Seismic Assessment and Retrofitting of Existing RC Building by Using Steel Braced Frames

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ABSTRACT

Antakya city is in danger as a result of solid seismic actions happening in the territory, and diverse soil conditions that can create a variety of the ground motion amplification. In recent years, scientists and engineers have started to assess the existing structures and their behaviors in resistance to lateral loading, potential earthquake hazard, and vulnerability. Existing structures can be retrofitted to incorporate new improvements and techniques to oppose quake and seismic burdens, which was the most efficient approach to shield against the financial and social disaster influenced by serious seismic action in urban areas.

This thesis presents a study on a five-storey reinforced concrete structure was built in 1988 and located in Antakya, Turkey. This work consists of three phases. The first stage, data collection which includes building plans, material properties, structural condition, and reinforcement details. Material properties are measured using non-destructive testing method called model calibration. The model calibration is obtained from building dominant periods and mode shapes of the existing building, which have been measured using forced vibration tests. In the second stage, the analytical modeling of the structure is made using SAP2000. After model calibration, the nonlinear static pushover analysis for the seismic performance evaluation based on the ATC-40 methods has been obtained. Finally, the existing building, which showed low performance according to code requirements, is strengthened by using two different types of external steel brace frames. They have been attached to Y-direction, which has poor performance for both sides until the second floor, and recommended that this strengthening technique is an appropriate method according to the performance and cost analysis.

Keywords: Evaluation earthquake, Pushover analysis, Retrofitting, Forced vibration

ÖZ

Antakya ili, bölgede meydana gelen güçlü depremler ve zemin hareket büyütmesini meydana getirebilecek farklı zemin koşulları nedeniyle risk altındadır. Akademisyenler ve mühendisler son yıllarda mevcut binaları, deprem yüküne, potansiyel sismik tehlikeye, hassasiyete ve yanal yüklere karşı dirençteki performanslarını değerlendirmeye başlamışlardır. Günümüzde, mevcut binalar, kentsel ortamlarda şiddetli sismik aktiviteden etkilenen ekonomik ve sosyal felaketi önlemenin en ekonomik yolu olan, deprem ve sismik yüklere karşı direnç gösteren yeni gelişmeleri ve yöntemleri içerecek şekilde yeniden güçlendirilebilir.

Bu tez, 1988 yılında inşa edilen ve Türkiye, Antakya'da bulunan beş katlı bir betonarme yapı ile ilgili yapılan çalışmayı sunmaktadır. Bu çalışma üç aşamadan oluşmaktadır. Bu aşamalar; ilk olarak, veri toplama (yapı planları, malzeme özellikleri, yapısal durum ve donatı detayları), malzeme özelliklerinin belirlenmesi için tahribatlı ve tahribatsız testlerin kullanılması, mevcut binaya zorlanmış titreşim uygulanarak bina hakim periyot ve mod şekillerinin belirlenmesi; Daha sonra, SAP2000 programı kullanarak yapının analitik modellemesinin yapılması ve model kalibrasyonundan sonra ATC-40 yöntemlerine dayanan sismik performans değerlendirmesi için doğrusal olmayan statik itme analizinin yapılması; Son olarak, performansı yetersiz bulunan mevcut binanın güçlendirilmesi için iki farklı tipte çelik çerçeve, binanın zayıf olduğu Y-yönünde ikinci kata kadar eklenerek güçlendirilmesi ve bu güçlendirme tekniğinin bina performansı ve fiyat analizi yönünden uygun bir yöntem olduğunun önerilmesi, kapsamaktadır.

Anahtar Kelimeler: Deprem, Statik itme analizi, Güçlendirme, Zorlanmış titreşim

DEDICATION

To My Family

My Father and My Mother

My Brother and Sisters

My Friends and Teachers

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I would never have been able to finish my dissertation without the guidance of my committee members, help from friends, and support from my family.

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LIST OF ABBREVIATIONS AND SYMBOLS

IO	Immediate Occupancy
LS	Life Safety
CP	Collapse Prevention
RC	Reinforced Concrete
ASCE	American Society Of Civil Engineers
FEMA	Federal Emergency Management Agency
TEC	Turkish Earthquake Code
TSC	Turkish Standard Code
ATC	Applied Technology Council (Seismic evaluation and retrofit of concrete buildings)
NCC	Network Control Centre
CSM	Capacity Spectrum Method
ADRS	Acceleration versus Displacement Response Spectrum
A_o	Seismic Zone Factor
I	Building Importance Factor
E_c	Modulus of elasticity of concrete
f_c'	Cylinder Characteristic Compression Strength of Concrete
δ_t	Target displacement
C_0	The coefficient correlating the displacement
C_1	Modification factor to relate expected maximum inelastic displacements to displacements
C_2	Modification factor to represent the effect of pinched

hysteretic shape, stiffness degradation and strength deterioration on maximum displacement response.

C_3	Modification factor to represent increased displacements due to dynamic P-Δ effects.
S_a	Response spectrum acceleration, at the effective fundamental period and damping ratio of the building in the direction under consideration, g
T_e	Effective fundamental period of the building in the direction under consideration
g	Acceleration of gravity
V_t	Target shear force
C_m	Effective mass factor to take in the account the higher mode mass participation effects obtained from Table 3-1 in FEMA 356.
W	Total building weight

Chapter 1

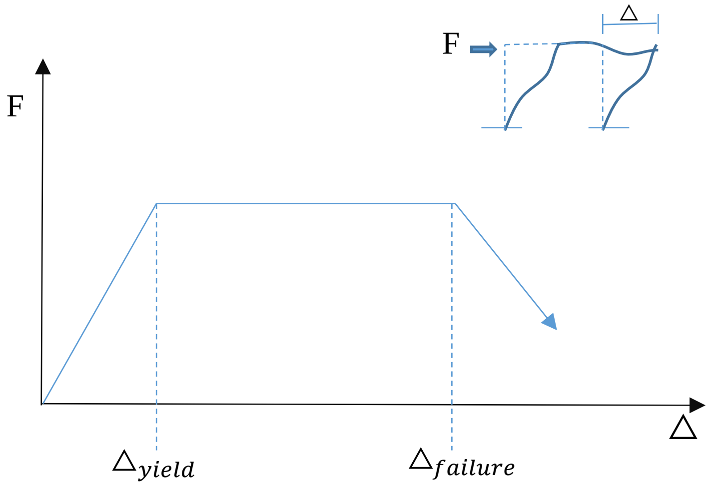
INTRODUCTION

1.1 Overview

Earthquakes are part of Earth's life and a tragic part of human history. Many people have lost their lives and homes because of the earthquakes activities. For example, 7.7 magnitude earthquake struck Kocaeli Turkey in 1999 were more than 17 thousand persons have died and around 44 thousand injured. Around 121 tent cities were needed for emergency housing for more than 250,000 people. Around 214 thousand residential buildings and 30,500 business buildings were light to heavy damaged. Turkey is in high earthquake active zone which requires buildings to follow the Turkish Earthquake Code (TEC) 2007. However, most of the residential units are designed without adequate detailing and reinforcement for seismic protection which increase the possibility of catastrophic collapse. All the buildings that have been designed before 1998 were following the old Turkish earthquake codes (TEC 1984 and TEC 1975) which are not valid anymore. Due to the cost efficient, and time associate with evaluating and building reinforcement, it is important to have fast, cost efficient and effective way of evaluating the existing buildings. The building evaluation is than used to strengthen a building structure. If the strengthen cost is 40% less than the cost of the building. The reinforced structure design should be able to:

- Resist lateral load of earthquake.
- Enhance buildings rigidity in order to resist ground acceleration.

- Guarantee good system strength.
- Consider the ductility of buildings for decreasing seismic action. Ductility is the ability of the structure to undergo large deformations without collapse (Fig.1).



$$Ductility\ Factor\ \mu = \frac{\triangle_{failure}}{\triangle_{yield}}$$

Figure 1: Perfectly plastic structure subject to monotonic loading

Earthquake damage can be affected by different parameters such as earthquake intensity, duration and frequency of ground motion, soil condition, and construction quality. In addition, the sociologic factors, such as population density and time of earthquakes effects the earthquake damage.

The TEC 2007 determined the seismic design of a building and suggest number of factors that should be considered during the structure design. Those factors and recommendations are:

- During minor earthquake, the buildings should have minimum damage (structure and non-structure damage).

- During moderate earthquake level, no structural damages. However, non-structural damages are acceptable.
- The design should prevent structural collapse during severe earthquake. However, structural and non-structural damage are acceptable.

The factors are considered in many old structural design are building weight and its effect on the structural behaviour, which does not include the seismic action factors. Therefore, performance evaluations are required for those buildings. The performance evaluation of existing building is done in two stages. In the first stage, building condition and data collection. The data includes material properties and structural design details of the building. The second stage is called performance level assessment which can be categorised to three levels based on it severity: immediate occupancy, life safety, and collapse prevention. There are four different methods to evaluate the structure performance of a building. However, in this study only non-linear static method called pushover is used. The pushover method is a simplified version of nonlinear seismic response spectrum analysis. In this method, the simulated lateral forces is inserted to the model and ramp up step by step to a target displacement where the structure seismic behavior is evaluated. More details about the method will be provided in the methodology chapter. The method can provide a seismic limit of a building, dynamic behaviour of plastic hinges, and possible element/component failure.

Once the performance level and properties of the building are determined, the structural reinforcement can be under taken to increase the building resistance against seismic activities. In this process a proper structural retrofitting technique must be chosen for the particular building with the consideration of earthquake and seismic specification. One of those methods of retrofitting is adding external steel frame with bracing to the existing building. The main advantages of the external frame with bracing are: it can be done with minimum interference with building tenant, required minimum damage to a building, and low cost. Several performance factors should be considered during retrofitting of existing buildings: shielding the reinforced materials, aesthetically plastering, prevent loss of substrates.

1.2 Problem Statement

Many buildings in Turkey do not follow the new TEC 2007 code which can lead to catastrophic collapse during earthquake activity. Most of house owner cannot afford the cost associated with the current evaluation and retrofitting process. Therefore, there is a necessity for fast and cost effective way to evaluate and retrofit the current structure of residential buildings. The first challenge in building evaluation is material properties measurement. Most of the current methods require using destructive testing methods to measure building material properties such as compressive strength of concrete (core test) and tensile strength of reinforcement. Those methods are not accepted by many of house owners due to the cost and damage that can be done to the building. In addition, core testing does not reflect the overall properties of the building material and just provide local material property. The second challenge is retrofitting the structure of existing building without evacuating resident from the building.

The aim of this study is to provide a methodology to estimate building material properties without damaging the building by using non-destructive testing method. The building material properties are than used to evaluate the building performance and design external steel frame with bracing to support the building. The study is done on an old five-story building in Antakya, Turkey.

1.3 Objective of Thesis

This study introduces a methodology for evaluating and retrofitting an existing building. The objective and scope of this study will include:

1- Estimate building material properties by calibrating simulated structure behaviour with real dynamic behaviours of the building obtained from forced vibration test results.

2- Reinforce the existing building by using external retrofitting technique.

3- Increase building safety and reduce economic losses.

4- Provide fast and cost effective way to evaluate and reinforce an existing building.

1.4 Content of Thesis

In this thesis, our main focus was on providing a fast, economic, and reliable methodology to evaluate and retrofit an existing building. The thesis includes five chapters as follows:

Chapter 1 is Introduction: this chapter provides short presentation of the problem, which is under study, the main objectives and scope of the work. In addition, the brief description of the main stages that have been used on building evaluation and retrofitting design.

Chapter 2 is Literature Review: this chapter is surveying different seismic performance and retrofitting methods to highlight the advantages of the methods that have been used on this study.

Chapter 3 is Methodology and Data Collection: this chapter shows the main information and data required about the building which include building plans, material properties, and structural condition. The methodology of building material properties measurement is highlighted. In addition, the chapter shows the main stages that have carried out on this study to evaluate and retrofit an existing building. This chapter also clarifies the main terminologies that have been used on this study.

Chapter 4 is Results and Discussion: this chapter shows the obtained results from the software SAP2000, and retrofitting solution of the existing building. Two different retrofitting frames are used and compared in terms of performance and cost.

Chapter 5 is Conclusion and Recommendations: this chapter provide a summary of this work and the obtained results with some recommendations and future work.

Chapter 2

LITERATURE REVIEW

2.1 Introduction

As mentioned in previous chapter, earthquake can results in large damage to buildings that have not been designed to stand up a seismic activity. In Turkey many buildings have not been updated to the new TEC-2007 code which required fast, reliable, and cost effective way to evaluate and strengthen the exist building. This chapter will introduce the risk level associated with earthquake and the main factors that has been considered on the risk level calculation, different modeling and non-destructive methods for seismic performance analysis, which includes the works that have been done on these areas, and survey of different retrofitting methods are presented in the literature. At the end, a short summary to highlight the methods and reasons for choosing those methods that have been used in this study is presented.

2.2 Risk Level of Earthquake

The main three factors on evaluating seismic risks are: level of seismic hazard, number of people and properties effected by the seismic hazards, vulnerability of these people and property to the seismic hazards.

Earthquake hazard level is a term used to estimate the probability of exceeding a ground shaking or motion a certain level within 50 years. There are number of parameters which are considered on the earthquake hazard level. Those parameters are: magnitudes and location of the earthquake, frequency of occurrence, and rocks and sediments properties. There are four different hazard levels based on Federal Emergency Management Agency (FEMA 356). Those levels are 50%, 20%, 10% and 2%. This is an example to illustrate those levels, a hazard level with a 50% on 50 years period will possibly return within 72 years. All of those factors have been take care of in SAP2000, software that is used on this study, by just providing the zone area of the building.

2.3 Modeling Method for Seismic Performance Evaluation

The modeling of seismic performance can be divided to two main categories: linear elastic and non-linear methods. There are number of linear methods such as equivalent lateral force analysis, and modal response spectrum analysis. The non-linear methods are pushover analysis, and non-linear time history analysis. Jeyasehar, C. *et al*. (2009) showed different computational and experimental methods to evaluate the seismic performance.

Applied technology council (ATC) and Federal Emergency Management Agency (FEMA) are recommended and provided methods to obtain level of performance of structure during seismic action. Pushover analysis is one of those methods that has been recommended by ATC 1996 and FEMA 1997, which has been followed in this study. The pushover method finds the performance point of a structure based on lateral force on structure and earthquake demand curves. The hypothesis behind pushover analysis are Liping, L., *et al*.(2008):

1- The structure seismic response is related to equivalent system single degree of freedom (DOF) which mean that, the seismic response is only controlled by first vibration mode.
2- The structure deformation is expressed by shape vector.

3- Infinite (rigid diaphragm) floor stiffness.

ATC 40 (1996) and FEMA 273 (1997) provided step by step report to use the non-linear static pushover analysis. Those reports highlight the modeling aspects of the hinge behavior, acceptance criteria, and procedures to locate the performance point.

Number of research studies have been conducted using pushover method and the pushover showed its capability to evaluate structural behavior. Some of those studies; Munshi and Ghosh (1997) conducted a study to determine the ductility of the structure using pushover method. The main reason of ductility was due to week coupling between walls which has been solved by increasing the wall strength. Helmut Krawinkler and Seneviratna (1998) compared pushover analysis with elastic analysis method. The study showed the capability of pushover method to identify the parameters that control the structure behavior during earthquake. Furthermore, they showed the effectiveness of pushover method when the structure is vibrating under a fundamental mode. Ashraf Habibullah and Stephen (1998) showed the flexibility and speed of using a pushover method on SAP2000 for analyzing a 3D structure. Mwafy and Elanashai (2000) compared inelastic static pushover analysis with dynamic pushover analysis. The modeling is done for 12 reinforced concrete buildings with different characteristics. The main finding is that the static pushover analysis is appropriate for low rise and short period framed structures. Elnashai (2001) showed the potential of using pushover analysis over inelastic dynamic time history analysis during seismic design and evaluation. In addition, he introduced a new adaptive pushover method which considers a spread of inelasticity, geometric non-linearity, full multi modal, spectral amplification and period elongation within a framework of fiber modeling of materials. Inel and Ozmen (2006) showed the effect of plastic hinges on a reinforced concrete buildings using nonlinear pushover analysis. The study is done on 7 stories reinforced concrete buildings with different heights. The results were in terms of shear capacity, displacement capacity, and deformation of hinges. Zine *et al.* (2007) conducted the Pushover analysis for reinforced concrete structures designed according to the Algerian code. Poursha *et al.* (2009) introduced a new pushover procedure to consider the higher-mode effects on the structure. The procedure is test on four special steel moment-resisting frames with different heights and compared with nonlinear response history analysis. Abdi *et al.* (2016) conducted a sensitivity study to show the effect of response modification factor on steel structures with soft story retrofitted building by using pushover analysis.

As a summary, our goal is to use fast, cheap, and reliable method to evaluate an existing RC building. Therefore, based on ATC & FEMA recommendations and the positive indications from the performed research studies the nonlinear pushover analysis will be used in this study to evaluate an existing building.

2.4 Non-Distractive Method for Seismic Performance Evaluations

Forced vibration tests performed with an eccentric-mass shaker after the retrofitting work was completed (Bayraktar, *et al*. 2013). During the forced vibration tests, the building was excited around its modal frequencies using an eccentric-mass shaker. It was found that the modal damping values increased with the amplitude of the excitation force.

Genes *et al.* (2011) conducted a project on the basis of field survey the local building stock of the study area (Antakya, Turkey). The building stock is classified with respect to the use, building type and parameters relevant for the response and damageability under seismic action. Sub-classes of the predominant RC frame building types were derived. Using a specific scheme of ranking criteria, representatives are selected for a multi-tasking instrumental testing procedure which in each phase is related to the outcome of parallel analytical investigation to calibrate the 3D-finite element model. The analytical investigation of the selected buildings have been done by pushover analysis to find the performance level of each building class.

In this study, the forced vibration test method is used to determine the dynamic behaviours parameters of the building. The analytical model which is prepared in SAP2000, is calibrated according to the vibration periods and mode shapes by changing the material properties of the concrete. This new method has been used recently and it has many advantages:

1- The core test method require damaging the building to determine the material properties of the building (i.e. compression strength).
2- Most of the methods measure the local material properties instead of the overall properties.
3- This method considers all of the building defects on the overall behaviour.
4- Easy and fast to implement.
5- Can be done with minimum interference with building residents.

2.5 Retrofitting Methods of Existing RC Building

Earthquake motions generate a horizontal load which act on the structural system as a respond to the motion. The structural deformation develops internal forces in cross section of the structural elements and overall displacement behaviour across the building. The displacement magnitude depends on mass and stiffness of the building.

Buildings with low mass and high stiffness will have smaller horizontal displacements. Each building has a particular displacement capacity. The goal of any strengthening method is supporting the building so that its displacement should not exceed the capacity (Kaplan, H. *et al.* 2011). The reinforcement can be achieved either by reducing the expected displacement of the whole structure system or increase the structural system capacity by providing ductility to the building.

Decreasing the total horizontal displacement of a building requires decreasing its natural period of vibration. The building natural period of vibration decreases by increasing the stiffness of the building. The building stiffness can be increased by adding new structural elements to the building. The new stiffness prevent the building from reaching its limited capacity under low lateral load and displacement.

The guidelines given by Rai (2005) provided a systematic methodology for the seismic evaluation of buildings and some cost effective strengthening schemes for existing buildings. Williams *et al.* (2009) showed an approach to make informative decisions on whether or not to retrofit structures for seismic events based on the expected economic payback. The approach has been test on two identical RC buildings, but at different locations, using different retrofitting methods. The comparison was based on the probabilities of failure and generalized reliability indices.

There are numbers of techniques to improve the strength of existing RC building. These techniques will be discussed in the following topics.

2.5.1 Improving Structural System

2.5.1.1 Infill Shear Wall

Infill shear wall is very popular retrofitting technique. Many researchers have been conducted to study the method and they showed how this method improves the lateral load capacity and stiffness of the structure. However, installing a shear wall is difficult, cause many damages to buildings and costly as shown in Fig. 2 (Baran, M., 2005).

White and Mosalam (2002) conducted study to evaluate and retrofit procedure of an existing RC building, which has been designed without considering seismic activity. The study showed the building behavior before and after masonry infilled frames.

Figure 2: Infill shear wall application in an RC building

2.5.1.2 Adding External Steel Bracing out of a RC Building

Adding steel bracing to RC frame reduces seismic action. This type of retrofitting is applied to outside of a structural system (Bush, T., *et al.* 1991). The main advantages of this method is that it allows easy installation across the axes on external facades. Fig. 3 presents the use of buttress type steel shear wall constructed on the building's external facade as a different example.

Prakash and Thakkar (2003) compared steel bracing method with infill masonry walls method on a 14 stories RC building located in seismic zone IV. Different steel bracing methods are used: V, diamond, and cross pattern. The cross patterns of steel bracing showed better seismic performance than the rest.

Figure 3: External steel bracing frame out of a structure

2.5.2 Strengthen based on Building's Connection (Beam -Column)

2.5.2.1 Reinforcement Concrete Jacketing

One common method for strengthening a concrete connection is the reinforced concrete jacketing as shown in Fig. 4. Jacketing can be defined as the restricting the connection with new and higher quality reinforced concrete elements.

Figure 4: Retrofitting the connection using jacketing by reinforcement concrete, (DEPTA Engineering)

2.5.2.2 Steel Jacketing

Steel jacketing is done by attaching steel jackets on the surface of connection as see in Fig. 5. The required thickness and the length of the steel jackets can be calculated. This method cannot be used in SAP2000 because the steel jacketing model is not available in it (Choi, E., *et al.* 2010).

Figure 5: Using steel jacketing for connection (RADYAB Engineering Solution)

2.6 Summary

As has been mentioned, our goal is to provide a procedure to measure building material properties, seismic performance evaluation, and retrofitting of an existence building using fast, cheap, and reliable way. There are different methods to evaluate and retrofit an existing building. Many of them require major change in the building and they are not easy to implement. For the purpose of this study, the material properties of the building are estimated by using a new method, which couples the results from force vibration method with analytical model from SAP2000. For seismic performance evaluation, nonlinear static pushover method is used. Because, it is fast, simple to use, and reliable. For retrofitting, external steel bracing is used. Because, it requires the minimum damage to the building, no need for people evacuation, and more cost efficient method.

Chapter 3

METHODOLOGY AND DATA COLLECTION

3.1 Introduction

This chapter shows the procedure, which has been used to conduct this study. The steps included data collection, estimation of building material properties, seismic performance analysis, and strengthening with external steel bracing. In addition, detailed descriptions and explanations about the methods and important terminologies are shown.

3.2 Research Methodology

This study consists of three stages. In the first stage, data and information about the structural plans, building frame, and material properties are collected. The model of elasticity is estimated by coupling the forced vibration data of the structure, with analytical modal behaviour from SAP2000. In second stage, a simulation model of the building has been created to evaluate the behaviour of various lateral loads resistance on existing RC frame. The analysis is carried out by using ATC 40 pushover methodology on SAP2000. The analysis outputs are: performance levels, behaviour of plastic hinges, and possible week elements location on the structure of the existing building. In the third stage, the seismic retrofitting design of the building is obtained and evaluated by using ATC 40 pushover analysis methodology. The strength of the building structure is increased by using steel braced frame which is linked to the RC frame of the building. The enhanced performance of the existing building was studied in detail in term of demands curves, hinges formation, momentrotation, capacity spectrum, and performance level of the building. Two different steel bracing frames have been studied in term of cost. The cost is based on the mount of steel material that have been used to strength the existing building. The procedure of those stages is shown in Fig. 6.

The aim of this procedure is to update the current existing building to meet the current TEC 2007.

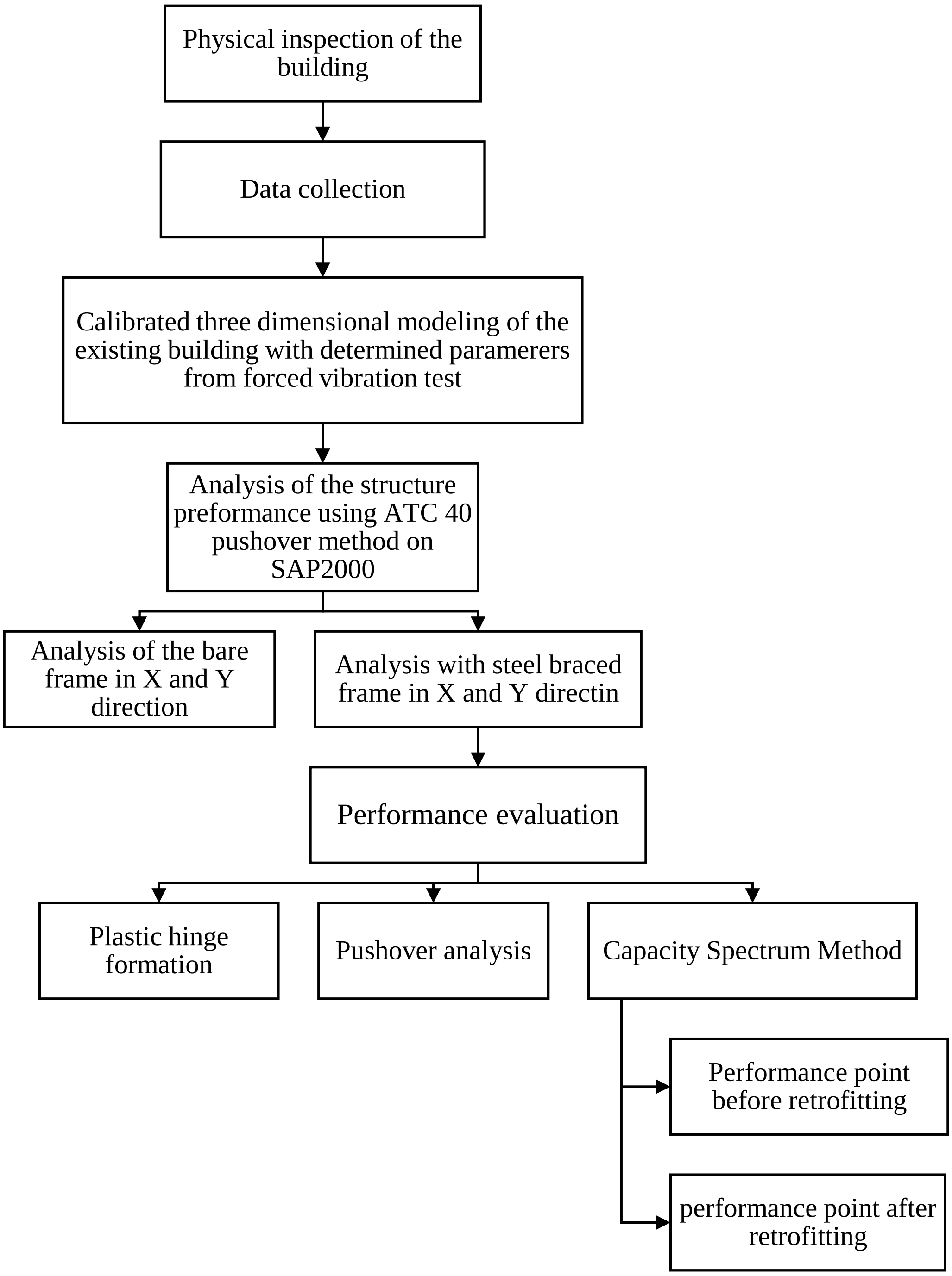


Figure 6: Analysis of the existing building

3.3 Data Collection

3.3.1 General Information about the Building under the Study

This study investigate the retrofitting and seismic evaluation of an existing building. The building is 5 story apartment, located in Antakya, Turkey. The building was built in 1988. This area is on the suburbs of Antakya, Turkey is located on 36°12'17.78''N 36° 8'39.67''E by Google Earth Map 2017 chronicles in (Fig. 7). The city is on seismic zone I.

Figure 7: Location of studied apartment in Antakya, Turkey

3.3.2 Building Details

The apartment is a five- story RC building**.** The building dimensions are 23 m length, 14.4 m width and 3 m height for each floor (Fig. 8). The whole designs of the building, floor designs, sections dimensions, shaft lengths, and stairs dimensions are given in Appendix A.

Figure 8: Studied apartment view in Antakya, Turkey

Those are the main building dimensions, which have been collected about the building:

1. Building plans and dimensions of span, height, length width which have been provided, are shown in Figures A1 to A7 in Appendix.

2. Columns design of reinforcement for each floor on plans design are shown in Figures A8 to A10 in Appendix.

3. Beams design of reinforcement for each floor on plans design are shown in Figures A20 to A24 in Appendix.

4. Marking short columns and other comparative issues.

5. Separation, contiguousness or the presences of joints are expressed.

3.3.3 Material Properties of the Building

The methods that have been used to measure modulus of elasticity are as follows: tension (or compression) test, bending test, and natural frequency vibration test. The tension and bending test are based on the principle of Hooke's law and they are called static methods.

In this study, the method that has been used to determine the modulus of elasticity is based on natural frequency vibration test. By using vibration generator and some sensors on the existing building, the dominant vibration period and the mode shape of the building can be determined. Performing modal analysis of a building in SAP2000 can provide a theoretical dynamic vibration behaviour of the building, which can be compared with real dynamic vibration parameters, by changing the Modulus of Elasticity or strength of the concrete in the model until the theoretical vibration period becomes equal to the real one obtained from the forced vibration test (Genes, M.C. *et al.* 2011). This method called, model calibration before performing Performance Analysis.

During the vibration tests at a given frequency both the applied load and the response are theoretically sinusoidal with the same frequency. The treatment of the recorded data is carried out by eliminating some noises from electrical, mechanical and ambient sources. Seismotec GmbH produces the vibration generator used in this test and BMR Ferra Automation GmbH has a working frequency of 0.5 to 15 Hz and is capable of producing a sinusoidal force as shown Fig. 9.


Figure 9: The shaker

It has two arms and each arm can be loaded with set of mass has weight 660 grams. The records obtained from shaking the building in x- and y-direction were used to determine dynamic properties of the building by averaging the records from the velocity meter's channels oriented in the same direction, and from the difference of these records, the torsion properties were determined. The records based on the resulting displacement-time responses at different stories and their phase difference, mode shapes could be obtained from the ratios of the amplitudes. As resonant frequencies were known at this stage, only responses at these frequencies were actually needed. As a data acquisition system, only six velocity meters (each one has three channel such as, x, y and z) could be connected to the Network Control Center (NCC) used, the locations of the sensors were selected in order to determine the first dominant lateral vibration periods in both directions (x and y) and the torsional period of the building (Fig. 10). The arrangement of the sensors is performed as, two sensors are located at the top floor and also two sensors are located at the third floor of the building at opposite corners. The fifth sensor is located at the corner of the ground floor. In addition, the 6th sensor is located at the entrance of the building. Since, all the sensors have three channels, there was no need to change the direction of the sensors during the tests. The data processing was performed by using View 2002 software performed by Syscom Instruments and developed by Ziegler Consultants, Zurich, Switzerland.

Figure 10: Sensors by Syscom

By applying the method, the modulus of elasticity of the building is obtain as 24855 MPa. The compression strength of the concrete can be obtained according to the Turkish Standard (TS500) using $E_c = 3250\sqrt{f_c'} + 14000 \quad (MPa)$. Where, f_c' is the cylinder characteristic strength of the concrete.

3.4 Seismic Assessment of the Building (Pushover Analysis)

In pushover analysis, the force from the earthquake is simulated as lateral force acting on the mass centre of each floor of the building according to different load patterns (i.e. uniform, triangular, and modal). The lateral force is increased gradually until the target displacement is reached. The target displacements depends on the design of the earthquake behaviour and building capacity. The amount of damage depends on the magnitude of ground shaking.

Relevant terms techniques are given in (ATC 40, 1996) which provided in SAP2000.

3.4.1 The Capacity Spectrum and Demand Spectrum (CSDS)

The capacity spectrum method (CSM) is a nonlinear static analysis method, which graphically compares the overall lateral resistance capacity curve with an earthquake demand curve. The lateral resistance capacity curve can be represented by force-displacement curve from a pushover analysis. On the other hand, earthquake demand is represented by the earthquake response spectrum curve. In addition, the acceleration and displacement spectral values are generated from the corresponding response spectrum for a certain damping ratio. Acceleration versus Displacement Response Spectrum (ADRS) graph shows all the curves on the same graph as shown in Fig. 11. The equivalent damping and natural period of a building increases by increasing the nonlinear deformation. The instantaneous demand point moves to a different response spectrum by changing the damping. Demand spectrum line is the track of the demand points in the ADRS plot.

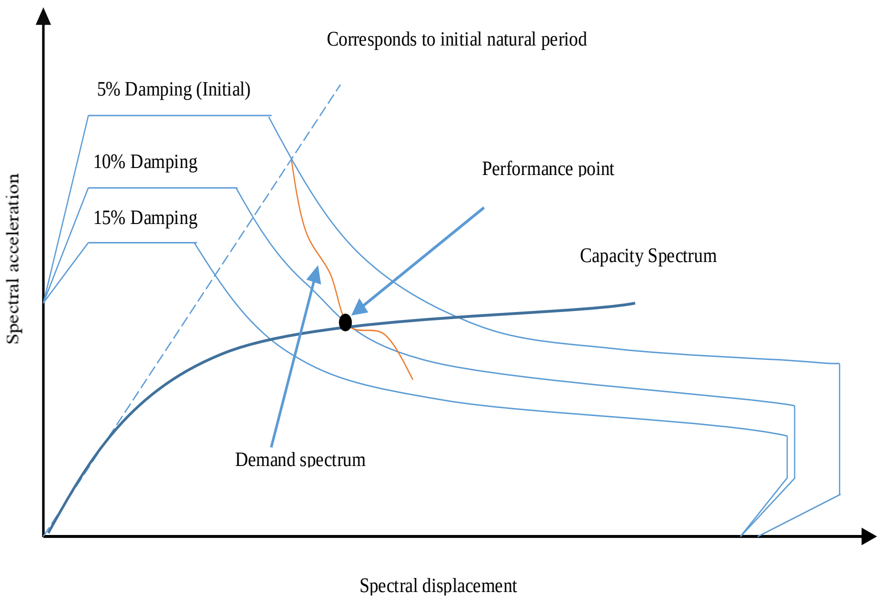


Figure 11: Demand and capacity spectrum (Vijayakumar, *et al.* 2012)

- **Displacement-Based Analysis**

Displacement-based analysis is a procedure, which estimates the expected deformation, lateral displacements, and inelasticity of a structure according to earthquake ground motion. The best example is nonlinear static pushover analysis.

- **Elastic Response Spectrum**

At 5% damped response spectrum for every signal seismic risk level of attention, represents the maximum response of the building, in terms of spectral acceleration (S_a), at any period throughout a shaking as a function of period of vibration T.

- **Performance Point**

The performance point is the point where the capacity spectrum curve crosses the demand spectrum, as shown in Fig. 11. If the performance point exists and the damage state at this point is satisfactory, then the building is considered to be acceptable for the design of lateral forces. To meet performance standards, the structure should be designed to tolerate this degree of loads.

3.4.2 Displacement Coefficient Methodology (FEMA 356)

This is the process of calculating a performance point by using the elastic spectrum from the capacity curve (FEMA 356). The top point of a building is targeted to a displacement (δ_t) which is consistent with the performance point that has been calculated by using:

$$\delta_t = C_0 C_1 C_2 C_3 S_a \frac{T_e^2}{4\pi^2} g \tag{3.1}$$

Where: C_0 is the coefficient correlating the displacement; $C_1$is Modification factor to relate expected maximum inelastic displacements to displacements; C_2 is the modification factor to represent the effect of pinched hysteretic shape, stiffness degradation and strength deterioration on maximum displacement response; C_3 is the modification factor to represent increased displacements due to dynamic P-Δ effects;

S_a is the response spectrum acceleration, at the effective fundamental period and damping ratio of the building in the direction under consideration; T_e is the effective fundamental period of the building in the direction under consideration, and g is the gravitational acceleration.

In addition the target shear (V_t) is calculated from FEMA 356 using the relation given below (EQ 3-10 in FEMA 356)

$$V_t = C_1 C_2 C_3 C_m S_a W \tag{3.2}$$

Where, C_m is the effective mass factor to take in the account the higher mode mass participation effects obtained from Table 3-1 in FEMA 356, and W is the total building weight.

3.4.3 Performance Levels of Building

FEMA 356 provides the limitation of acceptable damage which building can have during earthquake. The limiting condition considers: the building physical damage, life safety hazard because of the damage, and the post-earthquake serviceability of the building:

Yield Point (B)

Yield point is the ultimate capacity that structure can reach. It is also the end of linear elastic force-deformation relationship where the effective of stiffness begins to decrease.

Immediate Occupancy (IO)

The damages caused from the earthquake are very low. Horizontal and vertical loads resisting systems sustains most of their original characteristics. The risk of fatal harm from structural damage is irrelevant.

Life Safety (LS)

The vital structural elements are intact and remains while minor damage during the earthquake might happen. The risk is not life threatening even though rapid repairs to the structure are expected before reoccupation. Sometime, repairing could not be possible economically.

Collapse Prevention Level (CP)

This level of a building performance includes collapse of structural components, also excludes non-structural weakness, except parapet walls attachments.

Primary Elements and Secondary Elements (E)

Primary elements are that required to stand horizontal forces after many inelastic responses to earthquake movements. Secondary elements are that required to sustain gravity forces and some horizontal forces.

There are three structure performance levels based on TEC 2007: immediate occupancy (IO), life safety (LS), and collapse prevention (CP). The performance levels are defined based on the damage that they cause to structure as following:

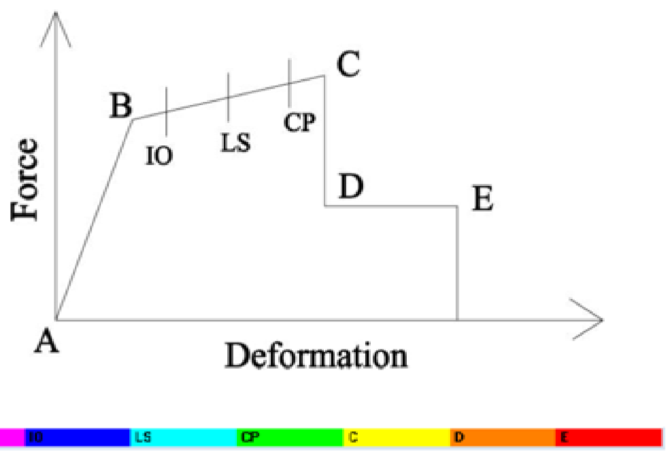
- Immediate occupancy (IO): the maximum allowed damage in each story is 10% of the beam sections. The beam damage must be between IO and LS limit. However, damage of the rest of the structural members should be less than IO.
- Life safety (LS): the limit is 30% of the beams can be between LS and CP except for the secondary members. However, the acceptable damage on columns must be between LS and CP for each story. The effect of shear forces should not stay any lower than 20% of the whole shear forces. For members between LS and CP levels, the total shear forces of columns on top story can be maximum 40% of the related story shear forces of all columns.

The rest of structural members should be lower than LS limits. For brittle members, the member can be assumed to be in the LS limit.

- Collapse prevention (CP): maximum 20% of the beams must be beyond the CP limit except the secondary members. The shear stress should be less than 30% all columns shear capacity when the column passes the IO limit.

3.4.4 Nonlinear Plastic Hinge Properties

This acquires the deformation curve of beam sections, column sections, and masonry by use of FEMA 356. The curve of shear force defamation are taken from details of reinforcement for components of the building. By using (section designer) in the SAP2000 model, all non-linear properties for columns and beams are evaluated. The shear hinge (V2) and the flexural hinge (M3) are applied to two ended beam. The hinge interaction of (P-M2-M3) are applied to upper and lower ended columns.



A to B	Elastic state
B to IO	below immediate occupancy
IO to LS	between immediate occupancy and life safety
IO to LS	between immediate occupancy and life safety
LS to CP	between life safety to collapse prevention
CP to C	between collapse prevention and ultimate capacity
C to D	between ultimate capacity and residual strength
D to E	between residual strength and collapse and greater than E – collapse

Figure 12: Different stages and definition of plastic hinge

The axial hinge (P) is applied to the stone elements. Then, pushover analysis has been controlled for the roof structures. As a result of position of hinges in several steps can be performed for hinges stages as shown in Fig. 12.

3.4.5 Pushover Analysis Applications in SAP2000

In order to apply the model in SAP2000 building properties and parameters must be collected. As stated before the modulus of elasticity of the concrete is determined during model calibration. The existing compressive strength of concrete is determined by the formula provided in TS500 and the existing steel reinforcement tensile strength is determined from the project of the building. The other seismic parameters such as seismic zone, factors, and building & soil condition are collected as shown in Table 1. (Özşahin, E., and Değerliyurt, M. 2013).

Table 1: Existing properties and code parameters of the building

Existing Building Properties	
Knowledge level	Moderate
Modulus of Elasticity	24,855 MPa
Knowledge level factor	0.9
Existing concrete compressive strength	9.5 MPa
Existing steel reinforcement tensile strength	220 MPa
Earthquake Code Parameters	
Seismic Zone	1
Seismic Zone Factor (A_o)	0.4
Building Importance Factor (I)	1.0
Soil Class	Z4
Live Load Participation Factor	0.6

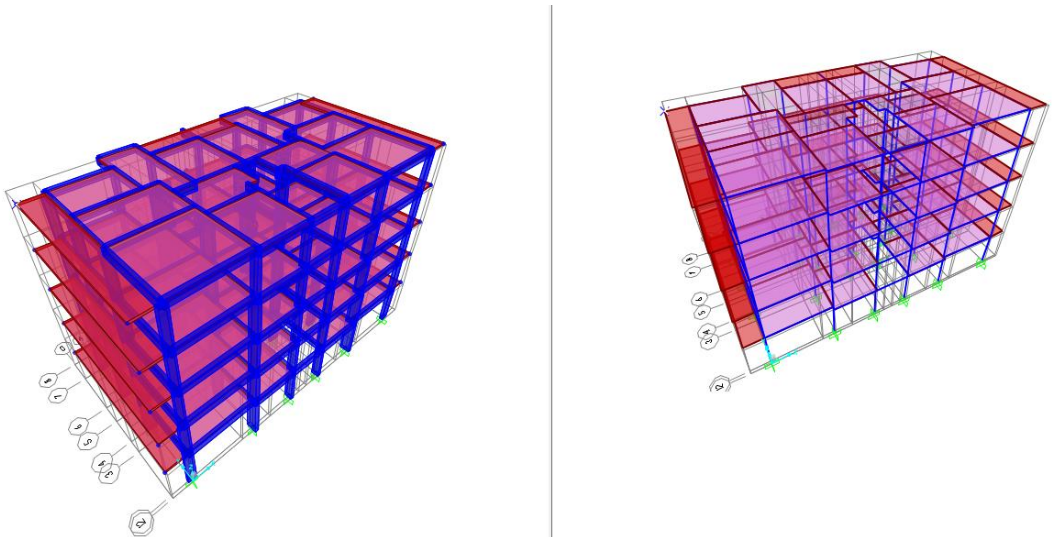


Figure 13: Three dimensional model of the building

- The basic 3D model of the building was created from original plan as shown Fig. 13.
- Definition of acceptance criteria and properties for the model.

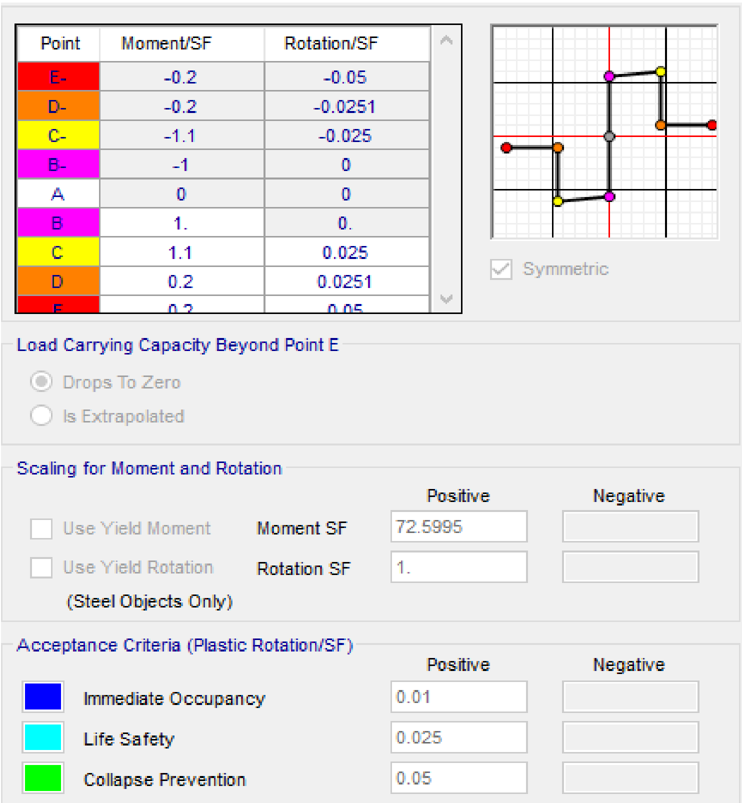


Figure 14: Default hinge properties of the frame

- The software offers many hinge properties that are undertaken for RC concrete elements in order to define during pushover analysis (ATC-40) as shown in Fig. 14:

- ✓ The pushover hinges on the model into SAP2000 were positioned by selecting the frame members then assigning them more hinge properties at hinge positions.
- ✓ The pushover cases were defined in SAP2000, where more than one pushover load case can be run in the same analysis. In addition, a pushover load case can be started from the final conditions of another pushover load case, which was before run in the same analysis. Classically, the first pushover load case is used to put on gravity load and then following lateral pushover load cases are quantified to start from the final conditions of the gravity pushover. Pushover load cases can be force measured, that is, pushed to a certain defined force level, under control the displacement that is, pushed to a specified displacement.
- ✓ The pushover curve is show, in the system when running the model. A table, which gives the coordination of each step of the pushover, the curve summarizes the amount of hinges in each state as defined, between IO and LS or between D and E).
- ✓ Next, the capacity spectrum curve displays. The performance point for a given set of values is defined by the intersection of the capacity curve and the single demand spectrum curve .
- ✓ The pushover displaced shape and sequence of hinge formation are reviewed on a step-by-step basis. Hinges appear when they yield and they are colour coded based on their state.

3.5 Retrofit the Building Using Steel Braced Frames

As mentioned in the previous problem statement, the need to rehabilitate the building with the least destructive intervention possible, also the process to be economic and accessibility for the existing building. Using steel braced frames system to strengthen the inadequate RC building; a diagonal bracing is the best solution for strengthening as well as stiffening the building for resisting lateral loads. To achieve the retrofitting objectives that are ranging from drift control to collapse prevention, The designer can control the force direction into the structure and the strength, analysis were performed to understanding the behaviour of a braced frame (Badoux, M., and Jirsa, J. O. 1990).

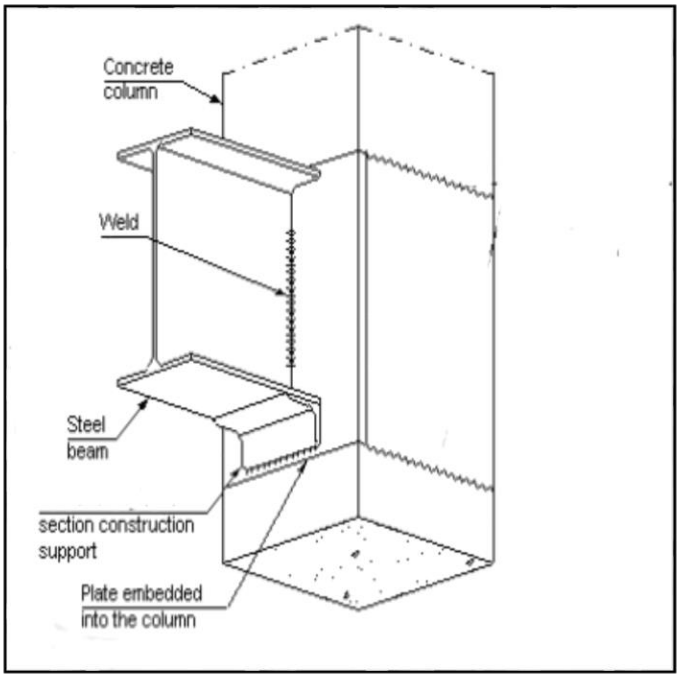


Figure 15: Connect the steel beam to column concrete by using plate around the column

During the installation period a problem arose, initially the ideal solution was to directly attach the steel braced frames to the existing skeleton of the structure, because the existing building has balconies extruding 1.5 m in the edges. In result, the steel braced frames had to be slightly shifted by steel beams to the end of the cantilevered balcony. After that, the frames had been modelled on SAP2000 together with concrete frames and performance analysis is done by using ATC-40 method in order to evaluated the results. The steel frames are connected to concrete structure according to the detail given in Fig. 15 (Ricles, J. *et al.* 2004).

Chapter 4

RESULTS AND DISCUSSION

4.1 Introduction

In the previous chapter, the 3D model of the building is created and the material properties are determined. This chapter presents seismic assessment and retrofitting results of the existing building. The analysis is done using capacity spectrum curves and demand curves of the earthquake. The performance points, obtained from CSM curves, are used to estimate the hinges state. Building retrofitting will be based on the hinges state. Two eternal steel brace frames are used to retrofit the building. The nominated frame is selected based on the stiffness that provides to the building and minimum amount of material used to design it. After retrofitting, the steel brace performance is simulated using CSM. All the modelling and analysis are carried over on SAP2000.

4.2 Performance of the Building before Retrofitting

The building is evaluated for both directions (X and Y) without any retrofitting by using ATC-40 capacity spectrum method (CSM). The method obtain target displacement and shear force curve, which dependents on the level of expected seismic activity in Antakya, the soil properties of the site, effective mass in the first mode, and amount of viscous damping capacity of the existing building. Target displacement and shear force curve is obtained by transferring the shear force to spectral acceleration and the displacement to the spectral displacement the performance curve can be compared with demand curve. Performance point was obtained according to capacity spectrum ACT-40 in X-direction as a shear force equals to 2303 kN and displacement equals to 0.156 m, and in Y-direction as a shear force equals to 2150 kN, and displacement equals to 0.167 m as shown in Figs. 16 and 17.

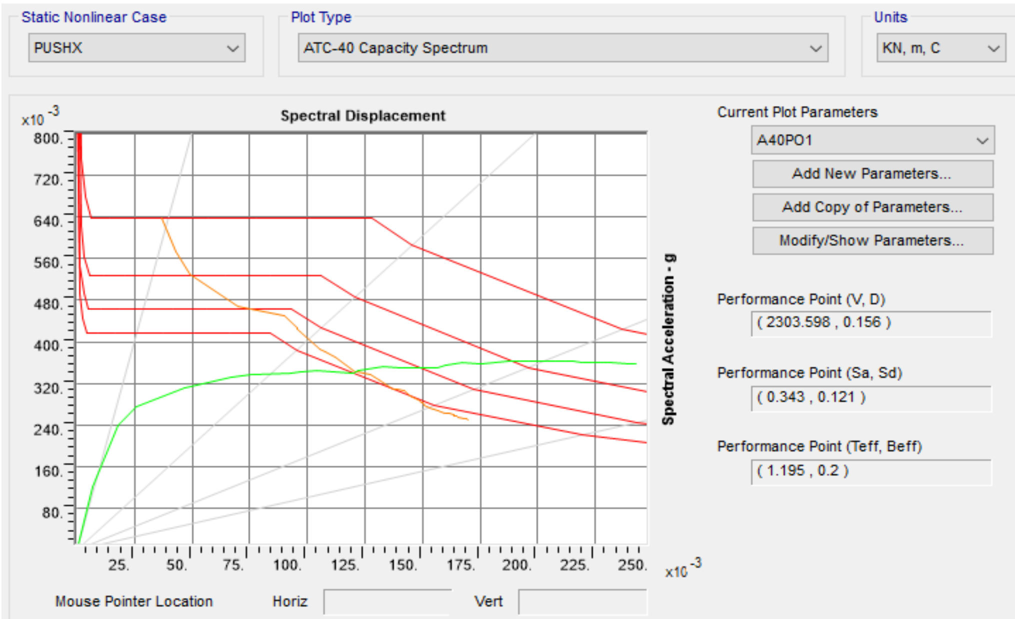


Figure 16: Demand and capacity spectrum in X-direction before retrofitting

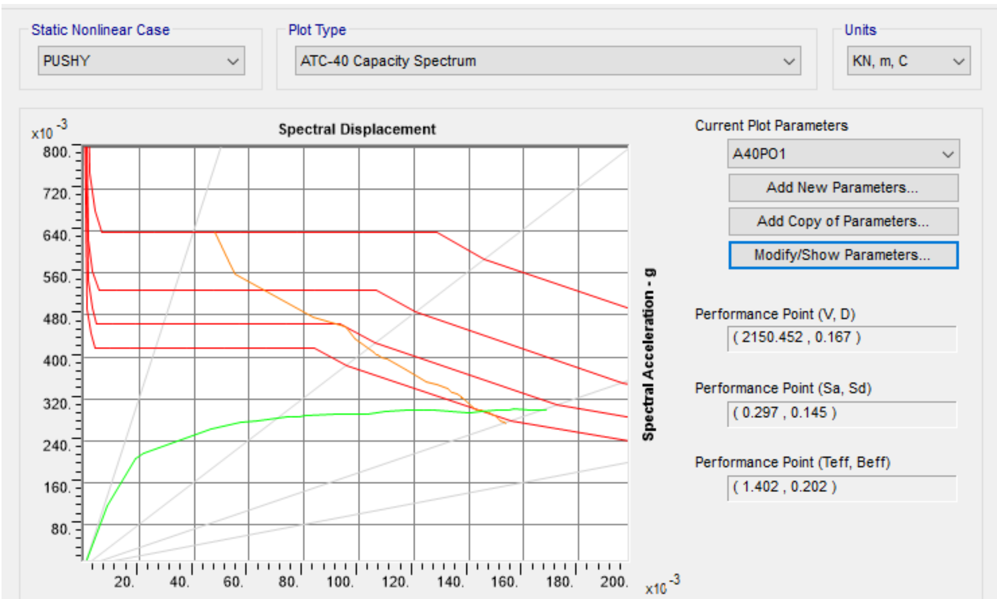


Figure 17: Demand and capacity spectrum in Y-direction before retrofitting

After getting the performances point for each direction, SAP 2000 provides table to evaluate the hinges performance for each step. The performance point is compared with base shear force from Table 2. In this case the performance point is between step 10 and 11. As conservative design, step 11 has been chosen to be the critical point which is still in immediate occupancy level. The same procedure has been done on the Y direction and step 25 is selected as the critical point, which was in collapse prevention level as shown Table 3.

Table 2: Plastic hinges state in X-direction before retrofitting

LoadCase	Step	Displacement	BaseForce	AtoB	BtoIO	IOtoLS	LStoCP	CPtoC	CtoD	DtoE	BeyondE	Total
Text	Unitless	m	KN	Unitless	Unitless	Unitless	Unitless	Unitless	Unitless	Unitless	Unitless	Unitless
PUSHX	0	0.000046	0	750	0	0	0	0	0	0	0	750
PUSHX	1	0.009675	759.883	749	1	0	0	0	0	0	0	750
PUSHX	2	0.023581	1530.891	687	63	0	0	0	0	0	0	750
PUSHX	3	0.032415	1787.539	644	106	0	0	0	0	0	0	750
PUSHX	4	0.058416	2070.51	575	175	0	0	0	0	0	0	750
PUSHX	5	0.083868	2219.141	530	206	14	0	0	0	0	0	750
PUSHX	6	0.094874	2251.254	517	208	25	0	0	0	0	0	750
PUSHX	7	0.095712	2251.352	516	207	27	0	0	0	0	0	750
PUSHX	8	0.096044	2252.491	516	207	27	0	0	0	0	0	750
PUSHX	9	0.117309	2287.654	494	220	36	0	0	0	0	0	750
PUSHX	10	0.122625	2301.358	493	220	37	0	0	0	0	0	750
PUSHX	11	0.133258	2311.176	485	207	58	0	0	0	0	0	750

Table 3: Plastic hinges state in Y-direction before retrofitting

LoadCase	Step	Displacement	BaseForce	AtoB	BtoIO	IOtoLS	LStoCP	CPtoC	CtoD	DtoE	BeyondE	Total
Text	Unitless	m	KN	Unitless	Unitless	Unitless	Unitless	Unitless	Unitless	Unitless	Unitless	Unitless
PUSHY	0	0.000246	0	750	0	0	0	0	0	0	0	750
PUSHY	1	0.011419	756.884	748	2	0	0	0	0	0	0	750
PUSHY	2	0.023523	1372.433	690	60	0	0	0	0	0	0	750
PUSHY	3	0.026803	1459.995	654	96	0	0	0	0	0	0	750
PUSHY	4	0.055408	1828.23	590	160	0	0	0	0	0	0	750
PUSHY	5	0.067805	1927.812	574	157	19	0	0	0	0	0	750
PUSHY	6	0.07437	1956.048	573	147	30	0	0	0	0	0	750
PUSHY	7	0.086598	2020.616	555	155	40	0	0	0	0	0	750
PUSHY	8	0.08848	2024.764	554	154	42	0	0	0	0	0	750
PUSHY	9	0.088902	2027.238	554	154	42	0	0	0	0	0	750
PUSHY	10	0.091662	2034.172	554	151	45	0	0	0	0	0	750
PUSHY	11	0.094559	2047.032	551	148	51	0	0	0	0	0	750
PUSHY	12	0.106419	2080.049	542	151	57	0	0	0	0	0	750
PUSHY	13	0.120147	2083.717	533	139	78	0	0	0	0	0	750
PUSHY	14	0.127011	2115.322	522	138	90	0	0	0	0	0	750
PUSHY	15	0.135591	2139.421	515	134	101	0	0	0	0	0	750
PUSHY	16	0.139023	2134.934	515	134	101	0	0	0	0	0	750
PUSHY	17	0.139452	2137.056	515	134	101	0	0	0	0	0	750
PUSHY	18	0.142884	2147.06	514	131	105	0	0	0	0	0	750
PUSHY	19	0.146316	2144.905	513	130	107	0	0	0	0	0	750
PUSHY	20	0.146424	2145.226	513	130	107	0	0	0	0	0	750
PUSHY	21	0.146853	2148.198	513	130	107	0	0	0	0	0	750
PUSHY	22	0.147711	2151.819	512	131	107	0	0	0	0	0	750

PUSHY	23	0.161439	2117.319	511	123	113	0	0	3	0	0	750
PUSHY	24	0.165729	2147.153	510	121	111	0	0	8	0	0	750
PUSHY	25	0.171735	2164.644	506	122	111	0	0	11	0	0	750

Figs. 18 and 19 show the hinge levels details during pushover analysis in both directions.

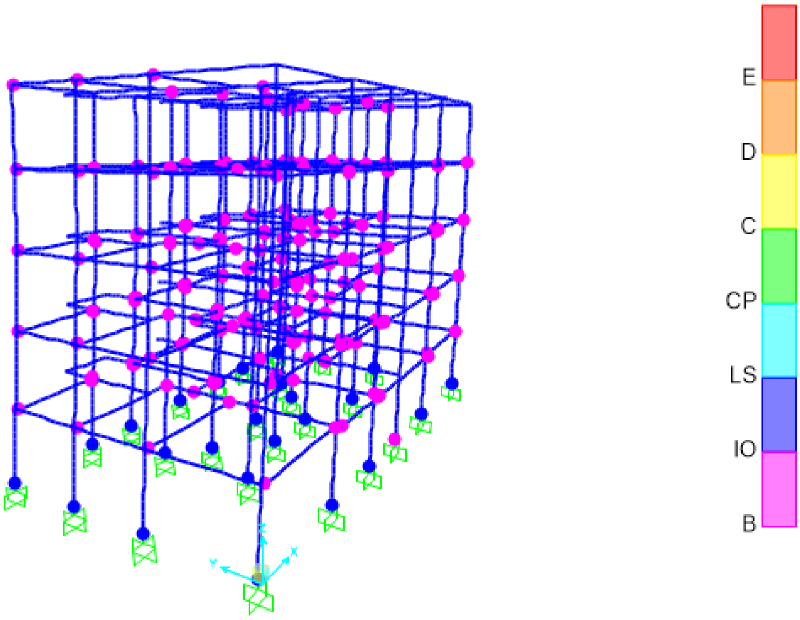


Figure 18: Hinges state in step 11 in X-direction

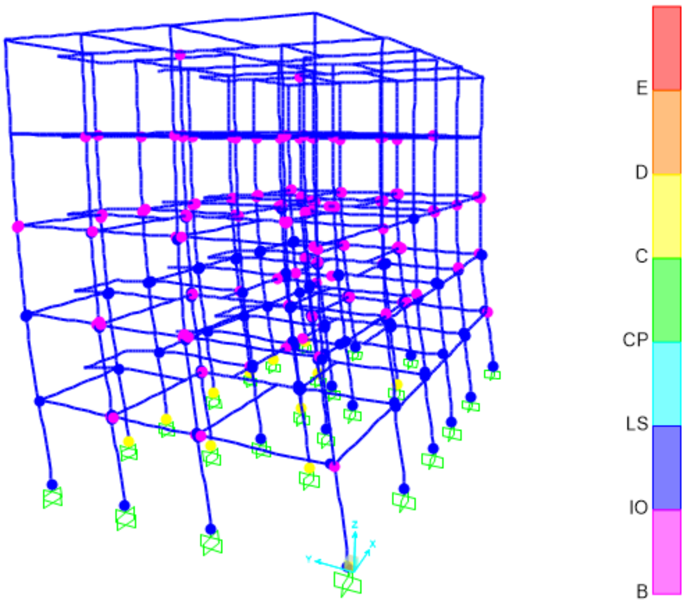


Figure 19: Hinges states in step 25 Y-direction

After performing pushover analysis and the Roof Displacement – Base Shear curve (Capacity Curve) obtained, ATC-40 for selected performance level has to be determined. The strategy depicted in the TEC 2007 was mentioned in Chapter 3. As indicated by the TEC 2007 earthquake code, onto the results of the calculations regarding all earthquakes applied in any floors, at most 10 % of the beams exceed the Significant Damage Zone and all other load-bearing components remain in the Minimum Damage Zone. Such buildings can be agreed to be in the Immediate Occupancy Level provided that the brittle damaged components, if any, are strengthened. For columns, The performance should be satisfied the immediate occupancy according to TEC 2007 and the collapse should not observed on columns which are bearing 30% of the shear force. Tables 4-7 summarise the results for nonlinear analysis procedure which was offered. As it can be observed, the existing system of the building under consideration cannot satisfy the performance levels because of 11 columns are in collapse stage (Table 7).

Table 4: Immediate occupancy performance level in X-direction for beams in existing structure

Story	Total number of beams	Beam not satisfying performance Level		
		Number	ratio	check
Second	34	0	0.00	<10%
First	34	0	0.00	<10%
Ground	34	0	0.00	<10%

Table 5: Immediate occupancy performance level in X-direction for columns in existing structure

Story	Total number of columns	Column not satisfying performance Level		
		LS	CP	>C
Second	26	0	0	0
First	26	0	0	0
Ground	26	0	0	0

Table 6: Immediate occupancy performance level in Y-direction for beams in existing structure

Story	Total number of beams	Beam not satisfying performance Level		
		Number	ratio	check
Second	34	0	0.00	<10%
First	34	0	0.00	<10%
Ground	34	0	0.00	<10%

Table 7: Immediate occupancy performance level in Y-direction for columns in existing structure

Story	Total number of columns	Columns not satisfying performance Level		
		LS	CP	>C
Second	26	0	0	0
First	26	0	0	0
Ground	26	0	0	**11**

4.3 Performance of the Building After Retrofitting

4.3.1 Braced Steel Frames

One general solution to the low performance of an existing building is adding braced steel frames on the outer frames in the weak direction. Steel frames were the most mutual technique used to strength the flexural, and shear strength of existing building. The building's ductility and stiffness are increasing while the cost of retrofitting is decreasing. Two types of braced steel frames are used in order to obtain an optimized frame in terms of ductility and cost. The noticed weak direction of the existing RC building has been retrofitted by using two different types of braced steel frames. First, frame-I is shaped as V and located, as shown in Fig. 20. Moreover, the frame has bracing section as HE160A, beams section as HE120A and columns section as HE180A. The details of the sections are given in Table 8. The sections are selected according to several pushover runs to obtain safe and also minimum section for degreasing the cost.

Table 8: Sections of frame-I

Section	Dimensions (m)	
Bracing HE160A	Outside height	0.152
	Top flange width	0.16
	Top flange thickness	0.0085
	Wed thickness	0.006
	Bottom flange width	0.16
	Bottom flange thickness	0.0085
Beam HE120A	Outside height	0.114
	Top flange width	0.12
	Top flange thickness	0.0080
	Wed thickness	0.005
	Bottom flange width	0.12
	Bottom flange thickness	0.008
Column HE180A	Outside height	0.171
	Top flange width	0.18
	Top flange thickness	0.0095
	Wed thickness	0.006
	Bottom flange width	0.18
	Bottom flange thickness	0.0095

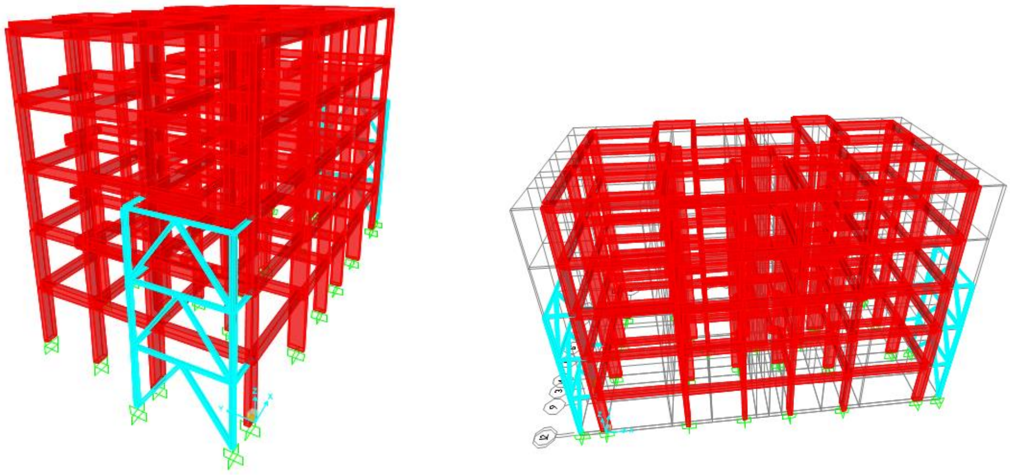


Figure 20: Steel frame used for retrofitting (Frame-I)

Second, frame-II is shaped X as shown in Fig.21. The frame has bracing section as HE160A, beam section as HE120A and column section HE180A.

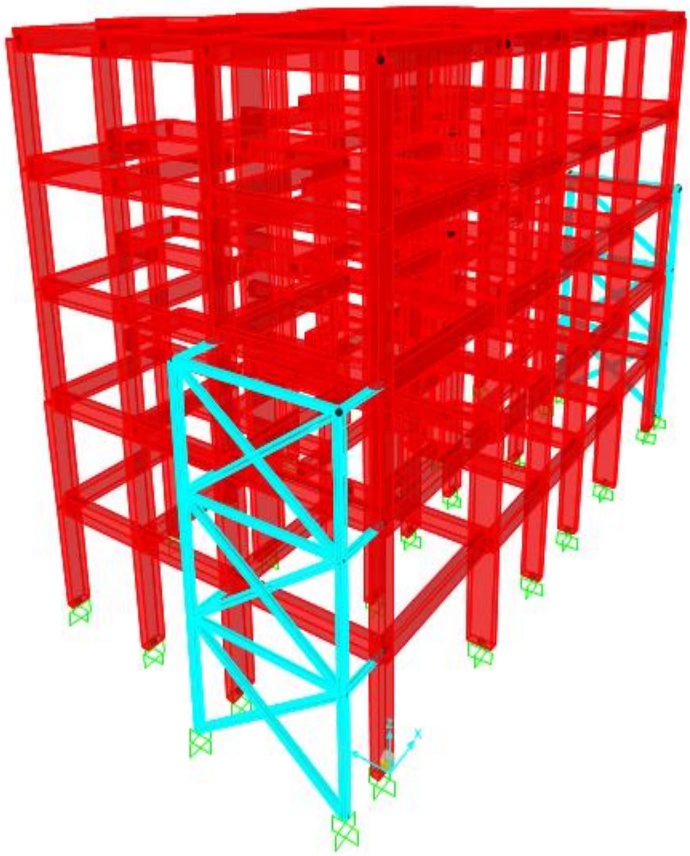


Figure 21: Steel frame used for retrofitting (Frame-II)

4.3.2 Performance in X-Direction after Retrofitting

After retrofitting In X-direction, in the capacity spectrum curve the load is increased from 2303 kN to 2516 kN and the displacement is decreased from 0.156 m to 0.109 m as shown in Fig. 22. Frames-I has a capability to take higher shear force and displacement than frame-II as shown in Table 9. There is a slight increase in the performance of the building in X-direction. Because the steel braced frame was installed in Y-direction.

Table 9: Performance points for both of frame (X-Direction)

	Shear Force (kN)	Displacement (m)	Weight (kN)
Frame I	2516	0.109	81.432
Frame II	2495	0.10	91.906

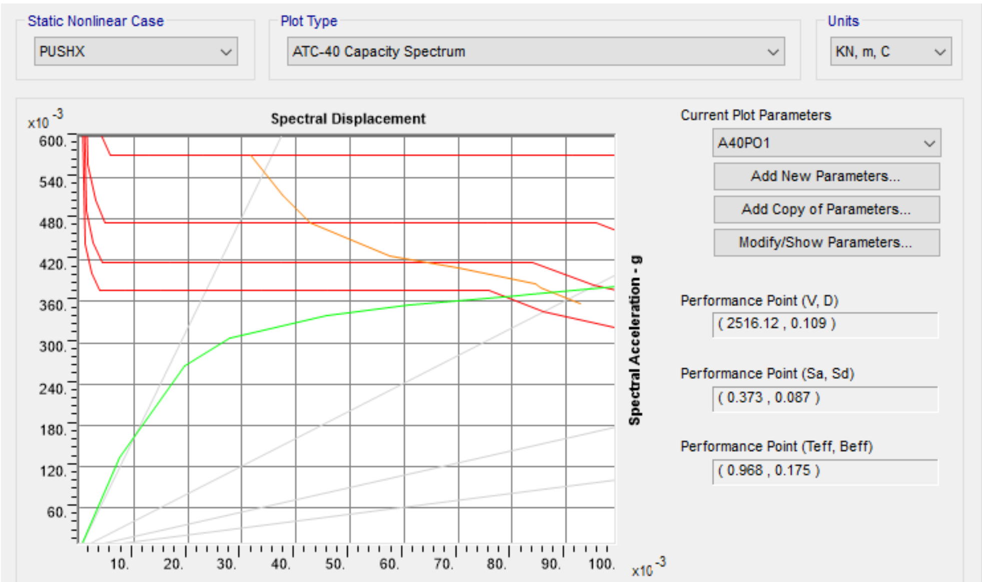


Figure 22: Demand and capacity spectrum for the building in X-direction after retrofitting

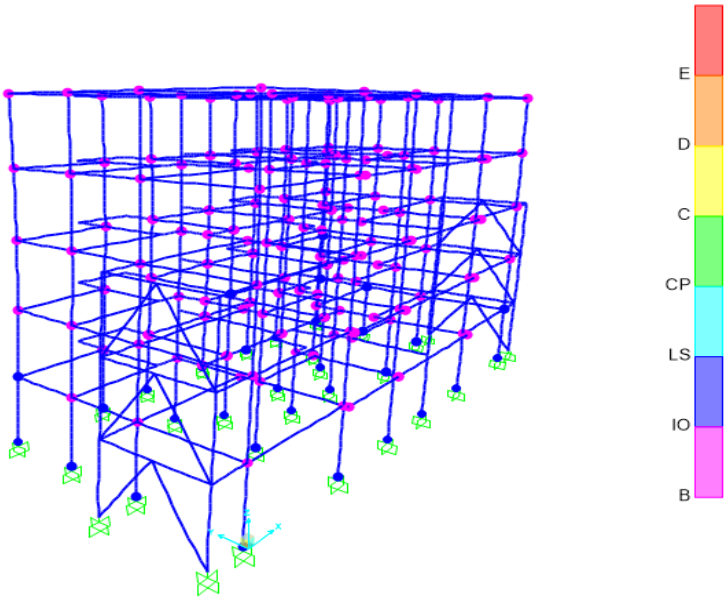


Figure 23: Hinges states in step 8 in X direction

Fig. 23 shows hinges state for each member of the model. As seen from the figure, some hinges are in yield level and the other are in immediate occupancy.

Table 10: Plastic hinges state in X-direction after retrofitting

LoadCase	**Step**	**Displacement**	**BaseForce**	**AtoB**	**BtoIO**	**IOtoLS**	**LStoCP**	**CPtoC**	**CtoD**	**DtoE**	**BeyondE**	**Total**
Text	Unitless	m	KN	Unitless	Unitless	Unitless	Unitless	Unitless	Unitless	Unitless	Unitless	Unitless
PUSHX	0	0.000045	0	834	0	0	0	0	0	0	0	834
PUSHX	1	0.010124	844.819	833	1	0	0	0	0	0	0	834
PUSHX	2	0.025322	1720.91	765	69	0	0	0	0	0	0	834
PUSHX	3	0.034903	2004.148	723	111	0	0	0	0	0	0	834
PUSHX	4	0.056829	2262.84	668	166	0	0	0	0	0	0	834
PUSHX	5	0.075537	2378.594	634	194	6	0	0	0	0	0	834
PUSHX	6	0.100606	2489.176	596	205	33	0	0	0	0	0	834
PUSHX	7	0.103522	2497.63	593	208	33	0	0	0	0	0	834
PUSHX	8	0.125018	2572.584	578	217	39	0	0	0	0	0	834
PUSHX	9	0.125068	2572.689	578	217	39	0	0	0	0	0	834
PUSHX	10	0.125097	2572.81	578	217	39	0	0	0	0	0	834

4.3.3 Performance in Y-Direction after Retrofitting

After retrofitting in Y-direction in the capacity spectrum curve the load is increased from 2150 kN to 2819 kN. In this direction, as it expected, the capacity spectrum curve has significant increase in performance point. The performance point is shown in Fig. 24 and Table 10. In both directions X and Y, the hinges state is satisfied as immediate occupancy as in Tables 11 and 12. As a result, frame-I is more efficient for retrofitting because of less weight and consequently less cost. On the other hand, the performance points are same for the both frames.

Table 11: Performance points for both of frames (Y-Direction)

	Share force (kN)	Displacement (m)	Weight (kN)
Frame I	2819.428	0.12	81.432
Frame II	2758.549	0.111	91.906

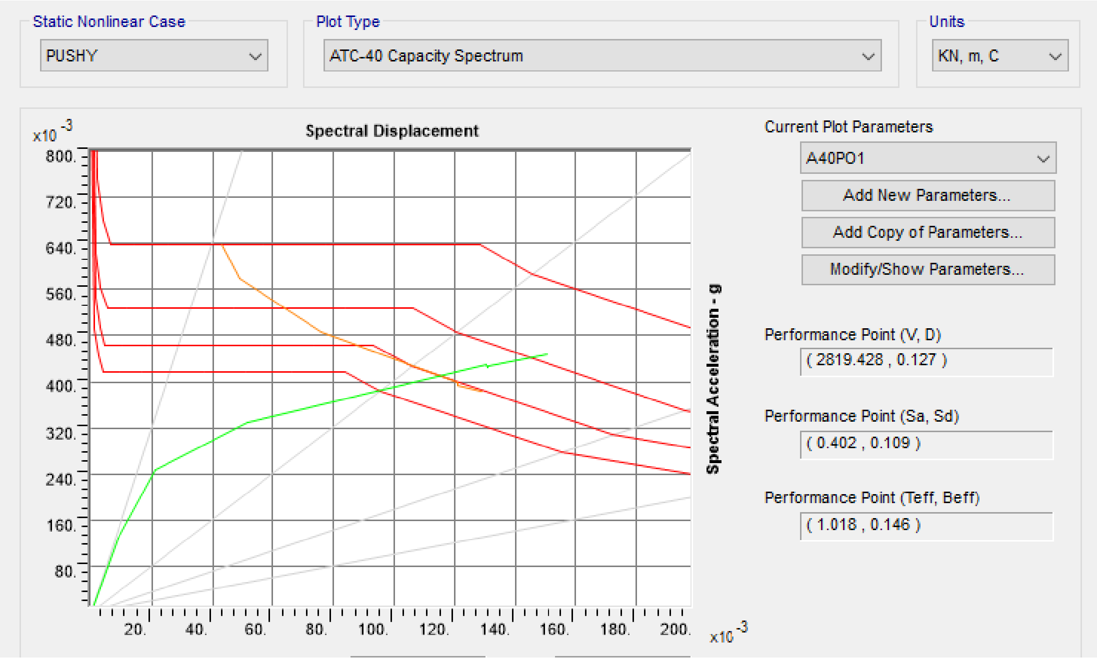


Figure 24: Demand and Capacity Spectrum for frames in Y-direction after retrofitting

Table 12: Plastic hinges state in Y-direction after retrofitting

LoadCase	Step	Displacement	BaseForce	AtoB	BtoIO	IOtoLS	LStoCP	CPtoC	CtoD	DtoE	BeyondE	Total
Text	Unitless	m	KN	Unitless	Unitless	Unitless	Unitless	Unitless	Unitless	Unitless	Unitless	Unitless
PUSHY	0	0.000225	0	834	0	0	0	0	0	0	0	834
PUSHY	1	0.011341	849.966	832	2	0	0	0	0	0	0	834
PUSHY	2	0.025544	1650.713	763	71	0	0	0	0	0	0	834
PUSHY	3	0.059958	2294.165	661	169	4	0	0	0	0	0	834
PUSHY	4	0.153016	2322.739	584	141	109	0	0	0	0	0	834
PUSHY	5	0.153016	2990.667	584	141	109	0	0	0	0	0	834
PUSHY	6	0.15343	2997.657	583	141	109	0	0	0	1	0	834
PUSHY	7	0.176873	3140.049	574	131	126	0	0	2	1	0	834

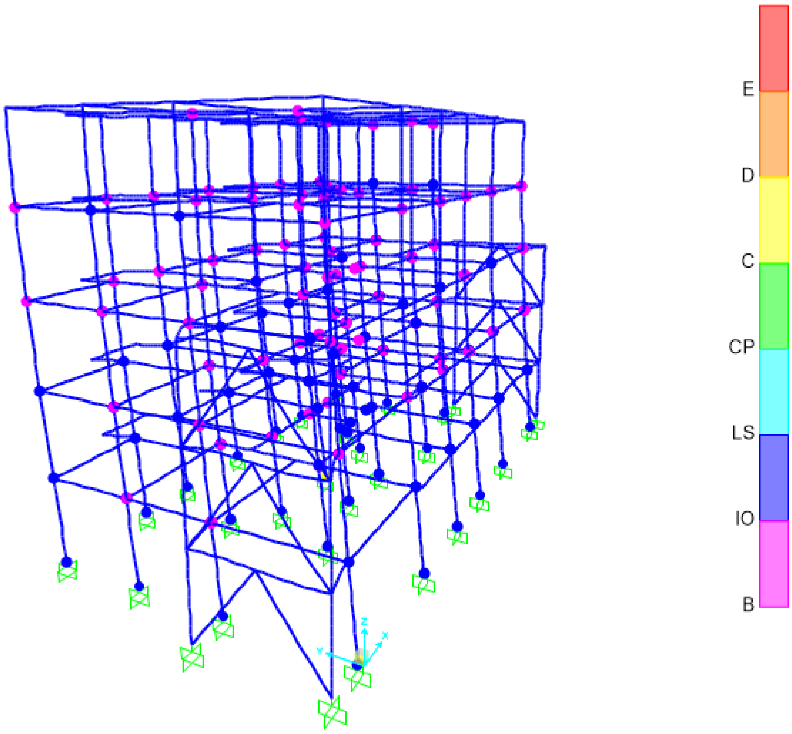


Figure 25: Hinges states in step 8 in X direction

4.4 Comparative Demand and Capacity Spectrum for X and Y-Direction before and after Retrofitting

As a result of the analysis, Frame-I is suggested to be more economical than the Frame-II (Table 13). However the demand curve showed a significant improvement in both x and y directions as shown in Figs. 26 and 27.

The performance point was obtained by using the capacity curves and the procedure is defined in the TEC 2007. Performance points are found at a shear force 2572 kN at displacement equals to 0.125 m in X direction (Table 10), and at a shear force 2990 kN at displacement equals to 0.153 m in the Y direction (Table 11), respectively.

Table 13: Cost price of braced frames

	Steel (ton)	Unite price ($)	Total cost ($)
Frame I	8.172	1200	9,800
Frame II	9.223	1200	11,000

Unit price = Steel Material + Installation

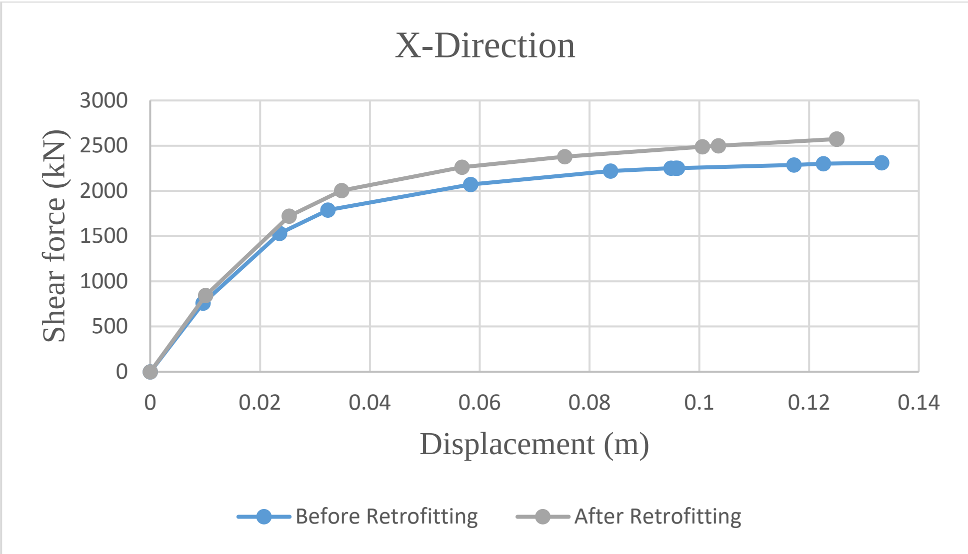


Figure 26: Capacity spectrum curve in X-direction before and after retrofitting

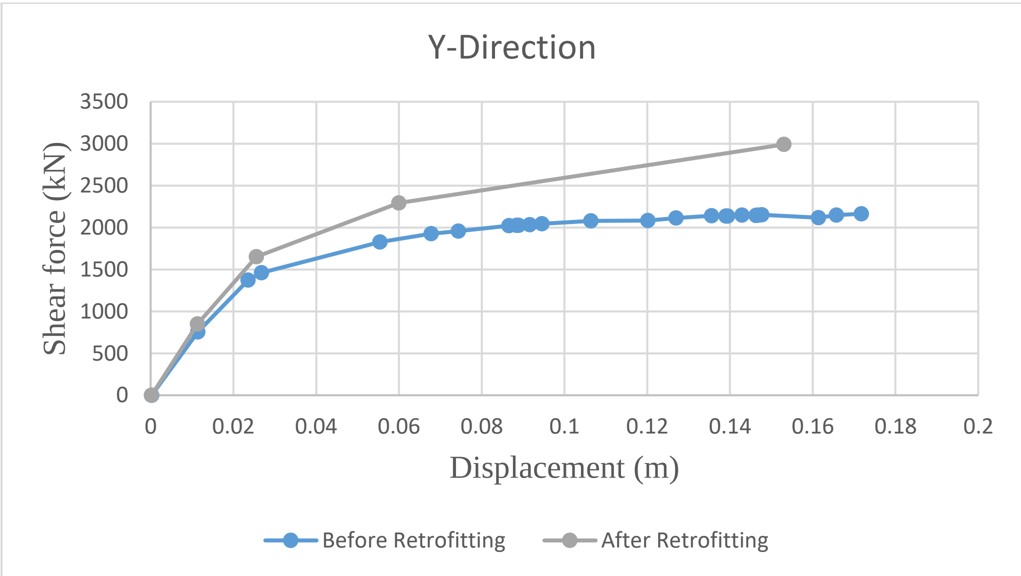


Figure 27: Capacity spectrum curve in Y-direction before and after retrofitting

Chapter 5

CONCLUSION AND RECOMMENDATIONS

5.1 Summary

The aim of this study is to provide a procedure for building performance evaluation and retrofitting that is fast, cheap, and reliable. The main parameters that have been considered on this study are: retrofitting with minimum intervention with building residents, minimum required damage to the building, and minimum retrofitting cost. The procedure has been tested on a residential apartment, which was built in 1988. The building is a five storeys RC apartment, located in Antakya, Turkey. The study has been conducted by utilizing distinctive codes and diverse investigation methodology in this examination.

The material properties of the building were obtained by coupling real vibration record results with building model in SAP2000. The material properties and other parameters are used to evaluate building performance. The method which has been used for building performance evaluation is a pushover analysis (nonlinear elastic analysis). The performance point is calculated before and after the retrofitting by using CSM. The performance point is used to assess the structural system. Based on the building performance, some hinges reached the collapse level. Therefore, the building structure has been supported with two different external steel bracing frames. The two external steel bracing frames have been compared in terms of building performance and material cost.

5.2 Conclusion

This study is conducted to provide fast, cheap, and reliable procedure for evaluating and retrofitting an existence building. The main conclusions are:

- Coupling real vibration records with building model in SAP2000 was successfully applied on existing building. The modulus of elasticity is obtained as 24855 MPa by using model calibration with real vibration data. The main advantages of this method are:
    - It is non-destructive method which requires no damage to the building with minimum intervention with building residences.
    - The modulus of elasticity represents the overall value of the building compared with other methods which use local material properties.
    - It counts for all of the building structure defects such as construction errors.
- Structure evolution before retrofitting showed that the building is near collapse based on Turkish Earthquake Code (TEC 2007). The results showed that performance level in X-Direction is in an immediate occupancy level and in Y-Direction 11 hinges are in collapse level. For this reason, the retrofitting was suggested.
- Two frame shapes of external steel bracing are used namely (X and V shape). The two frames are compared in terms of building performance and cost. The V shaped showed the highest performance and provided minimum material cost. The main advantage of using external bracing is that it requires minimum interference with building residents, and minimum building damage.

- The same evaluation method was applied after retrofitting. The results showed that all-plastic hinges performance level in Y-direction increased to immediate occupancy level as required to Turkish Earthquake Code (TEC 2007).
- The study showed that the external steel bracing does not need to be for all of the floors but up to the third floor is enough to rehabilitate the existing structure which can lowers the cost in terms of steel material and workmanship up to 30%. In addition, using V bracing frame instead of X bracing frame reduces the amount of steel material used in this study by 9%.

5.3 Recommendations for Further Studies

As it expected that, this investigation will be a perspective to the other private structures of Antakya city or some other cities has similar building stock in Turkey. The work will be done for retrofitting of the existing buildings; the proposed technique would not need any evacuation of the residence from the apartments. Because of this advantage, most of the other buildings should be investigated by the same methodology to determine the existing performance level. If they are not satisfying the required performance level according to Turkish Earthquake Code 2007, the buildings should be strengthened with this proposed simple and cost efficient method.

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APPENDIX

Appendix A: Data Collection of Studied Building

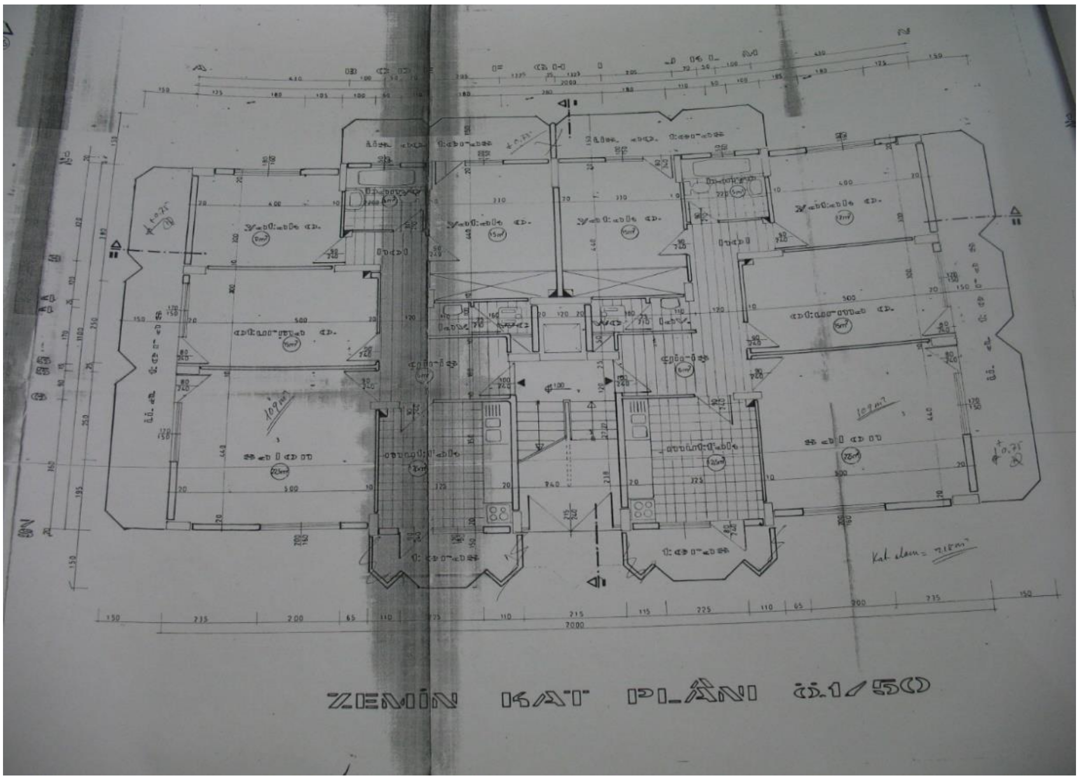

Figure A1: Plan of studied building

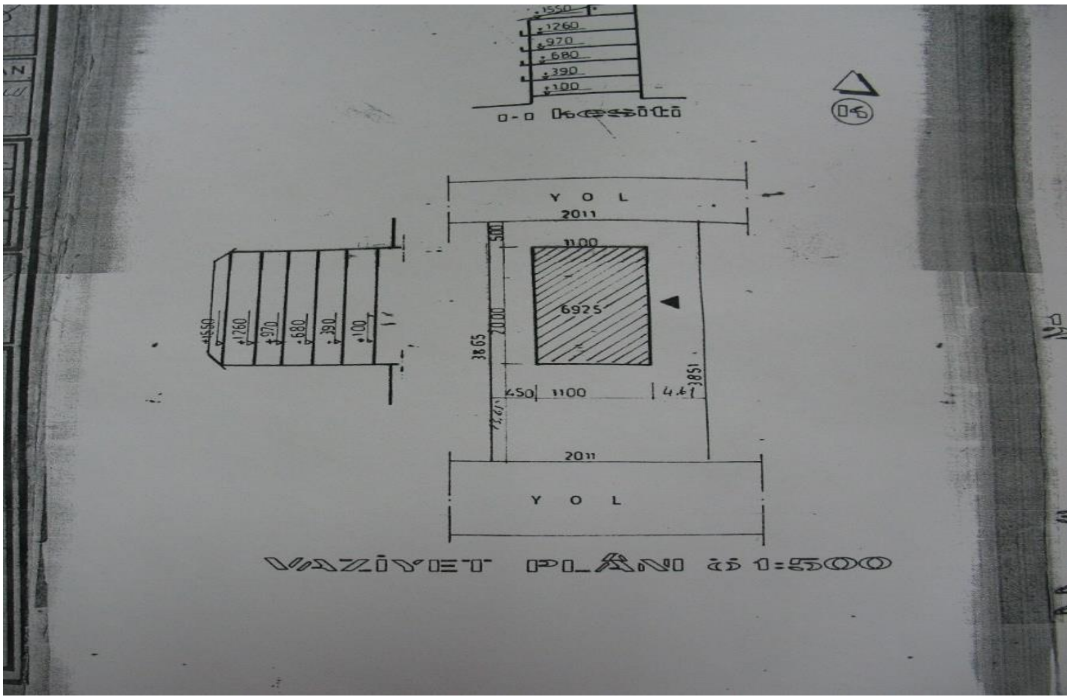

Figure A2: Site plan of the building

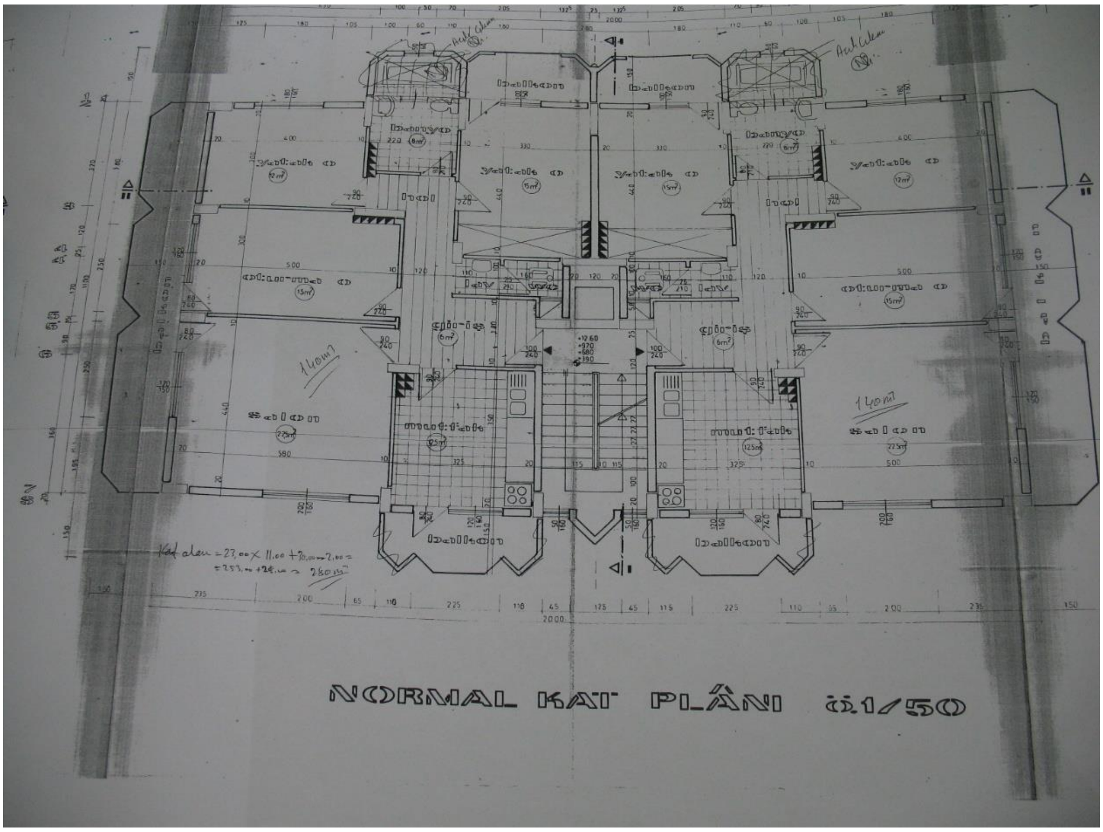

Figure A3: Grand floor of the studied building

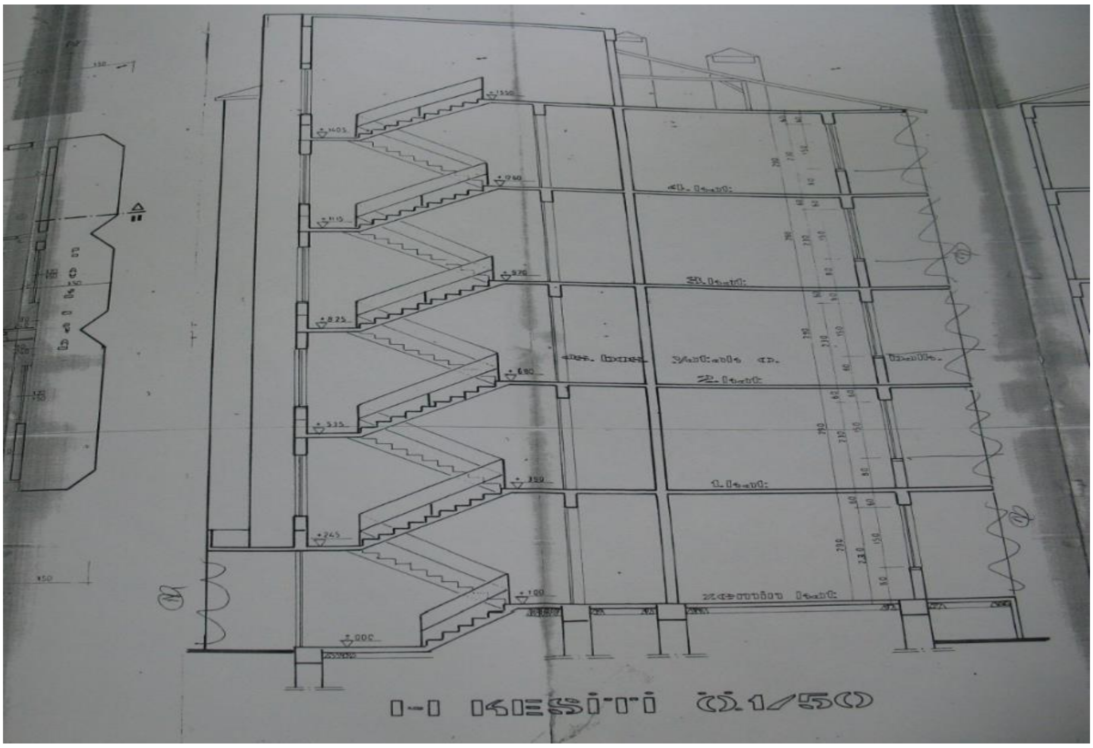
Figure A4: Elevation section X-direction

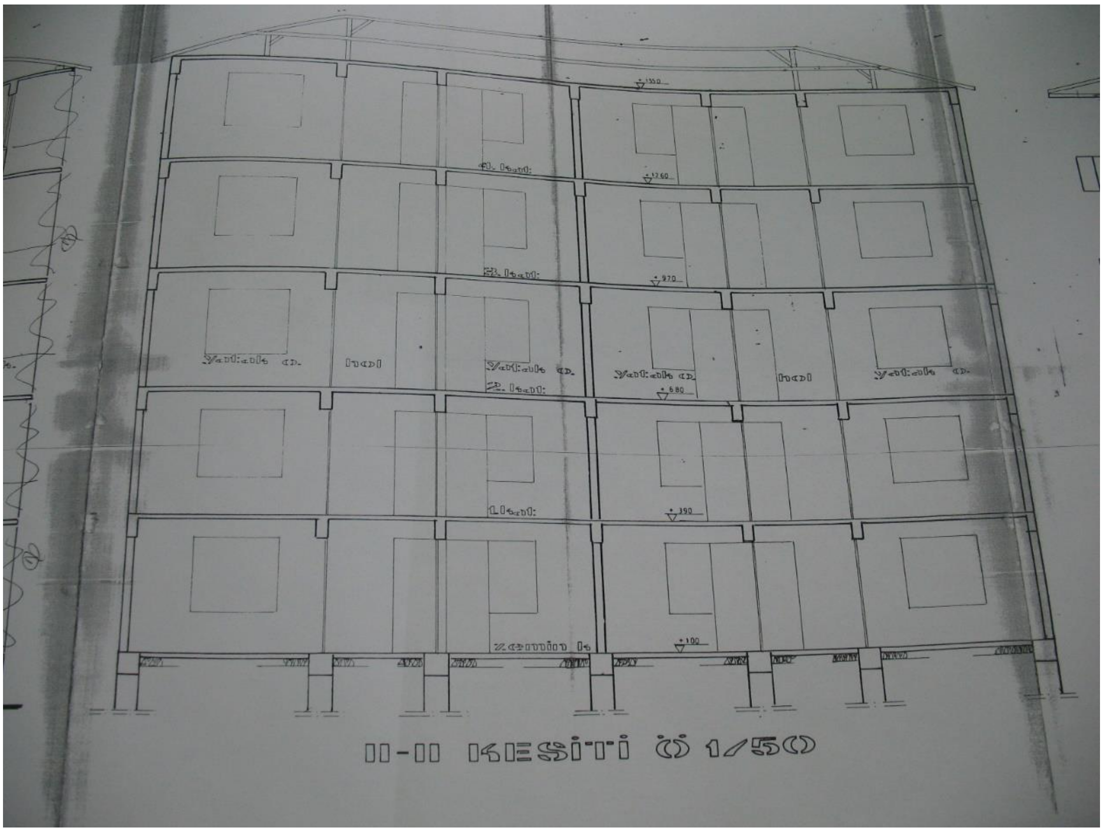

Figure A5: Elevation section at Y-direction

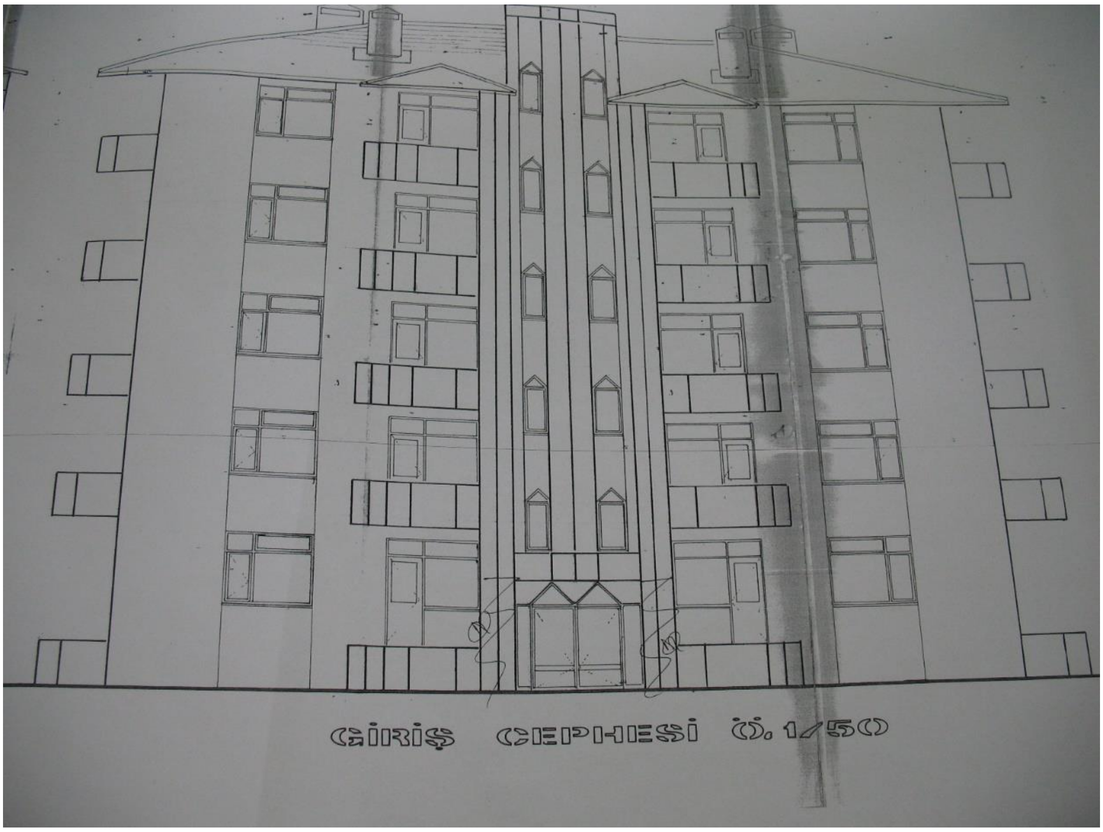

Figure A6: Entrance gate and side view of the studied building

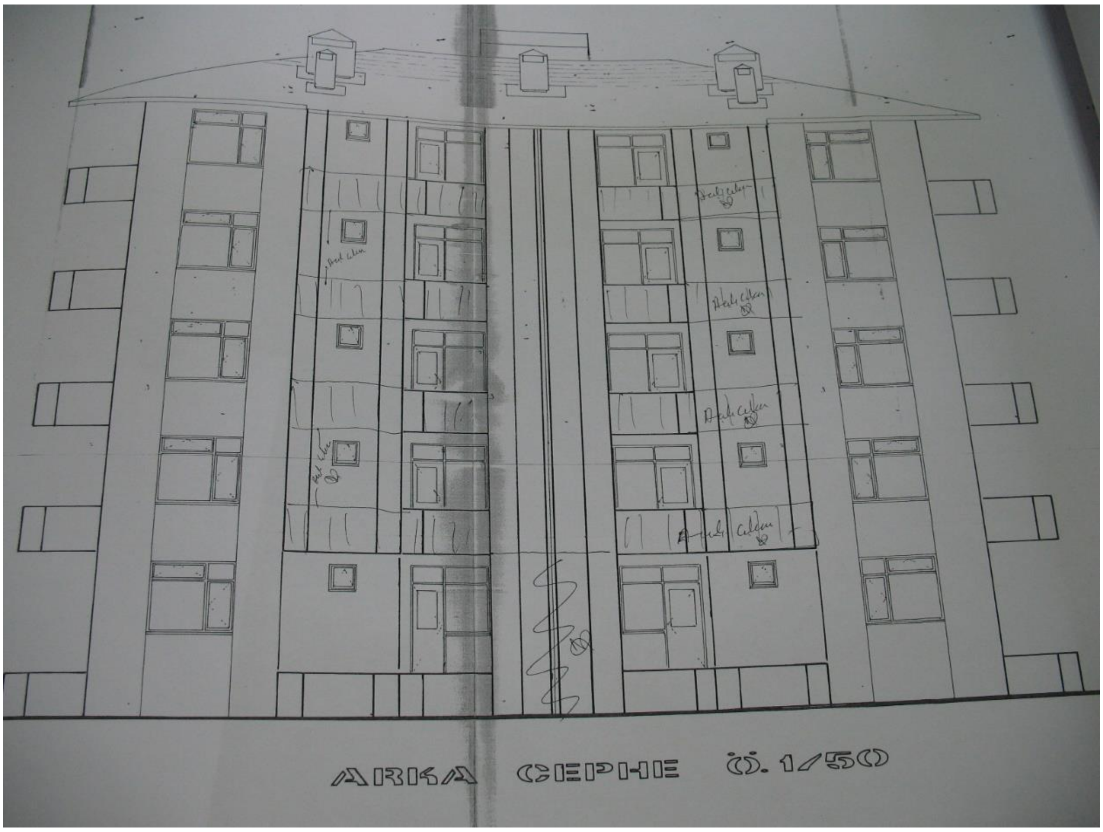

Figure A7: Backside view of the studied building

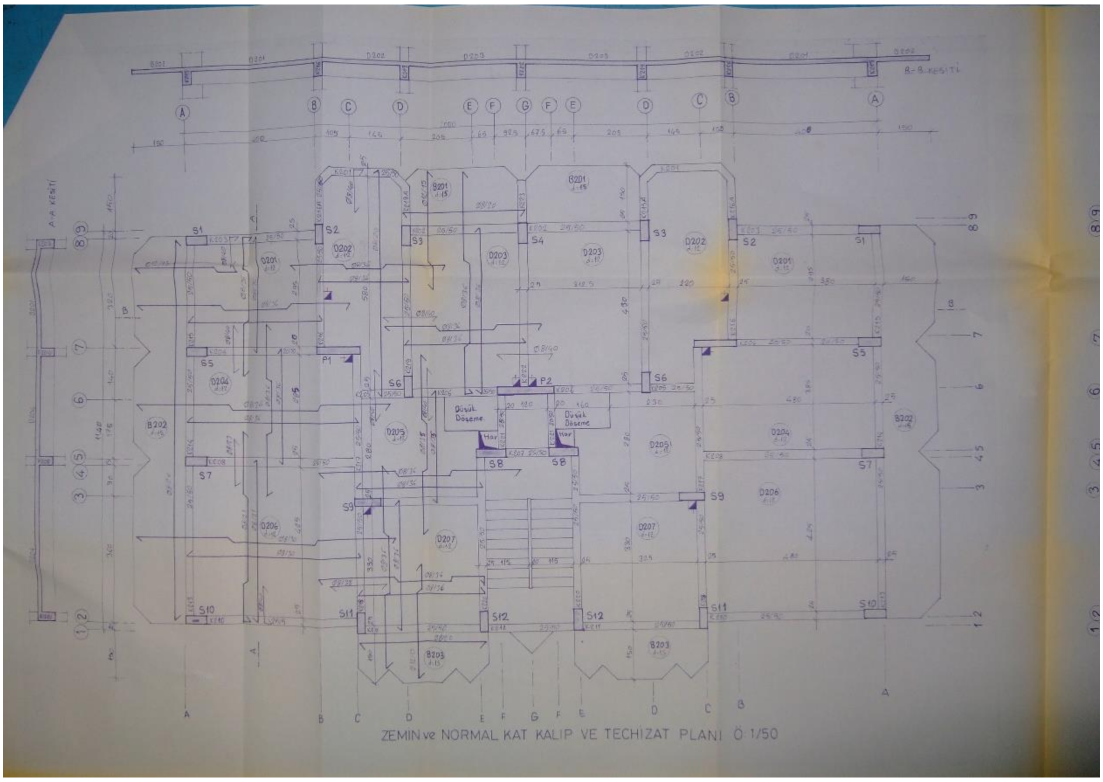

Figure A8: Reinforcement rebar of slab of the studied building

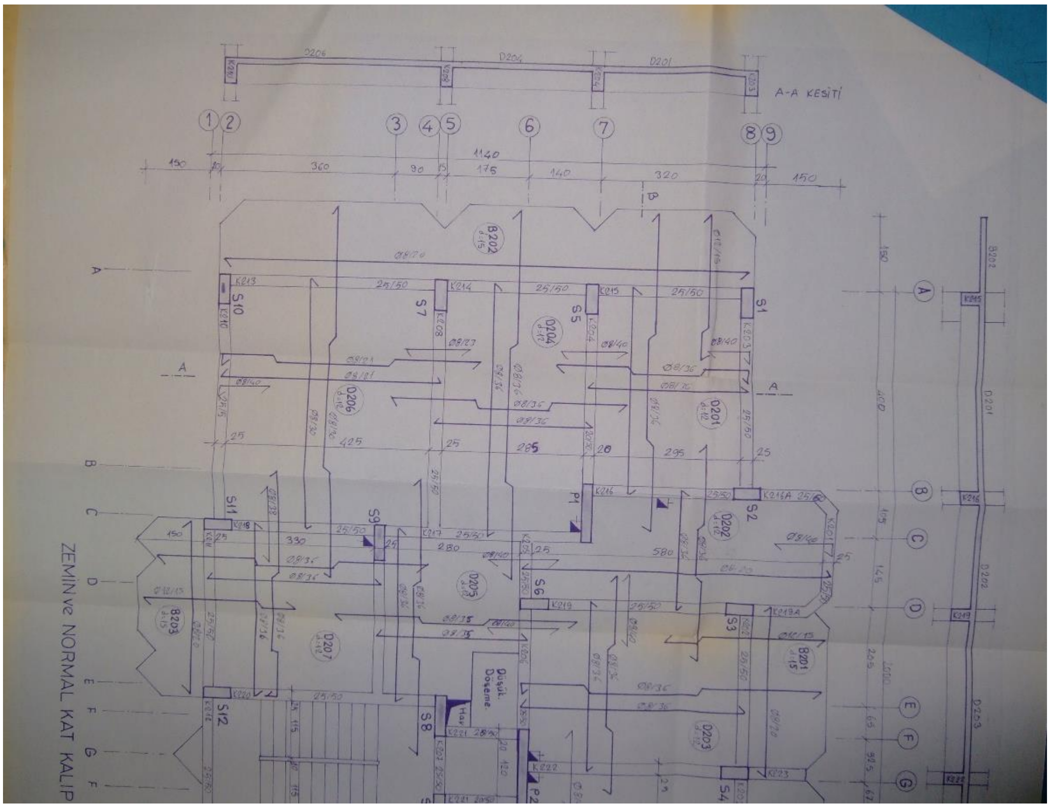

Figure A9: Reinforcement rebar of slabs of the studied building

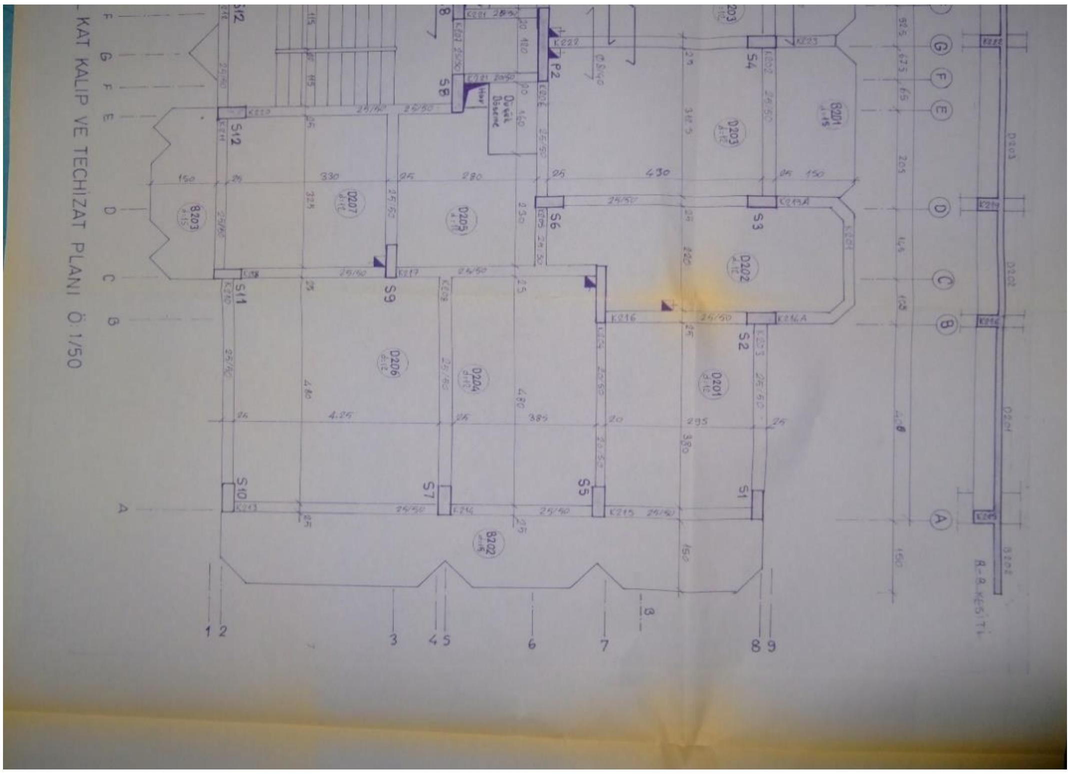

Figure A10: Reinforcement rebar of slabs of the studied building

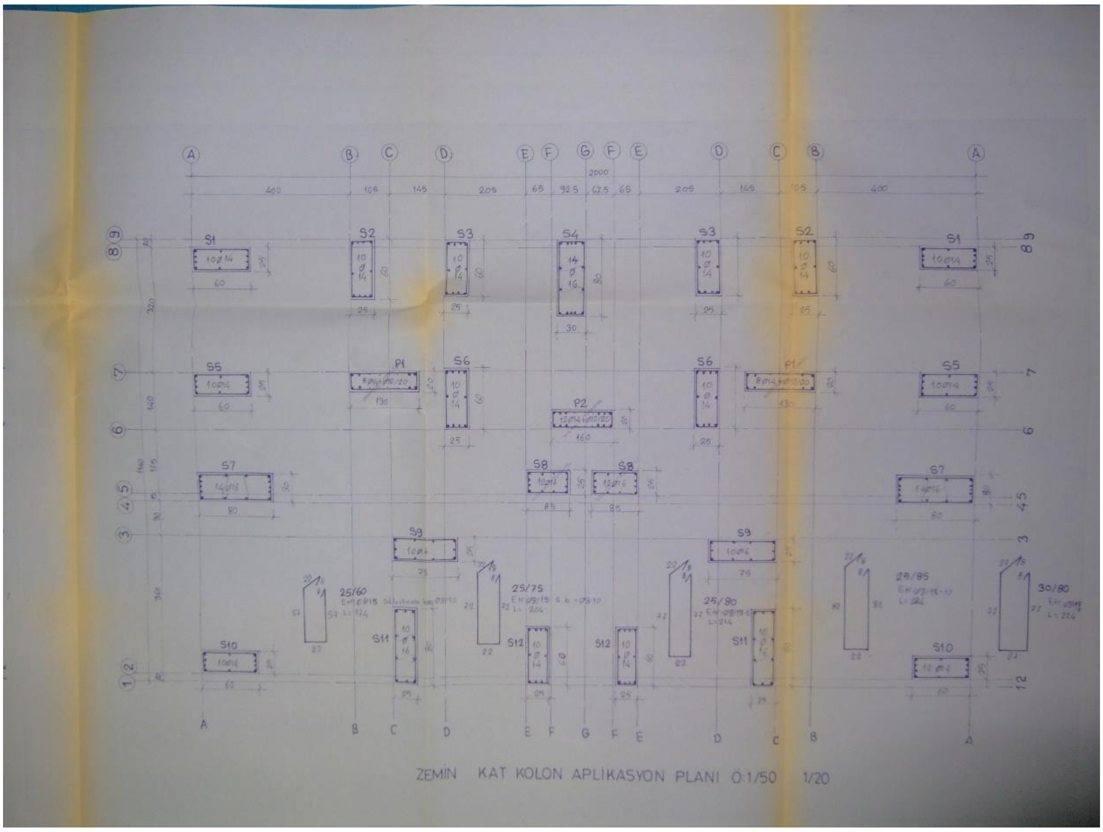

Figure A11: Reinforcement rebar of columns of ground floor

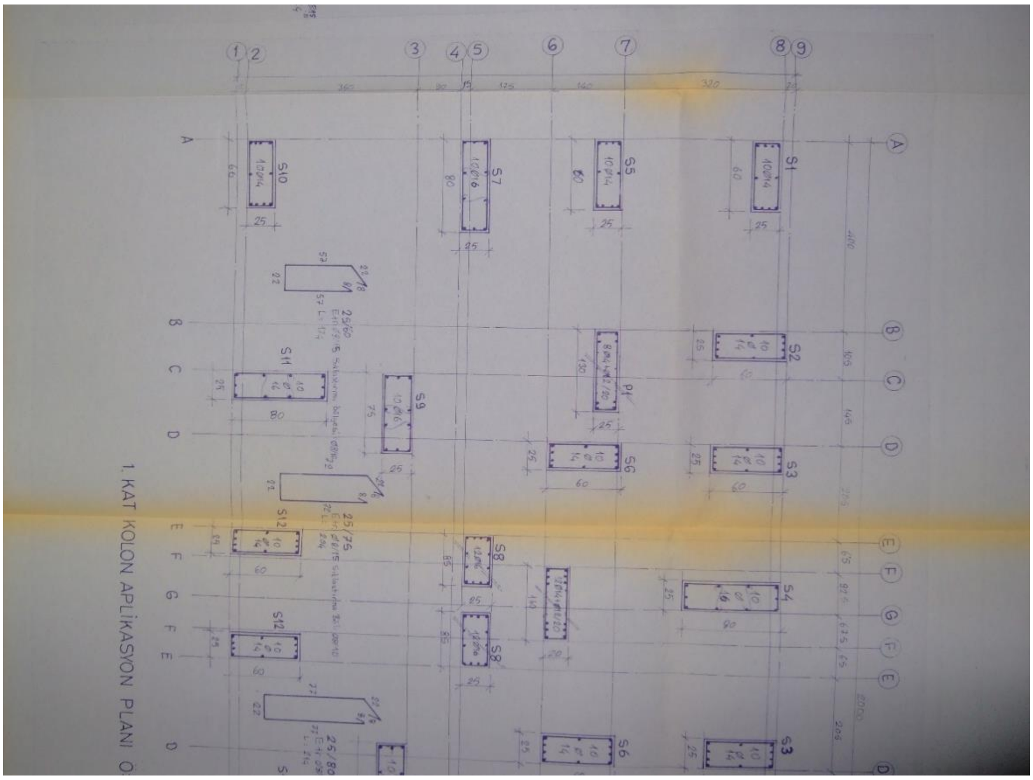

Figure A12: Reinforcement rebar of columns of the first floor

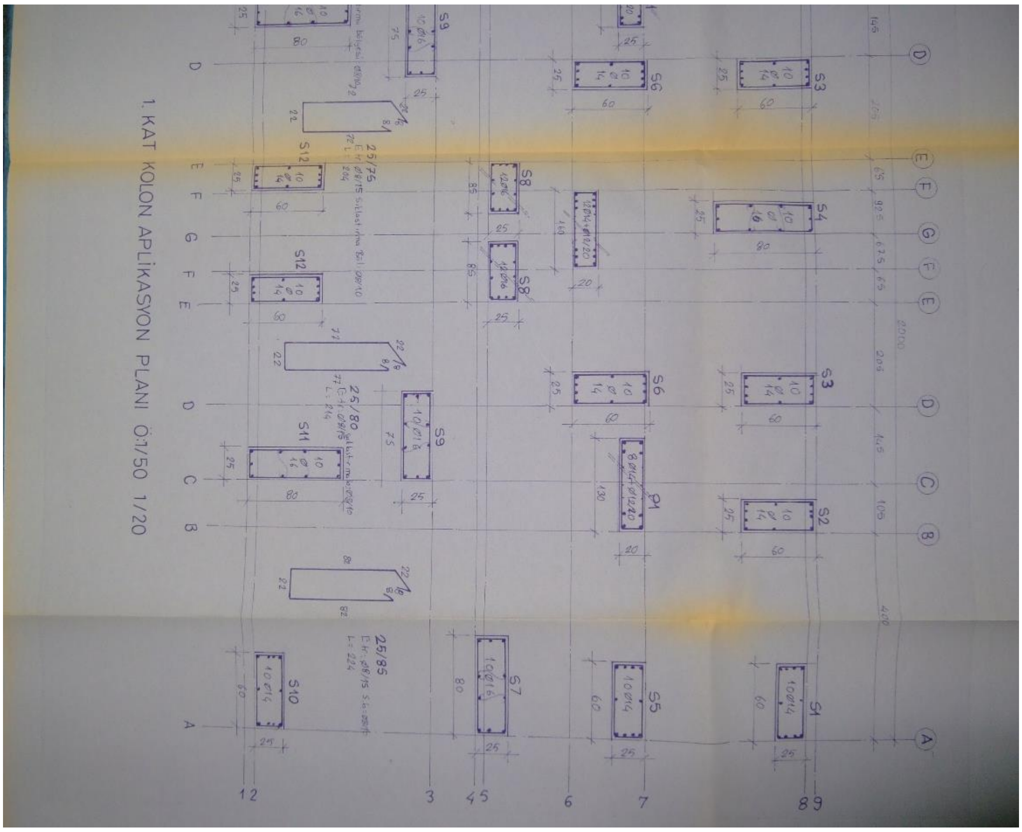

Figure A13: Reinforcement rebar of columns of the first floor

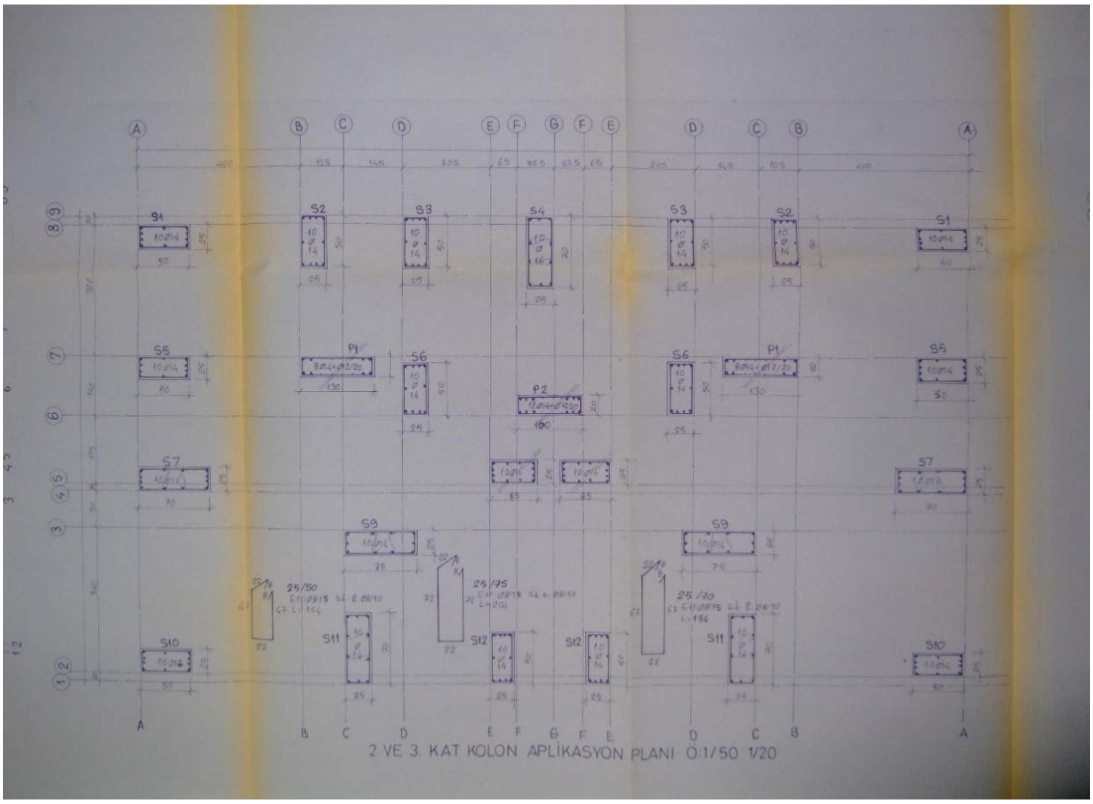

Figure A14: Reinforcement rebar of columns of second and third floor

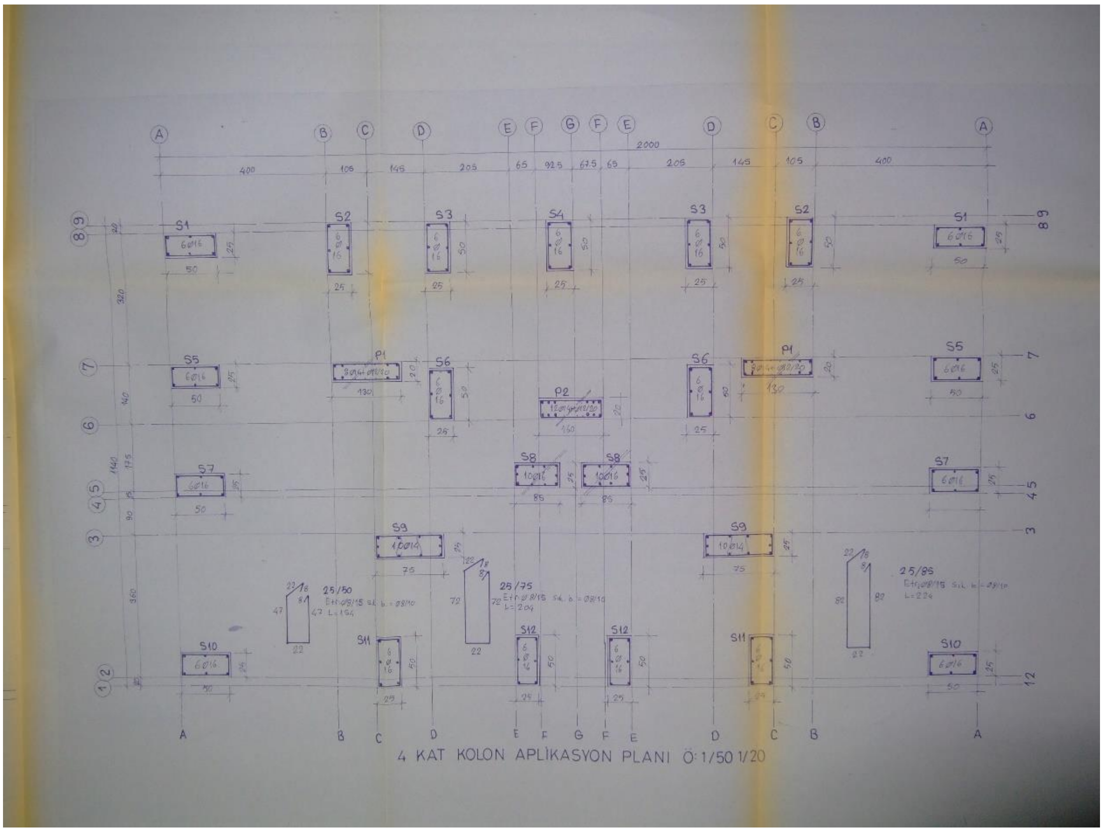

Figure A15: Reinforcement rebar of columns of the fourth floor

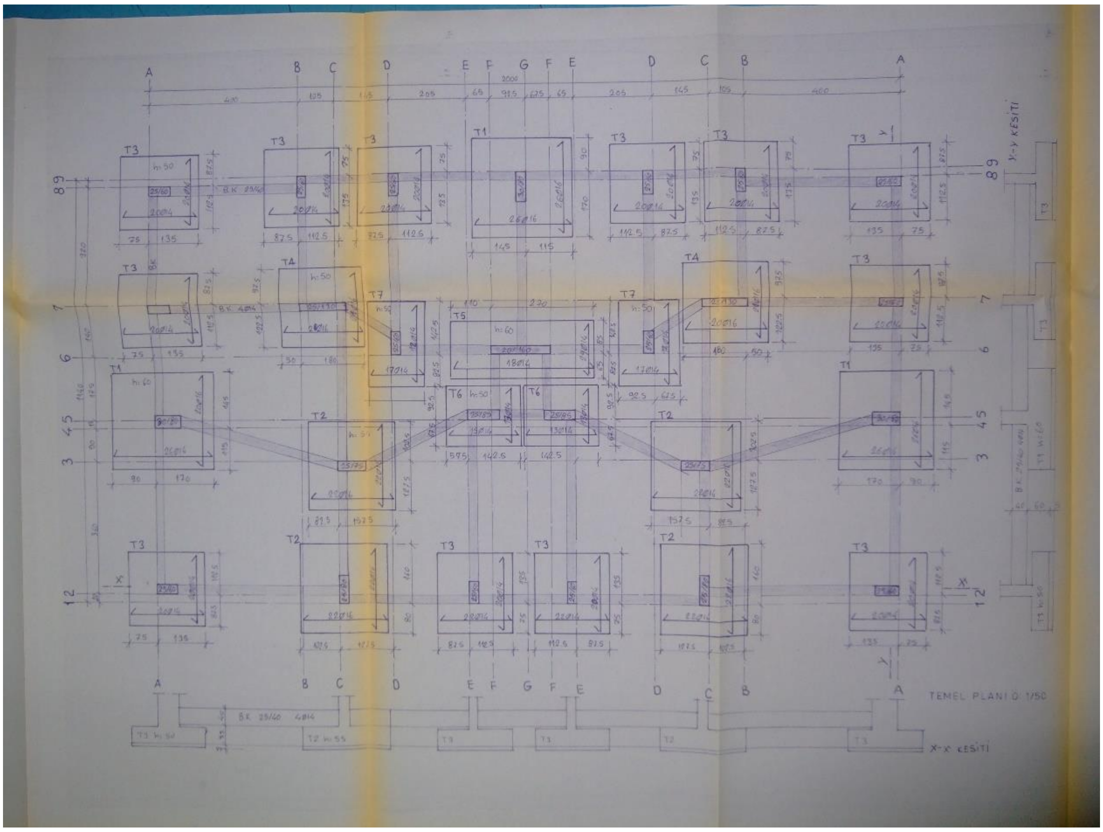

Figure A16: Reinforcement rebar of foundations of studied building

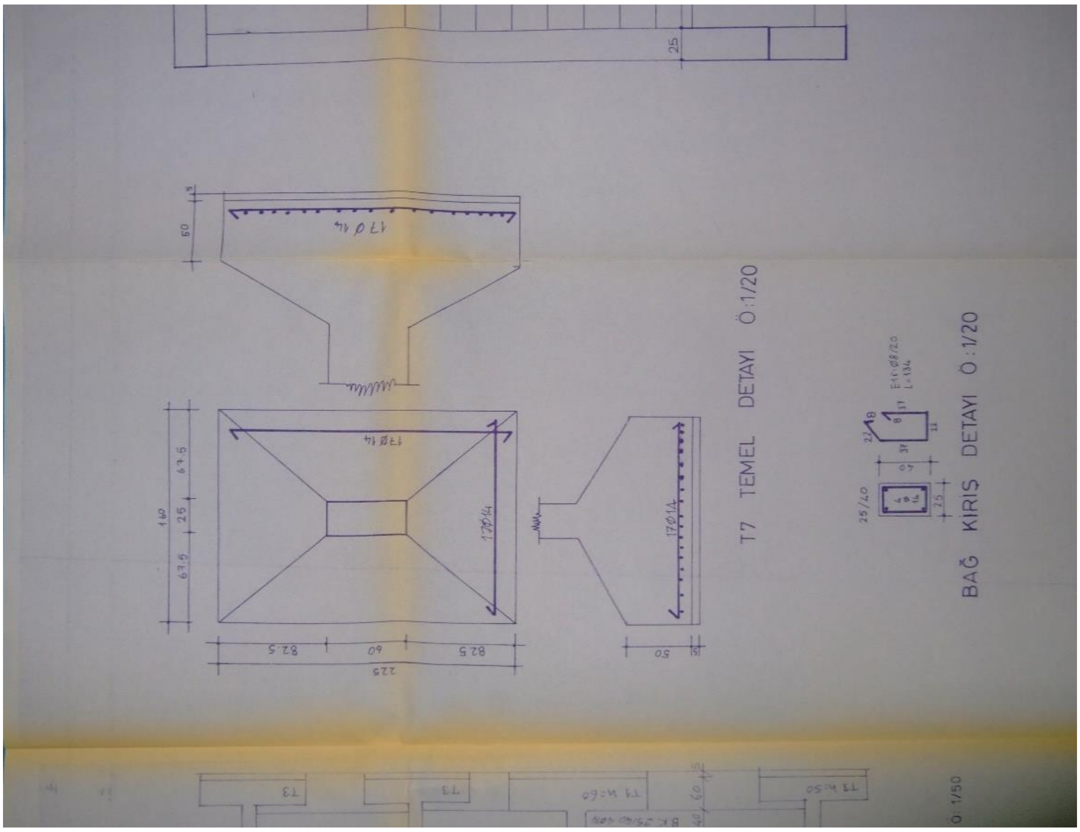

Figure A17: Reinforcement rebar of foundations of the studied building

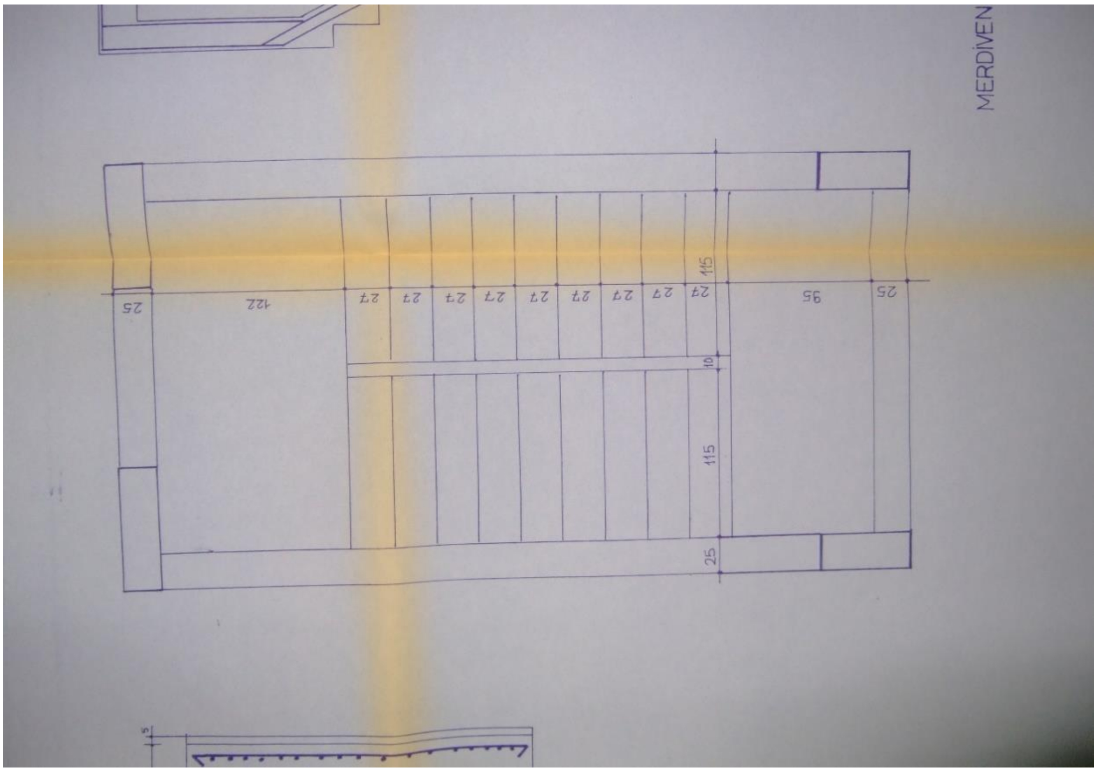

Figure A18: Section of stairs of the studied building

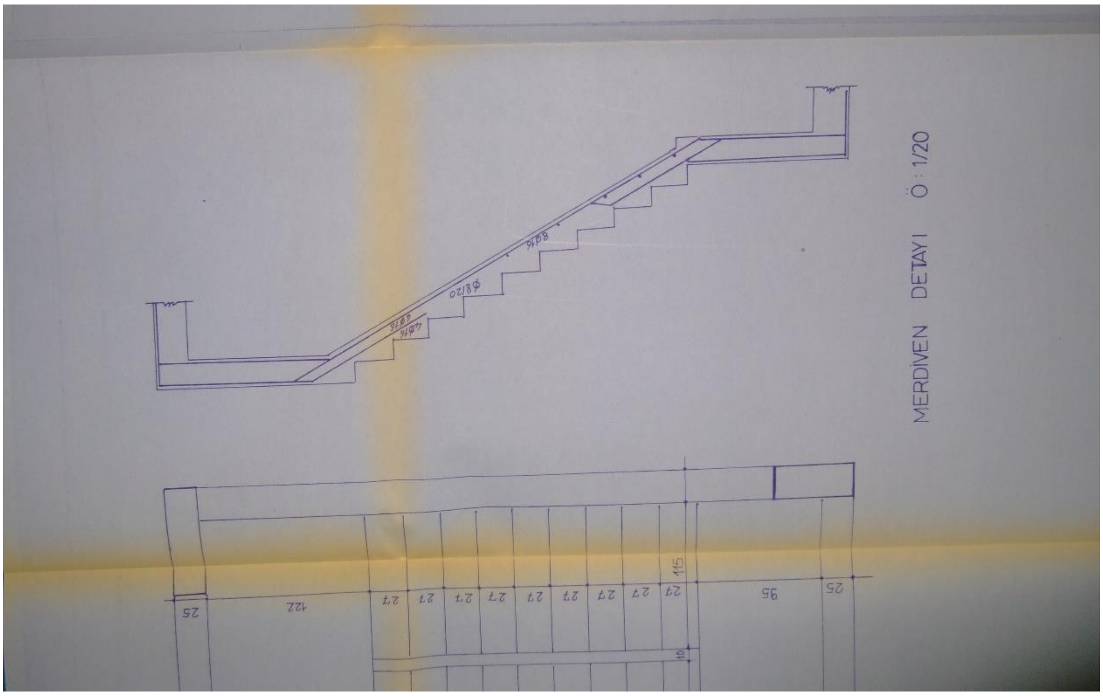
Figure A19: Reinforcement rebar of stairs of studied building

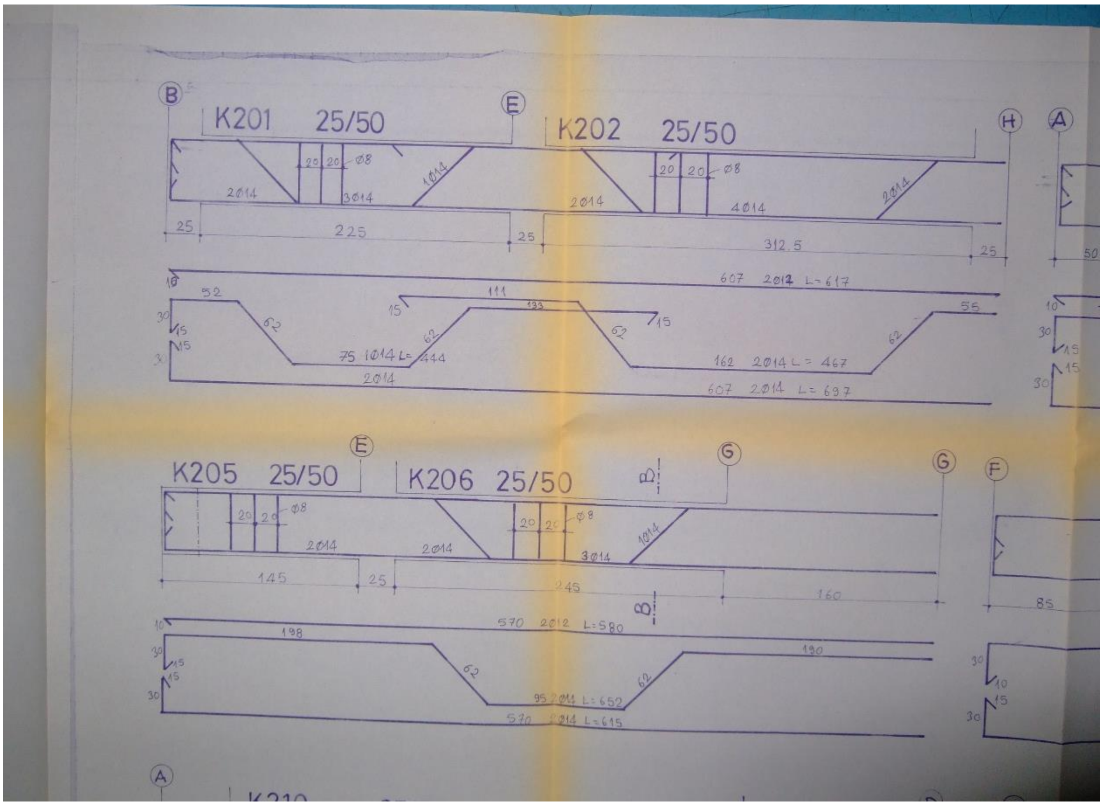
Figure A20: Reinforcement rebar of beams of studied building

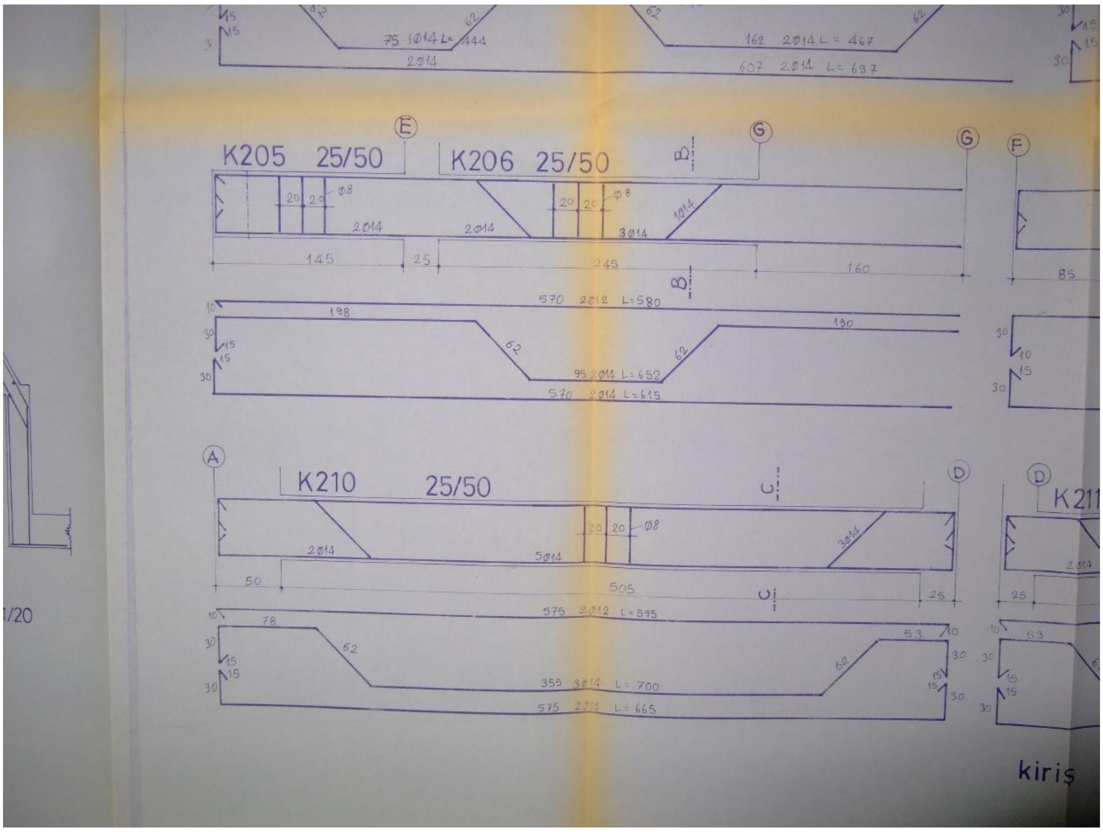

Figure A21: Reinforcement rebar of beams of studied building

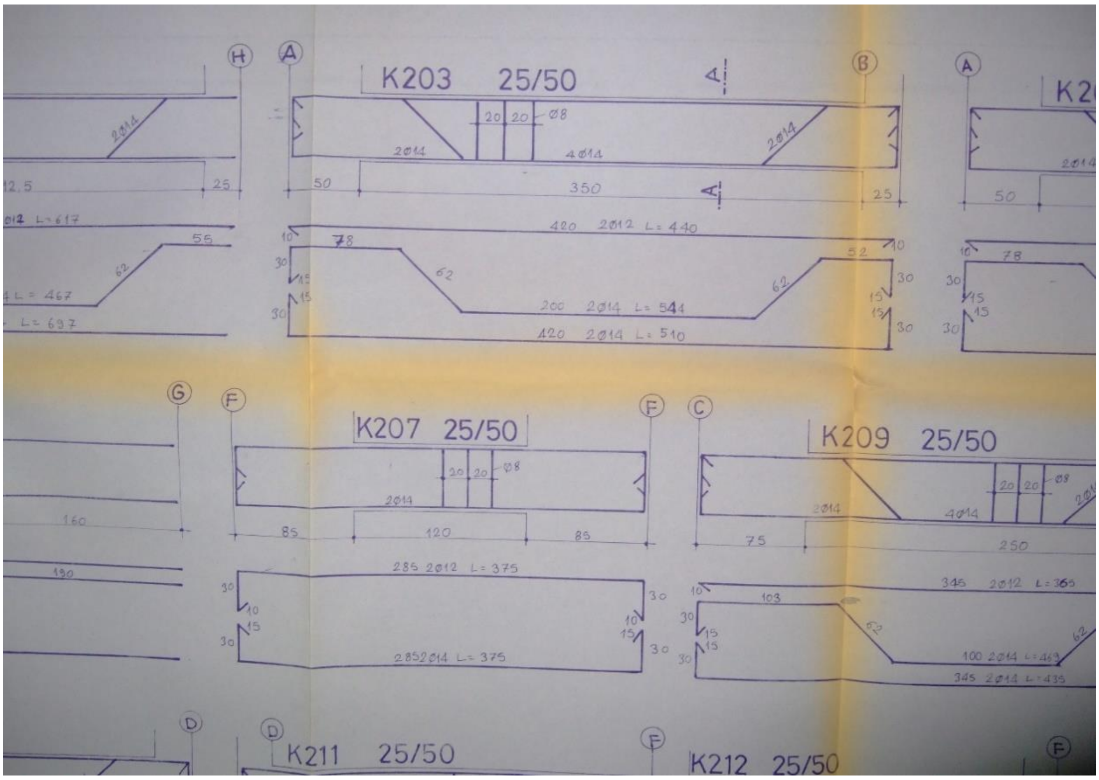

Figure A22: Reinforcement rebar of beams of studied building

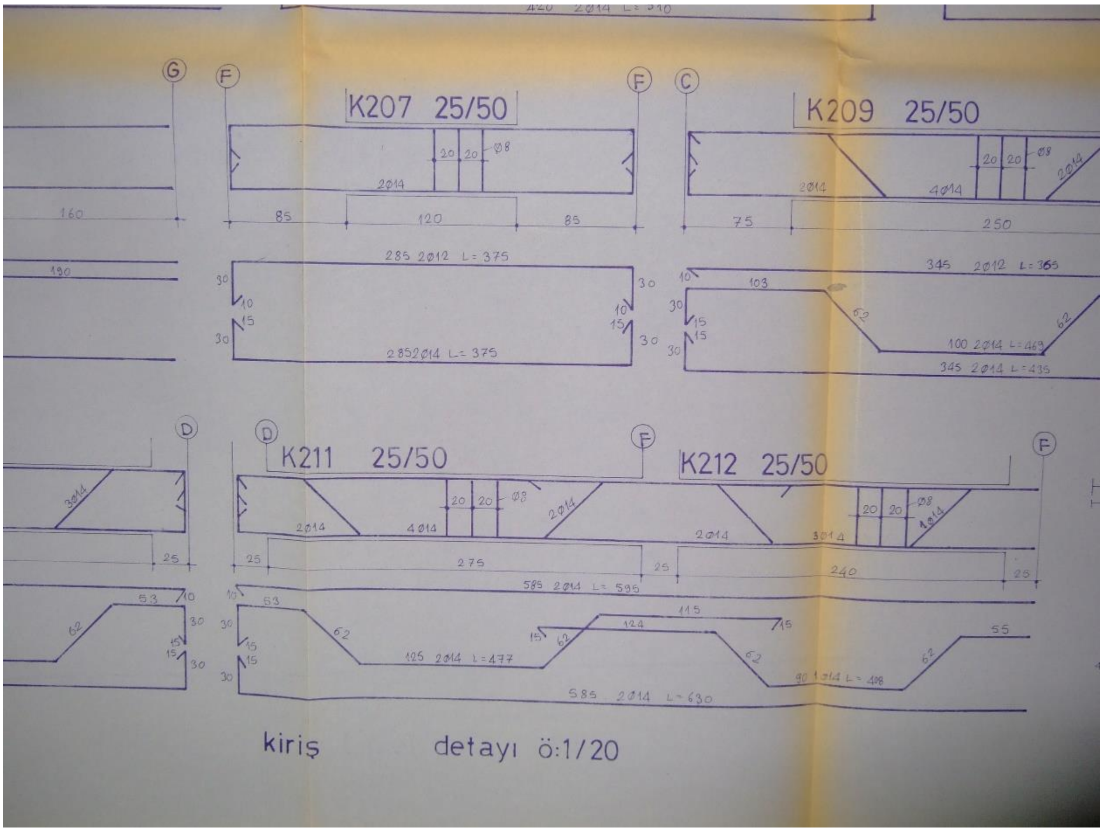

Figure A23: Reinforcement rebar of beams of studied building

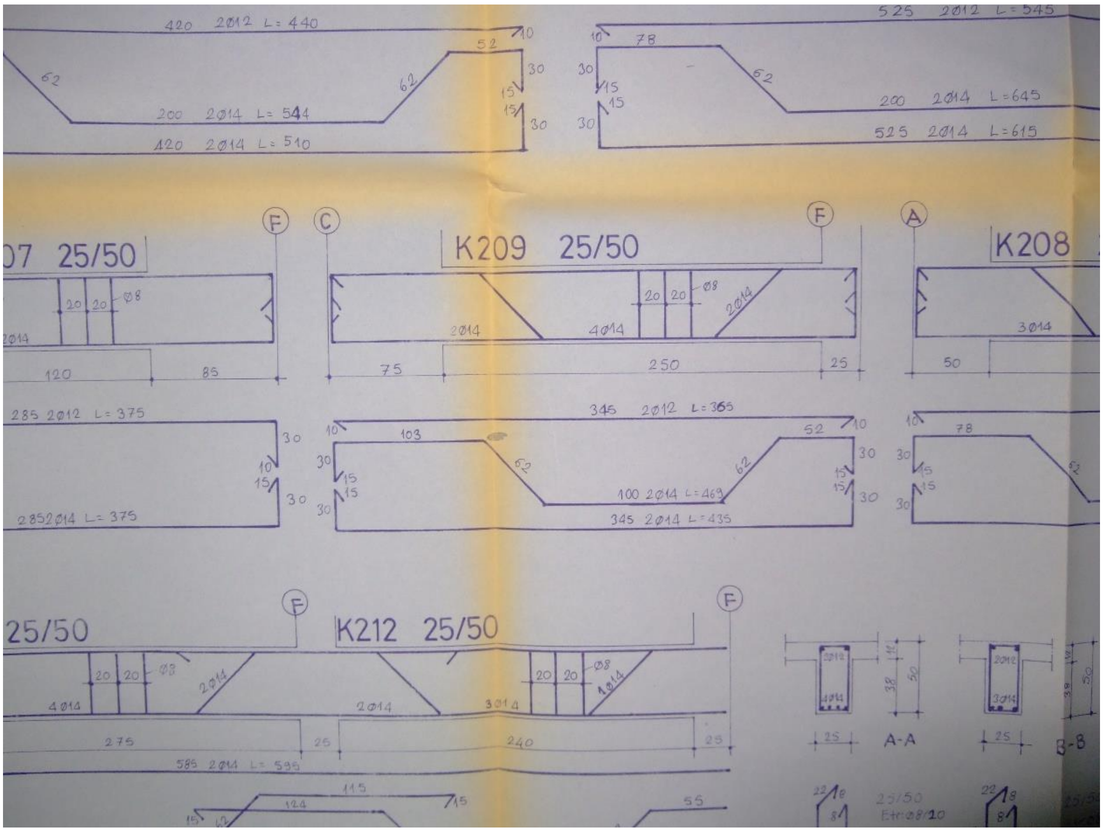

Figure A24: Reinforcement rebar of beams of studied building