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ANGIN

SEISMIC PERFORMANCE OF AN EXISTING REINFORCED
CONCRETE MASS HOUSING ACCORDING TO 2018 TURKISH
EARTHQUAKE CODE IN NORTH CYPRUS

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A THESIS SUBMITTED TO THE GRADUATE SCHOOL OF APPLIED SCIENCES OF NEAR EAST UNIVERSITY

By
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In Partial Fulfillment of the Requirements for the Degree of Master of Science in Civil Engineering

NICOSIA, 2019

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Approval of Director of Graduate School of Applied Sciences

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I hereby declare that all information in this document has been obtained and presented in accordance with academic rules and ethical conduct. I also declare that, as required by these rules and conduct, I have fully cited and referenced all material and results that are not original to this work.

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Dedicated

To my lovely father and mother

To my dearest brother...

ABSTRACT

Cyprus Island is under destructive earthquake risks through its history. The lack of seismic vulnerability knowledge about existing buildings increases importance of earthquake studies for Cyprus. Especially, building stock before 1990s constructed without horizontal loads consideration and it is recommended to investigate them. Vulnerable buildings may lead to loss of human lives. Local authority of North Cyprus constructed purchasable 2724 residential buildings for Turkish Cypriot community from 1986 to 1998 and that buildings are called mass housing. There are two kinds of structural systems in mass housing as prefabricated and reinforced concrete. The buildings were constructed only considering gravational forces. 3% of household population live in mass housing buildings in North Cyprus so it is important to know seismic performance of these buildings.

In this study, ground+three storey apartment type mass housing was selected as a representative on behalf of all mass housing. The selected building is in Nicosia and it was constructed in 1998. In the first step of the study, mass housing stock data was collected and existing structural plans with architectural plans were examined. Then, comprehensive investigation was followed for determination of material properties. At the end seismic performance of the selected mass housing was determined by Turkish made structural engineering software STA4-CAD V14.1. The performance analysis was performed within TEC-2018 regulations by using non-linear static analysis method.

Through this study, innovations in new Turkish Earthquake Code can be seen and seismic performance of other mass housing can be predicted.

Keywords: seismic vulnerability; mass housing; seismic performance; STA4-CAD V14.1; TEC-2018

ÖZET

Kıbrıs adası tarihi boyunca yıkıcı depremlerin riski altındadır. Mevcut binaların depreme karşı savunmasızlığı hakkındaki bilgi eksiklikleri Kıbrıs için deprem konusundaki çalışmaları önemli hale getirmektedir. Özellikle 1990'lı yıllardan önce inşa edilen binalar yatay yükler göz önüne alınarak inşa edilmedi ve bu binaların incelenmesi tavsiye edilmektedir. Depreme karşı savunmasız olan binalar can kayıplarına neden olabilir. Kuzey Kıbrıs yönetimi 1986 yılından 1998 yılına kadar Kıbrıs Türk halkı için ödeme planı uygun olan sosyal konutlar inşa etti. Sosyal konutlar betonarme karkas ve prefabrik olmak üzere iki tip taşıyıcı sisteme sahiptir. Bu binalar yalnızca düşey yük etkisi göz önüne alınarak inşa edildi. Kuzey Kıbrıs'taki hane halkının %3'ünün sosyal konutlarda yaşaması nedeniyle bu binaların deprem performansının bilinmesi önemlidir.

Bu çalışmada tüm sosyal konutlar adına zemin+üç katlı bir apartman temsili olarak seçildi. Seçilen bina Lefkoşa'da yer alıp, yapımı 1998 yılında tamamlanmıştır. Bu çalışmanın birinci aşamasında sosyal konutlar yapı stoğu hakkında bilgi toplandı ve statik projeler ile birlikte mimari projeler incelendi. Sonrasında ise malzeme özelliklerinin belirlenmesi için kapsamlı araştırma yapılmıştır. Çalışmanın sonunda temsili binanın Türk yapımı yapı mühendisliği yazılımı olan STA4-CAD V14.1 kullanılarak deprem performansı bulunmuştur. Performans analizi Türk Deprem Yönetmeliği 2018 ile birlikte doğrusal olmayan statik itme analizi yöntemi kullanılarak yapılmıştır.

Bu çalışma sayesinde yeni Türk Deprem Yönetmeliğinde gerçekleştirilen yenilikler görülebilir ve geriye kalan diğer sosyal konutların deprem performansı hakkında tahmin yürütülebilir.

Anahtar kelimeler: Deprem savunmasızlığı; sosyal konutlar; deprem performansı; STA4-CAD V14.1; Türk Deprem Yönetmeliği 2018

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LIST OF ABBREVIATIONS

TEC-2018:	Turkish Earthquake Code 2018
TEC-1975:	Turkish Earthquake Code 1975
NCEC-2015:	North Cyprus Earthquake Code 2015
TEC-2007:	Turkish Earthquake Code 2007
RC:	Reinforced concrete
3D:	Three dimensional
PGA:	Peak ground acceleration
EC8:	Eurocode 8 (Design of Structure for Earthquake Resistance)
FEMA:	Federal Emergency Management Agency
ATC:	Applied Technology Council
DC:	Demand capacity
RSA:	Response spectrum analysis
FNA:	Finite element analysis
IO:	Immediate occupancy
LS:	Life safety
CP:	Collapse prevention
TRNC:	Turkish Republic of Northern Cyprus
TS:	Turkish standards
DD:	Seismic ground movement level
BKS:	Building usage class
AFAD:	Disaster and Emergency Management Authority
DTS:	Seismic design classes
BYS:	Building height classes
CC:	Collapse case

LIST OF SYMBOLS

g:	Earth's gravity
A_0:	Effective ground acceleration coefficient
T:	Building period
T_S:	Solution period
Z:	Local site class
σ:	Standard deviation
N:	Number of elements of an array
x_i:	x th member of an array
$\bar{x}$:	Arithmetic average of numbers in an array
S_{DS}:	Short period design spectral acceleration coefficient
S_S:	Short period spectral acceleration coefficient
F_S:	Local site impact coefficient for the short period zone
S_1:	Spectral acceleration coefficient for T=1
S_{D1}:	Design spectral acceleration coefficient for T=1
F_1:	Local site impact coefficient for one second
R:	Structural behavior factor
D:	Overstrength factor
I:	Building importance factor
n:	Live load participation factors
Φ:	Diameter
G:	Dead load
Q:	Live load
S:	Snow load
E:	Earthquake load
W:	Wind load
$S_{ae}(T)$:	Horizontal elastic design spectral acceleration
$S_{de}(T)$:	Horizontal elastic design spectral displacement
V_e:	Shear force based on the calculation of transverse reinforcement in columns, beams, joints and shear walls

$\mathbf{f_{yk}}$: Characteristic yield strength of longitudinal reinforcement

$\mathbf{A_s}$: Total area of reinforcement

$\mathbf{V_{kol}}$: The smallest column shear force

$\mathbf{b_j}$: Column width is considered if the beam stuck in the junction area is of the same width as the column or protrudes from both sides of the column. Otherwise, twice the distance from the vertical mid-axis of the beam to the column edges in the considered direction of the earthquake. (Does not exceed the sum of the beam width and the height of the joint)

$\mathbf{h_c}$: Cross-sectional dimension of the column in the considered earthquake direction

$\mathbf{f_{ck}}$: Characteristic compressive strength of concrete

CHAPTER 1
INTRODUCTION

1.1 Problem Statement

Mass housing buildings are distributed in all districts of North Cyprus hovewer they are remarkable part of residential building stock. All mass housing buildings were constructed according to Turkish Earthquake Code 1975 (TEC-1975) and “Seismic Detailing Provisions” which is prepared by Chamber of Turkish Cypriot Civil Engineers in 1992. They are all considered vertical forces for designing. It is an advantage to identify the seismic response of old buildings accordingly improved new seismic codes. For this reason, this case study plays a key role in terms of human life. The results of this study is good to compute the situation and decide whether these structures can be retrofitted or demolished. The following cases are another encouragement factors of the study. North Cyprus Earthquake Code (NCEC-2015) is officially in use since 2015 in North Cyprus which is based on Turkish Earthquake Code 2007 (TEC-2007). Furthermore, Turkish Earthquake Code 2018 (TEC-2018) became an official code on 1 January 2019, in Turkey. Therefore, Turkish Cypriot authority can be inspired from TEC-2018 and will announce it as a new national earthquake code in near future. This study is an opportunity to see an availability of new Turkish Earthquake code for Northern Cyprus and to discover the shortcomings for usage if there are. Recently, sea side supplied aggregate was allowable for concrete mix so reinforcing steel corrosion is observed in a plenty of reinforced concrete (RC) buildings in North Cyprus. Corrosion is a stage which affect a reinforced concrete structure in a different kind of aspects as loss of steel-concrete bond strength, cover spalling and loss of reinforcement cross sectional area. Reduction of resistance and load bearing capacity in addition to failure mechanism transfer from ductile to fragile type. Corrosion depends on time which leads to reduction of strength and serviceability of structures. Environmental sources like sulphate and chloride ions, alkali-aggregate reaction, carbonation phenomenon and freeze-thaw cycles are some significant reasons for corrosion. When the chlorides or carbon dioxide concentrations overrun the critical value, passive layer destroys and corrosion process starts. For this reason, effective diameter of reinforcement bars start to reduce. Corrosion causes rust so it leads to concrete cracking. Finally, cracks cause loss of bond between concrete and steel. Moreover, microcracks lead to concrete strength reduction. Late adaptation of ready mix concrete facilities lead to low compressive strength concrete usage in North Cyprus especially, C14 concrete dominates old RC buildings. Moreover, the plain reinforcing steel (S220) was allowable until the end of 1990s (Safkan et al., 2017). Rainwater pipe usage was allowable inside columns until the end of 1990s so its application created holes inside of RC columns. Finally, most of the reinforced concrete buildings constructed without receiving any engineering attention. Therefore, improper configuration of structural and architectural system and insufficient detailing is observed in Cyprus.

1.2 Objective of the Study

- The fundamental objective of this study is to investigate the RC mass housing building, located in Nicosia city, using the new seismic design code (i.e TEC-2018).
- This study is intended to evaluate whether mass housing buildings are still safe or insecure against workloads and know the performance of the building structure when earthquake occurs. The three dimensional (3D) analysis is carried out under static analysis in both x and y directions. Pushover analysis has been used for seismic performance determination.
- To collect data about mass housing stock in North Cyprus.
- To see the innovations in new Turkish earthquake code.

1.3 General Concept of Earthquake

Earthquake is a natural event in the world which creates significant damage to human lives and structures. Earthquake prediction is a branch of the science of seismology dealing with the determination of time, location and magnitude of incoming earthquakes within stated limits. It defines parameters for future strong earthquakes to occur in a particular area. Prediction can be further distinguished from earthquake warning systems, which specify of an earthquake, provide a real-time warning of seconds to neighboring areas that may be affected. Many methods have been offered to predict the time and place in which earthquakes will occur. Scientifically, reproducible predictions cannot yet be made to a specific day or month although considerable research efforts spend by seismologists. United States of

America, Turkey, Japan, Italy, Indonesia, China and Iran are sample countries that are located on high seismic active zones (Rasol, 2014). The world's largest earthquake which occured on May 22, 1960 in Southern Chile was assigned a magnitude of 9.5.

1.4 General Concept of Seismic Performance Analysis

The latest concept of earthquake engineering is performance based engineering. Civil engineers and architects play a key role in improving the seismic design. Performance-based design is a common definition that design is explained in terms of achieving peformance target when the structure is exposed to seismic hazard. The target of performance levels shall be level of stress not be exceed target damage state, displacement, load or a limit state. Serviceability of limit strains let a stable level of assessment to be achieved to minimize the high expenses associated with loss of use and repair of heavily damaged structures. As a summary, performance based design means a technique that design criterias are based on achieving a performance objective (Ghobarah, 2001).

1.5 Importance of Seismic Performance Analysis

The awareness of the potential seismic vulnerability or seismic risk of existing building stock has increased cause of social and economic effects of previous earthquakes. Seismic risk analysis of such buildings is important for identifying the seismic vulnerability under the effect of potential seismic hazard. This approach is useful for disaster response planning, loss estimation, damage estimation and retrofitting decisions. Identifying potential hazards ahead of time and advance planning can save lives and significantly reduce injuries and property damage. Performance evaluation under the effect of expected seismic load is one of the main objectives of a performance based design (Maniyar et al., 2009).

1.6 Previous Studies

A review of the literature research was followed in the area of "Seismic Performance Evaluation" and "Seismicity of Cyprus Island" in the course of study. Since the past decade, many academic research work have been published, mainly as journal articles which have been reviewed as a part of this study.

- A. Yakut (2004), presents "Preliminary Seismic Performance Assessment Procedure for Existing RC Buildings". In this study, the beginning procedures to evaluate quickly the seismic performance of existent reinforced concrete buildings is presented. A capacity index is calculated taking into account size, orientation and material properties of the components including lateral load resisting system. Afterwards, the index is modified in accordance with different coefficients which shows the quality of architectural features, materials and workmanships. This method separates the building to structural performance levels.

- M. Inel, H. Baytan & H. Bilgin (2008), presents "Seismic Performance Evaluation of School Buildings in Turkey". This study includes seismic performance of the school buildings in Turkey in accordance with non-linear behavior of the reinforced concrete structural members. Six school buildings used which represents important percentage of school buildings in moderate-size cities which are in high seismic region of Turkey. Capacity curves was obtained by pushover analysis and TEC-2007 with Fema-356 used for this study.
- Z. Cagnan & G. B. Tanircan (2009), present "Seismic Hazard Assessment for Cyprus". This study focused on evaluation of probabilistic seismic hazard for Cyprus depending upon different new results: a new extensive catalog, seismic source models depending upon new investigation and new attenuation relations. Peak ground acceleration patterns achieved for rock statuses show high hazard along the southern shoreline of Cyprus where the expected ground motion is among 0.3 g and 0.4 g.
- I. Safkan (2012), presents "Comparison of Eurocode 8 and Turkish Earthquake Code 2007 for Residential RC buildings in Cyprus". In this study, ground+four storey building was selecected in Nicosia and Famagusta and different properties were used based on each code. The software model was established by Sap2000 program. Two different site conditions used for same building. At the end, it is concluded that results are similar for Nicosia and 30% base shear difference obtained in Famagusta.
- C. Z. Chrysostomou, N. Kyriakides, A. J. Kappos, L. Kouris, E. Georgiou & M. Millis (2013), present "Seismic Retrofitting and Health Monitoring of School Buildings of Cyprus". This study explains the whole assessment way, throughout with details of more than 10 year continuing school buildings retrofitting program of Cyprus, with program description and wireless monitoring system development.

Furthermore, mathematical models for selected school buildings are initiated and compared with in-situ measurement.

- H. Tekeli, H. Dilmaç, F. Demir, K. Güler & Z. Celep (2014), presents "A Simplified Procedure to Determine Seismic Performance of Residential RC Buildings". This conference study is about seismic performance evaluation of existing reinforced concrete frame buildings and includes a simplified approach for seismic performance estimation of buildings.
- M. Inel & E. Meral (2016), presents "Seismic Performance of RC buildings Subjected to Past Earthquakes in Turkey". This study evaluates the seismic performance of existing mid-rise and low reinforced concrete frame buildings by comparing their displacement capacities and demands in accordance with TEC-2007. 2, 4 and 7 storey reinforced concrete buildings picked to represent mid-rise and low buildings located in high seismicity region of Turkey. The buildings have no shear walls. Non-linear time history analysis used for the study.
- R. Reşatoğlu & R. S. Atiyah (2016), present "Evaluation of Reinforced Concrete Buildings in Northern Cyprus Using in TEC-2007 and EC8 in Respect of Cost Estimation". In this study, an ordinary reinforced concrete frame apartment building was selected in Nicosia, Cyprus. It is a residential building with 3 m typical storey height. STA4-CAD V12.1 package program was used to analyse the building. As it can be understood from the tittle of the study, TEC-2007 and EC8 were used as input data. In the conclusion, it is observed that usage of TEC-2007 incerased reinforcemet 3.45% compared to EC8.
- I. Safkan, S. Sensoy & Z. Cagnan (2017), present "Seismic Behavior of the Old-Type Gravity Load Designed Deteriorated RC Buildings in Cyprus". In this study, vulnerability of existing reinforced concrete building was investigated in respect of concrete strength and corrosion relation. This study concluded that brittle behavior showed due to strength and ductility reduction in non-seismically models. On the other hand, reducing in cover and bond strength, diameter, yield stress, buckling stress and shear strength causes decreasing global seismic performance. Moreover, a strong corrosion effect on the bound concrete strength models was obtained.

1.7 Organization and Scope

This thesis is organised in seven chapters. Need for the study, objective of the study, general concept of earthquake, performance analysis with its importance and previous studies are introduced in chapter 1. Chapter 2 focuses on seismicity of Cyprus. Chapter 3 presents seismic analysis methods regarding on FEMA 356 (Federal Emergency Management) and ATC-40 (Applied Technology Council). Chapter 4 focuses on mass housing stock data of North Cyprus which emphasises the importance of study. Chapter 5 includes the methodology of the study. Selection of building, determination of material properties and modelling of the building presented. Chapter 6 explains and discusses the results of analysis. Chapter 7 presents the conclusions of the study. On the other hand, appendix 1 includes significant features of new seismic design code of Turkey. Appendix 2 presents the concrete core report taken by Chamber of Turkish Cypriot Civil Engineers. Appendix 3 includes the architectural and structural plans that used for the study. Lastly, appendix 4 subjects the STA4-CAD calculations and reports.

CHAPTER 2

SEISMICITY OF CYPRUS

2.1 Overview

Cyprus is the third biggest island in Mediterranean Sea, located on between Eurasian and African tectonic plates that has high seismicity. The border between two plates is located on the south-west of Cyprus. Therefore, the island can be accepted as an earthquake vulnerable area. Historical reports and archeological findings are also contribute to support devastating effect of strong earthquakes in Cyprus. There were 16 destructive earthquakes between 26 B.C. and 1900 A.C. according to the historical data of island. As a summary, Cyprus was under the earthquake risk through its history and the historical background is a indicator of upcoming destructive earthquakes risk. The largest earthquake of Cyprus in the last century occurred on 9 October 1996 with magnitude of 6.5 at 50 km far away from the coast of Pafos (Geological survey department, Republic of Cyprus). Earthquakes in Cyprus are in a zone which is referred to as Cyprian Arc. The most earthquake prone area of Cyprus is the coastal zone that is shown in figure 2.1.

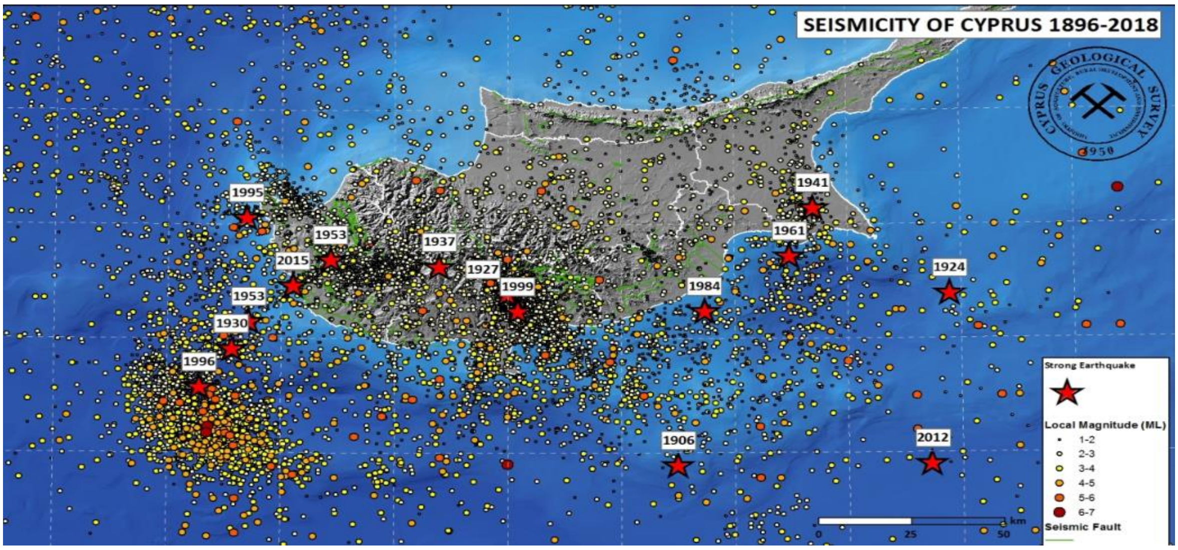

Figure 2.1: Seismicity of Cyprus 1896-2018 (Geological Survey Department, 2019)

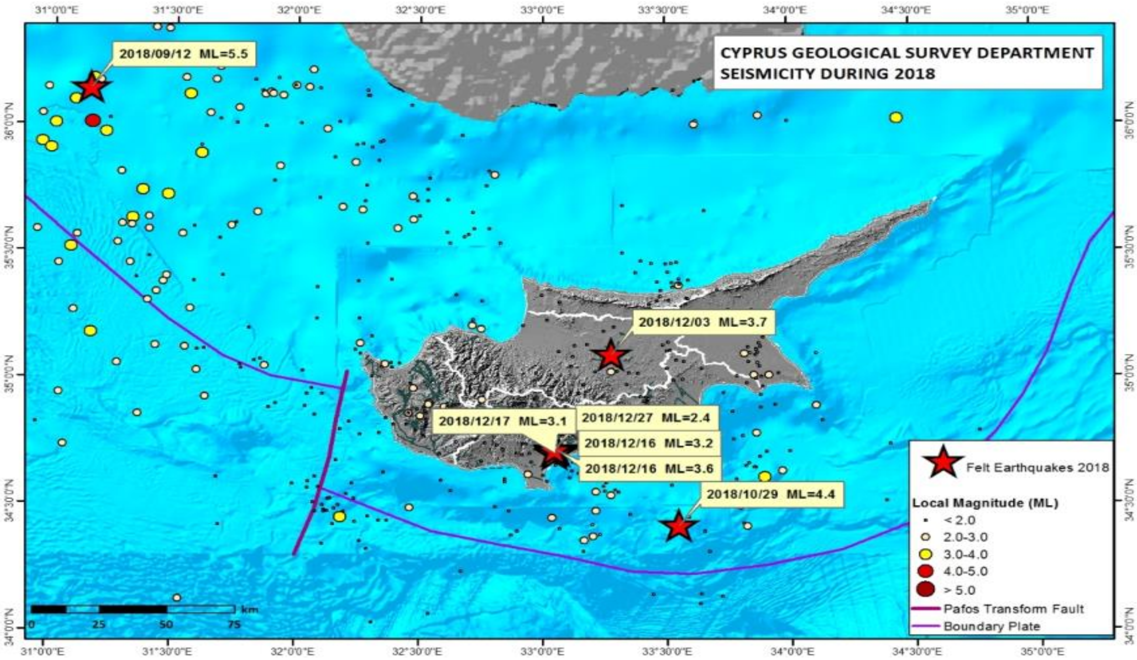

Figure 2.2: Seismicity of Cyprus in 2018 (Geological Survey Department, 2019)

Sixteen destructive earthquakes struck Cyprus in the 20th and 21st century. The largest earthquakes occured in 1918 and 1996. Table 2.1 lists the major earthquakes as it is shown below.

Table 2.1: List of earthquakes in 20th and 21st century in Cyprus (Geological Survey Department, 2019)

Date	Magnitude	Latitude	Longitude
23 February 1906	5.3	34.30	33.50
29 September 1918	6.3	35.10	34.80
18 February 1924	6.0	34.80	34.80
13 December 1927	5.0	34.80	33.00
9 May 1930	5.4	34.64	32.19
26 June 1937	4.7	34.88	32.80
20 January 1941	5.9	35.17	33.65
9 December 1947	5.4	36.46	34.66
10 September 1953	6.0	34.72	32.24
10 September 1953	6.1	34.80	32.78
15 September 1961	5.7	34.91	33.83

28 March 1984	4.5	34.75	33.58
23 February 1995	5.7	35.02	32.23
9 October 1996	6.5	34.53	32.10
11 August 1999	5.6	34.75	33.035
15 April 2015	5.6	34.8238	32.3690

The first seismic design code for buildings was established in 2015, in North Cyprus, which is called as "Regulation on buildings to be built in earthquake zones for Northern Cyprus". This was the first national code. This code will be nominated in the current work as Northern Cyprus earthquake code (NCEC-2015). NCEC-2015 provided seismic zoning map of Cyprus having the peak ground acceleration (PGA) values as shown below in figure 2.4.

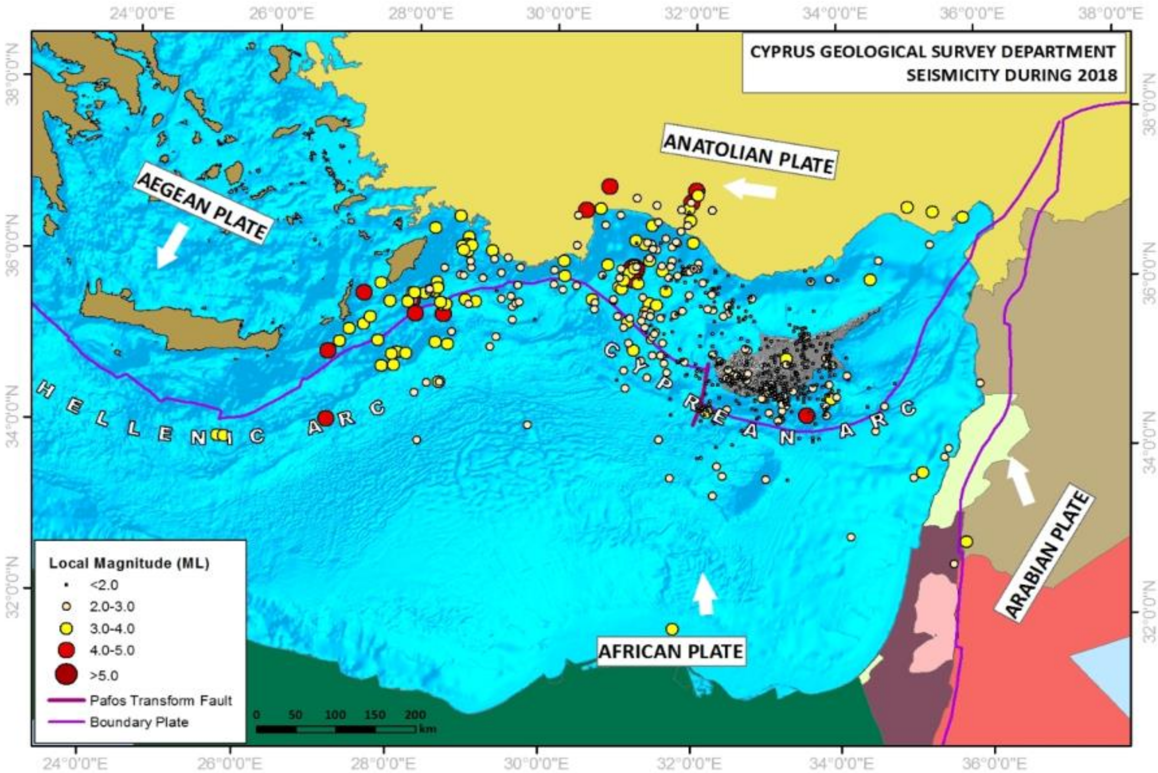

Figure 2.3: Tectonic plate boundaries around Cyprus (Geological Survey Department, 2019)

2.2 Seismic Zone Map and Peak Ground Acceleration (PGA) Values of Cyprus

Figure 2.4: Seismic zoning map of Cyprus from NCEC-2015

- 0.20g-0.30g
- 0.30g-0.35g
- 0.35g-0.40g
- 0.40g-0.45g

2.3 Earthquake Zones

- 1. Earthquake Zone: Ground acceleration value is greater than 0.4g
- 2. Earthquake Zone: Ground acceleration value is between 0.3g and 0.4g
- 3. Earthquake Zone: Ground acceleration value is between 0.2g and 0.3g
- 4. Earthquake Zone: Ground acceleration value is between 0.1g and 0.2g

2.4 Earthquake Zones by Districts in North Cyprus

Figure 2.5: Districts of North Cyprus until 2016 (Districts of Northern Cyprus, 2019)

Seismic zone coefficient details for NCEC-2015 is given in Table 2.2.

Table 2.2: Effective ground acceleration coefficient of districts in North Cyprus (NCEC-2015)

District	Earthquake Zone	Effective Ground Acceleration Coefficient (A_0)
Lefka (Lefke)	2	0.30
Morphou (Güzelyurt)	2	0.30
Famagusta (Gazimağusa)	2	0.30
Nicosia (Lefkoşa)	3	0.25
Trikomo (İskele)	3	0.25
Kyrenia (Girne)	3	0.20

CHAPTER 3
SEISMIC ANALYSIS METHODS

3.1 Overview

The main reason of existing building assessment is based on forecast of the performance for the retrofitting of building if it is needed under a future earthquake effect prediction. Many codes include specifications for analysis and calculation such as TEC-2018, FEMA 356, ATC-40 and EC8 for seismic performance and retrofitting of existent buildings. The analysis methods are separated into two groups as linear analysis and non-linear analysis. Linear analysis types are not recommended during highly irregular structural system analysis. However, it is useful while the building does not respond the design earthquake with respect to elastic behavior. If only elastic material behavior is thought, linear analysis types shall be enough, although P-Delta formulation can still be used. On the other hand, non-linear static procedure is the most safest method to identify the performance of building. Non-linear analysis procedures are the most appropriate options at the time of either material or geometric non-linearity is kept into account throughout modelling and analysis. The linear dynamic analysis type is also worked to prove the compatibility of the design when the non-linear static analysis is preferred to consider for a structure that has important higher response of mode.

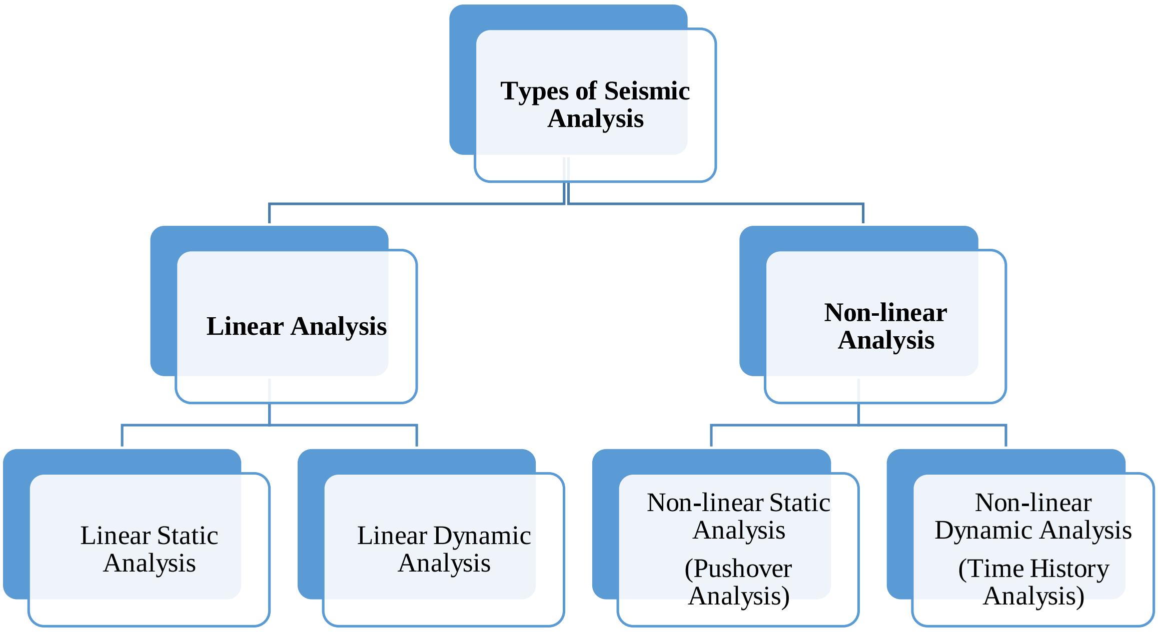

Figure 3.1: Seismic analysis types

3.2 Linear Analysis

3.2.1 Linear static analysis

Linear static analysis is based on strength analysis when the elastic capacity of structural components go beyond the demands of loading conditions. Strength-based demand-capacity (DC) ratios show the adequacy of every component. This method is the most easiest and least time-consuming way due to only the elastic stiffness chracteristics are used for the model.

3.2.1.1 Linear static analysis limitations

- The building period (T) is higher than or equal to 3.5 times solution periods (T_S).
- If the proportion of the horizontal dimension at any storey to the corresponding dimension at a neighbor storey goes beyond 1.4 (except penthouses).
- The building has an extreme irregularity of torsional stiffness in any storey. It happens if the diaphragm over the storey is inflexible and the analysis results shows that the drift along any side of the structure is higher than 150% of the mean storey drift.

- The building has an extreme stiffness or irregular vertical mass. That case is valid when the average storey drift in any storey exceeds the storey below or above by more than 150%.

3.2.2 Linear dynamic analysis

Response spectrum analysis (RSA) is a type of linear dynamic procedure that evaluates the natural vibration mode to notify the possible highest seismic reaction of an elastic structure. It helps to understand the dynamic behavior by measuring pseudo-spectral acceleration, displacement or velocity as a structural period function for a given time history and damping level. On the other hand, RSA is useful to make decision since it is interested in selection of structural type to dynamic performance. Consequently, structural performance objectives should be taken into account throughout response spectrum analysis and preliminary design.

3.3 Non-linear Analysis

3.3.1 Non-Linear static analysis (pushover analysis)

Structural system is subjected to monotonically increasing horizontal loads under constant gravity loads in non-linear static analysis. This method reflects building behavior more realistic during earthquake so it let engineers to make more accurate calculations. In this analysis, the deformation behavior of all elements in the building should be defined. As a result, this method is the examination of the situation at the point where the earthquake forces demand from building and building responses (capacity, force-displacement curve) to the earthquake forces. On the other hand, pushover analysis is helpful to predict earthquake damages by pushing building step by step by considering studied direction to observe plastic joints and to estimate damage occurrences which happens in sections. In other words, it is the divison of dynamic movement into static parts. As a result, they follow the joints until the last joint has been formed or the structure loses stability. It is the most commonly used calculation way in the performance evaluation of existing buildings. The spectral displacement must be converted into the pushover curve axis and the structure must be pushed at the actual displacement value to determine the displacement demand.

3.3.1.1 Plastic hinge formation

Plastic hinges are based on ductile design concept to design earthquake resistance building. Energy spread out throughout the plastic deformation of certain zones at the end of a member without impressing the rest of the building. The plastic hinge performance plays a key role for deformation and load carrying capacities of flexural members. Special attention should be paid for plastic hinge zone of reinforced concrete flexural members due to prevent the buildings from collapse mechanism. Hence, plastic hinge zones are important for seimic performance dertermination of existing buildings. Furthermore, plastic hinge occurs at the maximum moment region of reinforced concrete columns. The critical part of plastic hinge determination is the forecasting of lateral load drift of columns. Axial load level, concrete compressive srength, moment gradient, mechanical properties of transverse and longitudinal reinforcement steel bars, amount of transverse and longitudinal reinforcement steel bars and shear stress value in plastic hinge zone affect plastic hinge length (Narayanan, 2009).

3.3.2 Non-Linear dynamic analysis (time history analysis)

It is a procedure which needs either the FNA (finite element analysis) or the direct-integration. The equations of motion are combined at a series of time steps to characterize dynamic response and inelastic behavior. Loading is time-dependent hovewer it is appropriate for the ground-motion record application. Non-linear analysis can be used to count P-delta effects and material non-linearity.

3.4 Structural Performance Levels

Structural performance levels are categorized into six types in accordance with Fema 356. Performance level of a building is achieved by combining non-structural and structural performance levels. Afterwards, performance objectives are formed by combining performance level with earthquake ground motion.

3.4.1 Immediate occupancy structural performance (IO)

It can be introduced as the damage condition after earthquake that let the structures be safe to occupy. The structure keeps design stiffness before earthquake and strength of the structure. In other words, very limited structural damage is occured in immediate occupancy structural performance level. The basic vertical-and-lateral-force-resisting systems of the building retain nearly all of their stiffness and strength before earthquake. The risk of loss of life or life-threatening injury is negligible due to lack of structural failure.

3.4.2 Damage control structural performance

Damage control structural performance level defines a damage situation which is between immediate occupancy and life safety. It reduces repair time and operation interruption. Consequently, it is useful such a cases when valuable items or historical items are needed to keep in safe if the design budget is high.

3.4.3 Life safety structural performance level (LS)

It shows post earthquake situation of the structures which subjected to significant damages. Although significant damages are occured in the structures, some tolerance leaves against to either partial or total collapse. Main structural components have not displaced and fallen threatening life safety either outside or within the structure. Injuries can happen in the course of quake but life-threatening injury possibility is low. As a result, it is possible to make comprehensive repairs for structures although the damage is not economic to repair.

3.4.4 Limited safety structural performance level

Limited safety structural performance level mentiones the damage condition between life safety and collapse prevention structural performance level.

3.4.5 Collapse prevention structural performance level (CP)

It defines the damage state after earthquake which contains damage to structural elements. The structure still keep going to carry gravity loads but it has no lateral load resistance. Significant damage to the structure has happened that includes decreasing in stiffnes, lateral deformation of the structure and strength of lateral force resisting system. Significant injury cause of falling is possible. The structure can not repair and is not safe for usage as after earthquake may cause collapse.

3.4.6 Structural performance not considered

A building rehabilitation that does not subject the performance of the structure can be accept as structural performance not considered.

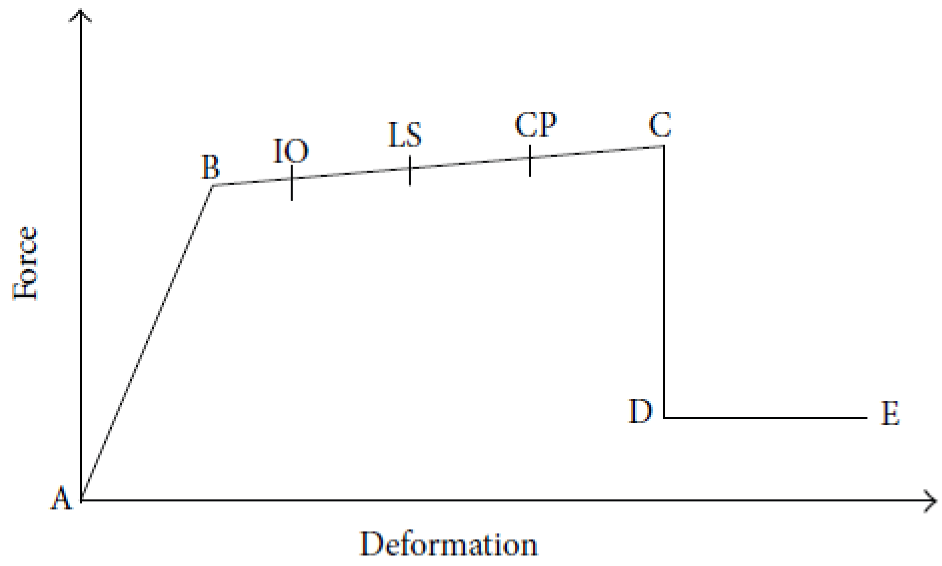

Figure 3.2: Force-deformation graph for plastic hinge formation (Yalciner, 2014)

CHAPTER 4
MASS HOUSING DATA IN NORTH CYPRUS

4.1 Mass Housing History in North Cyprus

Turkish Republic of Northern Cyprus (TRNC) authority constructed affordable reinforced concrete and prefabricated houses for local community due to increment in housing demand between the years of 1986-1998 that is called “mass housing” in North Cyprus. Mass housing projects were conducted by Mass Housing Department, Ministry of Interior in North Cyprus. Moreover, totally 2724 mass housing buildings were constructed in all districts including Nicosia, Famagusta, Kyrenia, Morphou, Trikomo and Lefka (Mass Housing Department, TRNC). Typical RC mass housing types are apartments (ground+three storey and ground+four storey without stairwell tower for both), single storey, two storey and prefabricated houses. Despite the different locations and construction end date of typical RC mass housing buildings, they all have the same architectural and structural plans. Apartment buildings are categorized into three groups according to their floor areas (60 m^2, 85 m^2, 100 m^2). Mass housing buildings constructed before 1998 were designed according to TEC-1975 and the rest was by Seismic Detailing Provisions. As it was mentioned before, Seismic Detailing Provisons was prepared by Chamber of Turkish Cypriot Civil Engineers in 1992. It was similar to TEC-1975 but it had some improvements. It was a kind of declaration. All mass housing data was generated based on information from “Ministry of Interior, North Cyprus”.

Figure 4.1: Two storey, semidetached mass housing buildings in Taşkınköy-Nicosia

4.2 Mass Housing Stock in North Cyprus

4.2.1 Numerical distribution of mass housing types in North Cyprus

Table 4.1: Mass housing types by numbers

Mass Housing Types	Number of Mass Housing
Single storey house	4
Two storey house	1112
Prefabricated	16
Apartment Flat	1592

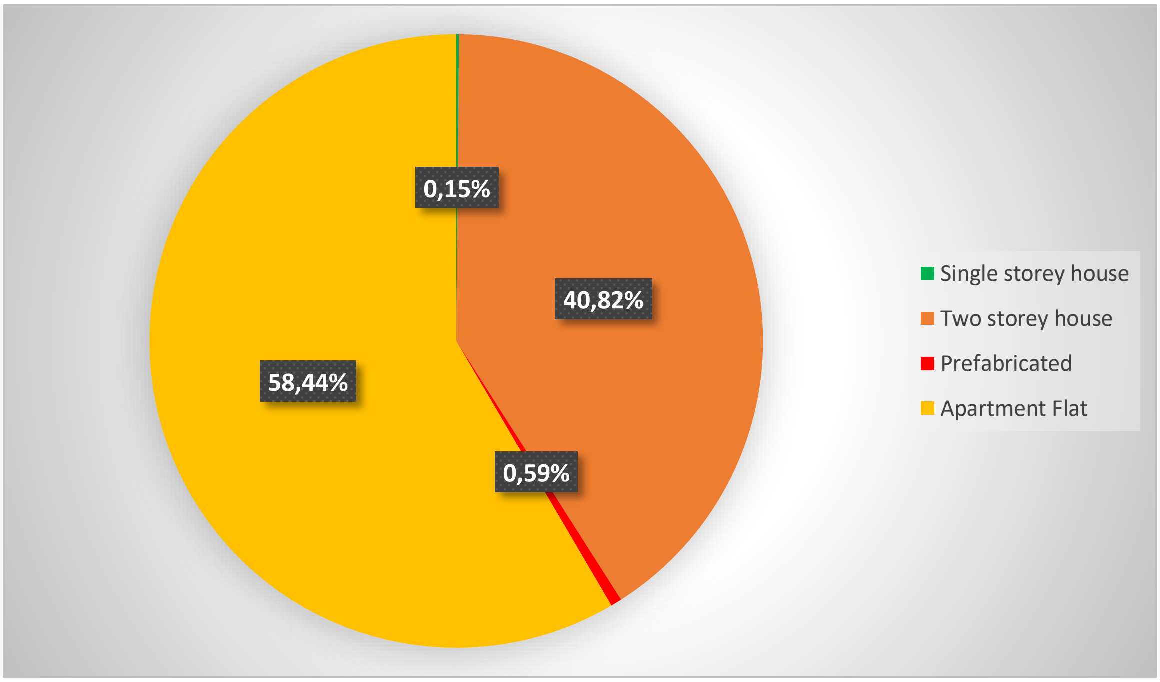

Figure 4.2: Mass housing distribution in North Cyprus

4.2.2 Numerical distribution of mass housing constructions by years

Table 4.2: Yearly mass housing stock

Year	Number of Completed Mass Housing Constructions
1986	288
1987	262
1988	244
1989	534
1990	16
1991	16
1992	208
1993	16
1998	1140

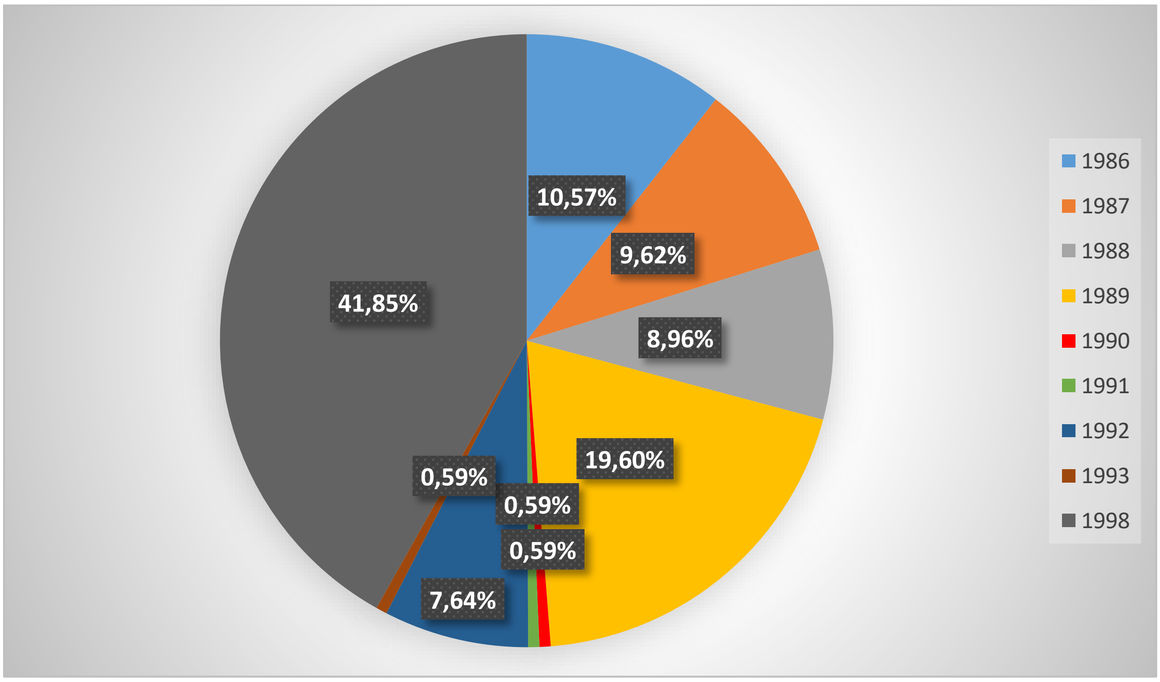

Figure 4.3: Yearly distribution of mass housing constructions

4.2.3 Numerical distribution of mass housing buildings by districts

Table 4.3: Mass housing stock by districts

District	Mass Housing Number
Nicosia	1502
Famagusta	724
Kyrenia	292
Morphou	148
Trikomo	10
Lefka	48

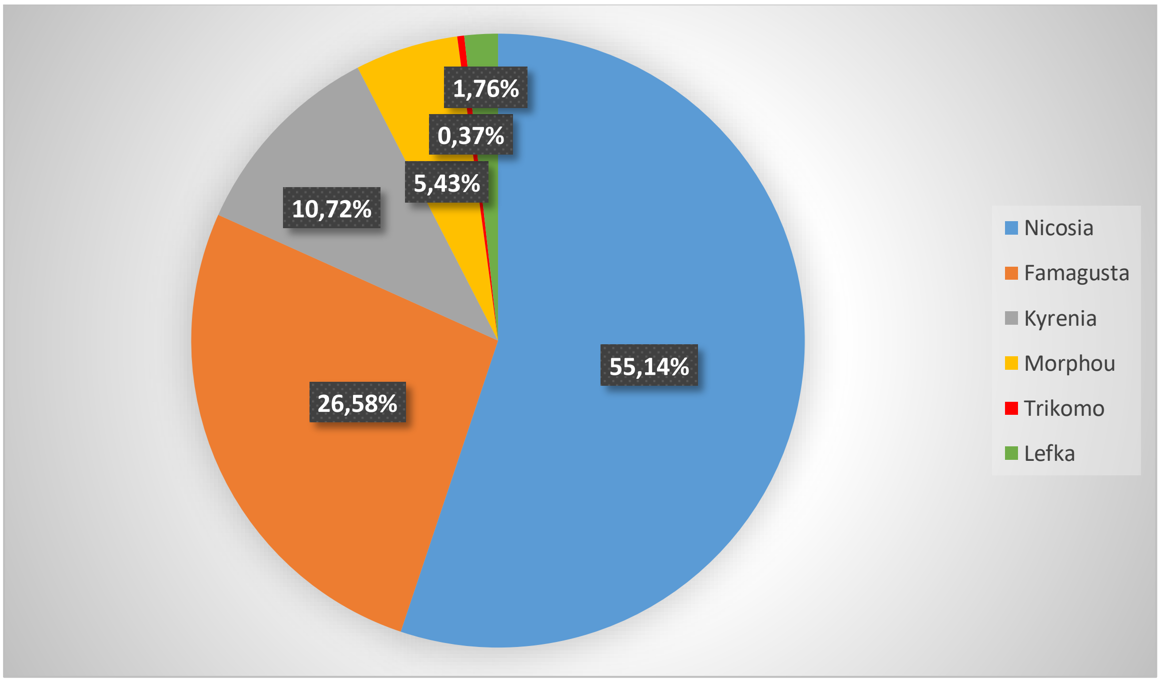

Figure 4.4: Mass housing distribution by districts

4.2.4. Yearly distribution of mass housing buildings by districts

Table 4.4: Yearly mass housing constructions in Nicosia

Year	1986	1987	1988	1989	1990	1992	1998
House Number	136	100	128	410	16	104	608

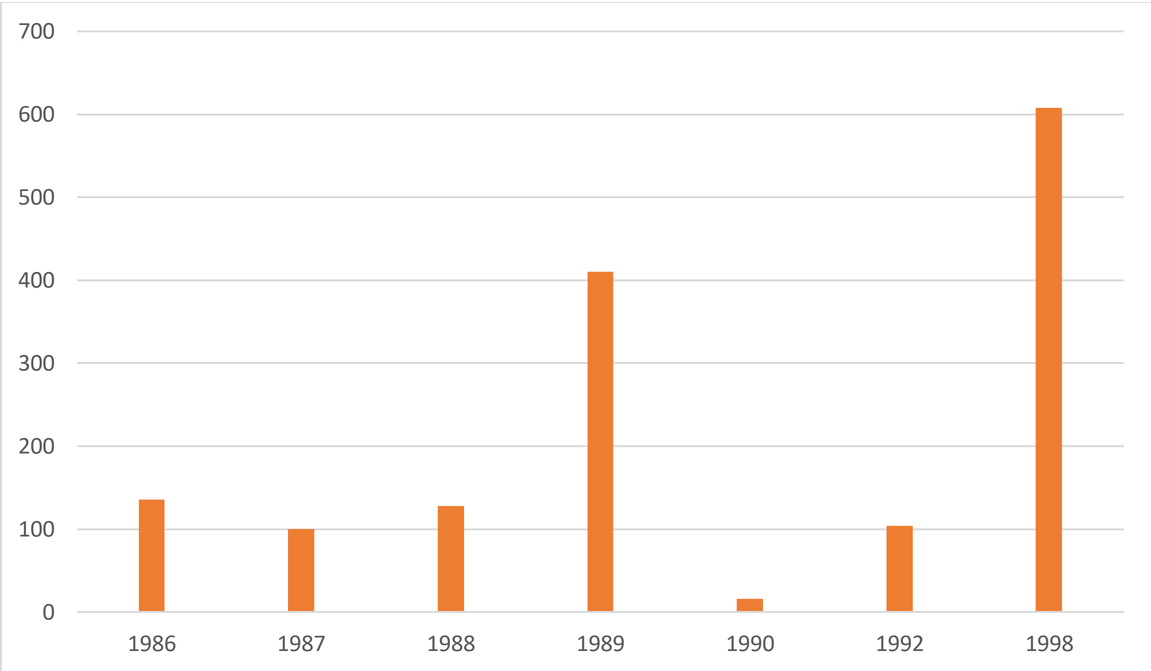

Figure 4.5: Yearly distribution of mass housing buildings in Nicosia

Table 4.5: Yearly mass housing constructions in Famagusta

Year	1986	1987	1988	1989	1992	1998
House Number	80	80	56	124	48	306

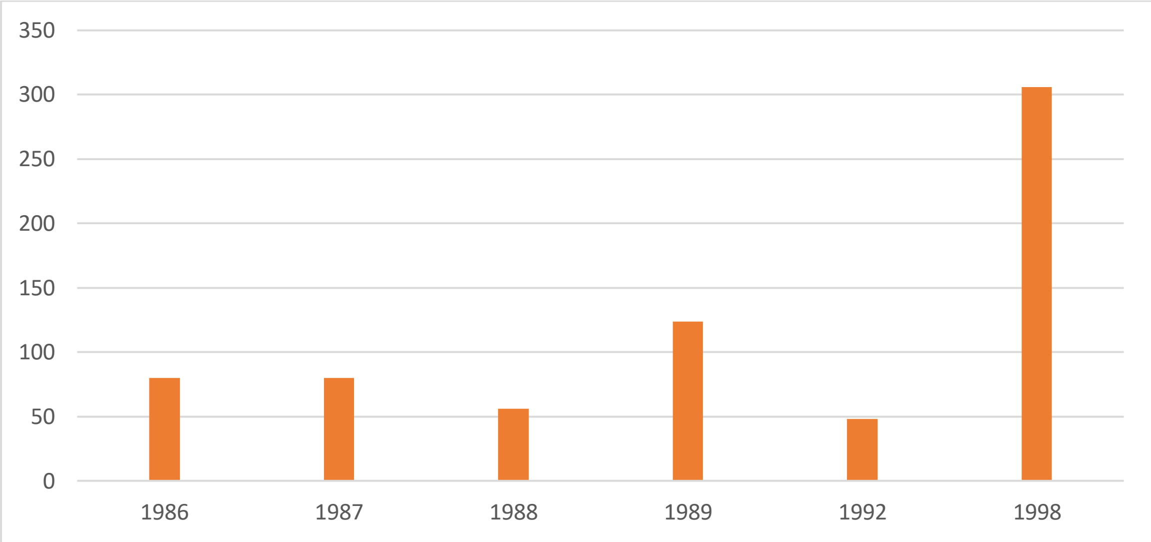

Figure 4.6: Yearly distribution of mass housing buildings in Famagusta

Table 4.6: Yearly mass housing constructions in Kyrenia

Year	1986	1987	1988	1992	1998
House Number	40	40	60	40	112

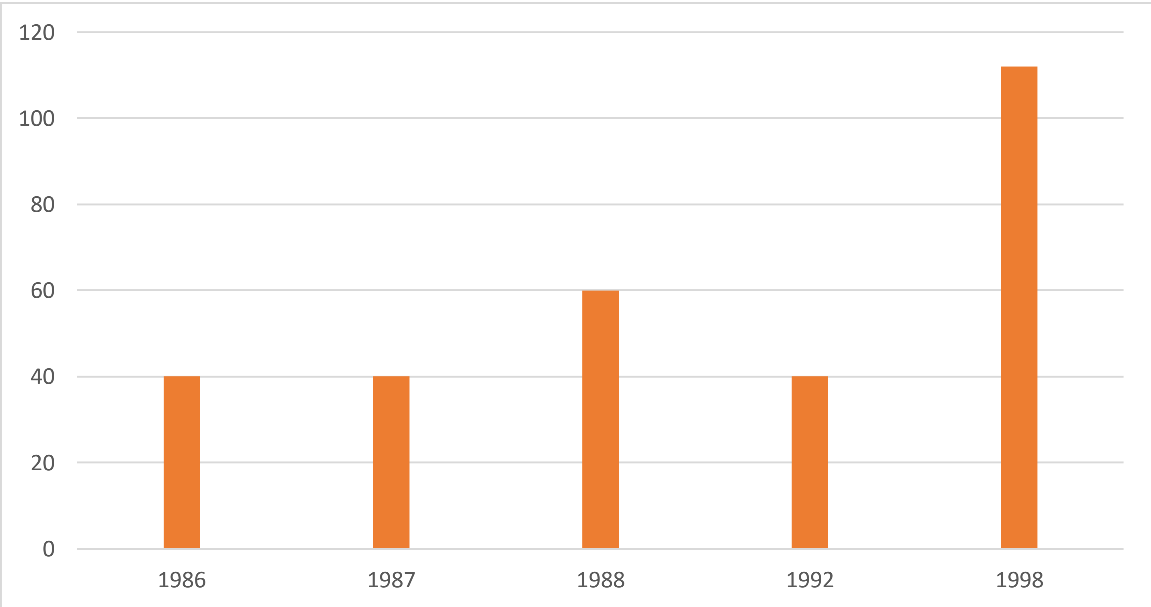

Figure 4.7: Yearly distribution of mass housing buildings in Kyrenia

Table 4.7: Yearly mass housing constructions in Morphou

Year	1986	1987	1992	1998
House Number	32	32	16	68

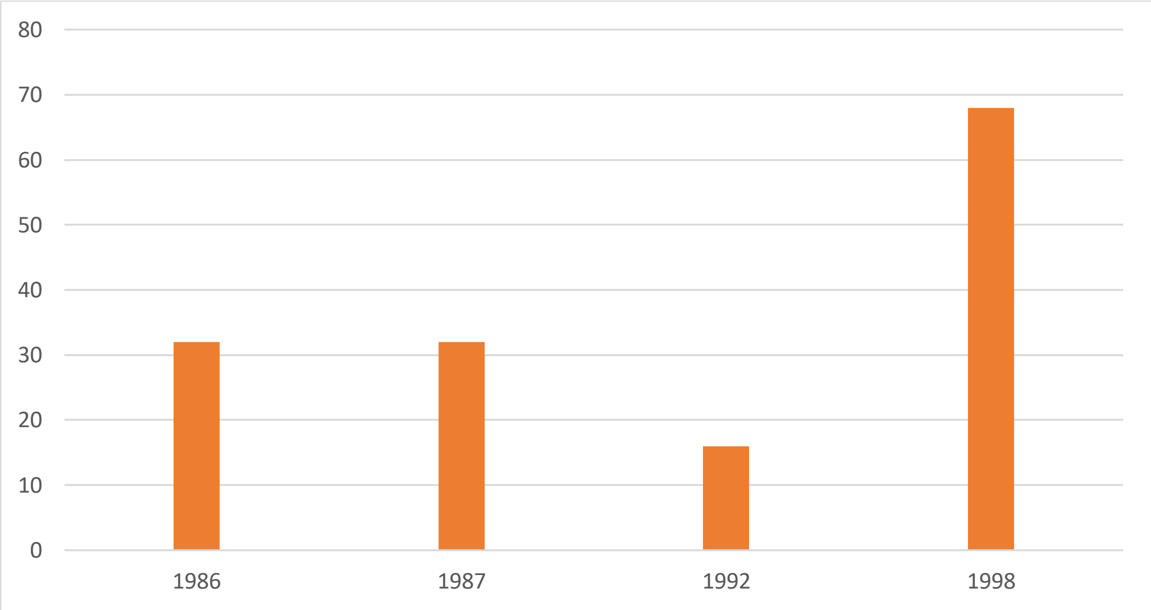

Figure 4.8: Yearly distribution of mass housing buildings in Morphou

Table 4.8: Yearly mass housing constructions in Lefka

Year	1991	1993	1998
House Number	16	16	16

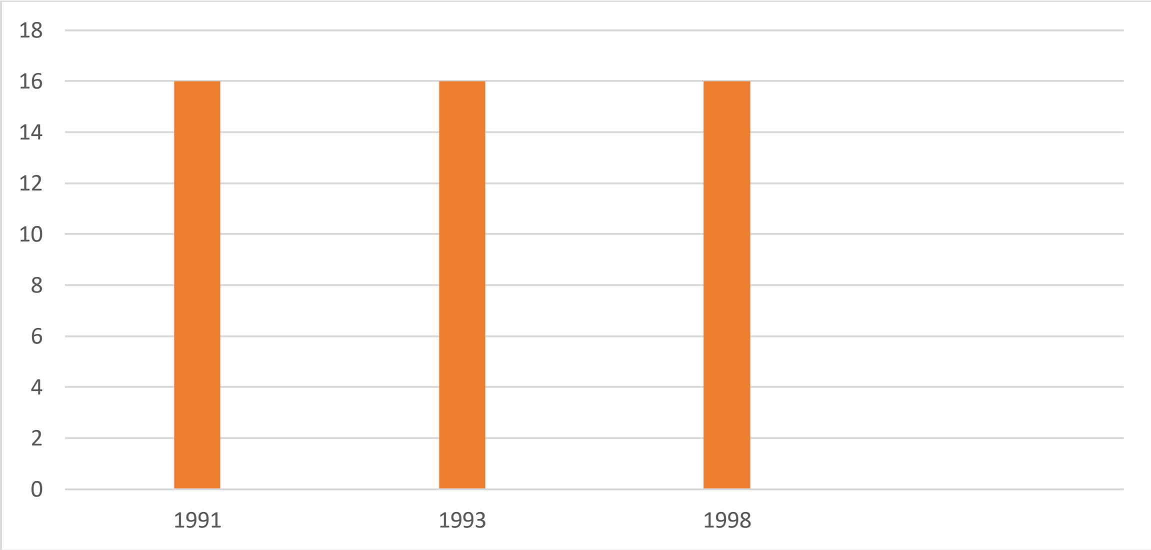

Figure 4.9: Yearly distribution of mass housing buildings in Lefka

Table 4.9: Yearly mass housing constructions in Trikomo

Year	1987
House Number	10

4.2.5 Classification of mass housing types by districts

Table 4.10: Mass housing types in Nicosia

House Type	Two storey	Apartment flat	Prefabricated
House Number	576	920	6

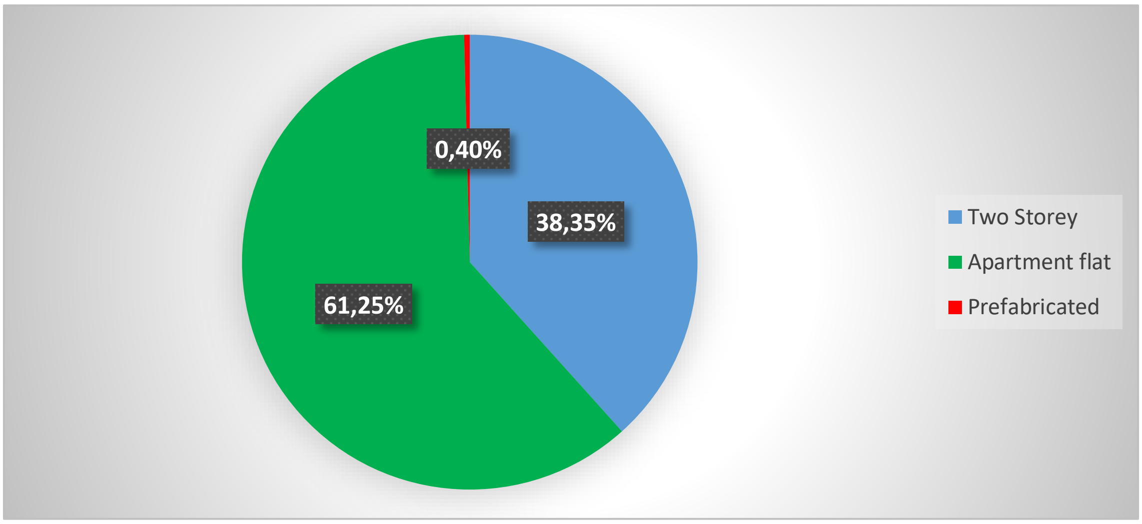

Figure 4.10: Distribution of mass housing types in Nicosia

Table 4.11: Mass housing types in Famagusta

House Type	Two storey	Apartment flat
House Number	332	392

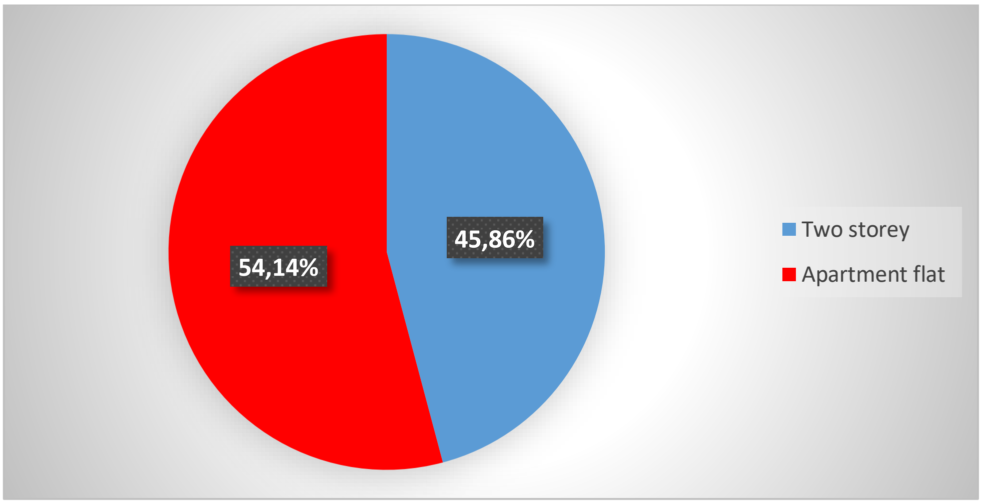

Figure 4.11: Distribution of mass housing types in Famagusta

Table 4.12: Mass housing types in Kyrenia

House Type	Two storey	Apartment flat
House Number	140	152

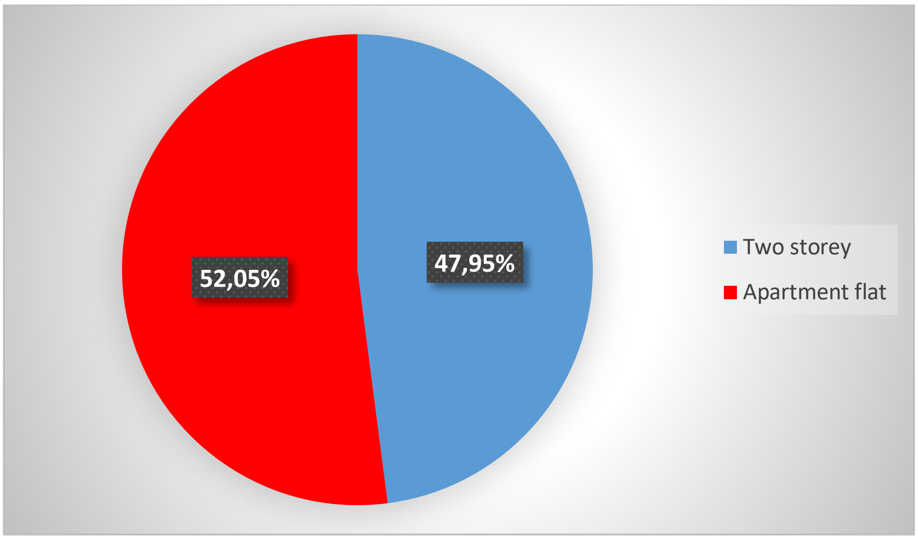

Figure 4.12: Distribution of mass housing types in Kyrenia

Table 4.13: Mass housing types in Morphou

House Type	Two Storey	Apartment flat	Single storey
House Number	64	80	4

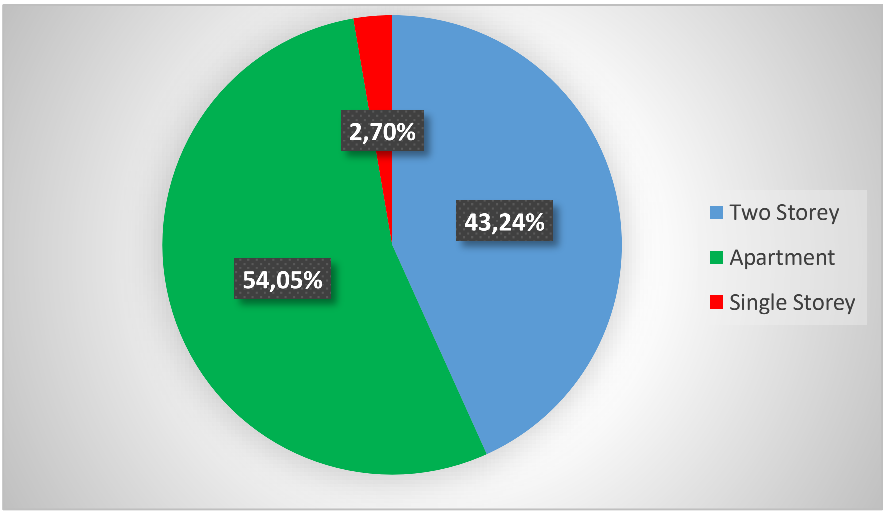

Figure 4.13: Distribution of mass housing types in Morphou

Table 4.14: Mass housing types in Trikomo

House Type	Prefabricated
House Number	10

Table 4.15: Mass housing types in Lefka

House Type	Apartment flat
House Number	48

4.3 Population Data of North Cyprus

De facto population of North Cyprus was 286,527 according to census conducted in 2011 (Statistical yearbook 2016, 2017). Although de facto population was 286,527, household population was 253,851 (Statistical yearbook 2016, 2017). On the other hand, total household size was 2.95. Hence, population data calculations are based on household population.

- Total household size: 2.95
- Total mass housing number: 2,724

For this reason, total household population in mass housing buildings calculated as $2.95 \times 2{,}724 = 8{,}036$.

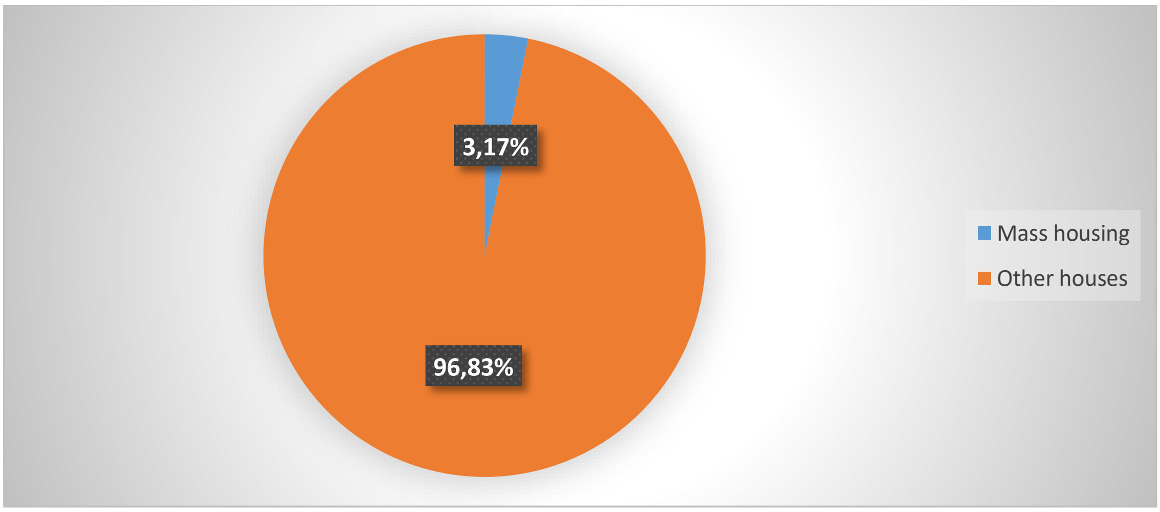

Figure 4.14: Household population distribution in North Cyprus

- Total Household population in Nicosia: 82,331
- Total Household size: 2.97
- Total mass housing buildings in Nicosia: 1,502
- Total household population in mass housing buildings: 1,502 X 2.97= 4,461

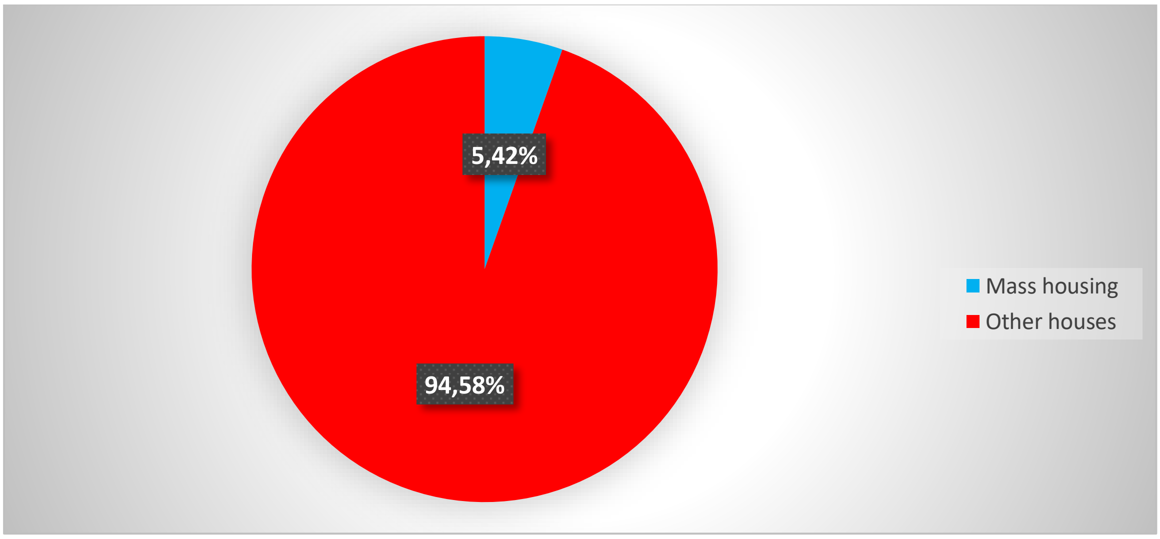

Figure 4.15: Household population distribution in Nicosia

- Total household population in Famagusta: 61,993
- Total household population: 2.93
- Total mass housing buildings in Famagusta: 724
- Total household population in mass housing buildings: 724 X 2.93= 2,121

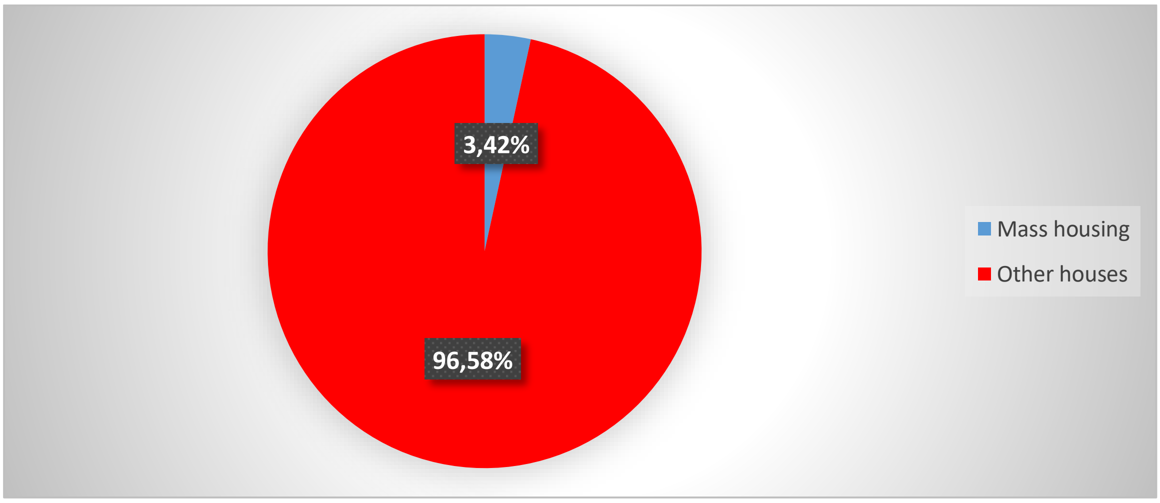

Figure 4.16: Household population distribution in Famagusta

- Total household population in Kyrenia: 61,585
- Total household population: 2.83
- Total mass housing buildings in Kyrenia: 292
- Total household population in mass housing buildings: 292 X 2.83=826

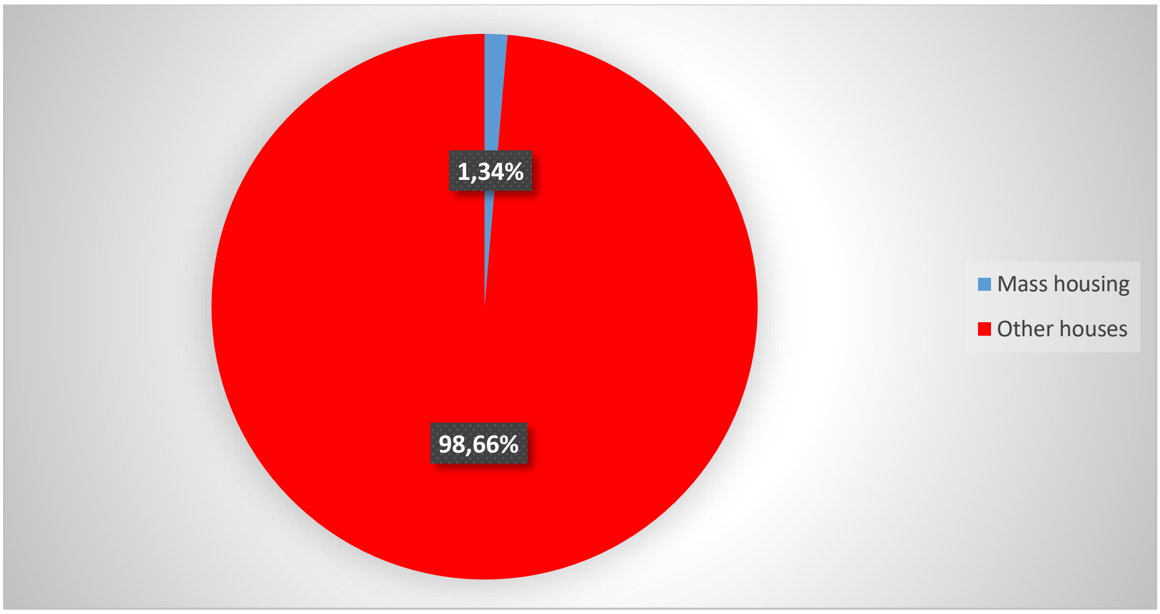

Figure 4.17: Household population distribution in Kyrenia

The statistical data was formed in 2011 and Lefka was not a district in 2011. It was a town in Morphou district. Hence, the population data includes both together Morphou and Lefka.

- Total household population in Morphou: 61,585
- Total household population: 2.96
- Total mass housing buildings in Morphou: 196
- Total household population in mass housing buildings: 196 X 2.96=580

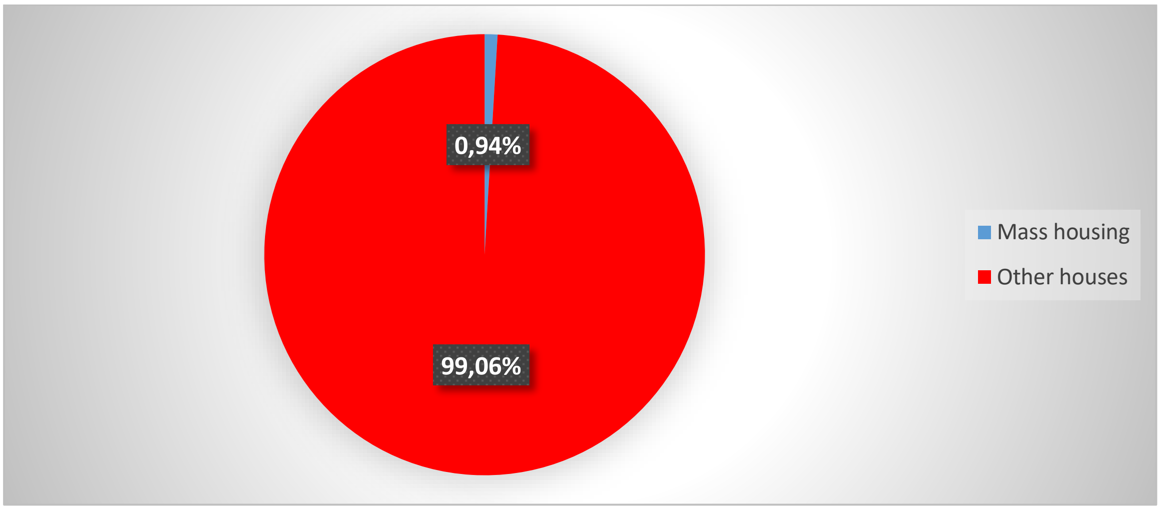

Figure 4.18: Household population distribution in Morphou and Lefka

- Total household population in Trikomo: 21,256
- Total household population: 3.29
- Total mass housing buildings in Trikomo: 10
- Total household population in mass housing buildings: 10 X 3.29= 329

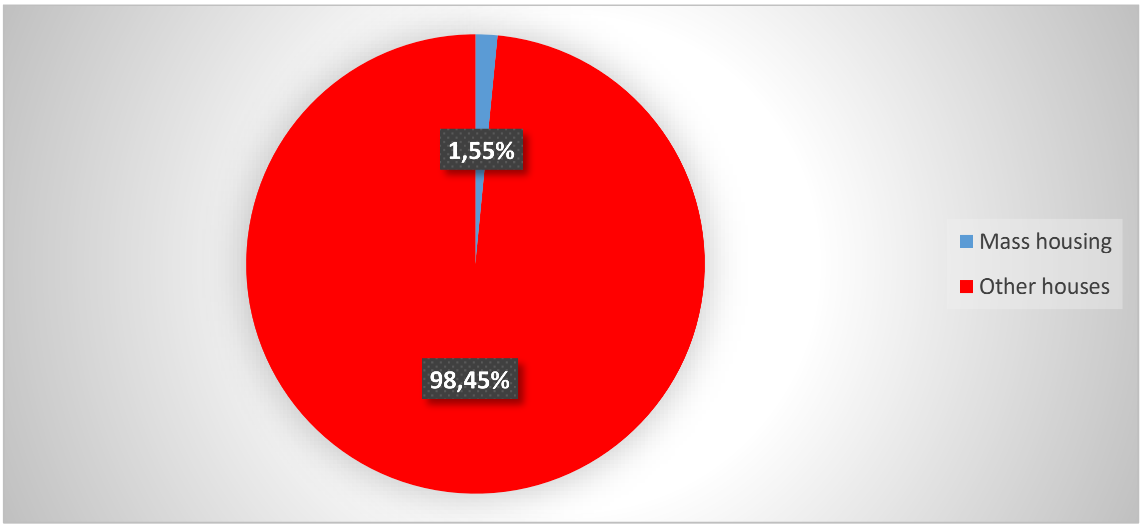

Figure 4.19: Household population distribution in Trikomo

4.4 Mass Housing Classification by Earthquake Code

Table 4.16: Mass housing numbers by earthquake code

Earthquake Code	TEC-1975	Seismic Detailing Provisions
House Number	1584	1140

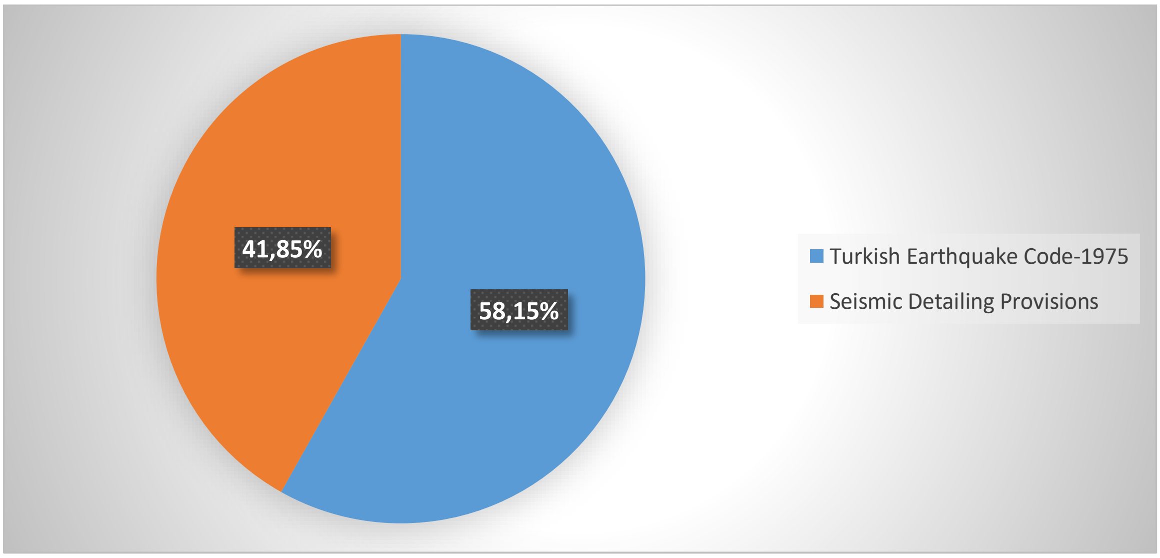

Figure 4.20: Mass housing distribution by earthquake code in North Cyprus

CHAPTER 5
METHODOLOGY

5.1 Assessment

5.1.1 Mass housing selection

One of the main objective of this study is to achieve the general idea of seismic performance of all existing mass housing buildings in North Cyprus. Therefore, one of them picked as a representative on behalf of all mass housing buildings in this study. The selected mass housing is located in Kermiya-Nicosia, North Cyprus as shown in figure 5.1. The location is close to Kermiya crossing gate. The building was chosen from Nicosia because 55.14% of mass housing buildings are in Nicosia (figure 4.4). On the other hand, 61.25% of mass housing buildings are apartment flat in Nicosia so apartment type building selected for this study. The building concerned in this study was designed and built in the 1990's. As mentioned in chapter 4, 41.85% of mass housing buildings completed in 1998 (figure 4.3). All mass housing buildings are apartment type in Kermiya but they are classified into two groups as ground+four storey (except stairwell tower) and ground+three storey (except stairwell tower). Ground+three storey apartments have three types of floor areas such as 60 m^2, 85 m^2 and 100 m^2. The concerned building is a four storey (ground floor plus three storeys above ground) RC apartment building which has 100 m^2 floor area.

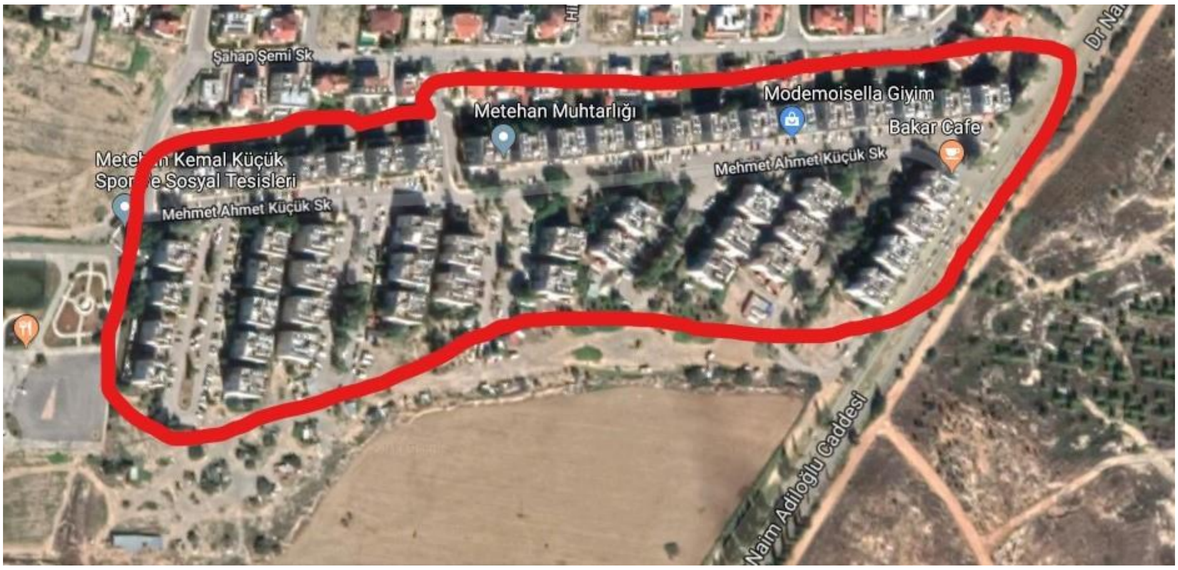

Figure 5.1: Location of selected mass housing

5.1.2 Collection of existing plans and design data

The existing architectural and structural plans were obtained from Mass Housing Department, Ministry of Interior, North Cyprus and all drawings are available in appendix 3. The structural system of the building is reinforced concrete frame, foundation type is continuous footing and slab type is ribbed. The height of the building is 15.65 m and typical storey height is 3 m. Moreover, the dimensions of the building are 15.30 m and 16.00 m within x and y direction respectively. Furthermore, the building was designed in accordance with "Seismic Detailing Provisons". Furthermore, reinforced concrete elements designed according to TS-500 requirements and design loads were defined considering TS-498 requirements. Additionally, local site class is Z2, soil ultimate stress is 20 t/m^2 and seismic zone is 3.

5.1.3 Field inspection

Site investigation is the part of this case study. For this reason, it is needed to determine the compatibility of existing buildings with plans and to observe current damages in buildings. Therefore, dimensions, locations and span length of columns and beams were checked and reinforced concrete structural elements were investigated for crack occurrence, reinforcement steel corrosion and presence of moisture. Figure 5.4 and figure 5.6 shows existing damages in site.

Figure 5.2: Back view of ground+three storey mass housing

Figure 5.3: Front and side view of ground+three storey mass housing

Figure 5.4: Observed cracks on column

Figure 5.5: Anchorage beam

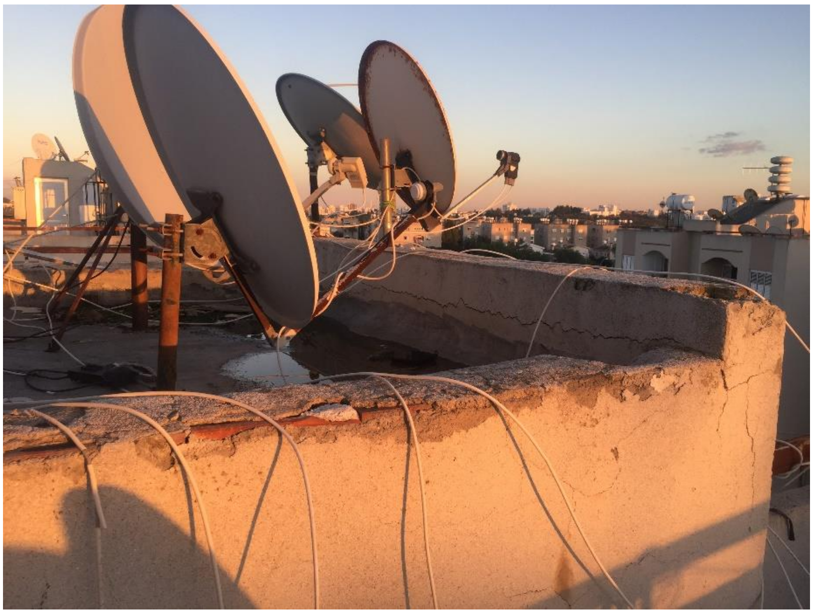

Figure 5.6: Cracks on stairwell tower

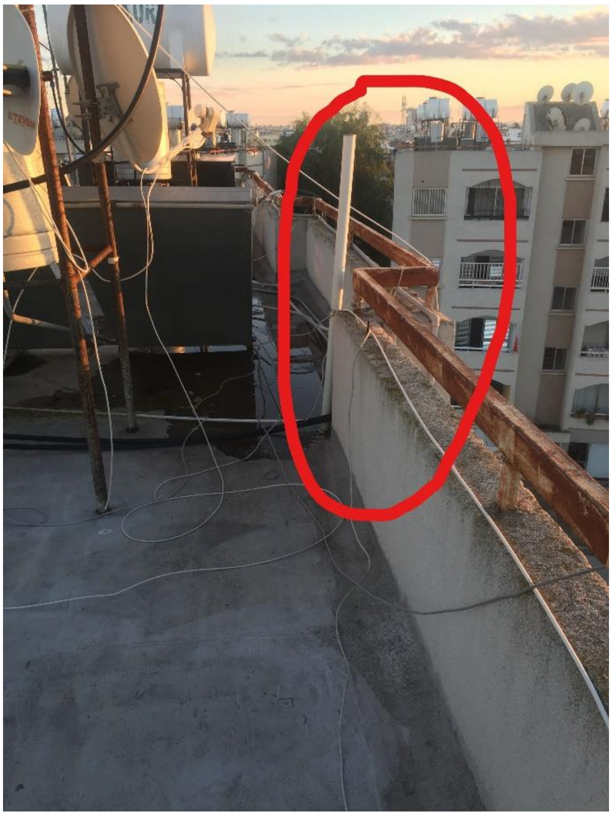

Figure 5.7: Rainwater pipe inside column

5.1.4 Material properties

Material property determination is the significant part of performance analysis. The structural and architectural plans are available for this study therefore material property determination follows comprehensive information level procedure of TEC-2018. Concrete coring is the most accurate way to achieve existing concrete compressive strength. If any performance analysis follow TEC-2018 requirements, concrete coring must be used. Hovewer, this study focused on destructive methods to detect concrete properties. People already live in mass housing buildings. For this reason, previous concrete cores obtained from mass housing buildings used for this study. Chamber of Turkish Cypriot Civil Engineers took concrete core samples from mass housing buildings in 2013 and this study kept into account that cores. Chamber of Turkish Cypriot Civil Engineers shared core report for this academic study and core results evaluated according to TEC-2018 regulations. Chamber of Turkish Cypriot Civil Engineers' laboratory is accrediated by international institutions so it proves the trustability of our scientific study. The core report is available in appendix 2. On the other hand, reinforcement steel details accepted as same as structral plans so S220 reinforcement steel was used in modelling and decreasing in diameter due to corrosion effect was not considered.

5.1.4.1 Determination of concrete compressive strength by destructive method

Since the ready mix concrete facilities were not good in the past, concrete properties can sometimes be incompatible with structural plans. Destructive method based on taking concrete cores from structural elements. Cores are used to identify current compressive strength of existing concrete but they are not only used for compressive srength determination. Additionally, it is applicable for determination of surface abnormalities and crack depth (Güçlüer and Günaydın, 2017). The cores should demonstrate the concrete strength and they souldn't reduce structural element's strength. On the other hand, cores should not be taken from high tension zones of structural elements. Care must be taken not to cut reinforcement during core taking and humidity of the core samples must be protected till the experimental study (Neville, 1995). Longitudinal and transversal steel reinforcements recommended to detect before coring. The cores are taken between reinforcement bars and they have 10 cm diameter and height. The holes must be infilled by using high-strength repair grout. Portable water-cooled drilling machine (concrete core drilling machine) uses for taking core between steel reinforcement bars (Kurtulus and Bozkurt, 2011). Many scientific studies proved that, coring is the most reliable method for concrete compressive strength determination. TEC-2018 requires to take test cores according to conditions specified in “TS EN 12504-1” (Testing concrete in structures – Part 1: Cored specimens – Taking, examining and testing in compression). TS EN 12504-1 includes comprehensive techniques for taking cores from existing reinforced concrete structural elements, their preparation for testing and determination of compressive strength.

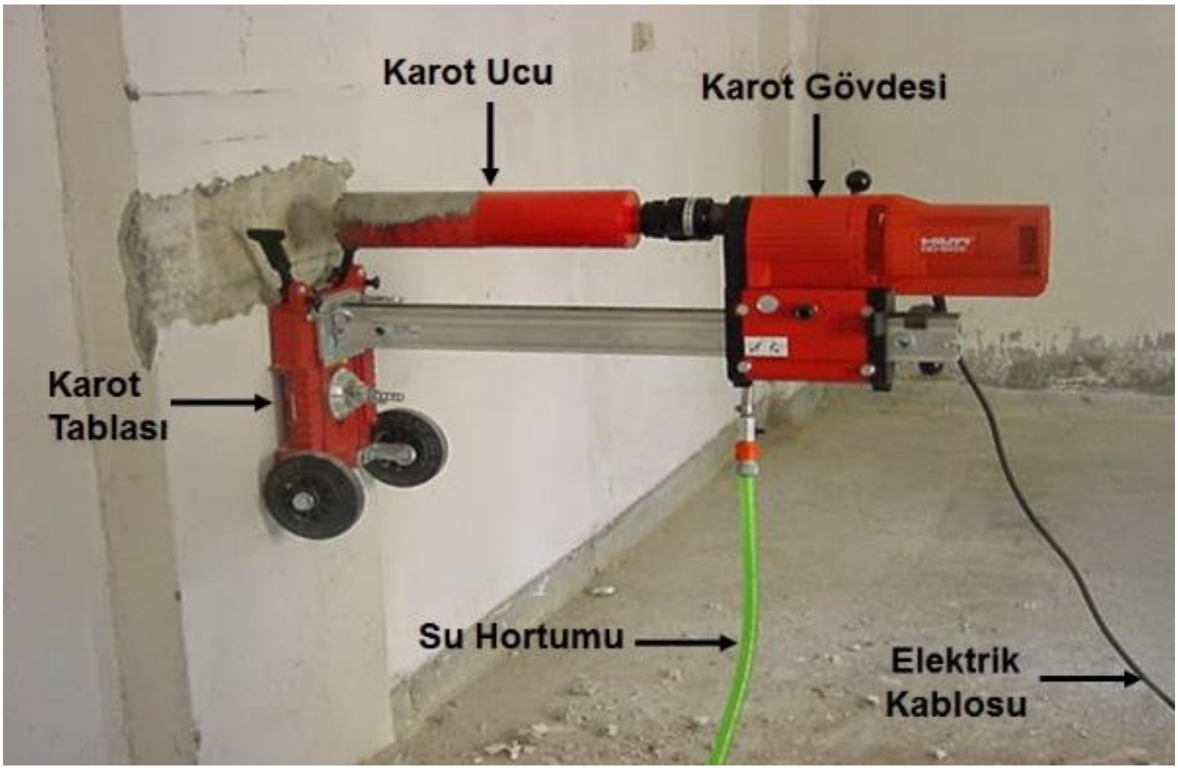

Figure 5.8: Concrete core drilling machine (Karot Nedir?. (2019).)

Table 5.1: Core data

Core location	Column
Number of cores	3
Dimensions of core (mm)	64x64

Table 5.2: Experimental data

Core Number	Place and Element Name of Core Sample		Cylinder Compressive Strength N/mm^2 (MPa)
1	Column 1	r=64 h=64	12.81
2	Column 2	r=64 h=64	11.28
3	Column 3	r=64 h=64	8.48

5.1.4.1.1 Evaluation of core samples

Core samples were evaluated according to appendix 1 Section A.1.13.2.3 based on comprehensive knowledge level.

1. Smallest compressive strength and the average of remaining compressive strength is compared to checked whether is deviating value or not.

   Smallest: 8.48 MPa

   Average of remaining: $\frac{12.81+11.28}{2} = 12.05$ MPa

2. If the smallest value is less than 75% of average of remaining results, it is not taken into consideration.

   75% of average: $\frac{75\times 12.05}{100} = 9.04$ MPa

   8.48 MPa<9.04 MPa, therefore 8.48 MPa was not taken into consideration.

3. The greatest value between (average minus standard deviation value) and (0.85 times average) is accepted as compressive strength of concrete.

   Standard deviation value: $\sigma = \sqrt{\frac{1}{N-1}\sum_{i=1}^{N}(x_i - \bar{x})^2}$ (5.1)

   σ: Standard deviation

   N: Number of elements of an array

   x_i: x th member of an array

   $\bar{x}$: Arithmetic average of numbers in an array

$$\sqrt{\frac{1}{2-1}}\left[(12.81-12.04)^{2}+(12.045-11.28)^{2}\right]=1.08\ \mathrm{MPa}$$

(Standard deviation value=1.08 MPa)

Average:12.045 MPa

$12.05-1.08=10.97$ MPa

$12.05\times 0.85=10.24$ MPa

Due to 10.97 MPa>10.24 MPa, existing concrete type took as 11 MPa. $\mathbf{(10.97\ MPa\cong\ 11\ MPa)}$.

5.1.5 Determination of target performance level

Target performance level was determined in accordance with Table A.1.9 in appendix 1. The following procedures reflect the required steps to identify the performance level respectively.

- Firstly, the representative building is a residential building therefore seismic ground movement level 2 (DD-2) was selected by Section A.1.2 in appendix 1. Hovewer, the possibility to be exceed in 50 years is 10%.
- Secondly, the building usage class is 3 (BKS=3) and building importance factor is 1 due to building purpose of occupancy. The details are available in Table A.1.4 in appendix 1.
- Thirdly, design spectral acceleration coefficient must be determined to pick the right performance level. The following mathematical calculation helps to achieve design spectral acceleration (S_{DS}).

$$S_{Ds}=S_S F_S \qquad (5.2)$$

S_S: 0.2 second, spectral response acceleration

F_S: Local site impact coefficient, 0.2 second

As it is understood from formula, S_S value is needed to determine design spectral acceleration. Disaster and Emergency Management Authority's (AFAD) hazard maps are used to identify S_S values regards to land coordinates. Although hazard maps are required to determine the value, there is no hazard map for Cyprus Island. Hovewer, literature research created another ways to use alternative methods for spectral response acceleration identification. Lubkowski and Aluisi proposed a formula to derive S_S and S_1 parameters from PGA maps (Lubkowski and Aluisi, 2012). PGA map of Cyprus is already available in chapter

2. Selected building is located on Nicosia and Nicosia takes part in seismic zone 3 with 0.25 A_0 value as shown in Table 2.2.

Lubkowski and Aluisi proposed formula is;

$$\frac{\mathrm{Ss}}{\mathrm{PGA}} = 2.265 \qquad (5.3)$$

$$S_S = 0.25 \times 2.265 = 0.57$$

F_S determination,

The existing mass housing was designed considering Z2 local site properties. Since Z2 local site class properties correspond to ZD in TEC-2018, ZD is taken as local site class. Thus, F_S value was determined by Table A.1.2.

Since 0.57 is not available in the table interpolation was used between 0.50 and 0.57.

Interpolation for 0.57

$$\frac{0.07 \times 0.2}{0.25} = 0.06$$

$$1.4 - 0.06 = 1.34$$

$$S_{DS} = S_S F_S \qquad (5.4)$$

$$0.57 \times 1.34 = 0.76$$

$$S_{DS} = 0.76$$

- Fourthly, earthquake design class was identified as DTS=1 by using Table A.1.5 considering design spectral acceleration and building usage class factors.
- Lastly, target performance level was found “controlled damage” according to Table A.1.9. Controlled damage occurs when earthquake damage level-2 meets with seismic design class 1 in existing reinforced concrete buildings.

5.2 Modelling

The modelling of selected building was made by Turkish code based software STA4-CAD V14.1. STA4-CAD is one of the most commonly used structural engineering software in Turkey and North Cyprus. It can be used to design new and existing reinforced concrete buildings. STA4-CAD includes TEC-2018 data base which made it fundamental software to prefer it for this study. In this section, input and structural section data of the selected building are presented.

5.2.1 General building data

- The selected building is a ground+three storey building except stairwell tower. The software model includes stairwell tower therefore it was modelled as five storey building.
- As mentioned in Section 5.1.5 seismic ground movement level is 2 (DD-2).
- Local site class is ZD.
- Short period region, spectral acceleration coefficient was calculated in accordance with Lubkowski and Aluisi proposed formula as described in equation 5.3 due to lack of seismic hazard maps of Cyprus. Thus, S_S value is 0.57.
- Spectral acceleration coefficient for T=1.0 second was calculated by using Lubkowski and Aluisi proposed formula which derive S_1 parameter from PGA maps.

  $$\frac{S1}{PGA} = 0.753 \qquad (5.5)$$

  A_0 for Nicosia: 0.25 (Table 2.2)

  $S_1 = 0.25 \times 0.753 = 0.19$
- Short period design spectral acceleration was already calculated in Section 5.1.5 so same value was used again (S_{DS}=0.76).
- Design spectral acceleration coefficient for T=1 was calculated according to TEC -2018 regulations.

  $$S_{D1} = S_1 F_1 \qquad (5.6)$$

  S_1=0.19

  F_1 was determined by applying interpolation according to values in Table A.1.3.

  Interpolation for 0.19

  $\frac{0.09 \times 0.2}{0.10} = 0.18$

  $2.4 - 0.18 = 2.22$, therefore F_1=2.22

  $0.19 \times 2.22 = 0.42$, therefore S_{D1}=0.42

- Structural behavior factor (R) is taken as 4 automatically when new retrofitting members are existing by STA4-CAD.
- Overstrength factor (D) was formed itself as 2.5 when structural behavior factor is 4.

- The building is residential type building so importance factor (I) is 1.
- Reduction factor of live load (n) is 0.3 for residential buildings.
- The building is located on Nicosia hovewer modulus of subgrade reaction, soil ultimate stress took as 3000 t/m^3 and 20 t/m^2 respectively.

Table 5.3: General building data

Story Number	5
Spectral Acceleration Coefficient (s_{ds}/s_{d1})	0.76/0.42
Structural Behavior Factor (R_x/R_y)	4
Overstrength Factor (D)	2.5
Seismic Importance Factor (I)	1
Live Load Seismic Reduction Factor (n)	0.3
Effective Seismic Load Level H_x/H_y (m)	0
Modulus of Subgrade Reaction K_o (t/m^3)	3000
Soil Ultimate Stress q_t (t/m^2)	20
Live Load Reduction Factor (C_z)	1
Seismic Load Eccentricity	0
Seismic Analysis min. Force Ratio (β)	0.8
Top Story No (TDY Code)	5
Application Relative Level (m)	0

5.2.2 Load combinations

Load combinations are ready defined in STA4-CAD software. In the model, 1.4G+1.6Q, 1.4G+1.6Q+1.6S, G+1.2Q+1.2T, G+Q+E, G+Q+S+E, 0.9G+E, G+1.3Q+1.3W, G+1.3Q+S+1.3W, 0.9G+1.3W and 0.9G+0.9S+1.3W combinations were used due to reinforced concrete structure type.

G: Dead load, **Q:** Live load, **S:** Snow load, **T:** Temperature changes, **E:** Earthquake load, **W:** Wind load

5.2.3 Column data

Columns were defined exactly same as structural plans with respect to column dimensions and steel reinforcement details. All columns have ϕ8/17 stirrup details as seen in existing plans. There are rain water pipes inside columns but all columns are full of concrete in software model. In other words, there is no pipe hole inside columns. The columns were sized considering seismic detailing provisions hovewer miminum dimension of columns is 25 cm. All column sections are available in Table 5.4.

Table 5.4: Column sections

Column Width (cm)	Column Height (cm)
25	30
25	40
25	50
25	60
25	70

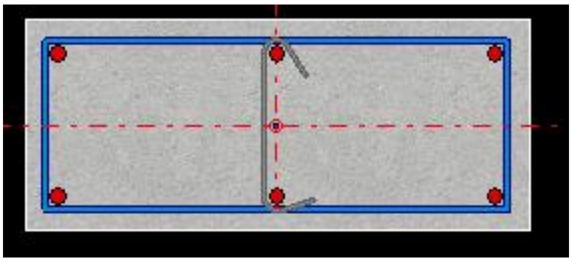

Figure 5.9: Existing steel reinforcement in 25 cm x 60 cm column

5.2.4 Beam data

Beam were defined exactly as same as with existing structural plans. Steel reinforcement details are also included in software model. All beams have 19 cm brick load as dead load and ϕ8/20 stirrups. The dimensions of all beams are available in Table 5.5.

Table 5.5: Beam sections

Beam Width (cm)	Beam Height (cm)
20	40
20	50
20	60
20	70
20	80

Figure 5.10: Existing steel reinforcement in beam 108

5.2.5 Slab data

Slab sections were defined as same as with existing structural plans. Typical slab types of this case study are divided into two groups as ribbed slab and cantilever slab. Ribbed slab was used in floors inside the building and cantiveler slab was used for balconies. The infill material of the ribbed slab is brick. Total thickness of ribbed slab is 17 cm including 10 cm brick and 7 cm concrete cover. Design loads were defined such as live load and dead load. Live loads were choosen as 0.2 t/m^2 for typical floors and 0.35 t/m^2 when any partial wall is existing on floor considering TS-498 regulations. Moreover, dead load also defined due to floor covering as 0.212 t/m^2 (mosaic). On the other hand, thickness of the cantilever slab is 17 cm as well. Staircases are also included in the STA4-CAD model as slab member. The thickness of the staircases is 20 cm. 0.50 t/m^2 and 0.35 t/m^2 live loads were defined for cantilever slab and staircases respectively. Additionally, floor covering load of cantilever slab and staircases are same with ribbed slab.

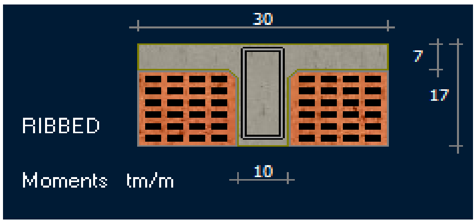

Figure 5.11: Ribbed slab section

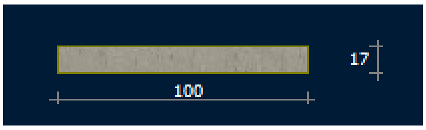

Figure 5.12: Cantilever slab section

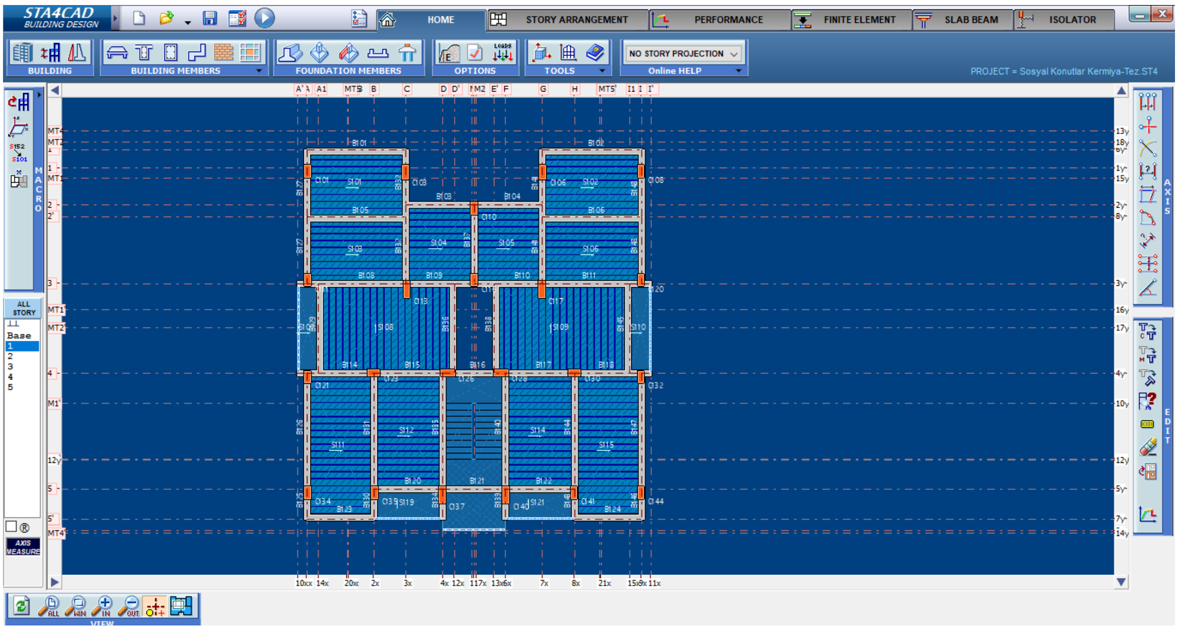

Figure 5.13: Typical floor plan of selected mass housing

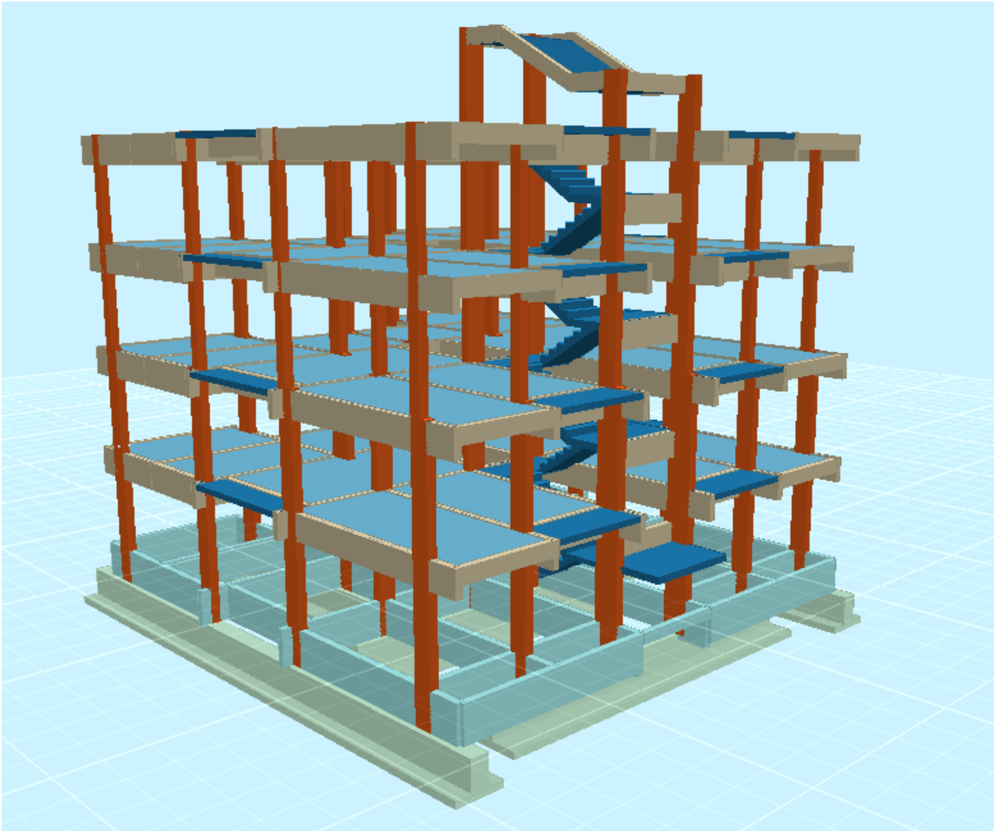

Figure 5.14: 3D view of model

5.2.6 Performance options

Building performance data is based on comprehensive information level since existing structural plans are available. Hovewer, information level coefficient is 1. On the other hand, concrete type and reinforcement steel was choosen as 11 Mpa and S220 respectively. Additionally, all reinforcement details of structural members were input to the model same as with structural plans.

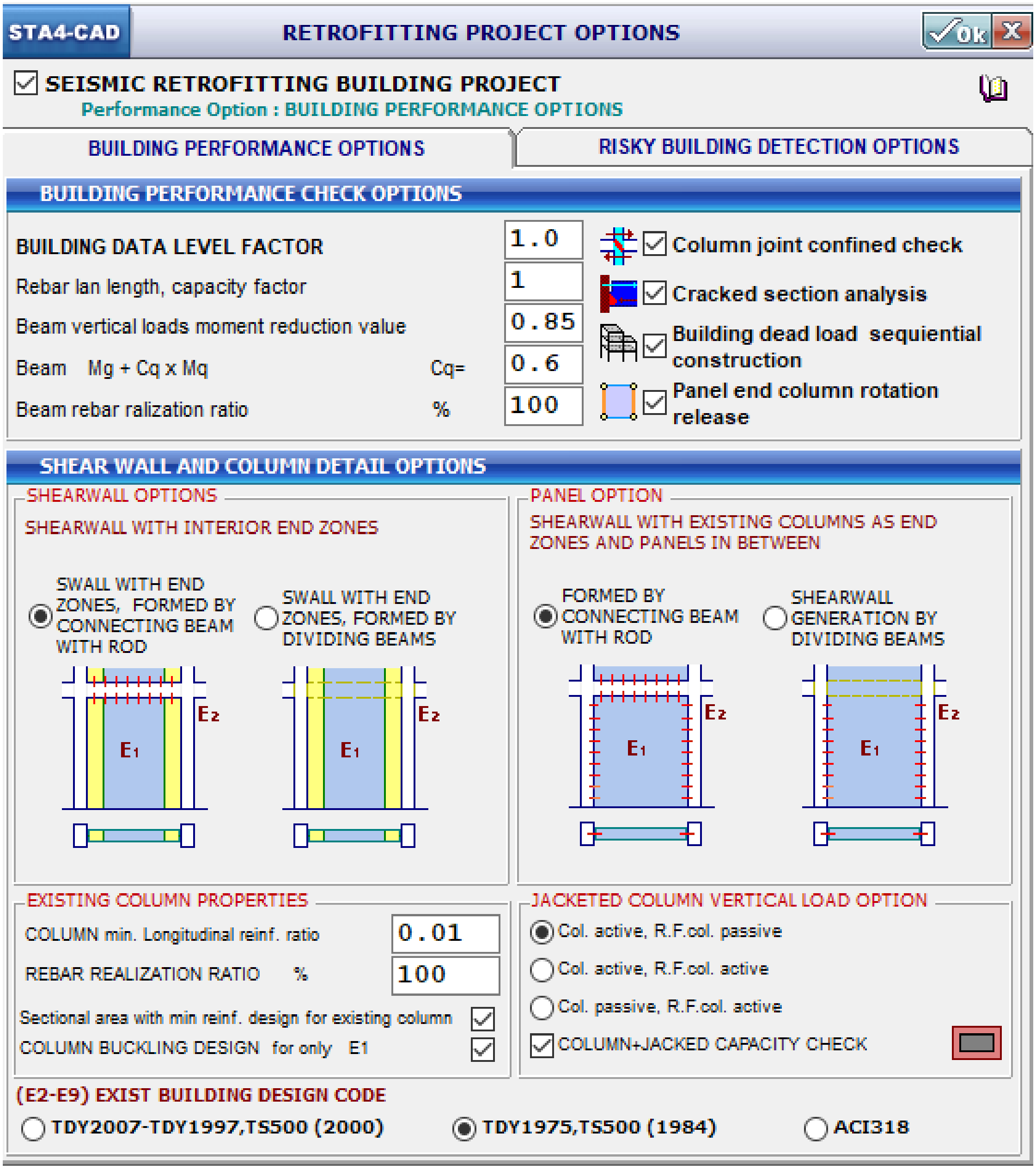

Figure 5.15: Building performance options

5.2.7 Project options

The scope of study is about seismic response of an existing reinforced concrete frame building hovewer "non-linear static analysis" was choosen. The main purpose of non-linear method is the calculation of plastic deformation and plastic rotation demands for ductile behavior and the internal force demands of brittle behavior for a given earthquake. Afterwards, the structural performance evaluation at the section and building level shall be done by comparing demand sizes with the deformation and internal capacities.

Seismic analysis type was chosen as multi mode analysis. Multi mode analysis can be used when building height class is higher than two (BYS≥2). In this method, the maximum values of the behavior magnitudes are calculated by using the modal calculation method when each vibration mode taken into consideration by using the earthquake design spectrum in the direction of an earthquake. The largest modal behavior magnitudes, which are calculated but not synchronous for sufficient vibration mode, are then combined in a statistical manner to obtain the largest approximate values.

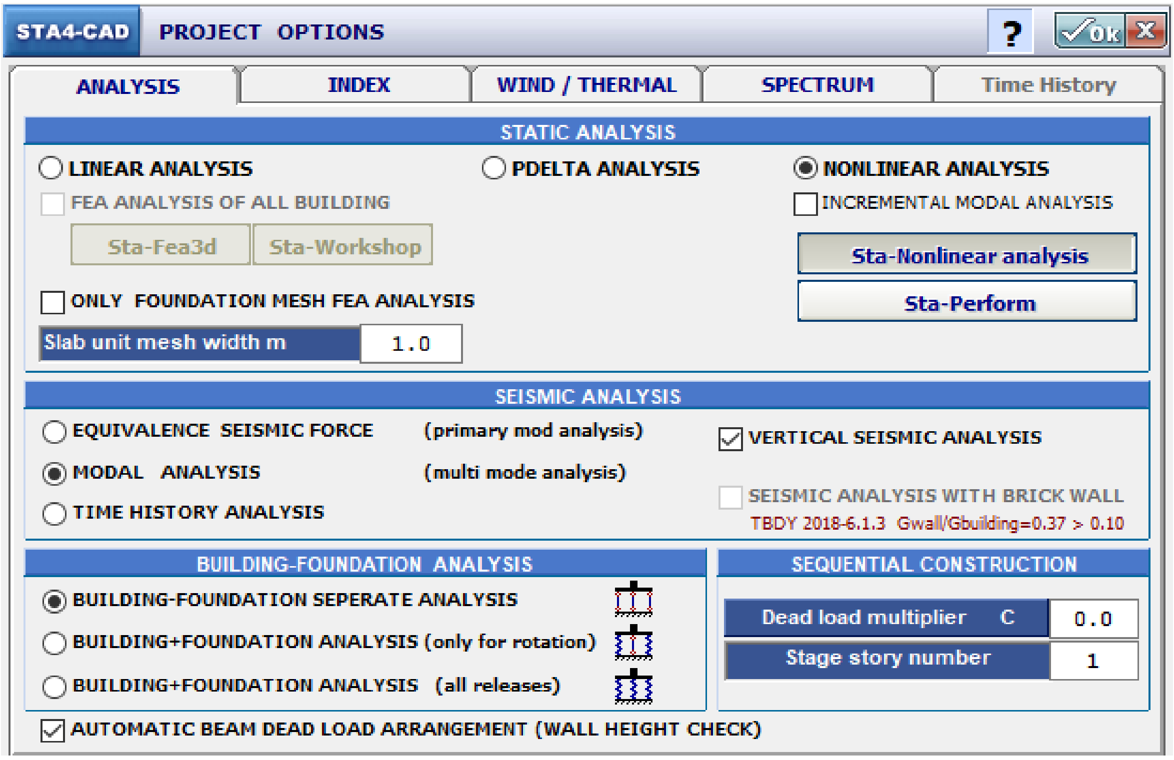

Figure 5.16: Analyse options

CHAPTER 6
RESULTS

Performance focused earthquake design is a modern approach for seismic resistant design. Capacity and demand are two important key terms of a performance focused design procedure. In this study, pushover analysis was used to achieve the seismic performance of the selected structures. Capacity curve is based on estimation of target displacement. As shown in the results given below, both capacity and demand curve are represented in response spectral variables. The intersection point of these curves is performance point according to literature review. Target displacements with respect to x and y direction are shown in figure 6.1 and 6.2.

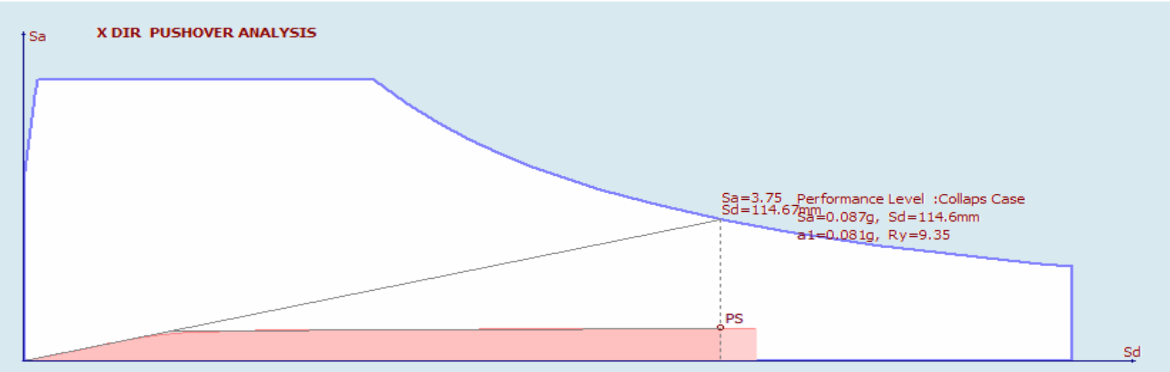

Figure 6.1: Target displacement with respect to X direction

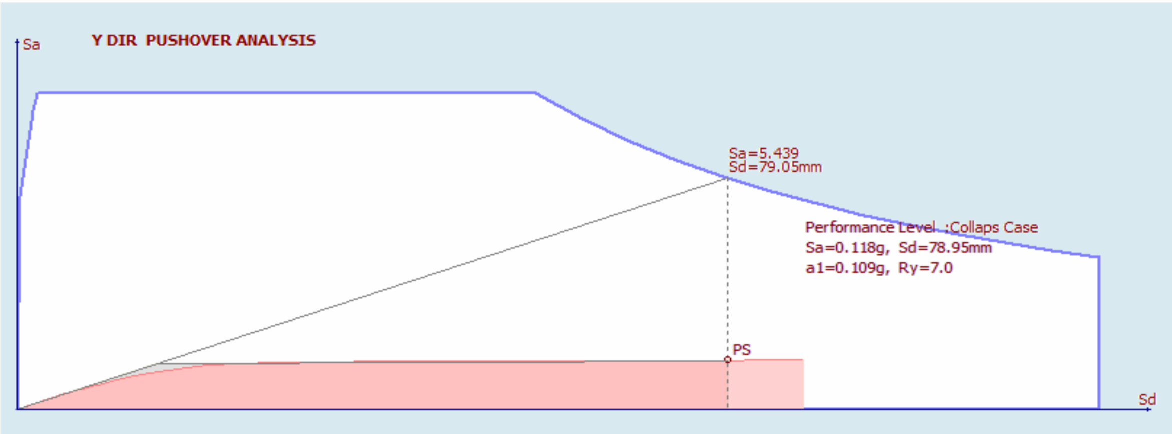

Figure 6.2: Target displacement with respect to Y direction

Pushover curves were achieved according to base shear and horizontal displacement iterations as shown in seismic report in appendix 4. Performance point is achieved when capacity spectrum curve is close to equivalent yield acceleration belonging to first mode accordingly "Turkish Earthquake Code". The displacement for X direction is 114.6 mm and 78.95 mm for Y direction according to figure 6.1 and 6.2. Plastic hinges start to occur when displacement exceeds 114.6 mm and 78.95 mm for X and Y directions respectively.

6.1 Spectral Displacement Check

Spectral displacements for X and Y directions are calculated employing the TEC-2018. The results obtained have been compared with those of STA4-CAD and a perfect match has been observed. The calculation details are given and explained below.

$$S_{ae}(T) = \frac{S_{D1}}{T} \tag{6.1}$$

$$S_{de}(T) = \frac{T^2}{4\pi^2} g S_{ae}(T) \tag{6.2}$$

S_{ae} (T): Horizontal elastic design spectral acceleration

T: Building period

S_{D1}: Design spectral acceleration coefficient for T=1 S

S_{de} (T): Horizontal elastic design spectral displacement

g: Earth's gravity

X-direction;

$$S_{ae}(T) = \frac{0.42}{1.099} = 0.3822$$

$$S_{de}(T) = \frac{(1.099)^2}{4\pi^2} x9.81x0.3822 = 0.1147 \text{ m} = 114.7 \text{ mm}$$

$$114.7 \text{ mm} \cong 114.6 \text{ mm}$$

Y-direction;

$$S_{ae}(T) = \frac{0.42}{0.757} = 0.5548$$

$$S_{de}(T) = \frac{(0.757)^2}{4\pi^2} x9.81x0.5548 = 0.0790 \text{ m} = 79 \text{ mm}$$

$$79 \text{ mm} \cong 78.95 \text{ mm}$$

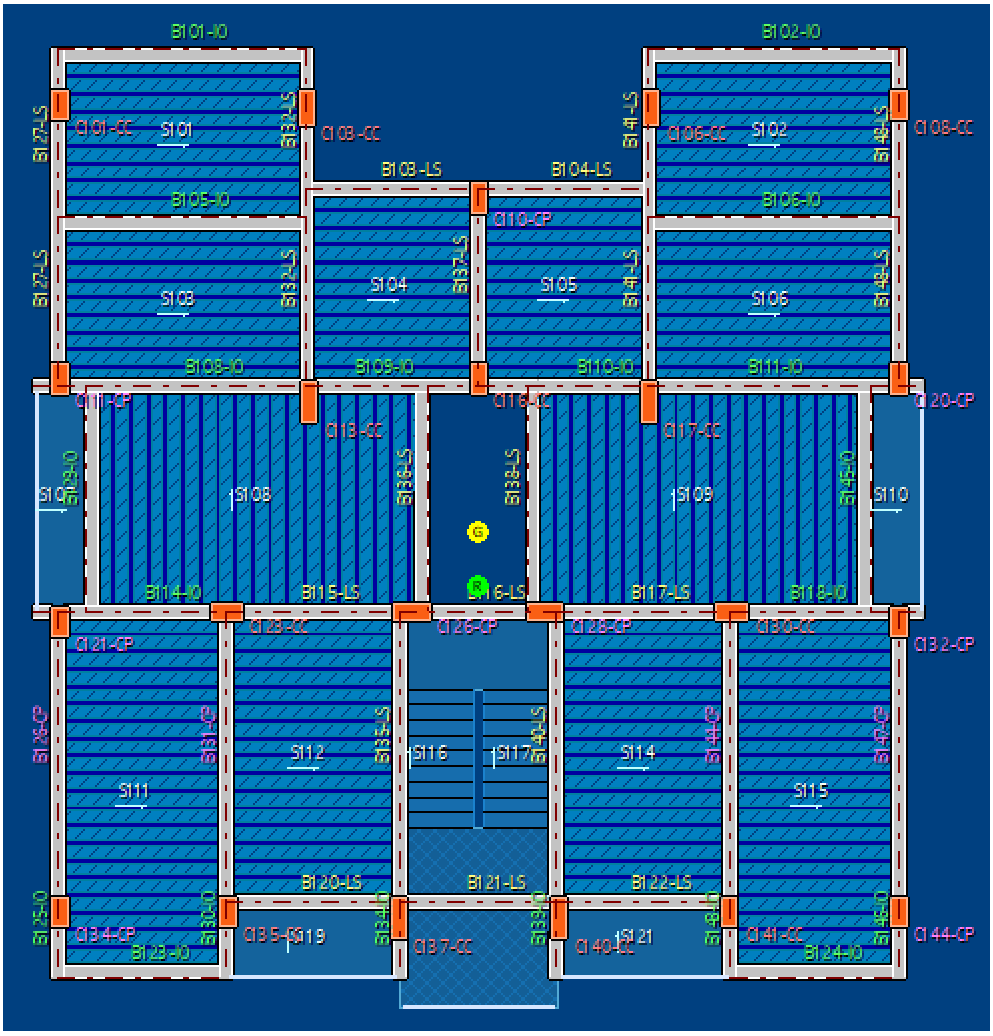

Figure 6.3: Section damages on ground floor for an existing apartment type mass housing structure

Section damages of columns and beams on ground floor of an existing apartment type mass housing are given in Table 6.1 and 6.2, respectively. 60% of ground floor columns are in collapse case.

Table 6.1: Section damages of columns

Column Names	Section Damages
C101, C103, C106, C108, C113, C116, C117, C123, C130, C135, C137, C140, C141	Collapse Case
C110, C111, C120, C121, C126, C128, C132, C134, C144	Collapse Prevention

Table 6.2: Section damages of beams

Beam Names	Section Damages
B101, B102, B105, B106, B108, B109, B110, B111, B114, B118, B123, B124, B125, B129, B130, B134, B139, B143, B145, B146	Immediate Occupancy
B103, B104, B115, B116, B117, B120, B121, B122, B127, B132, B135, B136, B137, B140, B141, B148	Life Safety
B126, B131, B144, B147	Collapse Prevention

6.2 Discussion of Results

Table 6.3: Beams damage percentage of an existing mass housing by TEC-2018

STOREY	-X Direction				+X Direction				-Y Direction				+Y Direction			
NO	IO	LS	CP	CC	IO	LS	CP	CC	IO	LS	CP	CC	IO	LS	CP	CC
5	100.0	0.0	0.0	0.0	100.0	0.0	0.0	0.0	100.0	0.0	0.0	0.0	100.0	0.0	0.0	0.0
4	100.0	0.0	0.0	0.0	100.0	0.0	0.0	0.0	100.0	0.0	0.0	0.0	100.0	0.0	0.0	0.0
3	100.0	0.0	0.0	0.0	100.0	0.0	0.0	0.0	100.0	0.0	0.0	0.0	100.0	0.0	0.0	0.0
2	100.0	0.0	0.0	0.0	100.0	0.0	0.0	0.0	56.0	44.0	0.0	0.0	56.0	36.0	8.0	0.0
1	70.8	29.2	0.0	0.0	70.8	29.2	0.0	0.0	56.0	44.0	0.0	0.0	48.0	36.0	16.0	0.0
Max.	100.0									44.0					16.0	

IO: Immediate occupancy, **LS:** Life safety, **CP:** Collapse prevention, **CC:** Collapse case

Table 6.4: Column shear force distribution of an existing mass housing by TEC-2018

STOREY	-X Direction				+X Direction				-Y Direction				+Y Direction			
NO	**IO**	**LS**	**CP**	**CC**	**IO**	**LS**	**CP**	**CC**	**IO**	**LS**	**CP**	**CC**	**IO**	**LS**	**CP**	**CC**
5	100.0	0.0	0.0	0.0	100.0	0.0	0.0	0.0	100.0	0.0	0.0	0.0	100.0	0.0	0.0	0.0
4	100.0	0.0	0.0	0.0	100.0	0.0	0.0	0.0	100.0	0.0	0.0	0.0	100.0	0.0	0.0	0.0
3	96.6	3.4	0.0	0.0	95.8	4.2	0.0	0.0	89.0	0.0	11.0	0.0	93.1	0.0	6.9	0.0
2	91.5	0.0	8.5	0.0	91.4	0.0	8.6	0.0	69.1	0.0	30.9	0.0	86.4	0.0	13.6	0.0
1	0.0	0.0	52.1	47.9	0.0	0.0	52.1	47.9	16.5	0.0	83.5	0.0	16.4	0.0	83.6	0.0
Max.	100			47.9		4.2									83.6	

IO: Immediate occupancy, **LS:** Life safety, **CP:** Collapse prevention, **CC:** Collapse case

Table 6.5: Shear force distribution of columns in exceeding minimum damage information in upper and lower sections

STOREY	-X Direction		+X Direction		-Y Direction		+Y Direction	
NO	**IO**	**LS+CP+CC**	**IO**	**LS+CP+CC**	**IO**	**LS+CP+CC**	**IO**	**LS+CP+CC**
5	100.0	0.0	100.0	0.0	100.0	0.0	100.0	0.0
4	100.0	0.0	100.0	0.0	100.0	0.0	100.0	0.0
3	100.0	0.0	100.0	0.0	100.0	0.0	100.0	0.0
2	100.0	0.0	100.0	0.0	100.0	0.0	100.0	0.0
1	86.2	13.8	86.2	13.8	100.0	0.0	100.0	0.0
Max.	100.0			13.8				

IO: Immediate occupancy, **LS:** Life safety, **CP:** Collapse prevention, **CC:** Collapse case

Controlled damage performance level was determined in accordance with TEC-2018 as target performance level for the building in Section 5.1.5. Therefore, all of the following criterias must be met.

- Maximum **35%** of beams can be in collapse prevention level in each earthquake direction (-X, +X, -Y, +Y) on any storey.
- The shear force ratio carried by columns in collapse prevention level to all shear force in each storey must be fewer than **20%**. On the other hand, the ratio of columns total shear force in collapse prevention level to total shear force of all columns in that storey can be maximum **40%** on top storey.
- Although the rest of the bearing elements must be in immediate occupancy and life safety level, the ratio of both lower and upper sections of the column in collapse prevention level to the total shear of that storey can be maximum **40%** on any storey.

i. All beams are in immediate occupancy and life safety level in respect of -X, +X, -Y and +Y directions respectively as shown in Table 6.3. Hence, 0% of beams are in collapse prevention and collapse case level. As a result, results provide first step.
ii. The column shear force distribution of first storey is being in collapse prevention level with the ratio of 52.1%, 52.1%, 83.5% and 83.6% is respect of –X, +X, -Y and + Y directions respectively according to Table 6.4. Moreover, 47.9% of them in collapse case (CC) level in respect of both –X and +X direction. Second rule of controlled damage level allows fewer than 20% of columns to be in collapse prevention level but 20% exceeded in all directions. Therefore, second step of controlled damage level can not be achieved.
iii. The addition of life safety, collapse prevention and collapse case level on first storey is 13.8% in respect of –X and +X direction and rest of them is 0% for lower and upper sections of columns as shown in Table 6.5. Controlled damage level allows maximum 40% of them to be in collapse prevention level hovewer third step of the controlled damage is provided.

The performance of the building does not provide controlled damage performance due to second reason given above. Long side of the most columns (18 out of 22) positioned in Y direction so collapse case percentage is zero in Y and -Y direction. Furthermore, there is no shear wall both in X and Y direction that increases collapse prevention and collapse case percentages. Lastly, low compressive strength concrete usage contributes to collapse mechanism by decreasing concrete shear force.

Table 6.6: Columns with insufficient cross section

Column Names	Storey Number
C110, C111, C113, C116, C117, C120, C121, C123, C126, C128, C130, C132, C135, C137, C140	1
C201, C208, C210, C211, C213, C216, C220, C221, C223, C226, C228, C230, C232, C235, C237, C240, C241	2
C302, C304, C305, C307, C309, C312, C314, C315, C318, C319, C322, C324, C325, C327, C329, C331, C333, C336, C338, C339, C342	3
C409, C414, C415, C418, C424, C425, C427, C429, C436, C442	4

Column beam connection check calculations are available in appendix 4. One of the column selected and all calculations are presented step by step in Section 6.3.

6.3 Column Beam Connection Check

$$V_e = 1.25 f_{yk}(As1 + As2) - v_{kol} \quad (6.3)$$

$$V_{max} = 1.0 x b_j x h_c x \sqrt{f_{ck}} \quad (6.4)$$

V_e: Shear force based on the calculation of transverse reinforcement in columns, beams, joints and shear walls

f_{yk}: Characteristic yield strength of longitudinal reinforcement

A_s: Total area of reinforcement

V_{kol}: The smallest column shear force

b_j: Column width is considered if the beam stuck in the junction area is of the same width as the column or protrudes from both sides of the column. Otherwise, twice the distance from the vertical mid-axis of the beam to the column edges in the considered direction of the earthquake. (Does not exceed the sum of the beam width and the height of the joint)

h_c: Cross-sectional dimension of the column in the considered earthquake direction

f_{ck}: Characteristic compressive strength of concrete

Column beam check for C312

X-direction;

$\mathrm{Ve} = 1.25\mathrm{x}220\mathrm{x}(800) - 13000 = 207000$ N

C11 concrete used in study thus, f_{ck}=11 MPa

$\sqrt{11} = 3.32$ MPa

$\mathrm{v_{max}} = 1.0\mathrm{x}3.32\mathrm{x}200\mathrm{x}250 = 166000$ N

V_e>V_{max}; insufficient cross section

Y-direction;

$Ve = 1.25\mathrm{x}220\mathrm{x}(960) - 19000 = 245000$ N

$Vmax = 1.0\mathrm{x}3.32\mathrm{x}200\mathrm{x}300 = 199200$ N

V_e>V_{max}; insufficient cross section

CHAPTER 7
CONCLUSIONS

7.1 Conclusions

This study assessed the seismic performance of an existing apartment type mass housing designed by “Seismic Detailing Provisions” considering TEC-2018 regulations. The analysis methods were non-linear static analysis and multi mode analysis. In chapter 6, results are available and STA4-CAD outputs with calculations are given in appendix 4. Mass housing stock data gathering including total house number, building type, house stock in each district, building distribution by earthquake code and population distribution are all collected and formed for North Cyprus through this study.

The target performance level is controlled damage but analysis result is in collapse case. Collapse case is achieved when collapse prevention performance level of building does not provide the following steps simultaneously;

- Maximum **20%** of beams can be in collapse case level in each earthquake direction (-X, +X, -Y, +Y) on any storey.
- The rest of the structural elements (columns & beams) must be in collapse prevention, life safety and immediate occupancy level.
- The ratio of both lower and upper sections of the column in life safety level to the total shear of that storey can not exceed **30%** on any storey.

As it can be understood from the second reason, none of the columns should be in collapse case. 47.9% of the column shear force is in collapse case on first storey, both in -X and X directions. For this reason, the building is in collapse case.

It is proved that studied building is dangerous for human life and it needs retrofitting.

7.2 Future Works

Some of the limitations of the adopted model are related to the lack of consideration of some effects that are observed in selected building. Although, there are rainwater pipes inside of the some columns, STA4-CAD model does not contain rainwater pipes inside the column sections. Same building might be analysed with new column sections including pipe holes in the future.

There is no seismic hazard map for Cyprus Island yet. For this reason, proposed formulas were used for spectral response acceleration coefficents determination. If any seismic hazard maps are prepared for the island in the future, same building can be analysed again.

This study prepared considering old concrete core samples hovewer concrete coring can be done again to improve the reliability of existing core results.

This study only focused on an existing reinforced concrete apartment building type mass housing. Other mass housing types can be studied in the future and this study offers an insight into future results. On the other hand, future and existing results can be compared and discussed for all mass housing types.

Finally, future studies may also focus on modelling a viable retrofitting strategy for the selected building.

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APPENDICES

APPENDIX 1

A.1.1 Guideline of New Seismic Design Code of Turkey

The first code for buildings was published in 1940 in Turkey, after the great Erzincan earthquake that occured in 1939. Turkish Earthquake Code had its later version approved and published in March 2018 and in force after January 2019. The new seismic design code is a comprehensive version of TEC-2007. The purpose of this regulation is to determine the rules and minimum conditions for the design, construction of all or part of all buildings, building types under the effect of earthquake, performance and strengthening of existing buildings under earthquake effect. Almost, 120 experts work for the whole document and permanent building code committee with 15 members coordinated all activities. The scope of new seismic code includes 17 chapters. Most of the chapters are rearranged where there are new chapters on seismically isolated, high-rise, cold-formed steel and wooden buildings. Chapter 15 of TEC-2018 includes special rules for evaluating and reinforcement design of existing building systems under earthquake effect. The scope of appendix 1 guides the new earthquake code. Many of the old concepts are not existing in this code anymore. For instance, the earthquake zones have been eliminated and every point is defined as earthquake zone. Hence, spectral acceleration coefficient (S_S and S_1) values are in use from now on. 16 Turkey earthquake hazard maps are available to find ground acceleration that belongs to Disaster and Emergency Management Authority (AFAD). Even the ground acceleration values of the two lands in the same neighborhood will be different with this system.

A.1.2 Seismic Ground Movement Levels (DD)

- **DD-1:** The possibility to be exceed in 50 years is 2%, repetition period is 2475 years, very rare but the greatest possible earthquake. It is used for high and special buildings. (Used for high and very special buildings)
- **DD-2:** The possibility to be exceed in 50 years is 10%, repetition period is 475 years and rare earthquake. It is called as standard design of seimic ground movement. (All buildings except tall buildings, insulated buildings and old buildings)

- **DD-3:** The possibility to be exceed in 50 years is 50%, repetition period is 72 years and ocurrence probability is very often earthquake.
- **DD-4:** The possibility to be exceed in 50 years is 68%, repetition period is 43 years and rare earthquake.

A.1.3 Design Spectral Acceleration Coefficients

$$S_{DS} = S_S F_S \tag{A.1.1}$$

$$S_{D1} = S_1 F_1 \tag{A.1.2}$$

S_S: Short period spectral acceleration coefficient

S_1: Spectral acceleration coefficient for T=1

F_S: Local site impact coefficient for the short period zone

F_1: Local site impact coefficient for one second

A.1.4 Local Site Classes

Determination of soil properties are required to design foundations of new buildings and evaluate the existing ones. Local site classes are classified considering site types that is available in Table A.1.1. The new earthquake code includes special cases that needs soil investigations before design.

Table A.1.1: Local site classes with ground parameters (TEC-2018)

Local Site Class	Type of Site	Upper 30 m on average		
		$(V_s)_{30}$ [m/s]	$(N_{60})_{30}$ [impact/30 cm]	$(C_u)_{30}$ [kpa]
ZA	Rugged, hard rocks	>1500	-	-
ZB	Slightly decomposed, moderately rugged rocks	760-1500	-	-
ZC	Very tight sand, gravel and hard clay layers or decomposed, very cracked weak rocks	360-760	>50	>250

ZD	Moderately tight-tight sand, gravel or very stiff clay layers	180-360	15-50	70-250
ZE	Loose sand, gravel or soft-stiff clay layer or PI>20 and w>40% Providing conditions for PI>20 and w>40% totally more than 3 m thickness soft clay layer (c_u<25 kpa) containing profiles	<180	<15	<70
ZF	Soil requiring specific research and evaluation 1. Soils with a risk of collapse and potential landslide under the effect of earthquake. (liquefaction soils, high sensitive clays, weakly cemented grounds) 2. Total thickness of more than 3 m peat and/or high organic content clays. 3. Clays with high plasticity (PI>50) with a total thickness of higher than 8 meters. 4. Very thick (>35 m) soft or moderate stiff clays.			

Table A.1.2: Local site impact coefficients for short period zone (TEC-2018)

Local site class	**Local site impact coefficient for the short period zone F_S**					
	S_S≤0.25	**S_S=0.50**	**S_S=0.75**	**S_S=1.00**	**S_S=1.25**	**S_S≥1.50**
ZA	0.8	0.8	0.8	0.8	0.8	0.8
ZB	0.9	0.9	0.9	0.9	0.9	0.9
ZC	1.3	1.3	1.2	1.2	1.2	1.2
ZD	1.6	1.4	1.2	1.1	1.0	1.0
ZE	2.4	1.7	1.3	1.1	0.9	0.8
ZF	Site-specific behavior analysis will be done					

Table A.1.3: Local site impact coefficients for one second (TEC-2018)

Local site class	Local site impact coefficient for one second F_1					
	$S_S \leq 0.25$	$S_S = 0.50$	$S_S = 0.75$	$S_S = 1.00$	$S_S = 1.25$	$S_S \geq 1.50$
ZA	0.8	0.8	0.8	0.8	0.8	0.8
ZB	0.8	0.8	0.8	0.8	0.8	0.8
ZC	1.5	1.5	1.5	1.5	1.5	1.4
ZD	2.4	2.2	2.0	1.9	1.8	1.7
ZE	4.2	3.3	2.8	2.4	2.2	2.0
ZF	Site-specific behavior analysis will be done					

A.1.5 Horizontal Elastic Design Spectrum

$$S_{ae}(T) = (0.4 + 0.6\frac{T}{T_A})S_{DS} \qquad 0 \leq T \leq T_A \qquad (A.1.3)$$

$$S_{ae}(T) = S_{DS} \qquad T_A \leq T \leq T_B \qquad (A.1.4)$$

$$S_{ae}(T) = \frac{S_{D1}}{T} \qquad T_B \leq T \leq T_L \qquad (A.1.5)$$

$$S_{ae}(T) = \frac{S_{D1}}{T^2} \qquad T_L \leq T \qquad (A.1.6)$$

$$S_{ae}(T) = 0.2\frac{S_{D1}}{S_{DS}}; T_B = \frac{S_{D1}}{S_{DS}} \qquad (A1.7)$$

- T_A and T_B values depend on local site class and land coordinates.

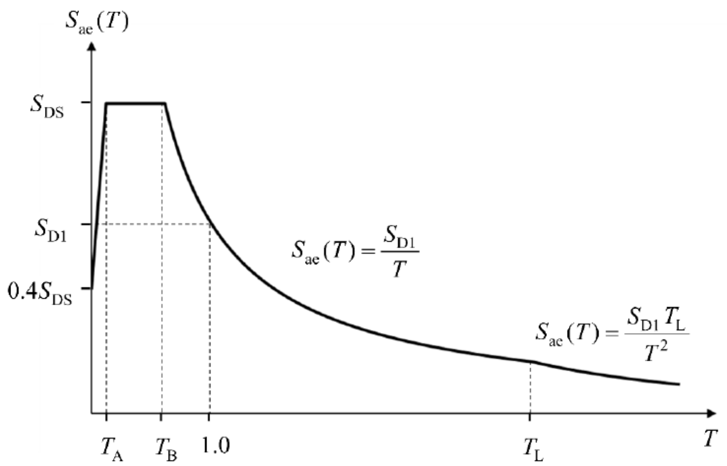

Figure A.1.1: Horizontal design spectrum by T_A and T_B

A.1.6 Vertical Elastic Design Spectrum

$$S_{aeD}(T) = (0.32 + 0.48\frac{T}{T_{AD}})S_{DS} \qquad 0 \le T \le T_A \qquad (A.1.8)$$

$$S_{aeD}(T) = 0.8S_{DS} \qquad T_{AD} \le T \le T_{BD} \qquad (A.1.9)$$

$$S_{aeD}(T) = 0.8S_{DS}\frac{T_{BD}}{T} \qquad T_{BD} \le T \le T_{LD} \qquad (A.1.10)$$

$$T_{AD} = \frac{T_A}{3} \qquad (A.1.11)$$

$$T_{BD} = \frac{T_B}{3} \qquad (A.1.12)$$

$$T_{LD} = \frac{T_L}{2} \qquad (A.1.13)$$

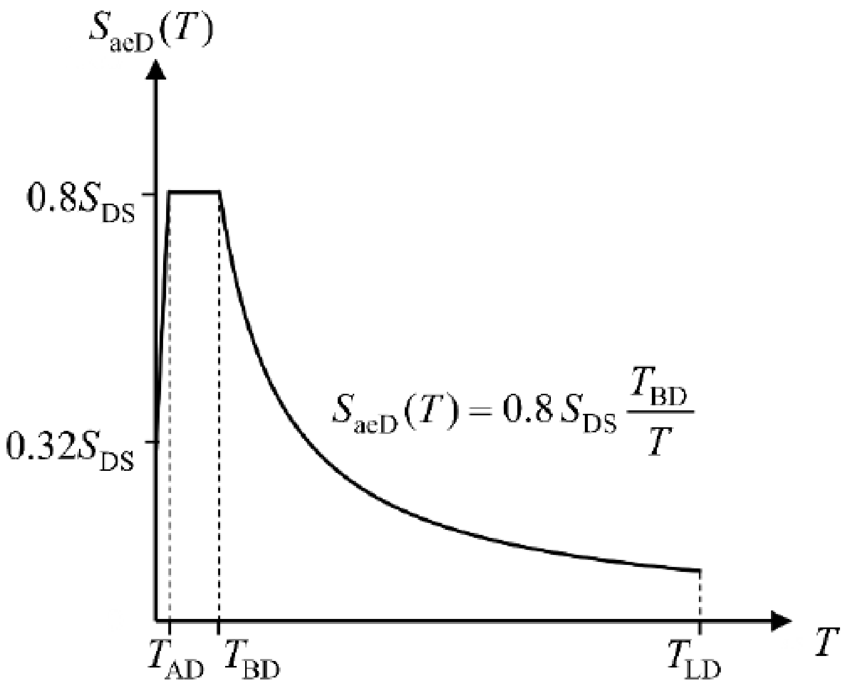

Figure A.1.2: Vertical spectrum by T_{AD} and T_{BD}

A.1.7 Purpose of Occupancy and Importance Factors of Buildings

In this section, building importance factors and building usage classes are presented for the purpose of design.

Table A.1.4: Building importance factors by building types (TEC-2018)

Building Usage Class	Purpose of Occupancy	Building Importance Factor
BKS=1	**Buildings to be used after the earthquake, intensively and long-term occupied buildings, building preserving valuable and dangerous goods.** a) Buildings required to be used immediately after the earthquake. (Hospitals, dispanseries, health wards, fire fighting buildings, PTT and other telecommunication facilities, transportation stations and terminals, power generation and distribution facilities, governorate, county and municipality administration buildings, first aid and emergency planning stations) b) Schools, other educational buildings and facilities, dormitories and hostels, military barracks, prison, etc. c) Museums d) Buildings containing or storing toxic, explosive and flammable materials, etc.	1.5
BKS=2	**Intensively but short-term occupied buildings** Shopping centers, sport facilities, cinema, theatre, concert halls, places of workship, etc.	1.2
BKS=3	**Other buildings** Buildings other than BKS=1 and BKS=2. (Residential buildings, places of workship, hotels, buildings-like industrial structures, etc.)	1.0

A.1.8 Seismic Design Classes (DTS)

Seismic design class depends on building usage class and short period design spectral acceleration coefficient in the level of seismic ground movement. Earthquake design class is a sample of innovation in TEC 2018.

Table A.1.5: Earthquake design classes (TEC-2018)

Short Period Design Spectral Acceleration Coefficient (S_{DS}) in the Level of Seismic Ground Movement DD-2	**Building Usage Class**	
	BKS=1	**BKS=2, 3**
$S_{DS}<0.33$	DTS=4a	DTS=4
$0.33\leq S_{DS}\leq 0.50$	DTS=3a	DTS=3
$0.50\leq S_{DS}\leq 0.75$	DTS=2a	DTS=2
$0.75\leq S_{DS}$	DTS=1a	DTS=1

A.1.9 Building Height Classes (BYS)

Buildings are separated into categories considering their height under the effect of seismic design.

Table A.1.6: Building height ranges (TEC-2018)

Building Height Class	**Building height ranges defined according to building height classes and seismic design classes [m]**		
	DTS=1, 1a, 2, 2a	**DTS=3,3a**	**DTS=4,4a**
BYS=1	$H_N>70$	$H_N>91$	$H_N>105$
BYS=2	$56<H_N\leq 70$	$70<H_N\leq 91$	$91<H_N\leq 105$
BYS=3	$42\leq H_N<56$	$56<H_N\leq 70$	$56<H_N\leq 91$
BYS=4	$28<H_N\leq 42$	$42<H_N\leq 56$	
BYS=5	$17.5<H_N\leq 28$	$28<H_N\leq 42$	

BYS=6	$10.5<H_N\leq17.5$	$17.5<H_N\leq28$
BYS=7	$7<H_N\leq10.5$	$10.5<H_N\leq17.5$
BYS=8	$H_N\leq7$	$H_N\leq10.5$

A.1.10 Building Performance Levels

- **Uninterrupted Usage (KK) Performance Level**

This performance level corresponds to a situation where there is no structural damage occurrence in the structural system of building components or the damage level remains negligible.

- **Limited Damage (SH) Performance Level**

This performance level corresponds to the level of damage in which a limited level of damage occurs in the building structural system elements. In other words, it corresponds to damage level where non-linear behavior is limited.

- **Controlled Damage (KH) Performance Level**

This performance level corresponds to the level of damage that is not very heavy and can often be repaired in the building structural system elements to ensure life safety.

- **Collapse Prevention (GÖ) Performance Level**

This performance level corresponds to the situation before collapse in which severe heavy damage occurs in building structural system elements. Partial or complete collapse of the building was prevented.

Table A.1.7: Performance level of new cast in place or precast reinforced concrete and steel buildings, except high buildings-BYS≥2 (TEC-2018)

Earthquake Ground Motion Level	DTS=1, 2, 3, 3a, 4, 4a		DTS=1a, 2a	
	Normal Performance Target	Evaluation/design approach	Advance performance target	Evaluation/design approach
DD-3	-	-	SH	ŞGDT
DD-2	KH	DGT	KH	DGT
DD-1	-	-	KH	ŞGDT

Table A.1.8: Performance level of new or existing high buildings-BYS=1 (TEC-2018)

Earthquake Ground Motion Level	DTS=1, 2, 3, 3a, 4, 4a		DTS=1a, 2a	
	Normal Performance Target	Evaluation/design approach	Advance performance target	Evaluation/design approach
DD-4	KK	DGT	-	-
DD-3	-	-	SH	ŞGDT
DD-2	KH	DGT	KH	DGT
DD-1	GÖ	ŞGDT	KH	ŞGDT

Table A.1.9: Performance level of existing cast in place reinforced concrete, precast and steel buildings except high buildings-BYS≥ 2 (TEC-2018)

Earthquake Ground Motion Level	DTS=1, 2, 3, 3a, 4, 4a		DTS=1a, 2a	
	Normal Performance Target	Evaluation/design approach	Advance performance target	Evaluation/design approach
DD-3	-	-	SH	ŞGDT
DD-2	KH	ŞGDT	-	-
DD-1	-	-	KH	ŞGDT

A.1.11 Ductility Levels of Structural Systems

Table A.1.10: Structural system behavior coefficient, strength excess coefficient and permitted building height for building structural systems (TEC-2018)

Building Structural System	Structural Behavior Factor (R)	Overstrength Factor (D)	Building Height Class (BYS)
A.Cast in place reinforced concrete building structural systems			
A1. High ductile structural systems			
A11. Buildings in that earthquake loads are fully resisted by moment transmitting high ductile frames	8	3	BYS≥3

A12. Buildings in that earthquake loads are fully resisted by high ductile coupled structural walls	7	2.5	BYS≥2
A13. Buildings that earthquake loads are fully resisted by high ductile solid structural walls	6	2.5	BYS≥2
A14. Buildings in that earthquake loads are resisted together by moment transmitting high ductile reinforced concrete frames and coupled structural walls	8	2.5	BYS≥2
A15. Buildings in that earthquake loads are resisted together by moment transmitting high ductile reinforced concrete frames and solid structural walls	7	2.5	BYS≥2
A16. Earthquake loads are resisted by single storey	3	2	-

buildings when their roof level connections are hinge and high ductile reinforced concrete columns are not exceeding 12 m			
A2. Mixed Ductile Structural Systems			
A21. Buildings in that earthquake loads are resisted together by moment transmitting limited ductile reinforced concrete frames and high ductile coupled structural walls	6	2.5	BYS≥4
A22. Buildings in that earthquake loads are resisted together by moment transmitting limited ductile reinforced concrete frames and high ductile solid structural walls	5	2.5	BYS≥4
A23. Buildings in that earthquake loads are resisted together by moment	6	2.5	BYS≥6

transmitting limited ductile ribbed slab or one way waffle slab with high ductile coupled structural walls			
A24. Buildings in that earthquake loads are resisted together by moment transmitting limited ductile ribbed slab or one way waffle slab with high ductile solid structural walls	5	2.5	BYS≥6
A3. Limited Ductile Structural Systems			
A31. Buildings in that earthquake loads are fully resisted by moment transmitting limited ductile reinforced concrete frames	4	2.5	BYS≥7
A32. Buildings in that earthquake loads are fully resisted by limited ductile coupled structural walls	4	2	BYS≥6
A33. Buildings in that earthquake	4	2	BYS≥6

loads are resisted together by moment transmitting limited ductile reinforced concrete frames and limited ductile coupled structural walls

A.1.12 Live Load Participation Factors

Table A.1.11: Live load participation factors by building types (TEC-2018)

Usage purpose of building	n
Warehouse, depot, etc.	0.8
Shop, restaurant, car park, concert hall, theatre, cinema, sport facility, dormitory, school, etc.	0.6
Hospital, hotel, office, residence, etc.	0.3

A.1.13 Special Rules for the Evaluation and Retrofitting of Building Systems Under Earthquake Effect

Data regarding the sizes and details of the elements to be used in the capacity determination of the supporting system elements of the existing buildings and data in accordance with the material and geometry characteristics of the supporting systems will be obtained from the reports and projects of buildings, from measurements and observations to be carried out on the building, and from trial experiments performed on the material samples taken from the building. Additionally, soil properties can be determined as well. After examination of buildings, knowledge level coefficients can be defined which are limited and comprehensive information level coefficients.

A.1.13.1 Limited knowledge level in reinforced concrete buildings

A.1.13.1.1 Building geometry

The measured drawings of the supporting system shall be prepared with field work. If the architectural projects are present, they may help to speed up the preparation process of measured drawings. Information obtained must contain the locations, materials, axis openings, dimensions and heights of all the reinforced concrete elements and non-bearing walls, and must be enough to set up the calculation model for the building. Foundation shall be determined by digging enough number of investigation holes in or outside the building. Short columns or similar irregularities in the building shall be added on the floor plan and sections. Relation of the building with neighboring buildings (separated, adjoining, jointing present / absent) shall be determined.

A.1.13.1.2 Details of elements

It is assumed that the amount of reinforcement and details in the reinforced concrete elements meet the minimum requirements for reinforcement with the date of building construction. With the purpose of confirming this assumption, or to determine what extent its true, at least one bulkhead or column must chosen in each floor then reinforcements shall be determined by scraping off the concrete covers of 5% of bulkheads and columns. Scraping off concrete cover must be applied at least one beam in each floor for the purpose of reinforcement determination. Such scraping must be performed on the one-third of the lengths of the columns and beams in the middle of the openings. Afterwards, scraped surfaces shall be recovered with high-strength repair mortar. Moreover, placement and number of longitudinal and transverse reinforcements of 20% of columns and shear walls that have not been scraped shall be determined using reinforcement-determining devices. The coefficient of reinforcement realization expressing the ratio of the actual reinforcement to the reinforcement found in columns and bulkheads. This coefficient must'nt be greater than 1. This coefficient shall be applied to all the other columns and bulkheads that reinforcement has not been determined so possible amount of reinforcement shall be determined. The necessary reinforcement only under vertical design loads will be used for beams.

A.1.13.1.3 Characteristics of materials

At least three concrete samples shall be collected from columns or bulkheads in each floor according to the conditions stated in TS EN 12504-1, and tests shall be performed. Testing of cores with a nominal diameter and length equal to 100 mm and strength values of existing concrete without any coefficient can be used in the determination. The conversion of the results should be based on the appropriate conversion coefficients obtained in the tests from cores with different length/diameter ratios. In case the total number of samples is three the lowest compressive strength number of samples obtained from the samples without statistical evaluation compressive strength shall be taken as the existing concrete strength. If the number of samples is more than three (average minus standard deviation) value with between (0.85 times mean) the larger one will be taken as the existing concrete strength. The difference between the smallest and the average of the remaining results with the smallest value is a statistically slinging or not shall be checked by the test results which belong to a group of concrete samples. For this purpose, the lowest single if the value is less than 75% of the average of the remaining is not considered. On surfaces scraped as described in reinforcement class A.1.13.1.2 the characteristic yield stress of this class of steel to be determined by visual inspection will be considered steel strength. Corrosion observed elements will be marked in the plan and this will be taken into account in the element capacity calculations.

A.1.13.2 Comprehensive knowledge level in reinforced concrete buildings

A.1.13.2.1 Building geometry

Compliance with the projects of the actual geometry is checked with the measurements performed in the building. If the projects show important conflicts with the measurements, the projects are ignored. If the project is not present, the building's structural system will be obtained. The information obtained on all floors of reinforced concrete elements and partition walls should contain location, openings, heights, dimensions and material. Short columns or similar irregularities in the building shall be entered on the floor plan and sections. Relation of the building with the neighboring buildings (separated, adjoining, jointing present/absent) shall be determined. Information regarding the geometry of the building must include the required details for a precise description of the mass of the building. Foundation system shall be determined by digging examination holes of sufficient number in-or outside the building.

A.1.13.2.2 Details of elements

If the detail projects of reinforcement of the building are present, procedures stated in A.1.13.1.2 for checking the compliance of reinforcement with the project shall be applied on the same number of reinforcement elements. Moreover, locations and numbers of longitudinal and transverse reinforcement elements of 20% of the bulkheads, columns and 10% of frame beams that have not been scraped shall be determined using devices for reinforcement-determining devices. In case there are any conflicts between the project and the application, then coefficient of actual reinforcement expressing the ratio of the amount of reinforcement the actually found in reinforced concrete columns and beams to the minimum reinforcement shall be separately determined for columns and beams. This coefficient used to determine the capacities of elements cannot exceed 1. This coefficient shall be applied to all the other elements that reinforcement has been determined, and the possible amount of reinforcement shall thus be determined. If the reinforced concrete projects or construction drawings are not available, at least two columns and bulkheads will be choose in each floor and 10% of concrete cover will be scraped to determine the reinforcement. Afterwards, scraped surfaces shall be recovered with high-strength repair mortar. Additionally, in non-scraped concrete cover of 30% of columns and bulkheads and 15% of the beams, the number and placement of longitudinal and transverse reinforcement elements shall be determined by reinforcement detection devices.

A.1.13.2.3 Characteristics of materials

Three concrete sample (borehole sample) shall be collected from columns or bulkheads on ground floor, not less than two samples in other floors, not less than nine for all building and one sample shall be taken from each 200 m^2 to performed tests according to the conditions stated in TS EN 12504-1. The samples which have equal length, nominal diameter and 100 mm diameter shall be tested to determine strength values in order to identify existing strength without applying any coefficient. In the procedure of converting the test results which have different length/diameter ratios obtained from experiments should be based on appropriate conversion coefficients. In the calculation of capacities of the elements obtained from the samples, the greater value between (average minus standard with the deviation value) and (0.85 times the average) will be taken as existing concrete strength. The results of the experiment of a group of concrete samples between the smallest value and the average of the remaining results should be evaluated to check whether the smallest value is a statistically deviating result or not. For this purpose, if the lowest single value is less than 75% of average the sample is not taken into consideration in the evaluation of samples of the group. Concrete strength distribution in building, shall be checked with experimental results adapted concrete hammer readings or similar undamaged inspection tools. Reinforcement class will be determined in scraped surface described as A.1.13.2.2 (TEC-2018), one steel sample will be taken and tested per steel class (S220, S420 etc), yield stress, rupture strength and deformation features will be determined and checked the suitability with the project. If it is suitable for the project, in the calculation of element capacity, yield stress of steel used in project shall be taken as current steel yield stress. If it not suitable at least three samples will be tested and the most unfavorable yield stress will be used as the current steel yield stress. In this review, the corrosion observed elements will be marked in the plan and this situation will be considered in the calculations of element capacities.

Table A.1.12: Building information level coefficients

Information Level	Coefficient of Information Level
Limited	0.75
Comprehensive	1.00

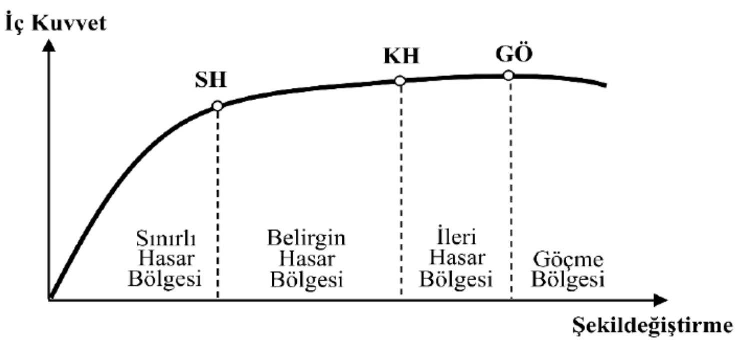

Figure A.1.3: Damage zone of sections (TEC-2018)

A.1.14 Most Striking Improvements in TEC-2018

- Minimum compressive strength of concrete is 25 MPa.
- Minimum steel yield strength is B420C or B500C.
- Minimum rectangular column dimension is 30 cm.
- Minimum circular column dimension is 35 cm.
- Ties bars are placed outer of stirrups in columns and shear walls.

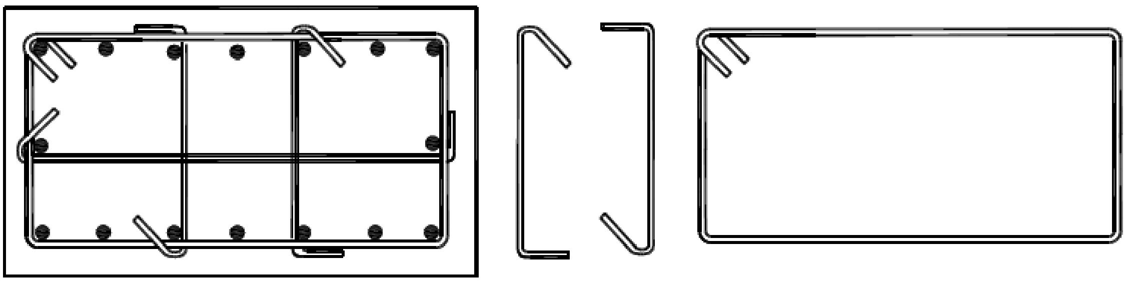

Figure A.1.4: Tie bar placement in respect of new regulations (TEC-2018)

- The ratio of long side to short side is 6 in shear walls.

APPENDIX 2

A.2.1 Core Report

K T M M O B

İNŞAAT MÜHENDİSLERİ ODASI
CHAMBER OF CIVIL ENGINEERS
Mehmet Göze (Asi)
YAPI MALZEMELERİ LABORATUVARI

İMO

Rapor Tarihi : 07/04/2013
Alınış Tarihi : 26/04/2013
Deney Tarihi : 30/04/2013
Rapor No : KR-13-04-06

Karot Raporu

NUMUNE BİLGİLERİ:

Temsil Ettiği Kütle	Kolon	Üretici Firma	-
Adet/Tip	3	Bet.Döküm Tarihi	-
Boyutları (mm)	64X64	Kıyaslanan Beton Sınıfı C...	C 8

NUMUNEYE AİT DENEY BİLGİLERİ:

Karot No	Karot Numunesinin Alındığı Yer ve Eleman Adı		Silindir Basınç Mukavemeti Ağırlık (Kg)	Kırım Kuvveti (kN)	Basınç Dayanımı N/mm² (Mpa)
1	Kolon 1	r=64 h=64	0.525	41.20	12.81
2	Kolon 2	r=64 h=64	0.535	36.29	11.28
3	Kolon 3	r=64 h=64	0.570	66.64	8.48
				Ortalama:	10.86

Sınır Değerleri:

Fort>0.85XFsk	Fek Betonun Eşdeğer Mukavemeti (N/mm²)	8.50 (N/mm²)
	Fsk Betonun Seri Mukavemeti (N/mm²)	11.05 (N/mm²)
Fmin>0.85xFek	Fmin Deney Sonucu Bulunan Minimum Mukavemet (N/mm²)	8.48 (N/mm²)
	Fort Deney Sonucu Bulunan Ortalama Mukavemet (N/mm²)	10.86 (N/mm²)

Not:
1-Deney ve hesaplamalar TS EN 13791 :2010 1992'ye göre yapılmıştır.
2-Numuneler (h=d) eşitliği esas alınarak deneye alınmıştır.
3-Numunelere % 70 kükürt ve %30 Grafit den oluşan karışım ile başlık yapılarak deneye alınmıştır.
4-Karot numuneleri laboratuarımız tarafından alınmış olup alınış tarihi,alınış yeri ve deney sonuçları dışındaki bilgiler kişinin beyanına göre yazılmıştır.

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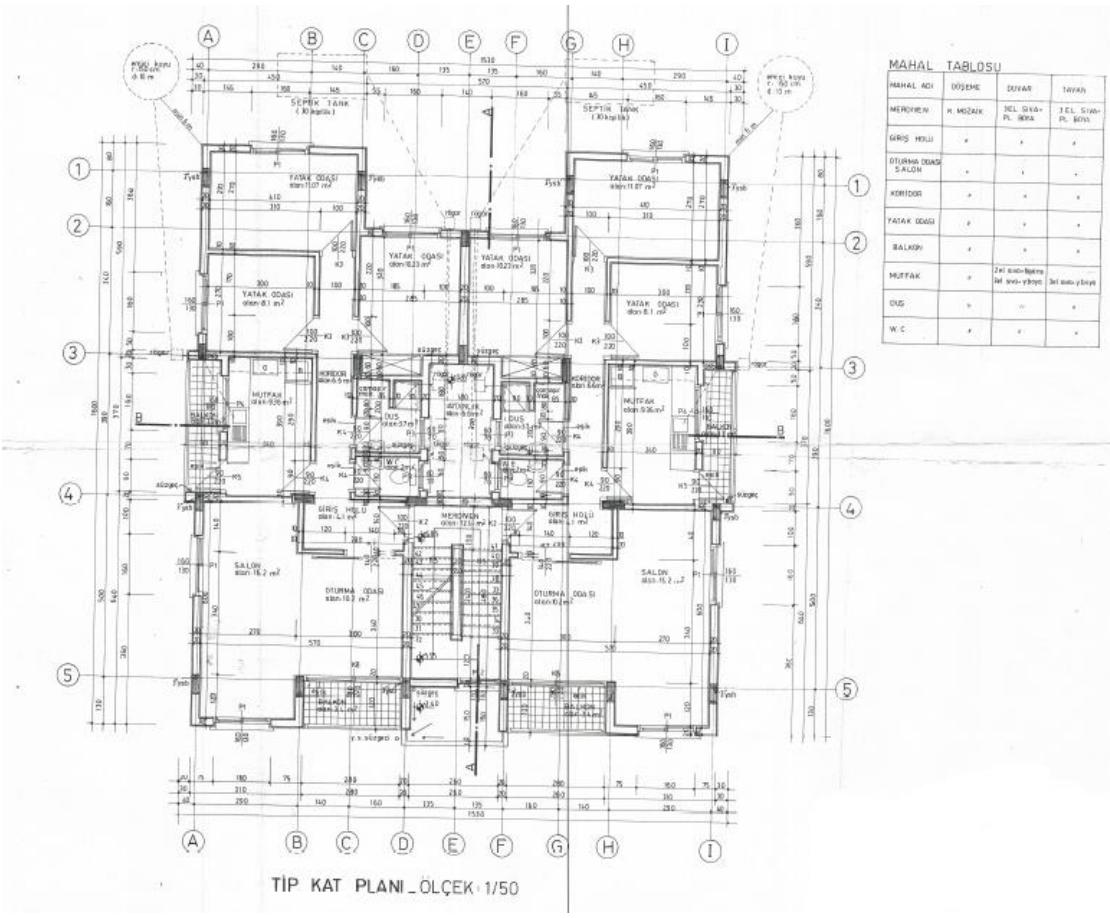
APPENDIX 3

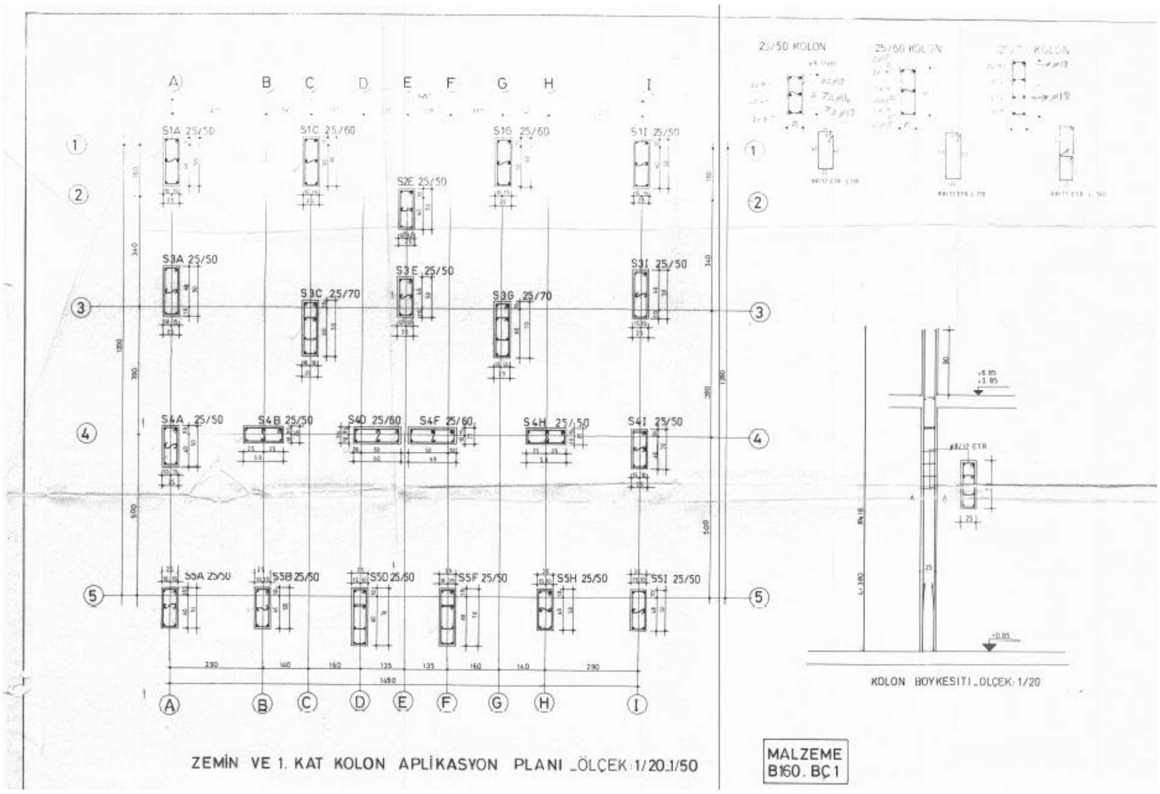
A.3.1 Typical Architectural Floor and Structural Plans

A.3.1.1 Typical architectural floor plan

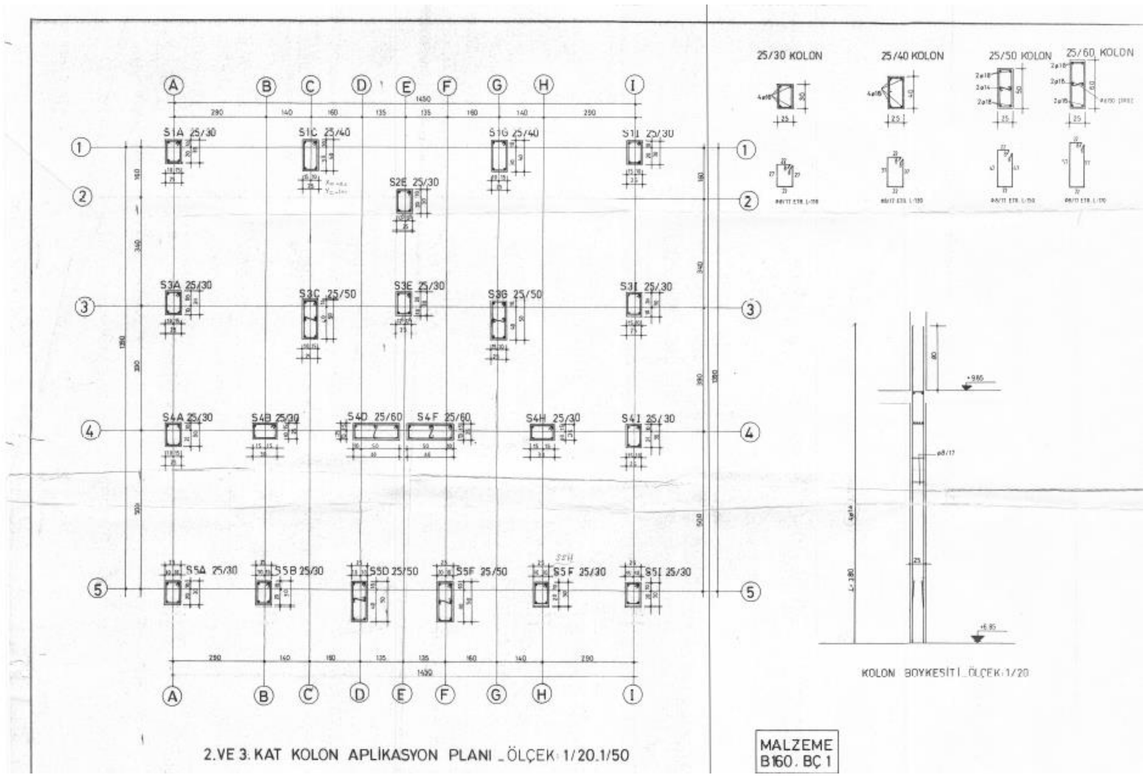


A.3.1.2 Building sections

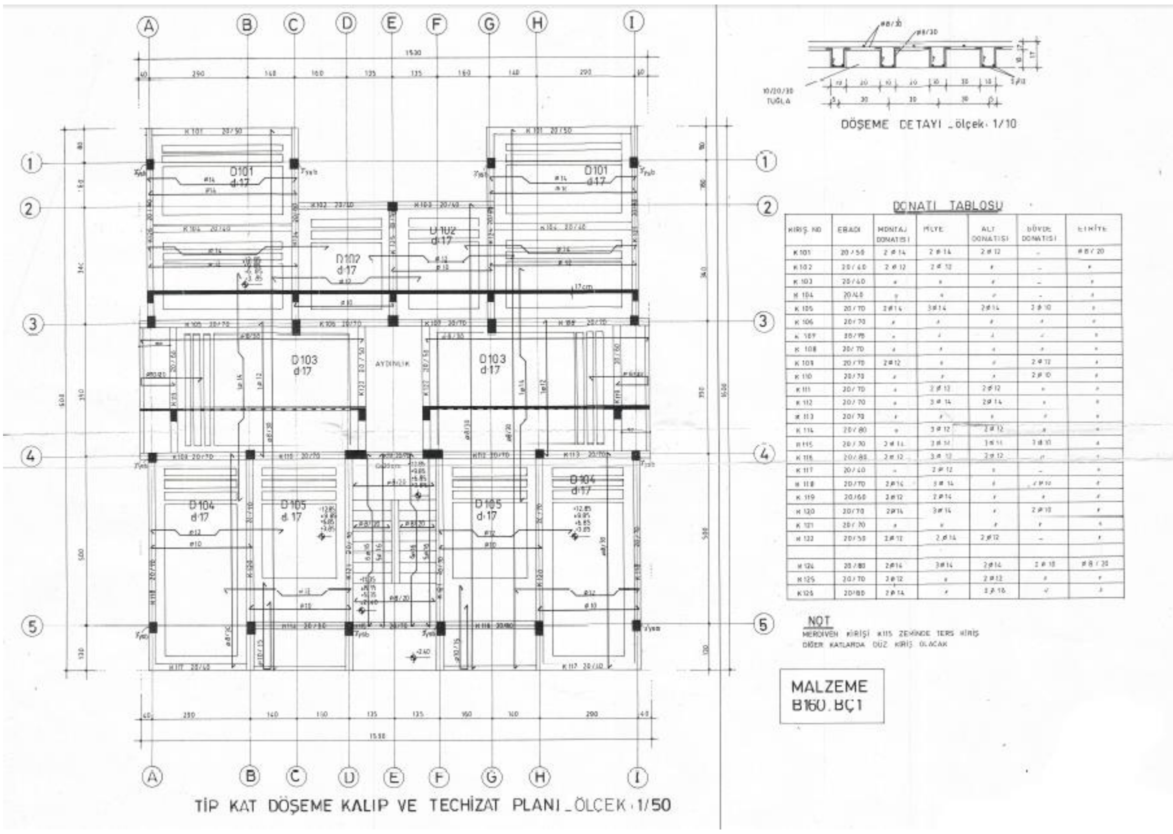
A.3.1.3 Ground and first floor column application plan



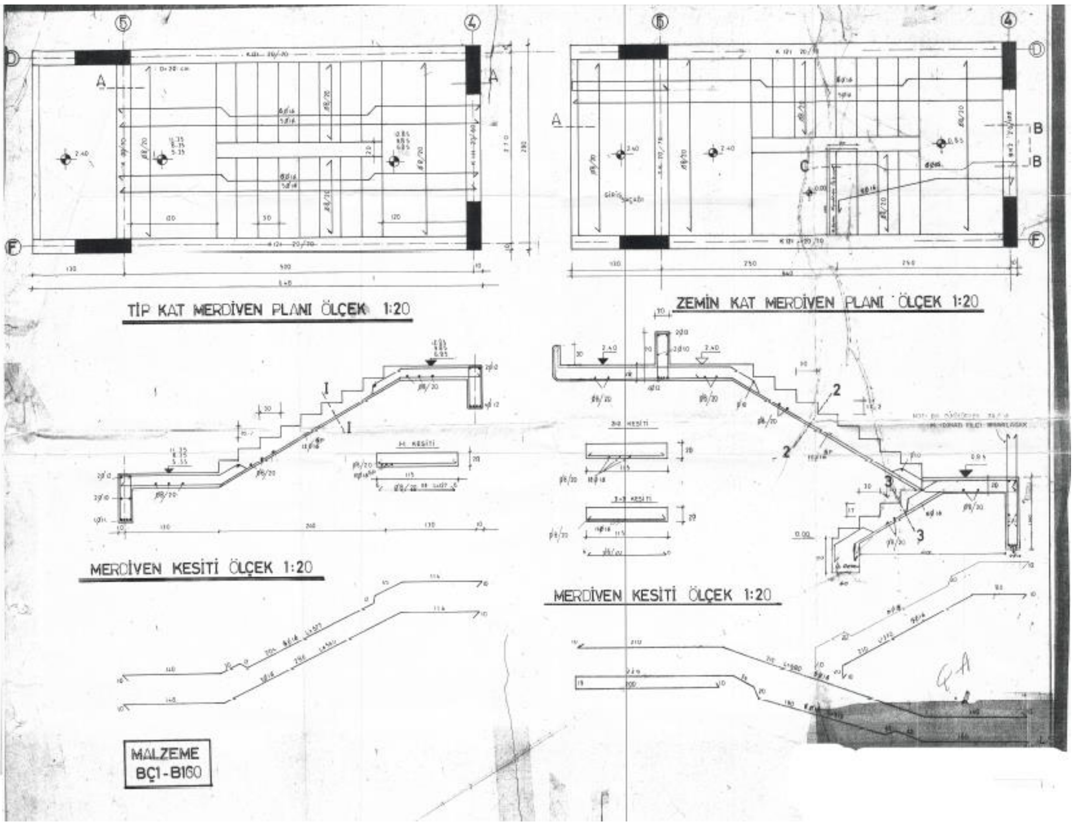
A.3.1.4 Second and third floor column application plan



A.3.1.5 Typical floor formwork plan



A.3.1.6 Typical staircase plan



APPENDIX 4

A.4.1 STA4-CAD Output