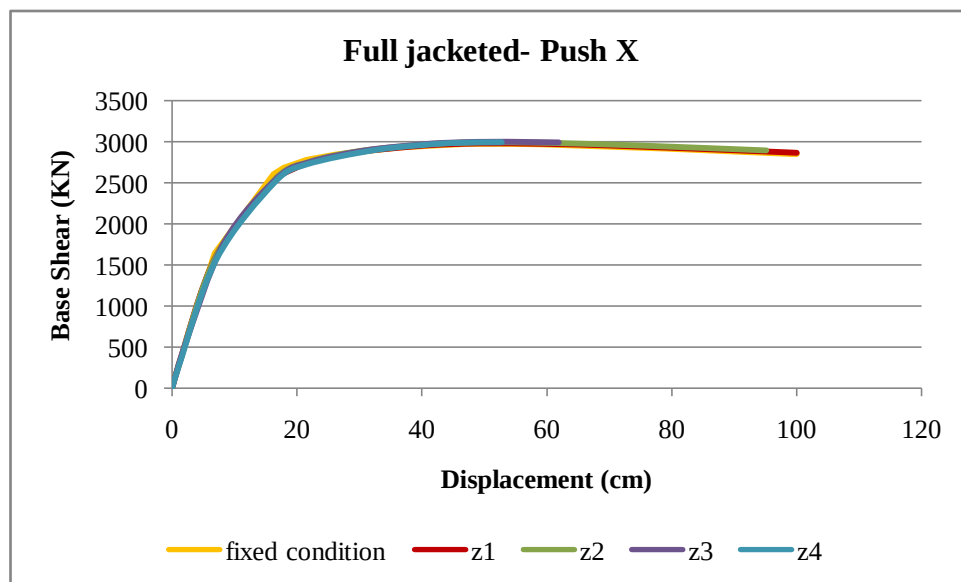


Figure 8. 5: Capacity curves of all columns jacketed case over "x" direction.

Over “y” direction:

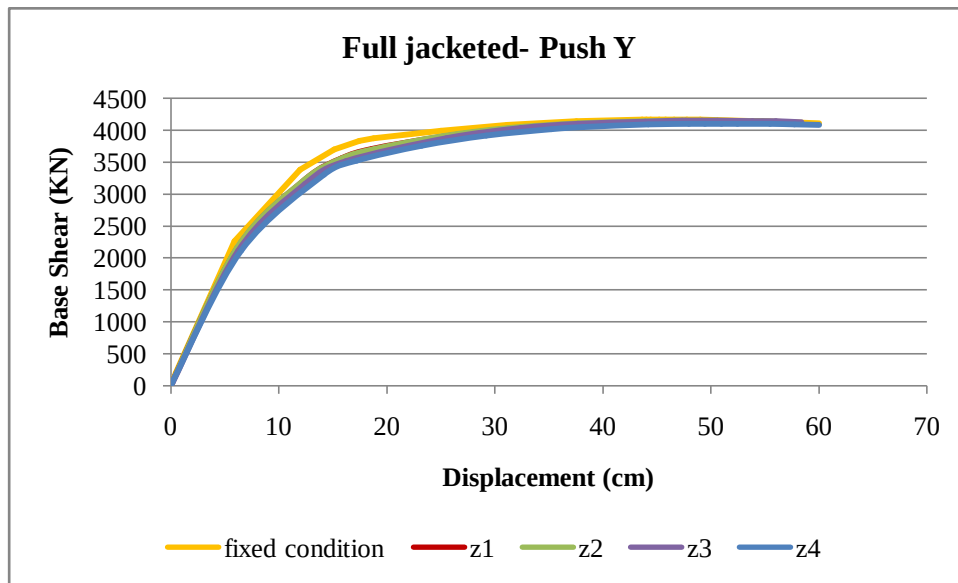


Figure 8. 6: Capacity curves of all columns jacketed case over "y" direction.

8.1.4 4x2 shear wall case

Over “x” direction:

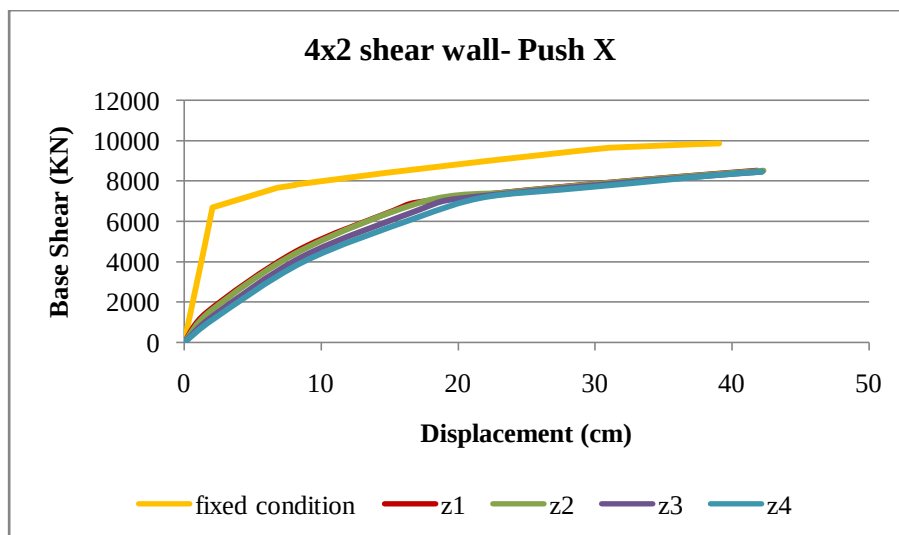


Figure 8. 7: Capacity curves of 4x2 Shear wall case over "x" direction.

Over “y” direction:

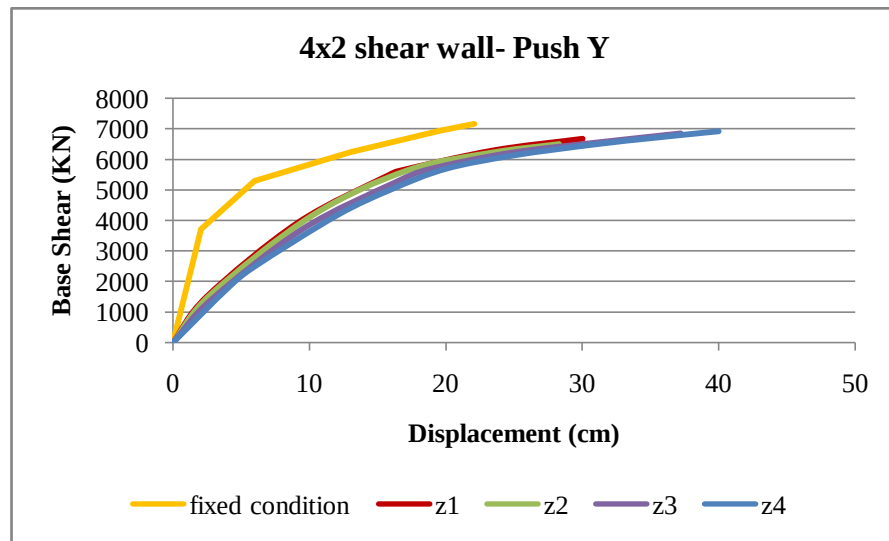


Figure 8. 8: Capacity curves of 4x2 Shear wall case over "y" direction.

8.1.5 4x4 shear wall case

Over “x” direction:

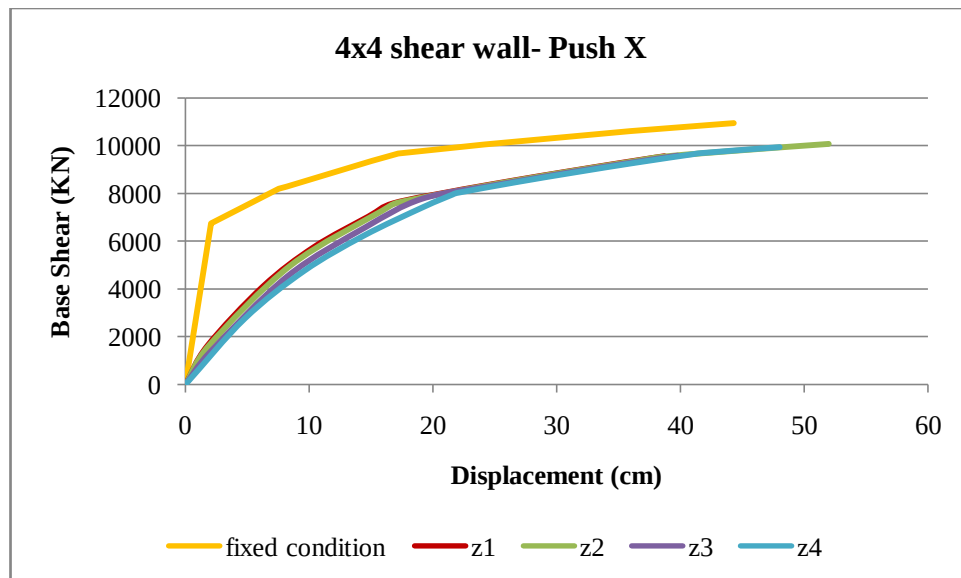


Figure 8. 9: Capacity curves of 4x4 Shear wall case over "x" direction.

Over “y” direction:

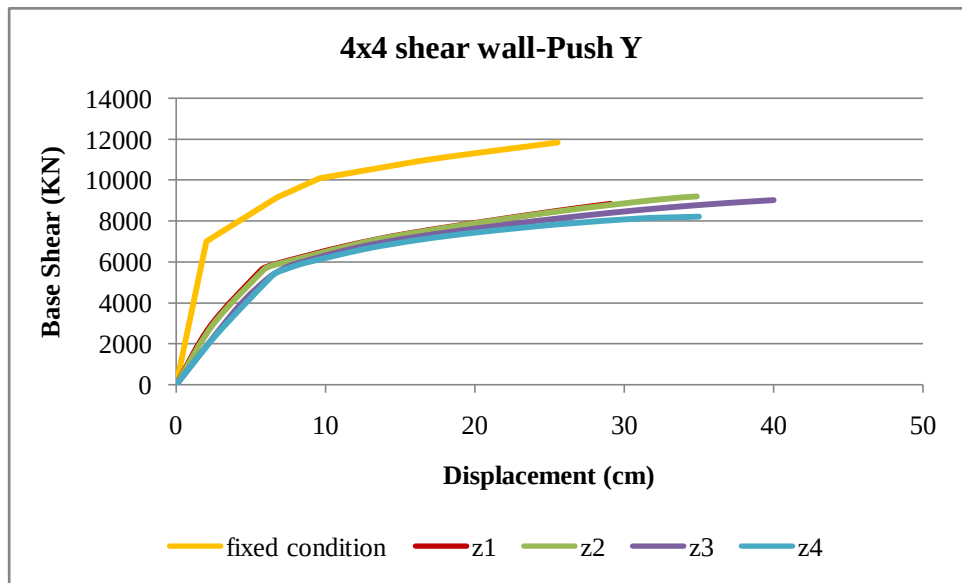


Figure 8. 10: Capacity curves of 4x4 Shear wall case over "y" direction.

8.1.6 Observations from capacity curves

In general, no considerable difference between the capacity curves is present. The soil modelization doesn't affect the capacity curve.

On the graphs representing the cases with shear walls, the soil modelization represents a change with the fixed condition case. With the representation of a shear wall under a fixed soil condition, uplifts are not represented. It is the reason why the shear wall behaves like a very rigid structure. Instead, with the soil modelization, there are uplifts around the shear wall.

Soil type is not an important factor of the response of the structure.

8.2 Capacity Curves depending on soil types

Due to the results obtained in the previous section, just two cases will be represented here. The first one will be the one representing the fixed condition soil. The other one will be the soil type Z2. The representation shows the pushover analysis in “x” and “y” direction. In this way, the difference that appears on the models that include shear walls is respected.

The model where the 1st floor is jacketed is just represented on the fixed condition case. During the analysis it has been observed that the jacketing technique just works

when all columns are jacketed. If the jacketing is done just on one or two floors, what happens is that the structural mechanism is transferred to upper levels where columns are not jacketed. This situation is presented in App. E.

8.2.1 Fixed foundation case

Over “x” direction:

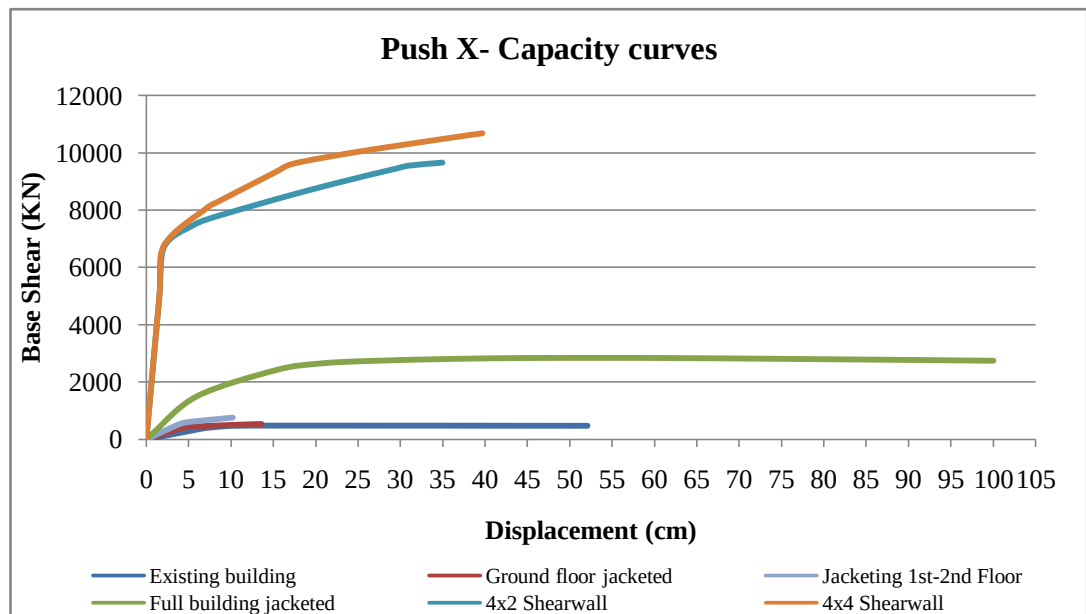


Figure 8. 11: Capacity curves of fixed foundation case over “x” direction.

Over “y” direction:

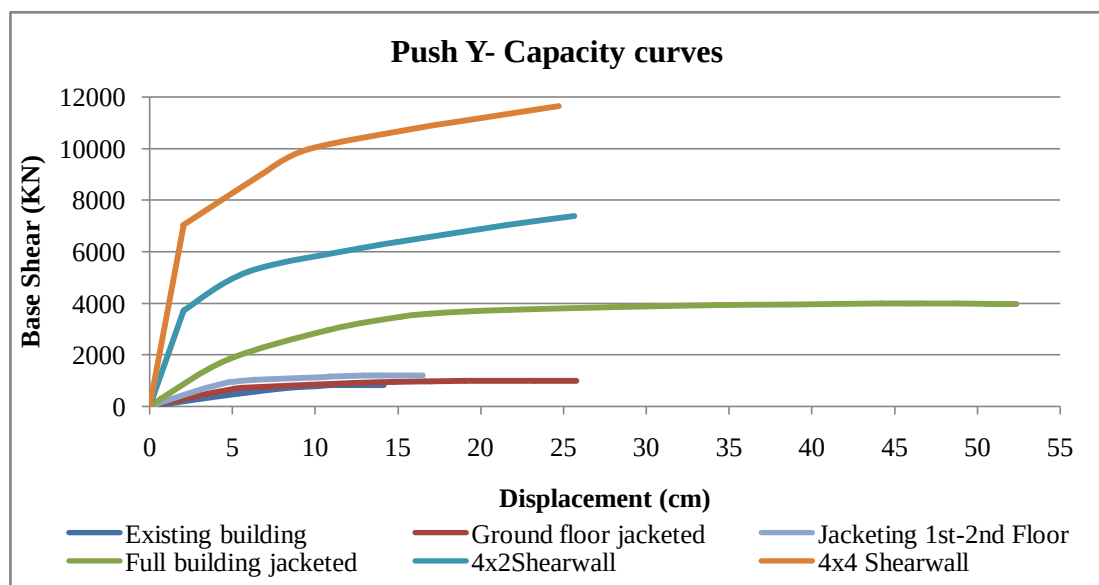


Figure 8. 12: Capacity curves of fixed foundation case over "y" direction.

8.2.2 Z2 type soil

Over “x” direction:

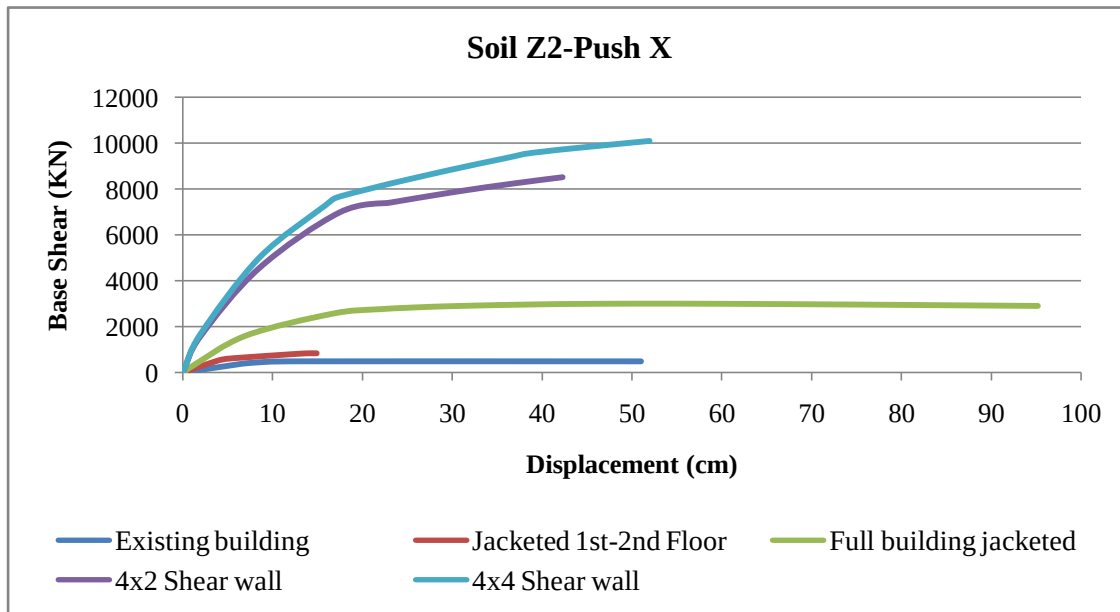


Figure 8. 13: Capacity curves of Z2 soil case over "x" direction.

Over “y” direction:

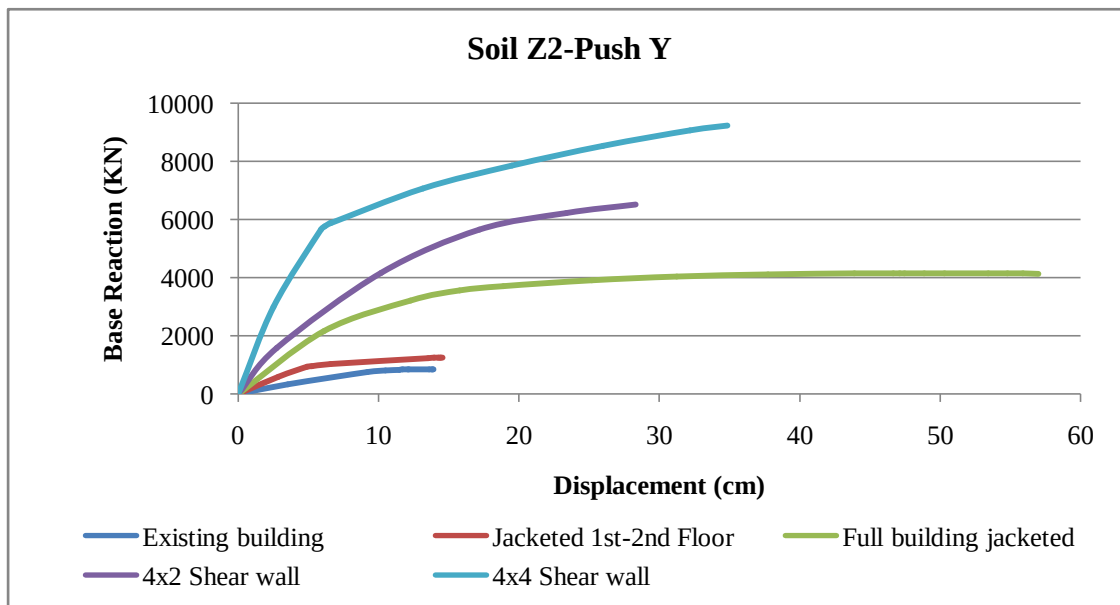


Figure 8. 14: Capacity curves of Z2 soil condition over "y" direction.

8.2.3 Observations by soil types

For the examining structure, the jacketing technique is useful when all the columns are jacketed, otherwise, as it has been already notified, what happen is that the unjacketed columns collapse. It can be seen at App. E.

For the fixed case:

Over “x” direction, shear walls cases have similar initial stiffness due to the same number of shear walls in this direction. Even though the stiffness is the same, the model with four shear walls in both directions represents an increment of the base reaction capacity (BRC).

Over “y” direction:

The jacketing retrofitting technique has the same observations as in direction “x”.

In general, BRC in “y” direction is higher in comparison with the “x” direction.

Shear wall cases present a significant difference due to the number of shear-walls. The model with four shear walls has a greater capacity than the model with two shear walls.

For the case representing the soil Z2:

All the BRC on shear wall models have decreased. Besides, the DC has generally increased.

4x4 Shear wall case has better general capacities than the 4x2 Shear wall case.

The models that have a good slope to make it able to reach a performance point are the ones with shear walls and the one with all the columns jacketed.

8.3 Natural Periods of The Structure Depending on Soil Types

In this section, the natural periods of each model under each kind of soil condition are presented (Tables 8.1 and 8.2). Natural periods are important for the determination of the performance point. As far as the period is short the performance point will be smaller.

Table 8. 1: Natural periods.

$T_0(s)$	fixed		Z_1		Z_2		Z_3		Z_4	
	x	y	x	y	x	y	x	Y	x	y
Actual building	2.14	1.65	2.15	1.66	2.15	1.66	2.16	1.68	2.17	1.69
1 st -2 nd jacketed	1.32	1.04	1.35	1.07	1.35	1.07	1.36	1.09	1.37	1.10
Full jacketed	1.00	0.80	1.02	0.83	1.04	0.85	1.06	0.87	1.07	0.89
4x2 SW	0.30	0.40	0.55	0.66	0.60	0.70	0.70	0.79	0.76	0.84
4x4 SW	0.30	0.30	0.56	0.46	0.61	0.50	0.70	0.56	0.75	0.61

The value for the Actual building period is different from the one calculated with the modal analysis done at the beginning of the study. It is due to the diminution of the stiffness of beams and columns, which is necessary to be able to run a non-linear analysis and to reach a coherent performance point.

Those natural periods confirm what is shown on the graphs, and what is known, that shear walls are more brittle than the jacketed columns. It is expected that the performance point's displacement will be higher for jacketed columns model than shear walls models.

8.4 Performance Point Calculation

As it has been explained in section 2.4, the obtaining of the performance point depends on the kind of soil and some other factors. The C_a and C_v seismic coefficients are directly related with the kind of soil.

For all those models that are fixed at base, we will assume that the soil consist to be the hardest type the turkish code has. It means it will be considered as a Z1 soil type.

The data that has to be introduced in SAP2000 are the C_a and C_v seismic coefficients as well as the interaction between the building and the soil. For this reason we have to define the Effective foundation size (b_e), the foundation embedment (e), the Shear wave velocity (v_s), the shear wave velocity reductor factor (n) and the modified damping ratio of the system (β_0). Those factors take the following values:

$$b_e = \sqrt{a * b} = \sqrt{2120 * 1495} = 1780.28 \text{ cm};$$

a, b are the dimensions of the foundation

$e = 0$; the default value

vs: from Table 5.3 from FEMA 440

$n = 0.65$; the default value

$\beta_0 = 0.05$; default value for the damping of 5%

While assuming that the stiffness degrading is around a 5%, from FEMA 440 is possible to obtain the parameters that will define the Effective Damping from Table 6.1 and the ones for the Effective Period from Table 6.2. Those parameters will be the same for all the other models, except the shear wave velocity that changes with the kind of soil.

With all those parameters defined is possible to determine the performance point of the structure. Those are presented in following table:

Table 8. 2: Performance points.

D(cm)	Fixed		Z ₁		Z ₂		Z ₃		Z ₄	
	x	y	x	y	x	Y	x	y	x	y
Actual building	31.21	N/A ¹	31.67	N/A	33.12	N/A	33.13	N/A	N/A	N/A
1 st	N/A	18.46	--	--	--	--	--	--	--	--
jacketed 1 st -2 nd	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
jacketed Full	20.26	15.80	29.59	23.93	33.51	31.99	35.72	29.09	40.87	34.21
jacketed	7.034	8.37	12.08	14.91	17.02	25.76	32.85	33.23	N/A	N/A
4x2 SW	7.74	7.08	11.14	14.04	16.66	15.81	32.33	22.35	43.78	28.51
4x4 SW										

As it is possible to observe, just cases where all the columns are jacketed and with four shear walls in both directions perform well in all the soil conditions. There is a huge difference between performance points due to strong soils (Z1 and Z2) and those due to soft ones (Z3 and Z4).

N/A¹: Not available.

Watching those results it seems that the most suitable cases to retrofit the building are: jacketing all the columns and 4 shear walls in both directions cases. To have a final decision it seems decisive to realize a damage study and also an economical comparison.

It is surprising to find out that the kind of soil is not a determinant parameter into the capacity of the structure but differences appear on the determination of the performance point. So soft is the soil so greater target displacement will be required. What is determinant, is the simulation of the foundation and the soil condition on models that have shear walls. The idealized method of simulating the structures as fixed on their base is questionable.

8.5 Uplift Comparison

The purpose of this section is to check out differences between soils comparing the uplift associated to shear walls during the soil modelization.

For this purpose, the uplift that comes from the shear wall located on axis “1” on “y” direction is considered. 4x4 Shear wall case will be the used case, pushing it on “y” direction till the performance point that corresponds to each kind of soil.

The next graph shows those uplifts:

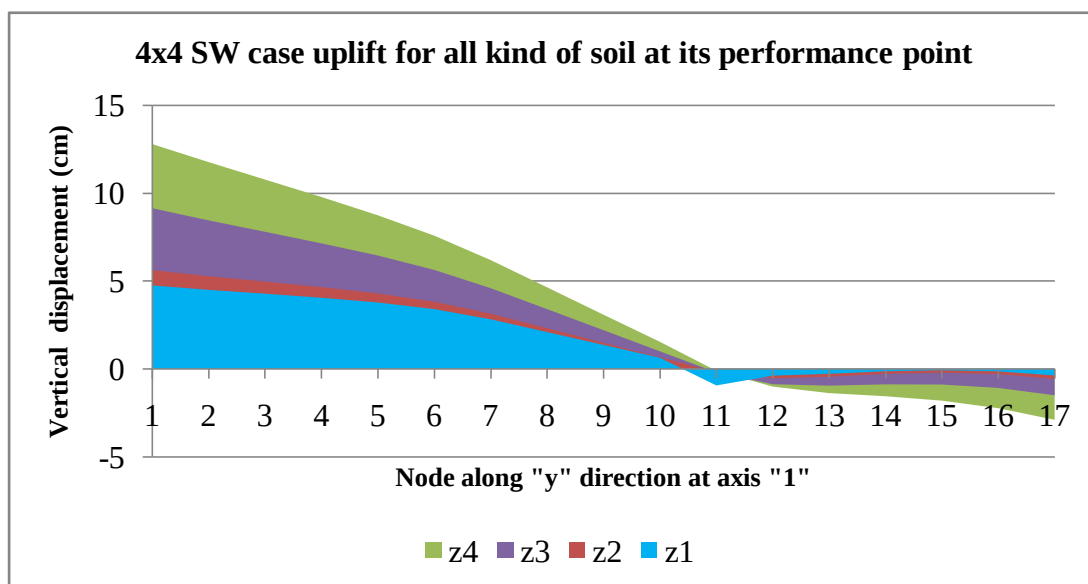


Figure 8. 15: Uplift comparison.

As it is possible to observe, softer is the soil, greater the uplift will be.

8.6 Hinge Configuration at the Performance Points

In this section, hinge configurations of the different models are presented. Those models are the full columns jacketed case and the two models containing shear walls. The others have been neglected due to their bad performance. Firstly, the performance criterion based on FEMA356 and taken from TEC2007 is presented:

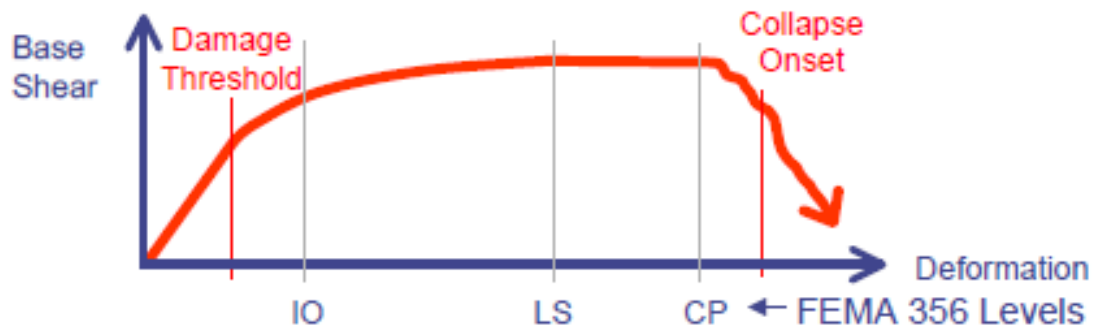


Figure 8. 16: Criteria for performance of the building [14].

At structure is classified as next:

IO: Immediate Occupancy performance level if at any storey, at most 10% of the beam's hinges are in IO level and the rest are in the lower level. Retrofitting is suitable.

LS: Life Safety performance level if at any storey, at most 30% of the beam's hinges are in LS as well as some columns. The rest are in lower levels. Retrofitting is suitable

CP: Collapse Prevention performance level if at any storey, at most 20% of the beam's hinges are in Collapse level and at most 30% of columns has IO or superior hinges in both extremities. Cost-effective analysis is recommended for rehabilitation

C: Collapse performance level corresponds to harder case in comparison to the ones above. Rehabilitation is not suitable.

Keeping in mind this classification, the studied cases hinge configurations are presented (Tables 8.3, 8.4, 8.5, 8.6, 8.7 and 8.8):

Firstly, the configuration for the actual building is presented on a picture. It corresponds to the performance point state over "x" direction under Z1 soil condition. Between the different soil types it is the one that requires less amount of displacement.

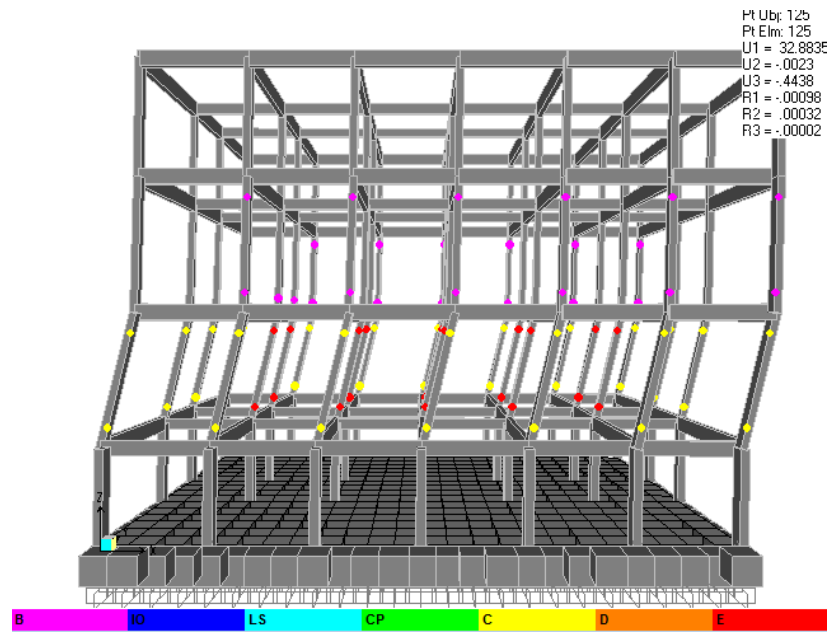


Figure 8. 17: Plastic Hinge configuration of the actual building under Z1 soil condition over "x" direction.

As it is possible to observe, the structure collapse. There is a structural mechanism at the second floor.

Table 8. 3: Hinge configuration of the full jacketed case over direction “x”.

% HINGES FULL JACKETED X		Worst storey	IO	LS	CP	C;D;E	TOTAL
Fixed case	Beams	2nd storey	50.0	50.0	0.0	0.0	100.0
	Columns		0.0	0.0	0.0	0.0	0.0
Z1	Beams	2nd storey	0.0	50.0	0.0	50.0	100.0
	Columns		0.0	0.0	0.0	0.0	0.0
Z2	Beams	2nd storey	0.0	50.0	0.0	50.0	100.0
	Columns		0.0	0.0	0.0	0.0	0.0
Z3	Beams	2nd storey	0.0	50.0	0.0	50.0	100.0
	Columns		0.0	0.0	0.0	0.0	0.0
Z4	Beams	2nd storey	0.0	33.4	0.0	66.6	100.0
	Columns		0.0	0.0	0.0	0.0	0.0

Table 8. 4: Hinge configuration of the full jacketed case over direction “y”.

% HINGES FULL JACKETED Y		Worst storey	IO	LS	CP	C;D;E	TOTAL
Fixed case	Beams	2nd storey	33.4	33.3	0.0	0.0	66.7
	Columns		0.0	0.0	0.0	0.0	0.0
Z1	Beams	2nd storey	33.4	54.8	0.0	11.8	100.0
	Columns		0.0	0.0	0.0	0.0	0.0
Z2	Beams	2nd storey	0.0	50.0	0.0	50.0	100.0
	Columns		0.0	0.0	0.0	0.0	0.0
Z3	Beams	2nd storey	16.7	33.3	0.0	50.0	100.0
	Columns		0.0	0.0	0.0	0.0	0.0
Z4	Beams	2nd storey	0.0	50.0	0.0	50.0	100.0
	Columns		0.0	0.0	0.0	0.0	0.0

There is no hinge on the columns. This is the ideal case. The fixed case has a weaker hinge configuration, when the ones where the soil is modelled present a big number of hinges representing the collapse of the building. Jacketing column technique is not enough and consequently, beams should be attended too. The structure would be in a Collapse Prevention performance level if the soil condition would be fixed. With any kind of soil it would be on Collapse performance level. Next picture shows how over “y” direction and under Z2 soil condition, half of the hinges of beams of the second storey are on collapse state.

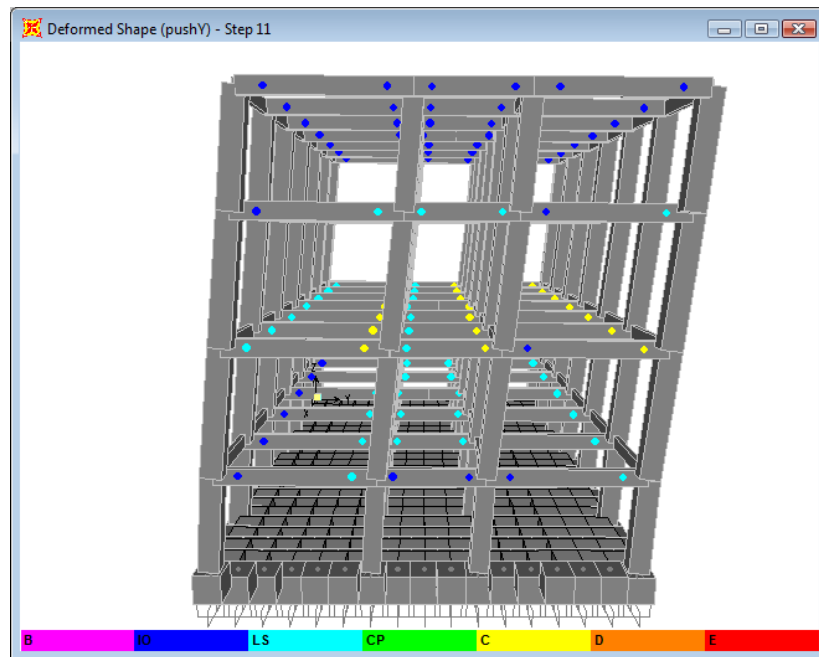
**Figure 8. 18:** Hinge configuration of all columns jacketed case over "y".

Table 8. 5: Hinge configuration of 4x4 Shear wall case over direction “x”.

% HINGES Shear wall 4x4 X		Worst storey	IO	LS	CP	C;D;E	TOTAL
Fixed case	Beams		0.0	0.0	0.0	0.0	0.0
	Columns	Ground floor	0.0	0.0	0.0	0.0	0.0
	SW columns		0.0	0.0	50.0	0.0	50.0
Z1	Beams		0.0	0.0	0.0	0.0	0.0
	Columns	Top floor	75.0	0.0	0.0	0.0	75.0
	SW columns		0.0	0.0	0.0	0.0	0.0
Z2	Beams		15.0	5.0	0.0	0.0	20.0
	Columns	Ground floor	66.7	8.3	0.0	0.0	75.0
	SW columns		0.0	0.0	0.0	0.0	0.0
Z3	Beams		10.0	25.0	0.0	10.0	45.0
	Columns	Ground floor	16.7	16.7	4.1	37.5	75.0
	SW columns		0.0	0.0	0.0	25.0	25.0
Z4	Beams		5.0	15.0	0.0	25.0	45.0
	Columns	Ground floor	25.0	0.0	8.3	50.0	83.3
	SW columns		0.0	0.0	0.0	25.0	25.0

Table 8. 6: Hinge configuration of 4x4 Shear wall case over direction “y”.

% HINGES Shear wall 4x4 Y		Worst storey	IO	LS	CP	C;D;E	TOTAL
Fixed case	Beams		0.0	0.0	0.0	0.0	0.0
	Columns	Ground floor	0.0	0.0	0.0	0.0	0.0
	SW columns		0.0	0.0	0.0	50.0	50.0
Z1	Beams		8.9	0.0	0.0	0.0	8.9
	Columns	Top floor	20.8	0.0	0.0	0.0	20.8
	SW columns		0.0	0.0	0.0	0.0	0.0
Z2	Beams		6.0	0.0	0.0	0.0	6.0
	Columns	Top floor	20.8	0.0	0.0	0.0	20.8
	SW columns		0.0	0.0	0.0	0.0	0.0
Z3	Beams		14.7	5.9	0.0	0.0	20.6
	Columns	Ground floor	25.0	0.0	0.0	0.0	25.0
	SW columns		25.0	0.0	0.0	0.0	25.0
Z4	Beams		3	17.6	0.0	0.0	20.6
	Columns	Ground floor	25.0	0.0	0.0	0.0	25.0
	SW columns		0.0	25.0	0.0	0.0	25.0

Shear wall hinges appear just at the softer soils. When there are shear wall hinges, they represent collapse. Under Z1 and Z2 soil condition, the structure could be classified at Life Safety performance level. For softer soils and fixed case, it would be at Collapse performance level.

Table 8. 7: Hinge configuration of 4x2 Shear wall case over direction “x”.

% HINGES Shear wall 4x2 X		Worst storey	IO	LS	CP	C;D;E	TOTAL
Fixed case	Beams		0.0	0.0	0.0	0.0	0.0
	Columns	Ground floor	0.0	0.0	0.0	0.0	0.0
	SW columns		0.0	0.0	50.0	0.0	50.0
Z1	Beams		5.0	0.0	0.0	0.0	5.0
	Columns	Ground floor	56.3	0.0	0.0	0.0	56.3
	SW columns		0.0	0.0	0.0	0.0	0.0
Z2	Beams		15.0	5.0	0.0	0.0	20.0
	Columns	Ground floor	75.0	6.2	0.0	0.0	81.2
	SW columns		0.0	0.0	0.0	0.0	0.0
Z3	Beams		10.0	20.0	0.0	5.0	35.0
	Columns	Ground floor	37.5	12.5	0.0	43.8	93.8
	SW columns		0.0	0.0	0.0	25.0	25.0

Table 8. 8: Hinge configuration of 4x2 Shear wall case over direction “y”.

% HINGES Shear wall 4x2 Y		Worst storey	IO	LS	CP	C;D;E	TOTAL
Fixed case	Beams		0.0	0.0	0.0	0.0	0.0
	Columns	Ground floor	0.0	0.0	0.0	0.0	0.0
	SW columns		0.0	0.0	0.0	50.0	50.0
Z1	Beams		15.8	5.3	0.0	0.0	21.1
	Columns	Ground floor	68.7	0.0	0.0	0.0	68.7
	SW columns		0.0	0.0	0.0	0.0	0.0
Z2	Beams		13.1	10.5	0.0	5.0	28.6
	Columns	2nd floor	62.5	12.5	0.0	0.0	81.2
	SW columns		0.0	50.0	0.0	0.0	50.0
Z3	Beams		10.5	23.6	0.0	0.0	24.1
	Columns	Ground floor	71.8	0.0	0.0	9.4	81.2
	SW columns		0.0	0.0	0.0	50.0	50.0

The same observations done for the 4x4 Shear wall case can be done here. Except under Z1 soil condition, where the structure can be classified at Life Safety performance level, all the others are on Collapse performance level.

Comparing the classifications, 4x4 Shear Wall case is better than 4x2 Shear wall case.

Next a briefing table is presented to show in a more clearly way how the cases are classified:

Table 8. 9: Performance classification of different cases.

	Fixed	Z1	Z2	Z3	Z4
4x2 SW	C	LS	C	C	-
4x4 SW	C	LS	LS	C	C
FJ	CP**	C**	C**	C**	C**

Full jacketed cases are classified considering the collapse of beams as a meaningful event. Maybe, if the collapse of the beams is not considered as a meaningful event, this technique would be a suitable solution.

9. CONCLUSION

In this thesis, an existing reinforced concrete building in Istanbul has been modelled and analysed with an informatic structural package. Retrofitting techniques as the jacketing of columns and the addition of shear walls have been introduced to the model to improve its performance against earthquakes. In a parallel way, the soil has been modelled too to find out if its simulation affects the response by adding the soil effect to the capacity curve. The study has been purchased using codes of FEMA 356, FEMA 440 as well as TEC 2007.

The hypotheses of this study were the followings:

- The actual building should have a very bad performance against earthquakes.
- The retrofitting techniques should contribute to a significant improvement to this performance.
- The soil modelization should suppose something to take in account during simulations.

Those hypotheses have been confirmed. The third one presents some details that should be taken in account.

Main conclusions of this thesis are the followings:

- 1) The actual building doesn't perform well against earthquakes.
- 2) Jacketing technique has good properties if all the columns are jacketed. Even though, the performance level of the building would be under collapse, which is due to the collapse of beams. It is why the beams should be treated too and perhaps it wouldn't be economically suitable. If the effect of the collapse of the beams on the general response is neglected, maybe it would be a suitable way to retrofit the building.
- 3) In general 4x4 shear wall case performs better than 4x2 shear wall case.

- 4) Depending on the kind of soil, shear walls are suitable:
- Normally for strong soils.
 - For fixed condition and soft soils it is not suitable.
- 5) The representation of the soil produces big differences at the capacity curve on those models that possesses shear walls. The difference is focused between the model without soil modelization and the one with.
- 6) Soil modelization should be taken in account when there are shear walls. It could be a mistake if it is not considered.

From this thesis, it seems that the soil is an important factor when a Performance-Based Design is being run and should be modelled too. It is a result that breaks the habits of engineers. It is why more studies, similar to this one, should be done, to check out if the same results are obtained or just to improve what has been done here.

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APPENDIX

APPENDIX A : Building plans and SAP 2000 representation

APPENDIX B : Loads

APPENDIX C : Section properties

APPENDIX D : Foundation modelization

APPENDIX E : Analysis pictures

APPENDIX A: BUILDING PLANS AND SAP 2000 MODELIZATION

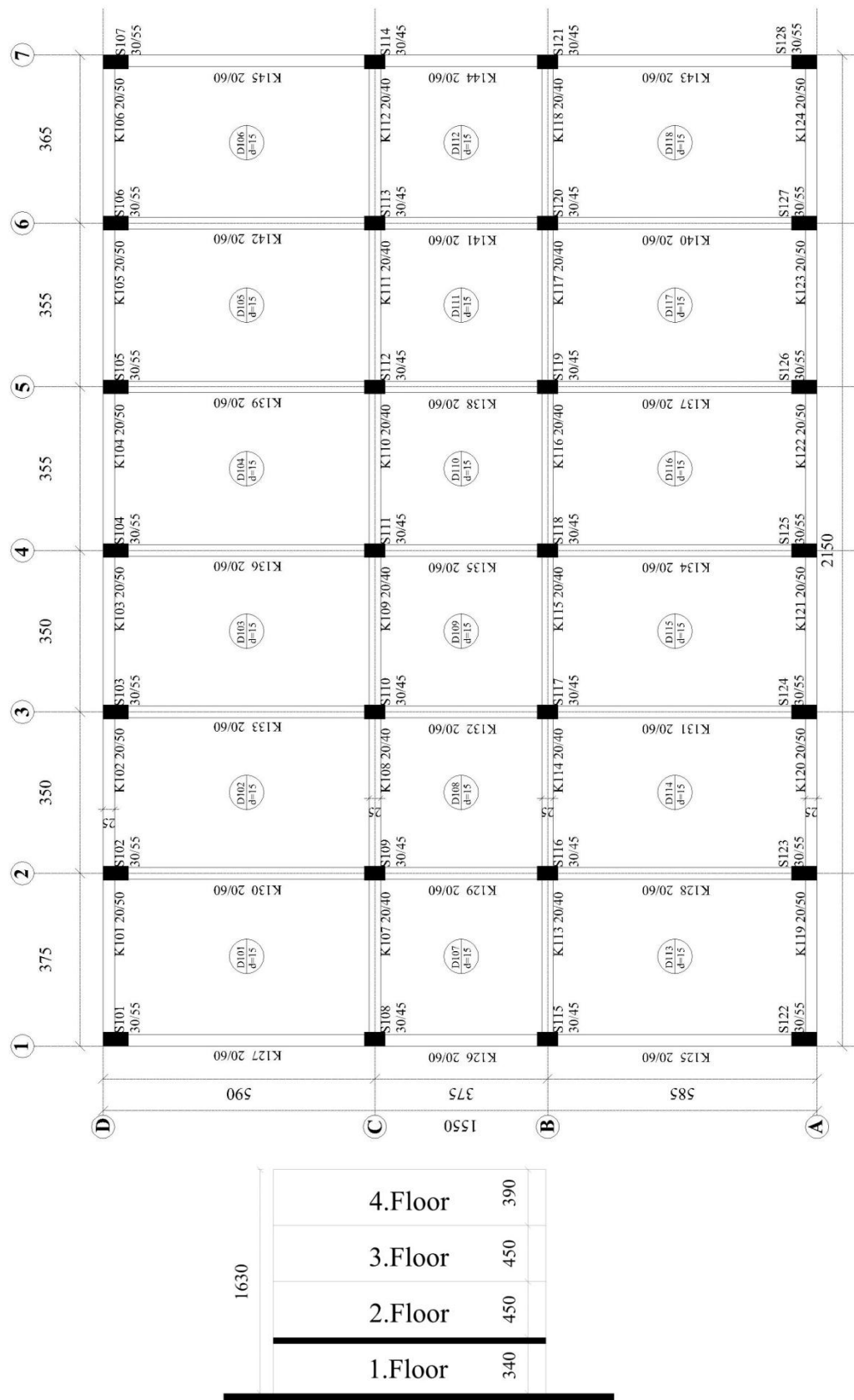


Figure A. 1: Typical plan of the first storey. (cm)

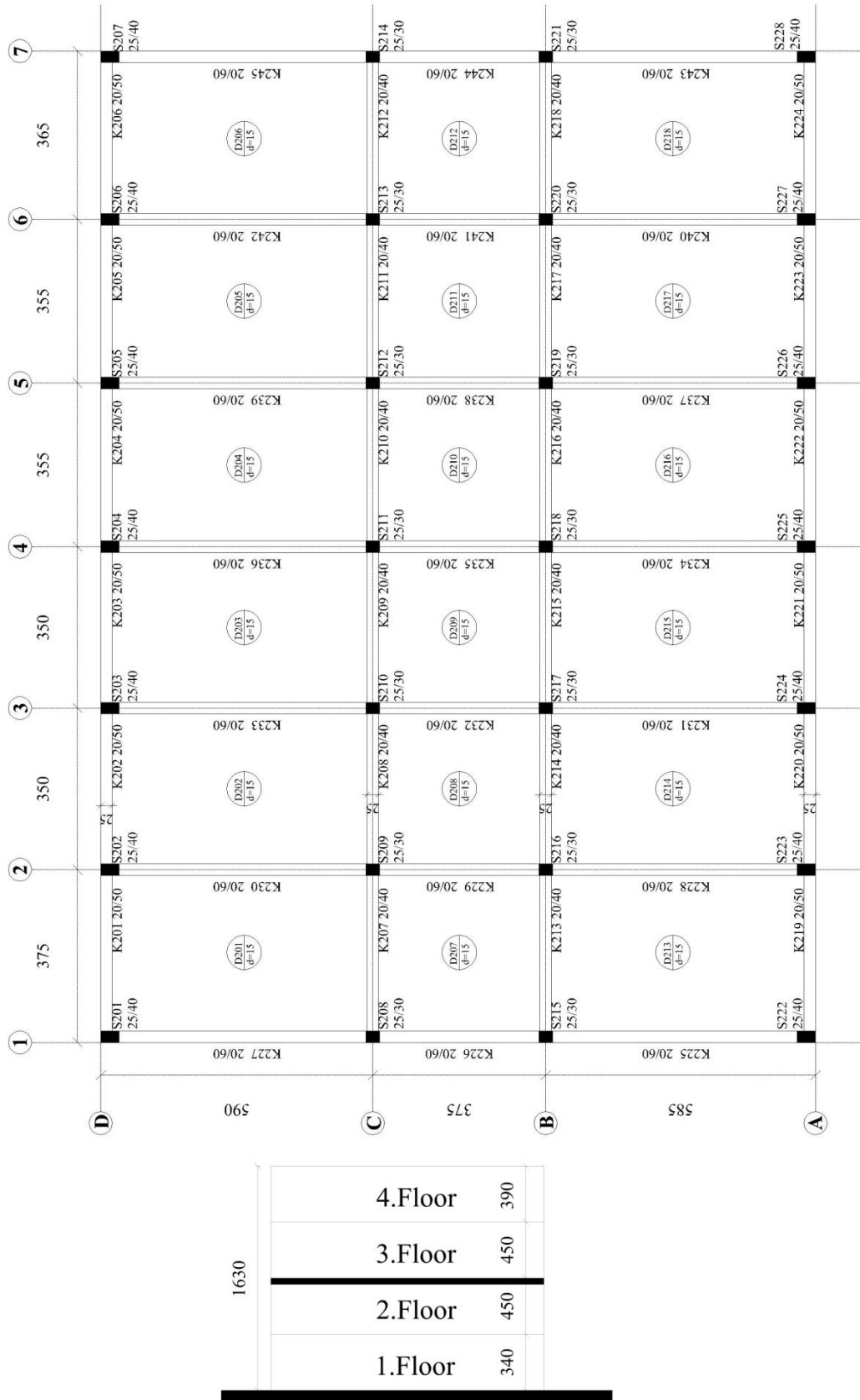


Figure A. 2: Typical plan of upper stories. (cm)

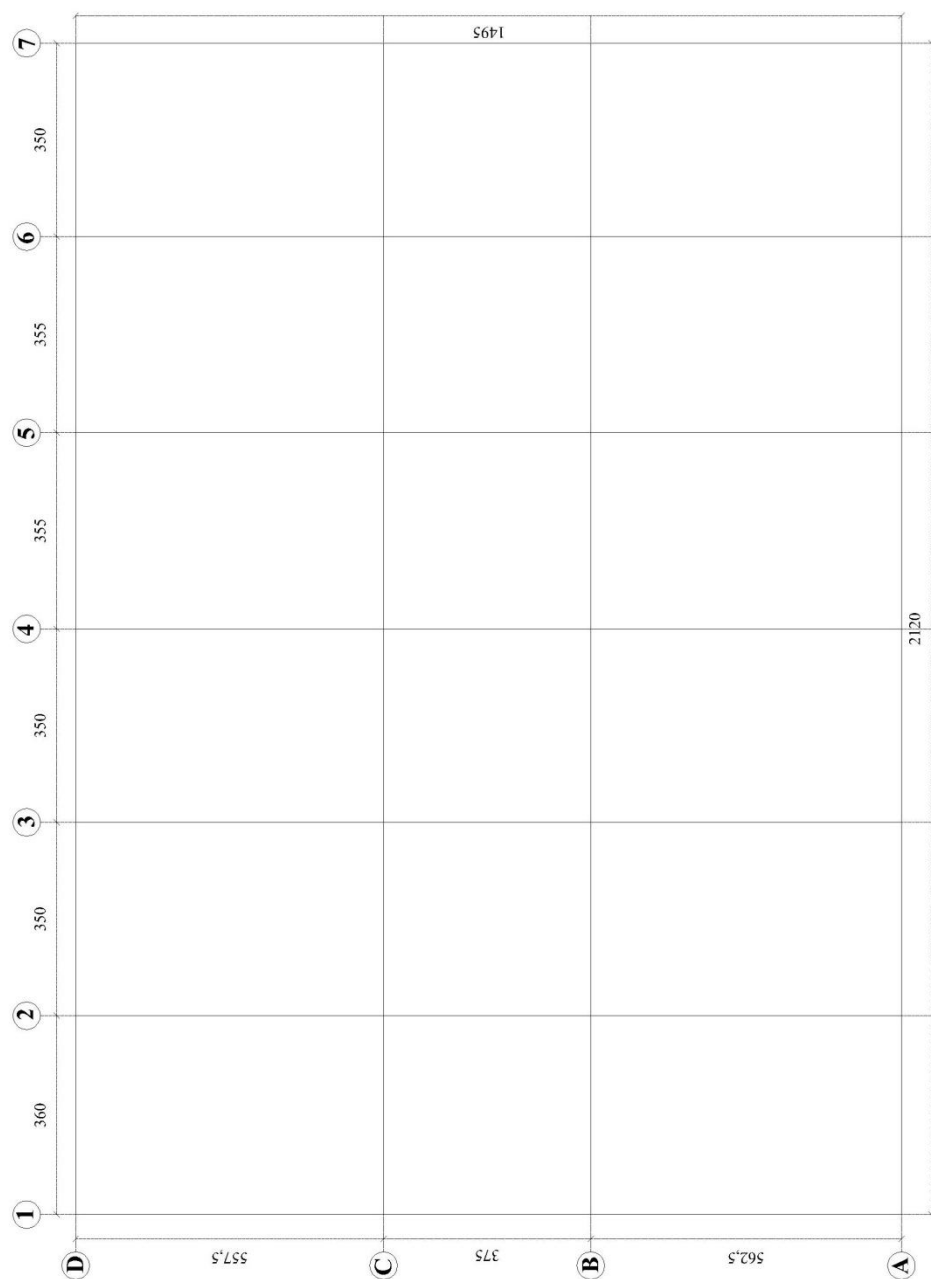


Figure A. 3: Plan of the skeleton of the mathematical model. (cm)

APPENDIX B: LOADS**APPENDIX B.1:** Results of distributed slab's load.**Table B.1:** Slab's load tables.

Slab between axis								
A-B $l_y=5.48\text{m}$	1-2 $l_x=3.38\text{m}$		2-3/3-4 $l_x=3.25\text{m}$		4-5/5-6 $l_x=3.30\text{m}$		6-7 $l_x=3.28\text{m}$	
Load KN/m	P_{ex}	P_{ey}	P_{ex}	P_{ey}	P_{ex}	P_{ey}	P_{ex}	P_{ey}
Dead	5.80	7.60	5.58	7.39	5.66	7.46	5.63	7.44
Live	2.20	2.89	2.13	2.81	2.16	2.84	2.16	2.84

Slab between axis								
C-D $l_y=5.53\text{m}$	1-2 $l_x=3.38\text{m}$		2-3/3-4 $l_x=3.25\text{m}$		4-5/5-6 $l_x=3.30\text{m}$		6-7 $l_x=3.28\text{m}$	
Load KN/m	P_{ex}	P_{ey}	P_{ex}	P_{ey}	P_{ex}	P_{ey}	P_{ex}	P_{ey}
Dead	5.80	7.62	5.58	7.40	5.66	7.48	5.63	7.46
Live	2.20	2.90	2.12	2.82	2.16	2.84	2.16	2.84

Slab between axis								
B-C $l_y=3.5\text{m}$	1-2 $l_x=3.38\text{m}$		2-3/3-4 $l_x=3.25\text{m}$		4-5/5-6 $l_x=3.30\text{m}$		6-7 $l_x=3.28\text{m}$	
Load KN/m	P_{ex}	P_{ey}	P_{ex}	P_{ey}	P_{ex}	P_{ey}	P_{ex}	P_{ey}
Dead	5.80	5.99	5.58	5.96	5.66	5.98	5.63	5.97
Live	2.20	2.29	2.13	2.28	2.16	2.28	2.15	2.28

Slab between axis						
A-B/C-D $l_y=5.48/5.53\text{m}$	1-2 $l_x=3.38\text{m}$		2-3/3-4 $l_x=3.25\text{m}$		4-5/5-6/6-7 $l_x=3.30\text{m}$	
Load KN/m	P_{ex}	P_{ey}	P_{ex}	P_{ey}	P_{ex}	P_{ey}
Dead	5.80	7.60	5.58	7.39	5.65	7.46
Live	2.20	2.89	2.13	2.81	2.16	2.84

Slab between axis						
B-C $l_y=3.5\text{m}$	1-2 $l_x=3.38\text{m}$		2-3/3-4 $l_x=3.25\text{m}$		4-5/5-6/6-7 $l_x=3.30\text{m}$	
Load KN/m	P_{ex}	P_{ey}	P_{ex}	P_{ey}	P_{ex}	P_{ey}
Dead	5.80	5.99	5.58	5.96	5.65	5.98

APPENDIX B.2: Calculation of the equivalent earthquake's load.

$$F_i = \frac{W_i * h_i}{\sum W_i * h_i} * F_{BASE}$$

$$F_{BASE} = A(t) * \sum W_i$$

From the new turkish code against earthquakes, it is assumed that $A(t)=0.08$.

$$W_i = g_i * A_i + n * q * A_i$$

Where: $W_i \rightarrow$ Weight of the floor

$g_i \rightarrow$ Total dead load of the floor (Tn/m^2)

$q \rightarrow$ Live load (Tn/m^2)

$n \rightarrow$ Reduction coefficient

$A_i \rightarrow$ Floor's area (m^2)

$n=0.4$

$A_i = 15.5m \cdot 21.5m = 333.25 m^2$

$$g_i = g + \frac{V_{Tb} * \delta_c}{A_i}$$

$V_{Tb} \rightarrow$ Floor's total beam's volume

Beam's section surface excluding the upper part of the beam:

T20/60 $0.2m * 0.45m = 0.09m^2$

T20/50 $0.2m * 0.35m = 0.07m^2$

T20/40 $0.2m * 0.25m = 0.05m^2$

Total longitude of beams per storey:

T20/60 $15.5m * 7 bays = 108.5m$

$$T20/50 \quad 21.5\text{m} \cdot 2\text{bays} = 43\text{m}$$

$$T20/40 \quad 21.5\text{m} \cdot 2\text{bays} = 43\text{m}$$

Storey's total volume:

$$T20/60 \quad 0.09\text{m}^2 \cdot 108.5\text{m} = 9.765 \text{ m}^3$$

$$T20/50 \quad 0.07\text{m}^2 \cdot 43\text{m} = 3.01 \text{ m}^3$$

$$T20/40 \quad 0.05\text{m}^2 \cdot 43\text{m} = 2.15 \text{ m}^3$$

$$V_T \quad \overline{14.925\text{m}^3}$$

$$g = 0.525 \text{ Tn/ m}^2$$

$$\delta_c = 2.5 \text{ Tn/ m}^3$$

Results are presented:

$$W_1 = W_2 = W_3 = W_4 = 237.607 \text{ Tn}$$

$$F_{\text{BASE}} = 0.08 \cdot 311.866 \cdot 4 \approx 76.034 \text{ Tn}$$

Table B.2: Floor's Static earthquake loads (Tn)

Floor	h_i (m)	F_i (Tn)
1	3.4m	6.463
2	7.9m	15.01
3	12.4m	23.665
4	16.3m	30.983

APPENDIX C: SECTION PROPERTIES

APPENDIX C.1: Frame's capacity curve and data used in M-KAPA program

End span's beam for M+:

T20/60

1

0.60 0.20

2000 0.00005 1 2 0.0

6800.0 6800.0 0.002 0.0035 0.00

191304.34 191304.34 239130.43 0.0011 0.011 0.16

0.025 0.001793 1

0.575 0.001256 1

25258330.0 200000000.0 680.0 0.10

2

1.00 1.00

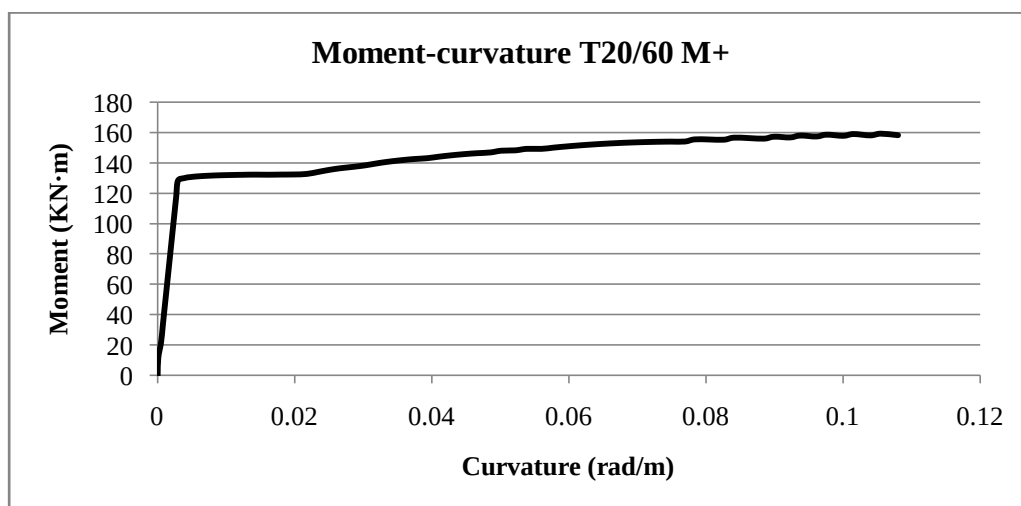


Figure C. 1: Equivalent section capacity of T20/60 at the end of span for positives moments.

T20/50

1

0.50 0.20
 2000 0.0001 1 2 0.0
 6800.0 6800.0 0.002 0.0035 0.00
 191304.34 191304.34 239130.43 0.0011 0.011 0.16
 0.025 0.001482 1
 0.475 0.001256 1
 25258330.0 200000000.0 680.0 0.10

2

1.0 1.00

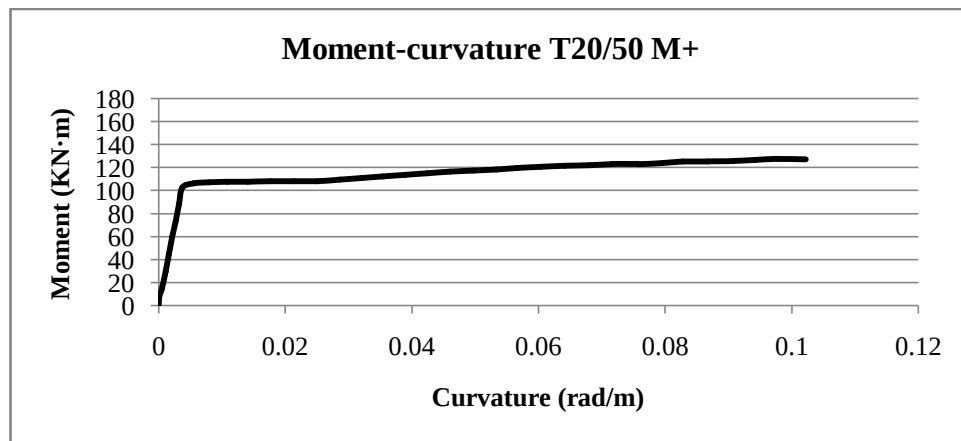


Figure C. 2: Equivalent section capacity of T20/50 at the end of span for positives moments.

T20/40

1

0.40 0.20

2500 0.00005 1 2 0.0

6800.0 6800.0 0.002 0.0035 0.00

191304.34 191304.34 239130.43 0.0011 0.011 0.16

0.025 0.001482 1

0.375 0.001256 1

25258330.0 200000000.0 680.0 0.10

2

1.00 1.00

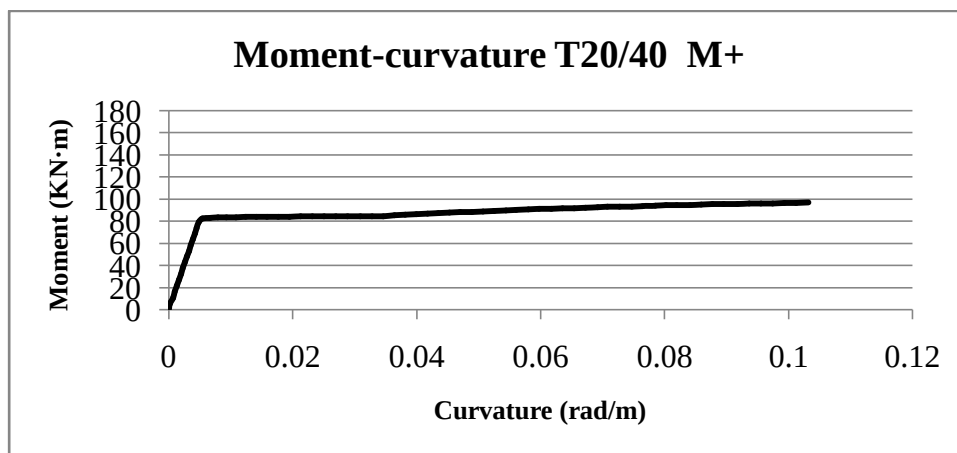


Figure C. 3: Equivalent section capacity of T20/40 at the end of span for positives moments.

End span's beam for M-:

T20/60

1

0.60 0.20

2000 0.00005 1 2 0.0

6800.0 6800.0 0.002 0.0035 0.00

191304.34 191304.34 239130.43 0.0011 0.011 0.16

0.025 0.001256 1

0.575 0.001793 1

25258330.0 200000000.0 680.0 0.10

2

1.00 1.00

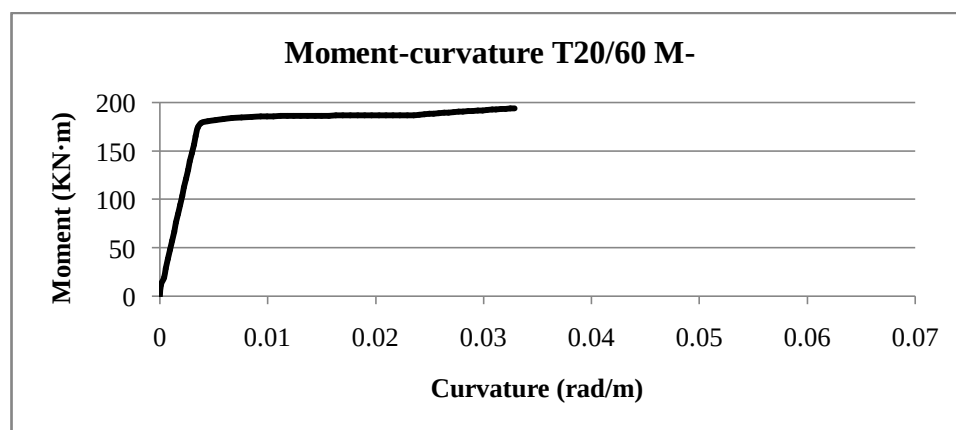


Figure C. 4: Equivalent section capacity of T20/60 at the end of span for negative moments.

T20/50

1

0.50 0.20

2000 0.0001 1 2 0.0

6800.0 6800.0 0.002 0.0035 0.00

191304.34 191304.34 239130.43 0.0011 0.011 0.16

0.025 0.001256 1

0.475 0.001482 1

25258330.0 200000000.0 680.0 0.10

2

1.00 1.00

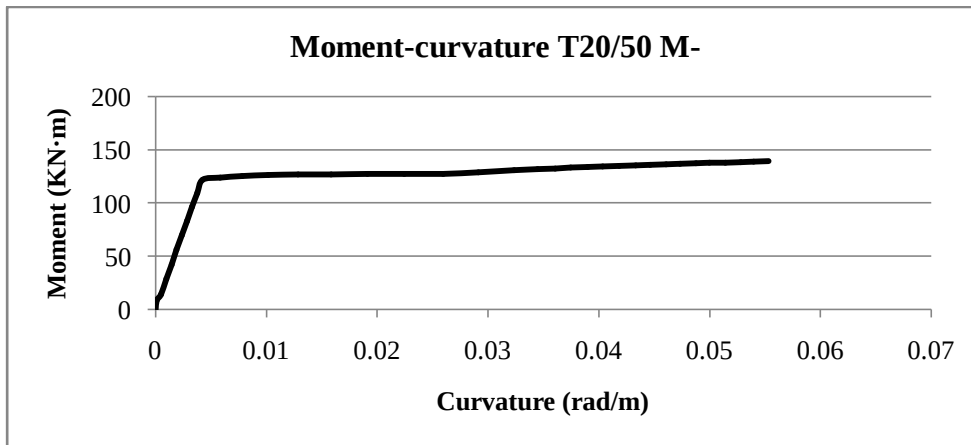


Figure C. 5: Equivalent section capacity of T20/50 at the end of span for negative moments.

T20/40

1

0.40 0.20

2500 0.00005 1 2 0.0

6800.0 6800.0 0.002 0.0035 0.00

191304.34 191304.34 239130.43 0.0011 0.011 0.16

0.025 0.001256 1

0.375 0.001482 1

25258330.0 200000000.0 680.0 0.10

2

1.0 1.00

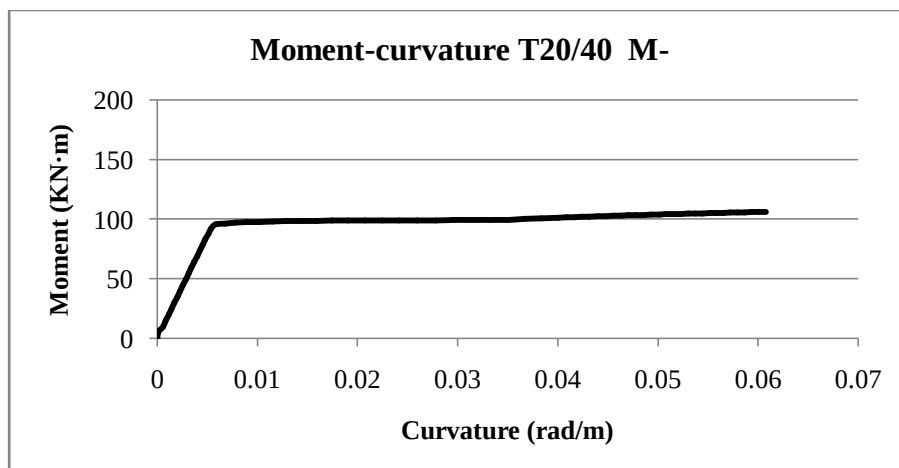


Figure C. 6: Equivalent section capacity of T20/40 at the end of span for negatives moments.

Appendix C.2: Beams and columns sections of the actual building:

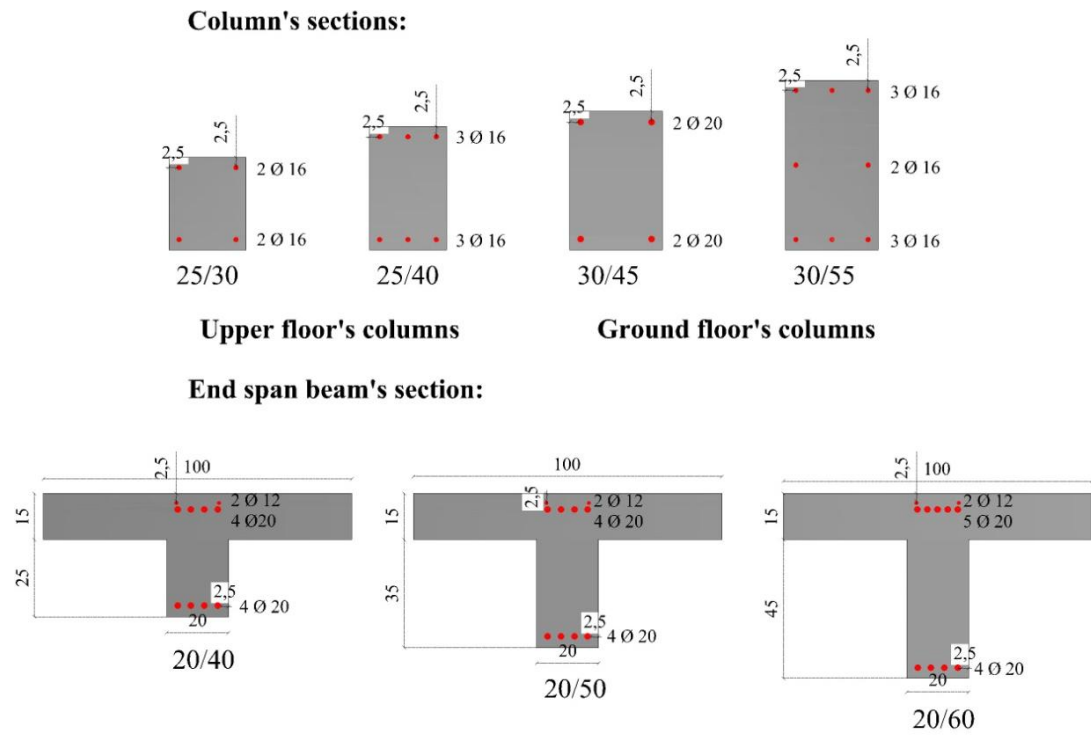


Figure C. 7: Sections of the actual building.

Appendix C.3: Jacketed columns:

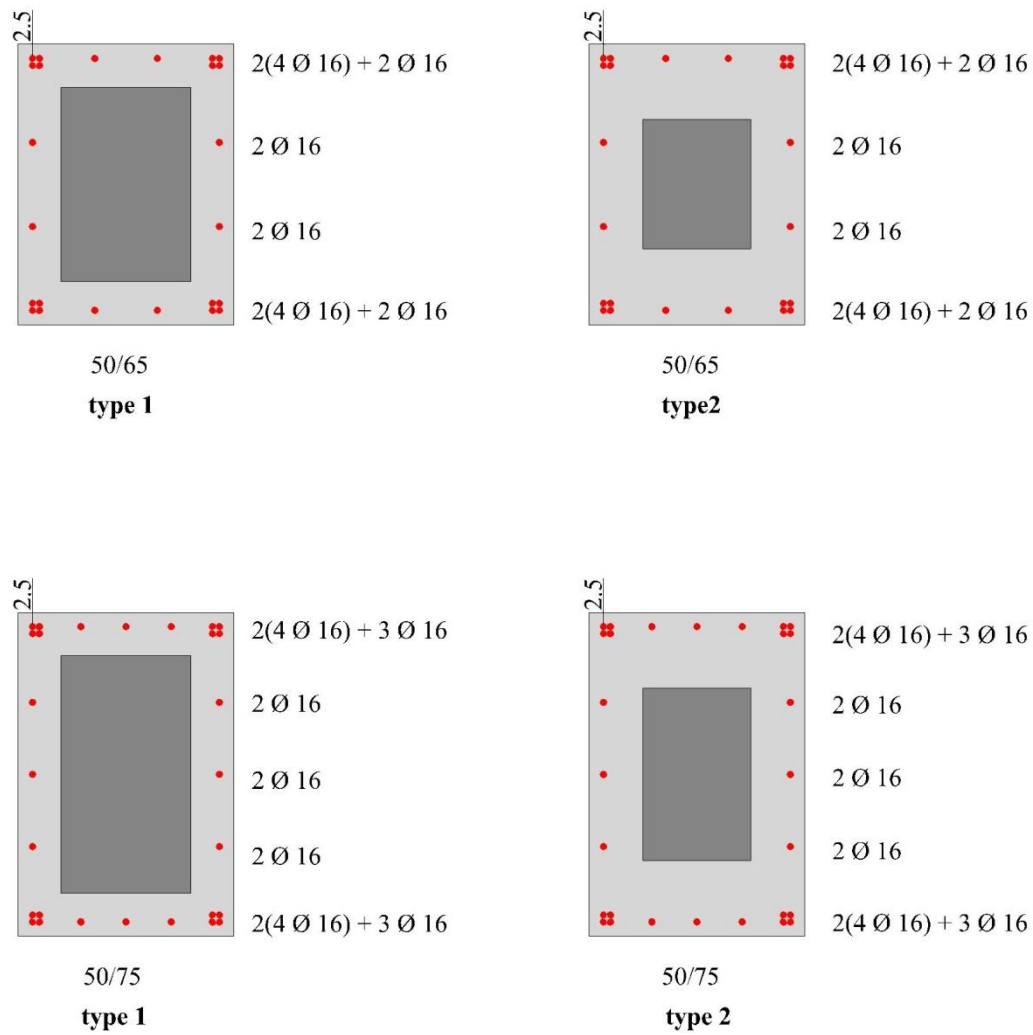


Figure C. 8: Jacketed columns sections.

Appendix C.4: Sections and capacities of Shear walls:

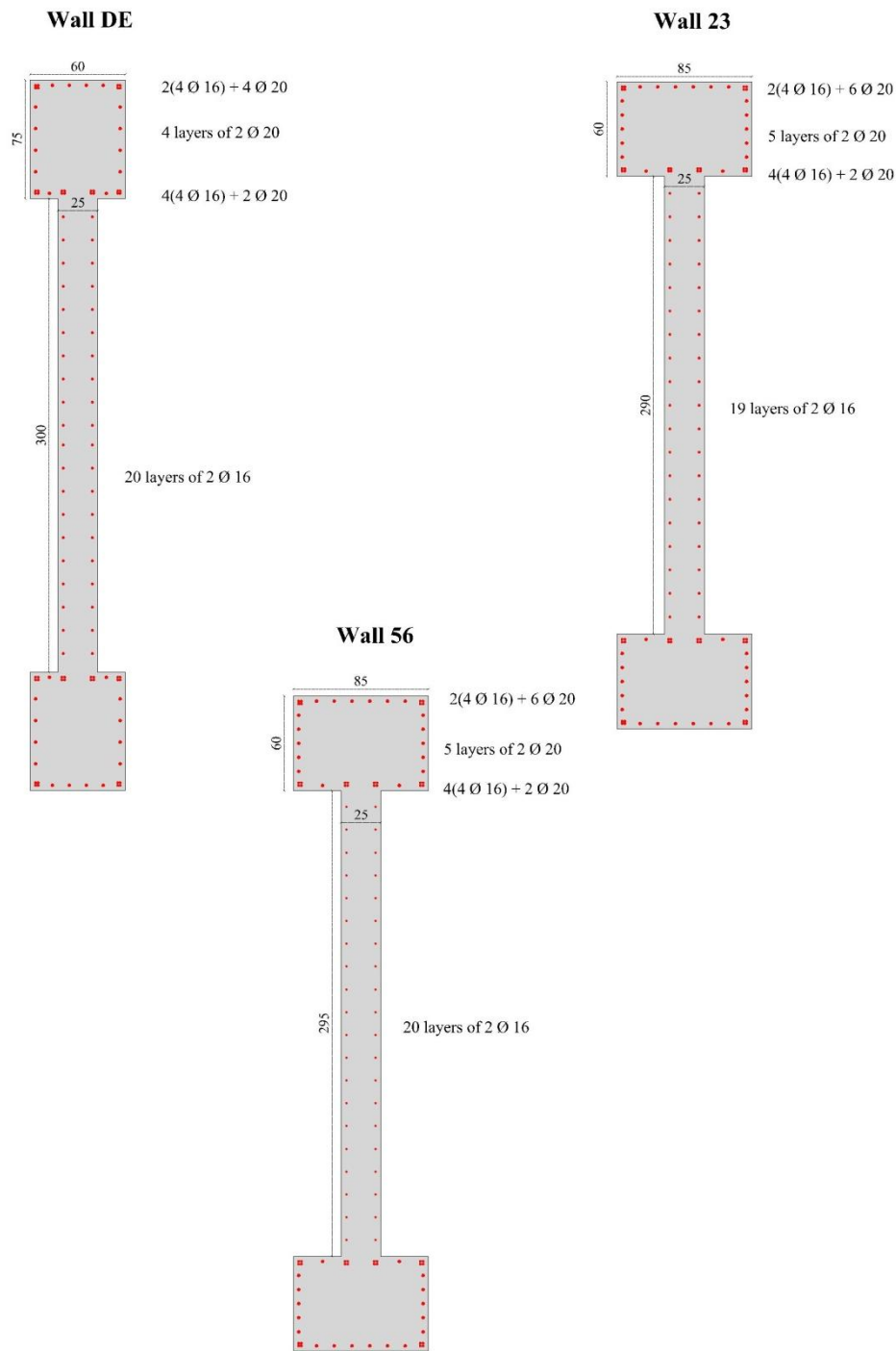


Figure C. 9: Sections of shear walls.

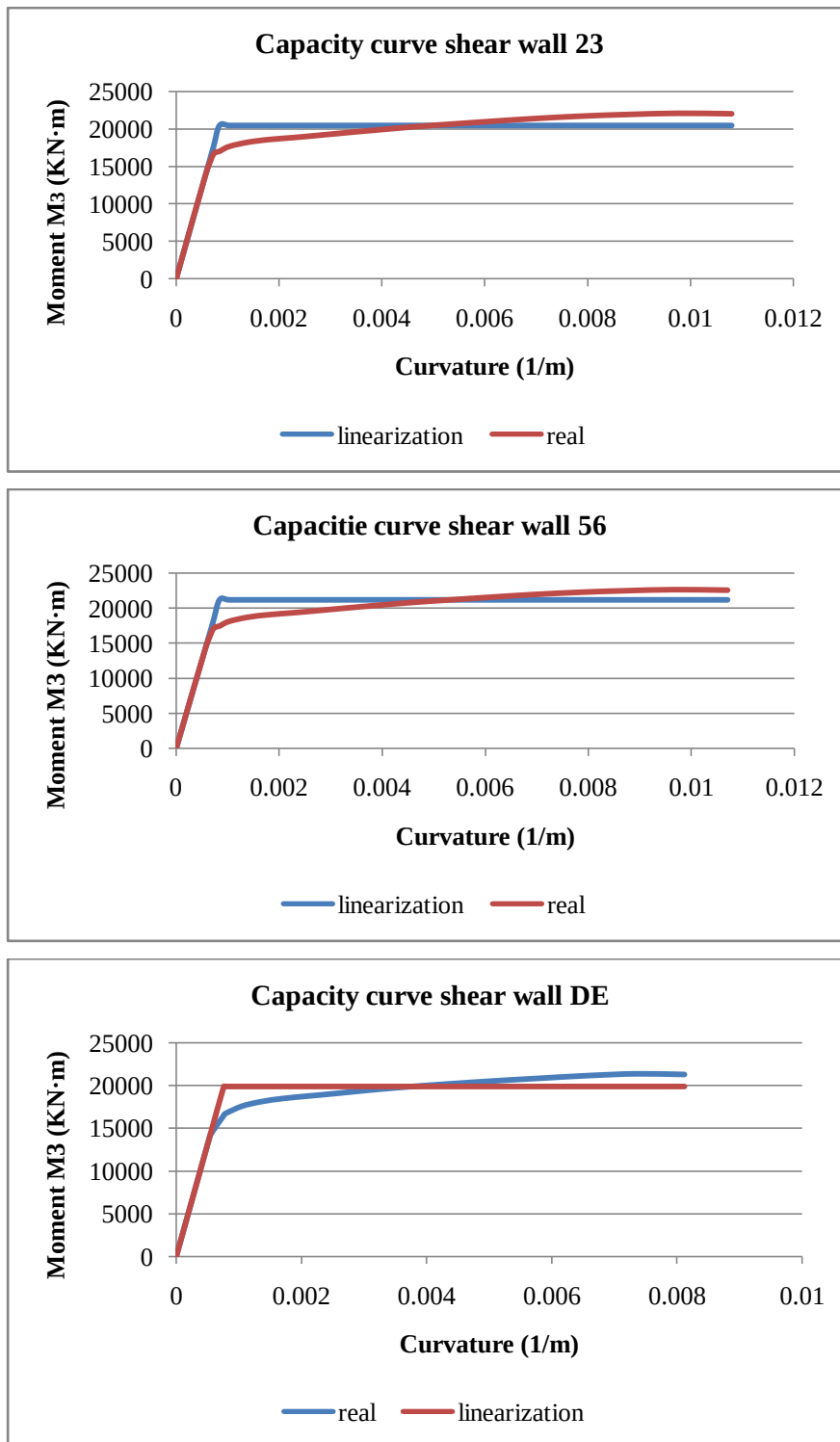


Figure C. 10: Shear wall capacities and their bilinearization.

The curve that represents the linearization of the real capacity is the one used to define hinges of shear walls.

APPENDIX D: DOUNDATION MODELIZATION

Appendix D.1: Foundation meshing and properties calculation

In this appendix it is possible to appreciate how the meshing has been done.

Firstly, the shell has been assigned where it is necessary. To mesh it, it has been done in the way that the elements will have more or less the same size. Each assigned area has been meshed in a grid. The element configuration of those grids is determined as on the following figures:

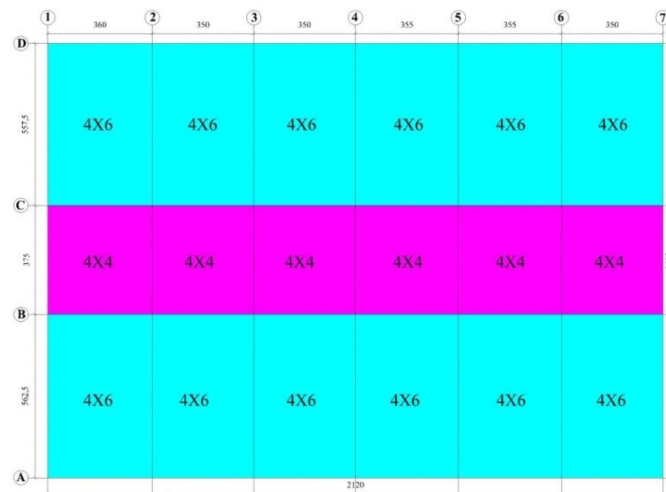


Figure D. 1: Normal mesh assignment.

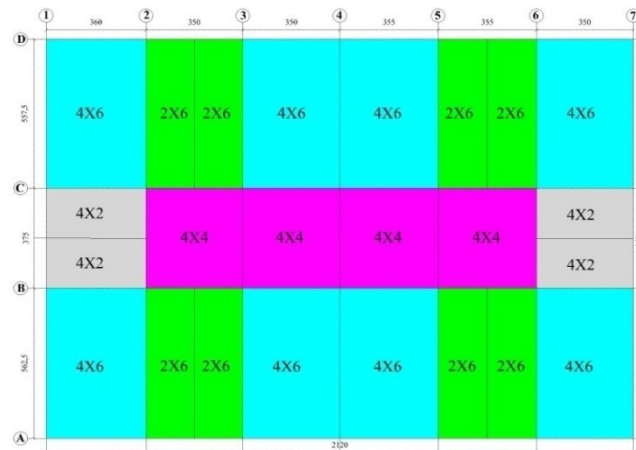


Figure D. 2: Mesh assignment for 4x4 shear wall case.

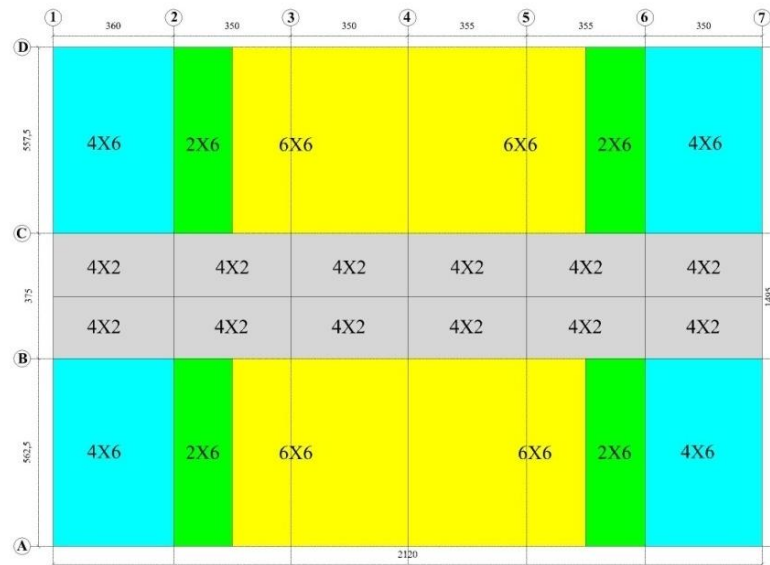


Figure D. 3: Mesh assignment for 4x2 shear wall case.

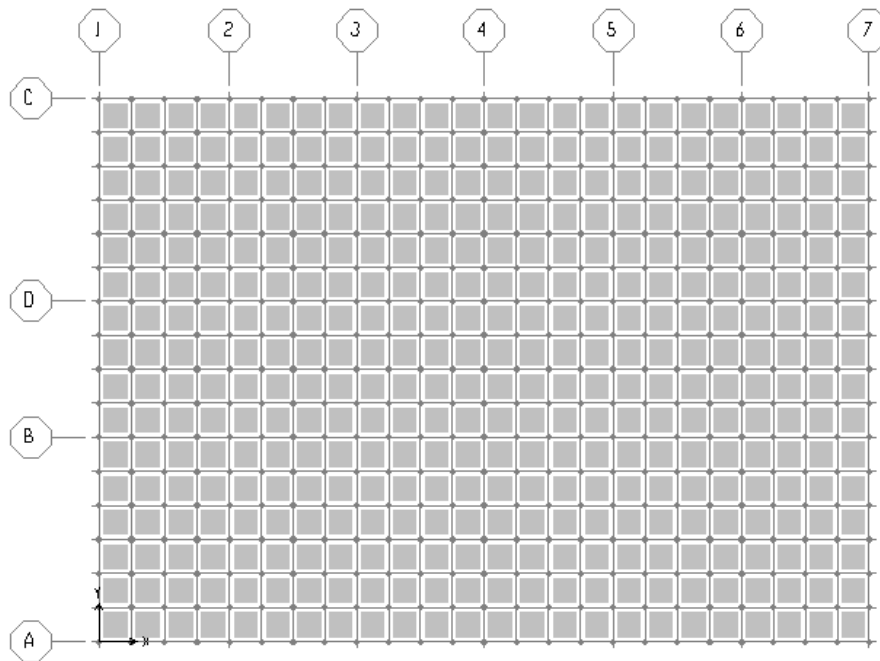


Figure D. 4: Meshed foundation.

Properties determination of the frame elements representing the soil:

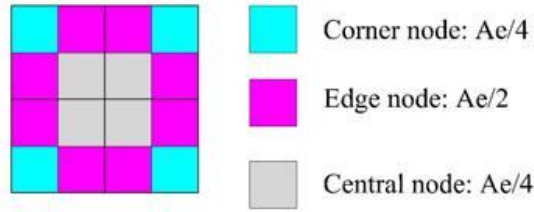


Figure D. 5: Effective area representation.

The cross section of each frame element will be different depending on which node it has been assigned because the effective area varies depending of the node. As it is possible to see in the figure below, a central node works on a effective area A_e , while an edge node works on a half one, when a corner node work on a quarter one.

The effective area is the same as the area of one element. All the elements have more or less the same dimensions, so their area has been generalized to: $A_e = 0.83 \text{ m}^2$.

Using the following formulae that have already been presented:

$$EF = k_s * A_e * \text{Unit lenght}$$

$$E \rightarrow \text{Elastic modulus of C12}$$

$$F \rightarrow \text{Cross section}$$

$$k_s \rightarrow \text{soil modulus}$$

$$A_e \rightarrow \text{Effective influence area of the frame members}$$

$$\text{Unit lenght is taken as } 1\text{m}$$

$$C_{lim} = q_u * A_e$$

$$C_{lim} \rightarrow \text{compression limit}$$

$$q_u \rightarrow \text{bearing capacity}$$

The following properties are gotten:

Table D. 1: Properties of the foundation elements.

Cross section (m ²)	Z1	Z2	Z3	Z4
Edge points	0.00328604	0.00164302	0.00049291	0.00026288
Middle points	0.00657209	0.00328604	0.00098581	0.00052577
Corner point	0.00164302	0.00082151	0.00024645	0.00013144
C _{lim} (KN)	747	498	311.25	174.3

APPENDIX E: Analyses pictures

The two next pictures are at their ultimate step. Their objective is to show how the structural mechanism is being translated to upper stories while columns are jacketed.

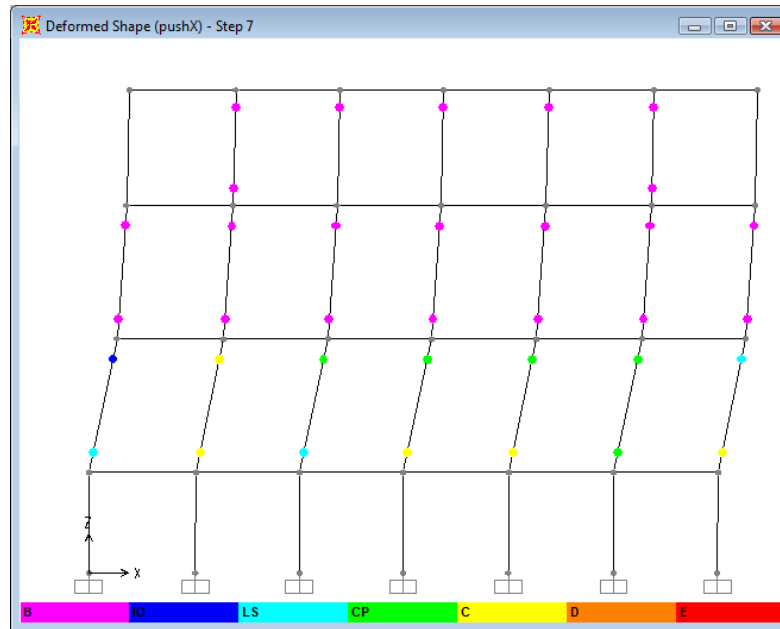


Figure E. 1: Deformed shape of jacketed columns of the first floor case with fixed soil condition case.

To increase dimensions and material qualities of columns of ground floor, doesn't contribute in improving the performance. The structural mechanism continues existing. Hinges are not so critic but there are more than in Actual building case.

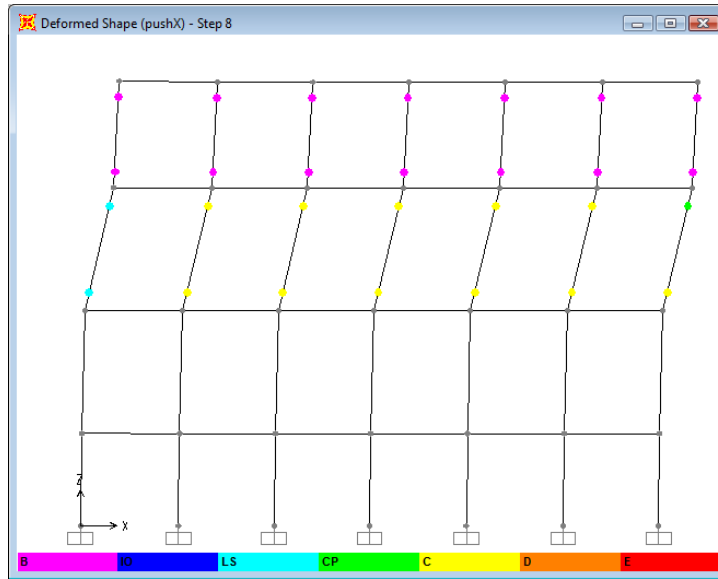


Figure E. 2: Deformed shape of jacketed columns of 1st-2nd case over "x" direction, on fixed case soil condition.

The structural mechanism is transferred to a upper level. This solution is invalid.

Following pictures are at their performance point:

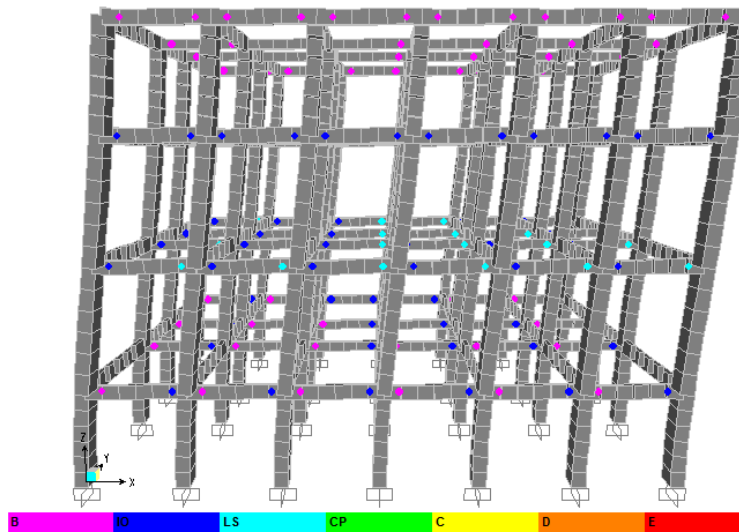


Figure E. 3: Deformed shape of all columns jacketed case over "x" direction with fixed soil condition.

The structural mechanism has disappeared. Hinges of the building appear at the beams. It is one of the expected cases. No hinges are present at the feet of the columns.

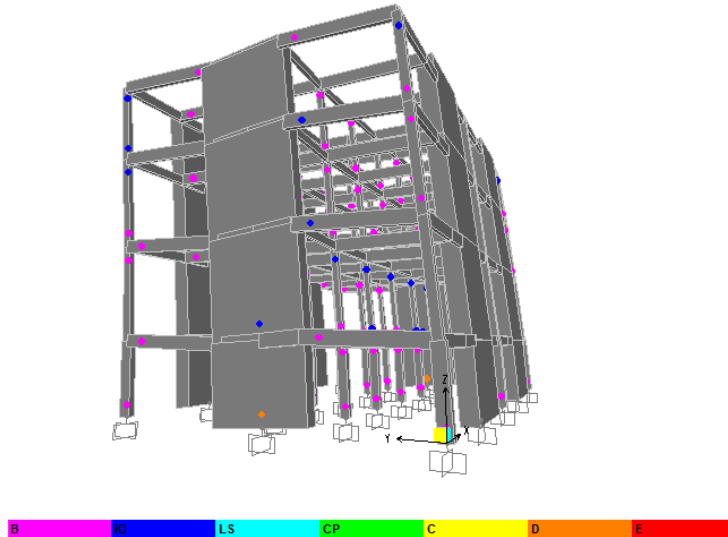


Figure E. 4: Deformed shape of 4x2 shear wall case over "y" direction with fixed soil condition.

The situation is the one expected. There is ultimate hinges on some columns and also on feet of shear walls. They are located on the two lowest floors as it was expected.

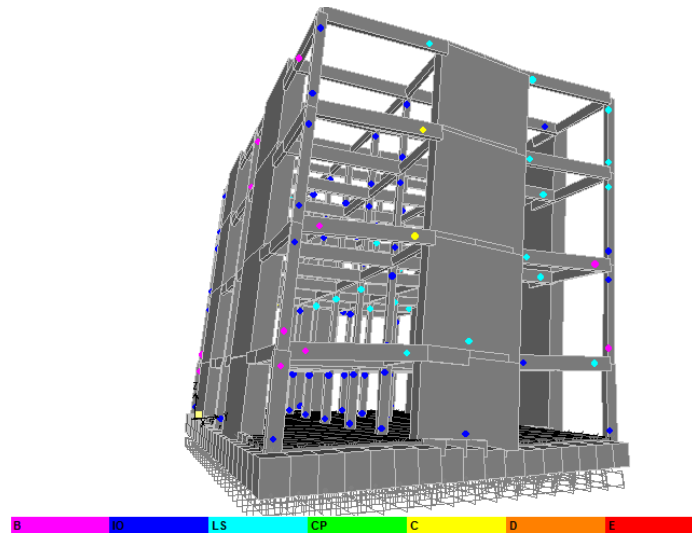


Figure E. 5: Deformed shape of 4x2 shear wall case over "y" direction with Z2 soil condition.

Hinges appear where expected. Uplifts around shear walls appear. With positive vertical displacement on left side of the shear wall and negative one on the right side. As it is possible to observe the hinge distribution differs meaningfully.

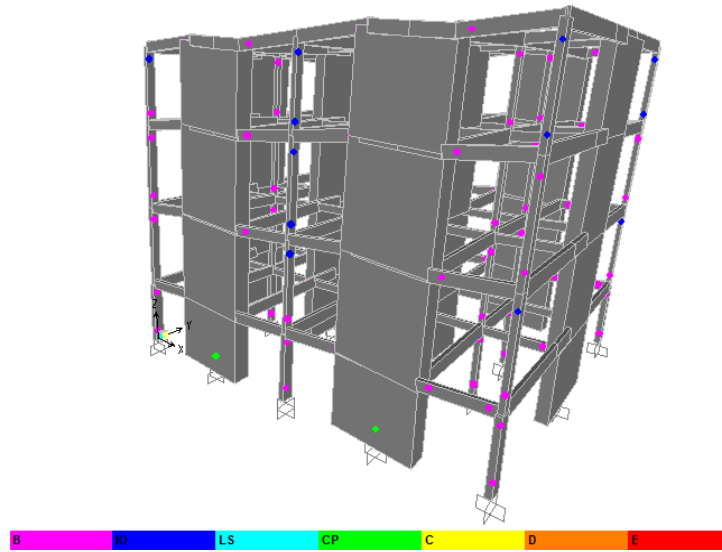


Figure E. 6: Deformed shape of 4x4 shear wall case over "x" direction with fixed soil condition.

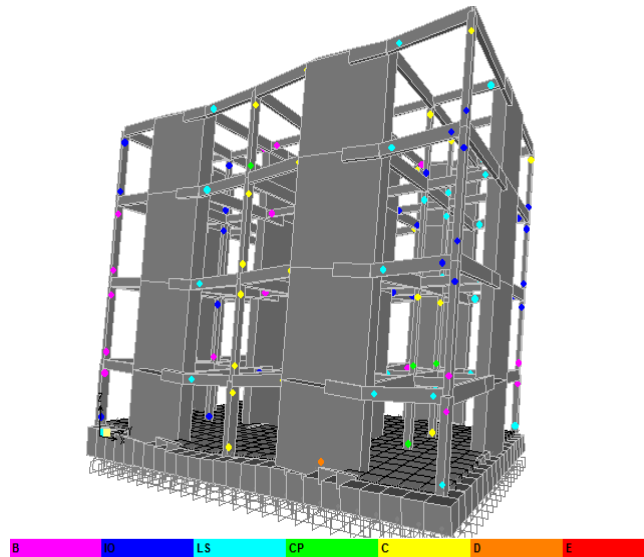


Figure E. 7: Deformed shape of 4x4 shear wall case over "x" direction with Z3 soil condition.

There is less hinges on the shear walls but hinges on columns more critical. It is possible to observe the uplift around shear walls.

CURRICULUM VITAE

Francesc Jané Figuerola was born in Valls (Tarragona, Catalunya, Spain) in 1986. He graduated from IES Pons d'Icart in 2004. He had been admitted to Escola Tècnica Superior d'Enginyeria Industrial de Barcelona (ETSEIB, Universitat Politècnica de Catalunya) as a Industrial engineer student in 2004. In 2009 he has been admitted by Istanbul Technical University to realize his Master thesis under the Erasmus Program at the Faculty of Civil Engineering. During his stage in Istanbul, he was invited to participate at the STEEL seminar.