

T.C.
GEBZE TECHNICAL UNIVERSITY
GRADUATE SCHOOL

**VERIFICATION OF RAPID EARTHQUAKE RISK
IDENTIFICATION STUDIES WITH EARTHQUAKE
PERFORMANCE ANALYSIS**

KADİR AKMAN

**A THESIS OF MASTER OF SCIENCE
DEPARTMENT OF CIVIL ENGINEERING
EARTHQUAKE AND STRUCTURAL ENGINEERING
PROGRAM**

ADVISOR: ASSIST. PROF.DR. ÜLGEN MERT

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Co-ADVISOR: DR. ONUR AVCIOĞLU

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T.C.
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LİSANSÜSTÜ EĞİTİM ENSTİTÜSÜ

HIZLI DEPREM RİSK TESPİT ÇALIŞMALARININ
DEPREM PERFORMANS ANALİZİ İLE
DOĞRULANMASI

KADİR AKMAN

YÜKSEK LİSANS TEZİ
İNŞAAT MÜHENDİSLİĞİ ANABİLİM DALI
DEPREM VE YAPI MÜHENDİSLİĞİ PROGRAMI

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OCAK 2026



MASTER of SCIENCE JURY APPROVAL FORM

A thesis submitted by Kadir AKMAN defended on 22/01/2026 before the jury formed with the 20/01/2026 date and 2026/5 numbered decision of the GTU Graduate Administration Board, has been accepted as a MASTER of SCIENCE thesis in the Department of Civil Engineering, Earthquake and Structural Engineering Program.

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To my brother, Dr. Nurettin Akman ...

ABSTRACT

This thesis examines the significance and application of visual-based rapid seismic risk assessment methods for the swift evaluation of Turkey's building stock. An original Rapid Seismic Risk Assessment (RSRA) method was developed, drawing on several established rapid screening approaches (FEMA P-154, Canadian Seismic Screening, P-25, and PERA2019) as reference points. The RSRA method prioritizes in-situ evaluation of material quality and incorporates engineering-based parameters – such as concrete compressive strength, stirrup spacing, anchorage length, section forces (M, N, V), and fundamental period – into its scoring system. In this way, the method provides a more comprehensive framework that integrates both visual observations and analytical data. The RSRA system was applied to 513 branch buildings of a public bank in Turkey, and its risk predictions were validated against detailed earthquake performance analyses. Linear performance analyses were conducted on simplified analytical models for these buildings, and the resulting spectral demand reduction factors were compared to the RSRA scores. Regression analysis revealed a strong inverse relationship, with a second-degree polynomial fit achieving approximately $R^2 \approx 0.605$. Furthermore, an RSRA score threshold of 500 (indicating a building may require evacuation) was statistically tested using Receiver Operating Characteristic (ROC) analysis. The ROC evaluation yielded an Area Under the Curve (AUC) of 0.964, with an overall accuracy of 85.7%, sensitivity of 91.7% (correct identification of weak buildings), and specificity of 81.3% (correct identification of safe buildings) at the proposed threshold. These results confirm that the preliminary risk classification provided by the RSRA method is consistent with structural performance outcomes. In conclusion, the RSRA approach is demonstrated to be a reliable and efficient tool for large-scale seismic risk screening of existing buildings, offering rapid identification of potentially vulnerable structures.

Keywords: Rapid Seismic Risk Assessment, Earthquake Performance Analysis, Rapid Screening Methods, Structural Index Score, ROC Analysis

ÖZET

Bu tezde, Türkiye yapı stoğunun depreme karşı hızlı değerlendirilmesi için kullanılan görsel tabanlı hızlı risk değerlendirme yöntemlerinin önemi ve uygulama alanları incelenmiştir. Çalışma kapsamında, literatürde yaygın olarak kullanılan FEMA P-154, Kanadalı Hızlı Tarama, P-25 ve PERA2019 gibi yöntemler referans alınarak özgün bir Hızlı Risk Tespiti (HRT) yöntemi geliştirilmiştir. HRT yöntemi, saha gözlemlerine dayalı hızlı değerlendirme parametrelerinin yanı sıra beton basınç dayanımı, etriye aralığı, kenetlenme boyu, kesit tesirleri (M, N, V) ve doğal titreşim periyodu gibi mühendislik verilerini puanlama sistemine dâhil etmektedir. Bu sayede hem görsel hem de analitik verileri bütünleştiren daha kapsamlı bir risk değerlendirmesi hedeflenmiştir. Geliştirilen HRT sistemi, Türkiye’de bir kamu bankasının 513 şube binasında uygulanmış ve elde edilen sonuçlar detaylı deprem performans analizleri ile doğrulanmıştır. Binaların lineer performans analizlerinden elde edilen spektral talepleri azaltma katsayıları ile HRT puanları arasındaki ilişki regresyon modelleriyle incelenmiş; ikinci derece polinom model için yaklaşık $R^2 = 0.605$ gibi anlamlı bir korelasyon değeri elde edilmiştir. Ayrıca, $RSRA \geq 500$ puan eşik değeri “tahliye gerekebilir” kriteri olarak önerilmiş ve bu eşik, ROC (Receiver Operating Characteristic) analizi ile test edilmiştir. Analizler, HRT puanları kullanılarak zayıf binaları tespit etmede %85.7 doğruluk, %91.7 duyarlılık, %81.3 özgüllük sağlamış ve $AUC = 0.964$ değeri ile yöntemin ayırt ediciliğinin çok yüksek olduğunu göstermiştir. Elde edilen bulgular, HRT metodunun saha gözlemlerine dayalı öngörülerinin yapısal performans analizleriyle tutarlı olduğunu ve büyük ölçekli bina stoklarında hızlı, güvenilir ve maliyet-etkin bir deprem riski tarama aracı olarak kullanılabileceğini ortaya koymaktadır.

Anahtar Kelimeler: Hızlı Risk Tespiti, Yapı Performans Analizi, Hızlı Tarama Yöntemleri, RSRA, ROC Analizi

ACKNOWLEDGEMENTS

I would like to express my sincere appreciation to my advisor, Assist. Prof. Ülgen Mert, and my co-advisor, Dr. Onur Avcıoğlu, for their invaluable guidance, constructive remarks, and continuous academic support throughout the preparation of this thesis. Their expertise and perspectives have significantly contributed to the development and refinement of this work.

I also acknowledge the institutional and professional environment that supported my research journey. In this regard, I would like to recognize the senior management of Ziraat Bank for the opportunities they provided. I am especially grateful to our Head of Construction Management Department, Architect Serap Baydar Çalış, for her leadership, support, and the professional possibilities granted under her supervision, together with all my colleagues whose collaboration and commitment created an environment conducive to progress and perseverance.

Furthermore, I would like to extend my thanks to Civil Engineer İbrahim Can Beziklioğlu for his technical support and assistance during the editing and final preparation stages of this thesis for publication. I am also profoundly grateful to my family for their enduring encouragement and presence, which sustained my motivation during the most demanding stages of this study.

Finally, the completion of this thesis reflects not only academic effort but also the collective influence of everyone who believed in my capacity to conclude this work and accompanied me throughout this journey.

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LIST OF SYMBOLS AND ABBREVIATIONS

A_0	: Effective ground acceleration coefficient
A_c	: Gross concrete cross-sectional area
A_{sw}	: Cross-sectional area of shear reinforcement
AUC	: Area Under the ROC Curve
E_c	: Modulus of elasticity of concrete
E/C	: Demand-to-capacity ratio
f_{ck}	: Characteristic compressive strength of concrete
f_{ctd}	: Design tensile strength of concrete
f_{yd}	: Design yield strength of reinforcement
f_i	: Parameter multiplier
g	: Acceleration of gravity
h_i	: Height of storey i
I	: Moment of inertia / Importance factor
k_c	: Stiffness coefficient of column/shear wall
L	: Length / Characteristic span
M	: Bending moment
M_e	: Moment demand
M_r	: Moment capacity
N	: Axial force
P	: Performance score / Axial load
P_0	: Composite structural index term in P-25 method
P_{min}	: Minimum parameter value in scoring
r_1	: Shear demand-to-capacity ratio (V_e / V_r)
r_2	: Moment demand-to-capacity ratio (M_e / M_r)
R^2	: Coefficient of determination
$S_{ae}(T)$	: Elastic spectral acceleration at period T
S_{DS}	: Short-period design spectral acceleration coefficient
S_{D1}	: 1-second period design spectral acceleration coefficient
SIS	: Structural Index Score (RSRA)
T	: Fundamental period
V_e	: Shear demand
V_r	: Shear capacity
$V_{s,30}$	: Average shear-wave velocity in the top 30 m
δ	: Lateral displacement
δ_i/h_i	: Interstorey drift ratio
σ	: Normal stress
ε	: Strain
λ	: Load or scaling parameter
AD	: Advanced Damage (performance level)
ATC	: Applied Technology Council
C	: Collapse (performance level)
CD	: Controlled Damage (performance level)
CSI	: Computers and Structures, Inc.
ETABS	: Extended Three-Dimensional Analysis of Building Systems
FEMA	: Federal Emergency Management Agency
FEMA P-154	: Rapid Visual Screening of Buildings for Potential Seismic Hazards

GWT	: Groundwater Table Depth
LD	: Limited Damage (performance level)
MASW	: Multichannel Analysis of Surface Waves
MRF	: Moment-Resisting Frame
MVLEM	: Multiple-Vertical-Line-Element Model
NBCC	: National Building Code of Canada
NSI	: Nonstructural Index
OpenSees	: Open System for Earthquake Engineering Simulation
P-25	: Turkish rapid seismic screening method
PERA	: Rapid seismic assessment method family
PERA2019	: 2019 update of PERA method
	: Reinforced Concrete
ROC	: Receiver Operating Characteristic
RSRA	: Rapid Seismic Risk Assessment
SD	: Significant Damage (performance level)
TBEC	: Turkish Building Earthquake Code (2018)
TS 500	: Turkish Standard for Reinforced Concrete Design and Construction
URM	: Unreinforced Masonry
USGS	: United States Geological Survey

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1. INTRODUCTION

1.1 Subject

Turkey is geographically situated in a region that has historically been exposed to frequent and severe earthquakes. Major seismic events such as the Erzincan Earthquake (1939), the Marmara Earthquake (1999), the Van Earthquake (2011), and the Kahramanmaraş Earthquake (2023) have tragically revealed the extent to which this natural disaster poses a significant threat to the country. Each major earthquake in the past has not only resulted in the loss of lives and extensive material damage but has also heightened public and institutional sensitivity regarding structural safety. Considering potential future hazards, such as the anticipated major Marmara Earthquake, it is of vital importance to rapidly and effectively assess the safety of the existing building stock in order to minimize the impacts of possible large-scale earthquakes. However, the vast number of existing structures and the time-consuming nature of conventional detailed assessment methods constitute a major obstacle to this process.

As will be further discussed in later sections, traditional detailed assessment methods thoroughly examine the structural characteristics of a building. These methods often require advanced analyses, such as modeling the nonlinear behavior of the structure, determining its plastic deformation capacities, and defining its limit performance levels. Nevertheless, since this procedure is considerably lengthy and comprehensive, there is an urgent need for rapid risk identification methods that enable the examination of a large number of buildings within a limited time frame. Such methods typically involve practical and swift evaluation processes based on visual inspection and provide a crucial tool for large-scale screening efforts.

1.1.1 Purpose of the Thesis

Within the scope of this thesis, the significance and areas of application of visual-based rapid seismic risk assessment methods used for the swift evaluation of Turkey's building stock against earthquake hazards are examined in detail. The advantages,

limitations, and potential for improvement of these methods are discussed, emphasizing their contributions toward creating a more resilient built environment in the face of possible seismic disasters.

The study focuses on an original system developed to determine the earthquake resistance level of the existing building stock through a field-based and time-efficient approach, namely the Rapid Seismic Risk Assessment (Hızlı Risk Tespiti, RSRA) method. During the development of the RSRA method, several rapid screening methodologies available in the literature were reviewed, including FEMA P-154, the Canadian Seismic Screening Method, the P-25 Method, and Pera2019, all of which served as major reference points. The parameters commonly used in these methods primarily aim to systematically identify structural irregularities based on field observations. Considering the characteristics of Turkey's building stock, determining the quality of materials through field investigations is of critical importance. Therefore, in the developed method, the assessment of material quality was prioritized. Furthermore, in addition to the parameters employed in existing methodologies, the RSRA method incorporates engineering-based quantities such as concrete compressive strength, stirrup spacing, anchorage length, section effects (M, N, V), and natural period into the scoring system, all derived from field data. In this respect, the RSRA method provides a more comprehensive assessment framework that integrates both visual and analytical data.

The RSRA method was designed to be applicable under real field conditions, aiming to understand building behavior through on-site observations and experimental data. In this method, weaknesses in the structural system are numerically evaluated under specific parameter groups, and these numerical scores are multiplied to obtain the Structural Index Score (SIS). The final score quantitatively defines the risk level of the structure.

Additionally, in order to calculate certain parameters on a more scientific basis, a simplified equivalent static model was created for each structure through an interface software developed on the OpenSees platform. This allowed for a more realistic determination of the fundamental dynamic properties of the structures—such as period, mass, and stiffness—and incorporated analytical results into the overall risk scoring process.

The RSRA method was developed by Dr. Onur Avcıoğlu, Abdullah Tokmak, and Kadir Akman, and was applied to 513 branch buildings of a public bank in Turkey. The conceptual foundation of the RSRA method, however, was established individually within the scope of this thesis.

Although the RSRA study was conducted on a total of 513 branches, the statistical evaluations presented in Chapter 5 were carried out on 427 branch buildings. Detailed explanations regarding the 86 buildings excluded from the evaluation are also provided in Chapter 5.

Before implementing the RSRA applications, buildings in the bank's inventory that had previously undergone performance analysis or structural strengthening were reviewed. As a result of these preliminary investigations, it was observed that in regions where the spectral acceleration coefficient ($S_d \geq 0.6$), most buildings exhibited insufficient seismic performance, revealing the necessity for strengthening. This observation constituted the primary rationale for selecting $S_d = 0.6$ as the critical threshold in the RSRA studies.

Therefore, during the fieldwork, buildings located in regions with $S_d \geq 0.6$ were designated as priority inspection areas. Additionally, buildings constructed after 2010 or previously retrofitted were excluded from the study. The year 2010 marks the nationwide implementation of Turkey's Building Inspection Law No. 4708, and it is assumed that structures built after this date were more likely to have been inspected and designed in accordance with contemporary seismic codes.

As a result, a total of 638 buildings were selected as candidates for evaluation based on these criteria, and RSRA studies were conducted on this dataset.

To determine the soil properties of the buildings, available geotechnical survey reports were used. In the absence of such reports, publicly available data sources such as those provided by the United States Geological Survey (USGS) were utilized to estimate the soil shear-wave velocity ($V_{s,30}$). The spectrum generated according to the soil conditions played a significant role in the scoring process.

Within the scope of this thesis, the design drawings of 34 buildings evaluated through the RSRA method were obtained and examined; however, only 28 of them were subjected to a more detailed evaluation using nonlinear performance analyses. The exclusion of the remaining six structures was due to incomplete design documentation,

inconsistencies between field and analytical data, or the lack of sufficient information for reliable analysis. Furthermore, it was determined that past renovations had significantly altered the dynamic behavior of certain structures. For instance, a steel story was later added atop a five-story building; in another, basement shear walls were largely removed; and in yet another, the concrete exhibited highly irregular aggregate distribution and extensive corrosion. Such conditions hindered the accurate interpretation of analyses conducted using the RSRA method and therefore led to the exclusion of these structures from detailed evaluation.

For each of the 28 evaluated structures, nonlinear static pushover analyses were conducted to determine whether the buildings achieved the “controlled damage” performance level. For those that did, the spectral acceleration coefficients (S_{ds} and S_{d1}) were gradually increased, and pushover analyses were repeated at each increment to identify the point at which the structure exceeded its capacity limit. This approach revealed the capacity thresholds of the structures under increasing seismic demands. Conversely, for buildings that did not reach the controlled damage level in the initial analysis, the spectral demand levels were gradually decreased, and the nonlinear performance levels were examined at each stage to determine the demand level corresponding to controlled damage.

Through this bidirectional analytical approach, a “spectrum multiplier” corresponding to the controlled damage performance level was obtained for each building. These multipliers were compared with the Structural Index Scores (SIS) calculated using the RSRA method to evaluate the method’s ability to predict the seismic performance of structures.

Furthermore, this thesis includes a comprehensive review of similar rapid screening methods available in the literature and studies correlating these methods with structural performance. In the following sections, the FEMA P-154, Canadian Seismic Screening Method, P-25, and Pera2019 methodologies are introduced sequentially, followed by a detailed discussion of the development, implementation, and comparative framework of the RSRA method supported by the results of the pushover analyses.

1.2 Nonlinear Behavior of Structural Systems

Structural systems generally exhibit linear behavior under service loads. However, since their behavior under earthquake loads is considerably more complex, a more comprehensive analysis is required. When analyzing the behavior of linear systems, the stress–strain relationships of materials are assumed to be linearly elastic, and displacements are considered small enough that second-order effects can be neglected. As external loads exceed service levels and approach the ultimate strength of the structure, stresses surpass the linear-elastic limit and displacements increase significantly.

Although structural design codes permit analysis based on linear theory, they indirectly incorporate the effects of nonlinear behavior into structural analysis or limit this behavior to mitigate its impact. For instance, according to the earthquake codes published in 1997, 2007, and 2018, the seismic forces acting on a structure are obtained using the elastic spectrum in conjunction with the structural behavior factor, denoted as R . In this context, the nonlinear structural behavior during an earthquake is indirectly accounted for through the use of the R factor. Similarly, the TS500 standard proposes the moment magnification method to consider second-order effects. Moreover, by limiting the slenderness ratio (λ) to a maximum value of 100, the code ensures that the assumptions adopted in the analysis remain valid. In the calculation of soil stresses, large safety factors are applied, thereby assuming that the soil behaves in a linear-elastic manner.

When analyses are performed under the assumption of linearly elastic materials, large safety factors are employed, and numerous simplifications are made. However, designing structural systems by considering their behavior beyond the linear-elastic range enables more realistic and economical solutions. Accounting for nonlinear behavior requires modeling the structural response more accurately, which often involves complex computations. Additionally, solving nonlinear systems of equations introduces further numerical challenges.

1.2.1 Conditions That Must Be Satisfied in Structural Analysis

A structural system must be in equilibrium under external loads and internal forces. The analysis of structural systems must satisfy the following three fundamental conditions:

1. Constitutive Conditions: These depend on the material properties and define the stress–strain relationships.
2. Equilibrium Conditions: These are derived from the equilibrium equations of the structural members and the joints connecting them.
3. Geometric Compatibility (Continuity) Conditions: These refer to the continuity relationships between the members and joints, as well as the geometric constraints at the supports.

1.2.2 Factors Causing Nonlinearity in Structural Systems

The main factors contributing to the nonlinear behavior of structural systems are as follows:

1. The stress–strain relationship is nonlinear because the material does not behave in a linear-elastic manner.
2. The equilibrium equations become nonlinear due to large displacements.

In systems where displacements are not small, the equilibrium equations are written with respect to the deformed geometry. Similarly, when displacements are large in the geometric compatibility conditions, the continuity equations must also be expressed with respect to the deformed configuration (see **Table 1.1**)

The comparison of field data obtained through the Rapid Seismic Risk Assessment (RSRA) method with the results of nonlinear analyses is essential for a more comprehensive understanding of the behavior of structural systems beyond their elastic limits. In this context, the theoretical basis of the nonlinear behavior of structural systems is addressed in the following sections.

Table 1.1: Causes of nonlinear behavior in structural systems.

Conditions Required for Solution	Linear Systems	Nonlinear Systems				
		In Terms of Material	In Terms of Geometric Changes		In Terms of Both	
			Second-Order Theory	Finite Displacement Theory	Second-Order Theory	Finite Displacement Theory
Constitutive Equations (Stress–Strain Relationships)	Linear-elastic	Not linear-elastic	Linear-elastic	Linear-elastic	Not linear-elastic	Not linear-elastic
Displacements in Equilibrium Equations	Small	Small	Not small	Not small	Not small	Not small
Displacements in Geometric Compatibility Conditions	Small	Small	Small	Not small	Small	Small

1.2.3 Nonlinear Behavior of Structural Systems under External Loads

The load parameter–displacement (P – Δ) relationships of a structural system subjected to vertical and lateral loads, obtained according to both linear and nonlinear theories, are illustrated schematically in **Figure 1.1**.

For a structural system in which the material is assumed to behave in a linear-elastic manner, its response under increasing external loads, based on the first-order theory, is represented by the straight line (I) in the figure. In the second-order theory, where the effects of geometric changes on the equilibrium equations—that is, the second-order effects induced by axial forces—are taken into account, different behavioral patterns may occur depending on whether the axial force is compressive or tensile. For example, when the axial force is compressive, as seen in curve (II), increasing external loads cause displacements to grow at a faster rate. When the load parameter, representing the intensity of the external loads, reaches a value denoted as P_B —known as the linear-elastic buckling load—the displacements increase indefinitely, and the system collapses through buckling. In certain specific cases, following buckling, increasing displacements may correspond to decreasing load parameters. Conversely, in systems where the axial force is tensile, such as suspension structures, the P – Δ diagram exhibits a strain-hardening behavior, as illustrated by curve (IIa). In systems not subjected to lateral loads and therefore not deformed prior to buckling, a phenomenon known as bifurcation buckling occurs when the load parameter reaches

a critical value, P_{cr} . As seen in the diagram (IIb), displacements increase suddenly and approach infinity. The load that causes bifurcation buckling is termed the critical load, which is generally equal to or slightly greater than the buckling load. Bifurcation buckling can also occur in systems that experience deformation prior to buckling, as indicated by curve (II).

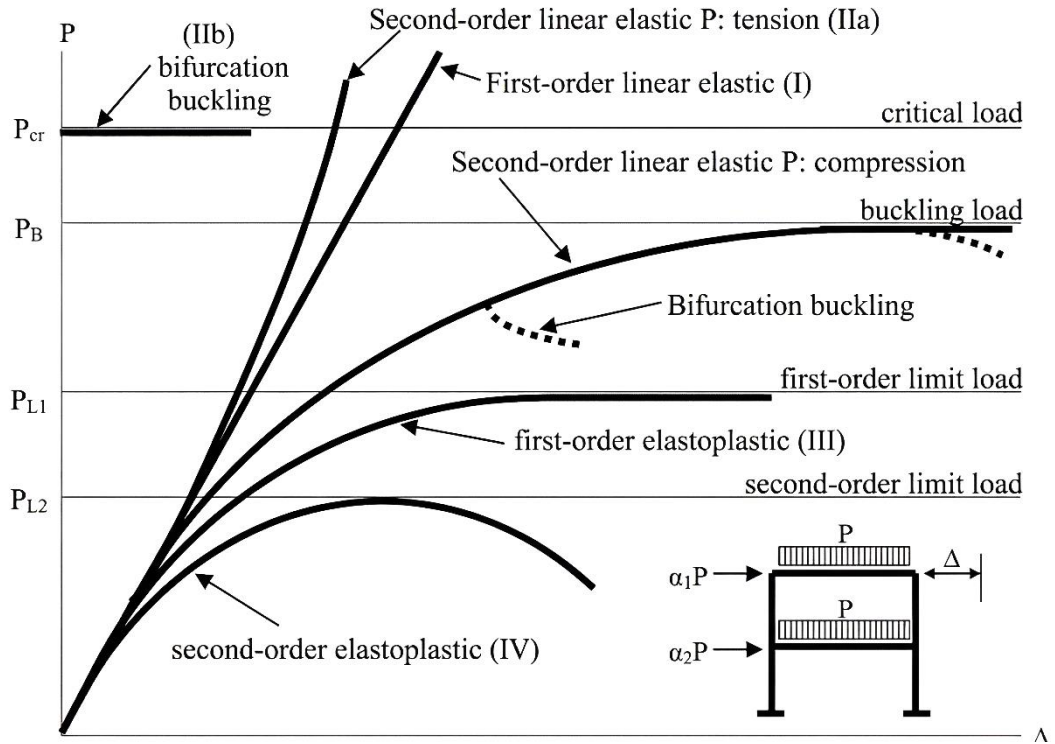


Figure 1.1: Load parameter–displacement relationships obtained according to various theories.

In systems composed of nonlinear materials, internal forces increase with the applied external loads, and the linear-elastic limit is exceeded in certain sections, resulting in nonlinear (plastic) deformations. These nonlinear deformations may spread continuously throughout the system. However, in structures made of ductile materials—where the ratio of total deformation at failure to elastic deformation is large—the nonlinear deformations are typically assumed to concentrate at specific sections, referred to as plastic hinges (or, more generally, plastic zones), while the rest of the system remains linearly elastic. This assumption is known as the plastic hinge hypothesis.

For a structural system analyzed under the first-order theory based on the plastic hinge hypothesis, as represented by curve (III), the formation of plastic hinges causes part or

all of the structure to behave as a mechanism, indicating that the load-bearing capacity has been exhausted. The corresponding load is termed the first-order limit load.

When both sources of nonlinearity are considered together—that is, when the structural system is analyzed according to the second-order elastoplastic theory—the resulting $P-\Delta$ relationship is depicted by curve (IV) in the figure. In this diagram, the behavior initially follows curve (II) up to the point where the first critical section exceeds the linear-elastic limit; thereafter, due to plastic deformations, displacements increase more rapidly. In structural systems analyzed under the plastic hinge hypothesis, as the external loads increase and reach a limiting value PL_2 , the formation of plastic hinges causes the system stiffness to decrease, and the buckling load of the structure drops below the applied load parameter. In other words, in the $P-\Delta$ diagram, increasing displacements correspond to decreasing loads. The load parameter at which the structure loses its load-bearing capacity due to stability deficiency is defined as the second-order limit load.

In some cases, even before reaching the limit load, large displacements, extensive plastic deformations, or severe cracking in reinforced concrete systems may render the structure unserviceable or lead to collapse.

Understanding these modes of nonlinear structural behavior is particularly important for interpreting the engineering rationale behind the scoring mechanism embedded within the Rapid Seismic Risk Assessment (RSRA) method. This theoretical framework provides the conceptual foundation for modeling the performance classes of buildings evaluated using the RSRA method within the scope of this thesis.

1.3 Earthquake Risks and Study Approach

A significant portion of Turkey's land area and population resides in regions with high seismic hazard. When combined with the insufficient structural performance of the existing building stock under expected seismic demands, this situation poses an extremely high earthquake risk for the country. Numerous buildings are seismically vulnerable due to deficiencies in load-bearing systems, inadequate concrete quality, irregularities, and construction errors.

Conducting detailed performance analyses for every building with unknown structural adequacy across the entire building stock is neither economically feasible nor

operationally practical. As a solution, rapid assessment methods have been developed to prioritize buildings according to their seismic risk levels and to support decisions regarding evacuation or the necessity for detailed performance evaluations.

The primary objective of this thesis is to examine the prioritization capability of the newly developed RSRA method and to enhance its efficiency through necessary improvements. By analyzing the correlation between the RSRA scores derived from field data and the spectrum multipliers obtained from nonlinear analyses, this study aims to demonstrate that the RSRA method serves as a field-based yet analytically verifiable screening tool.

1.4 Scope

In the second chapter, the rapid assessment methods forming the basis of the RSRA study—namely FEMA P-154, the Canadian Seismic Screening Method, the P-25 Method, and Pera2019—are introduced. Their field implementation procedures and the parameters they employ are summarized. These methods are compared with the RSRA method to evaluate its contribution to the literature.

In the third chapter, the development, application principles, parameter structure, and version updates of the RSRA method are detailed. Additionally, supporting documents from a previously conducted field study and examples demonstrating how certain parameters were calculated are presented in Appendix 1.

In the fourth chapter, the relationship between RSRA scores and nonlinear analysis results, obtained using the ETABS software and the spectrum-based multiplier approach, is examined, and the data produced for each building are interpreted.

In the fifth and final chapter, the overall findings of the study are discussed, along with an evaluation of the RSRA method's effectiveness as a seismic screening tool and its potential areas for improvement.

1.5 Studies Comparing Rapid Screening Methods with Performance Analyses

This section provides a summary of existing studies in the literature that compare rapid assessment methods with the seismic performance of buildings. Following this

overview, the four principal rapid assessment methods used in this thesis—FEMA P-154, the Canadian Seismic Screening Method, the P-25 Method, and Pera2019—are introduced and described in detail.

1.6 Comparative Studies in the Literature

The question of how closely the scoring or prioritization results obtained from rapid assessment methods align with the outcomes of detailed structural performance analyses has been the subject of numerous studies in the literature. Within the scope of this thesis, the degree of correlation between seismic risk scores obtained using various rapid assessment methods and the results of linear or nonlinear analyses has been examined.

Özmen B. H. (2005), in the thesis titled *Investigation of the Effects of Parameters Used in Rapid Assessment Methods on Structural Performance*, first modeled two reinforced concrete buildings with four and seven stories without any irregularities. Subsequently, common irregularities such as short columns, soft stories, enclosed cantilevers, and wide stirrup spacing were introduced, and performance analyses were conducted for each scenario. This study demonstrated which types of irregularities most significantly affect structural performance and to what extent rapid assessment methods are capable of predicting such effects.

Yakut (2004), in the paper titled *Preliminary Seismic Assessment Procedure*, proposed a rapid assessment model that considers parameters such as soft stories, short columns, plan irregularities, vertical irregularities, and structural quality. Coefficients influencing structural behavior were defined for each parameter, and a scoring framework was developed accordingly.

Dilan K. (2019), in the thesis *Comparison of Parameters Used in Rapid Screening Methods with Structural Earthquake Performance*, selected a five-story reinforced concrete frame building as a reference structure. Each irregularity parameter (such as insufficient stirrup spacing or vertical irregularity) was applied individually, and variations in the obtained rapid assessment scores were analyzed to determine the sensitivity of the scoring system to specific deficiencies.

Polat N. (2025), in the thesis *Comparison of P25, Ryteie 2019, and Pera Methods with TBEC 2018 Performance Analysis Results for an Existing Reinforced Concrete*

Building, performed a nonlinear multimodal pushover analysis in accordance with the 2018 Turkish Building Earthquake Code (TBEC 2018) to determine the performance level of a three-story reinforced concrete building located in Serdivan, Sakarya. The same building was analyzed using the Pera and RYTEIE 2019 rapid risk assessment methods, and the compatibility between these methods and the code-based analysis results was investigated.

Al-Nimry et al. (2015), in their paper Rapid Assessment for Seismic Vulnerability of Low and Medium Rise Infilled RC Frame Buildings, presented a rapid assessment approach for low- and mid-rise reinforced concrete buildings. The proposed method incorporated parameters such as heavy overhangs, plan irregularities, and soft/weak stories. The influence of these parameters on visual screening and capacity indices was integrated into the model through performance multipliers and coefficient factors.

Vulaş Y. (2014), in the thesis Performance Analysis of Existing Reinforced Concrete Buildings Using the PERA Method, compared the Rapid Performance Evaluation Method (PERA) with the Regulation on Structures to be Built in Seismic Zones and the Principles for the Determination of Risky Buildings applicable in Turkey.

Ergun M. (2007), in the thesis Comparison of the Japanese Seismic Index Method and Nonlinear Incremental Pushover Analysis Results in Determining the Seismic Performance of Buildings, analyzed two reinforced concrete school buildings using the Japanese Seismic Index Method and subsequently performed Incremental Pushover Analyses on the same structures. The results obtained from both approaches were compared to assess consistency.

Özkul B. and Gülgeç E. (2022), in the paper Comparison of the Seismic Performance of a Reinforced Concrete School Building Using Four Different Rapid Assessment Methods, evaluated the earthquake performance of a previously studied reinforced concrete frame-type school building using four different and one revised rapid assessment methods. The results obtained from each method were compared, identifying the relative strengths and limitations of each approach.

Finally, **Çırak Ç. (2016)**, in the thesis Evaluation of Seismic Performance Determined by Rapid Assessment and Detailed Investigation Methods in Existing Buildings, examined the consistency between commonly used rapid assessment methods in

Turkey and nonlinear analysis results, assessing which methods are more applicable and reliable in practice.

The aforementioned studies collectively reveal the extent to which rapid assessment methods correspond with actual structural behavior observed in the field. Furthermore, they provide valuable insight into the relative significance of various parameters under different conditions, highlighting which factors are most decisive in rapid seismic evaluation.

1.6.1 Rapid Assessment Methods Introduced in This Thesis

In this thesis, four rapid assessment methods—FEMA P-154, the Canadian Seismic Screening Method, P-25, and Pera2019—are comparatively analyzed. Although these methods rely on different data sources and parameter priorities, their common objective is to rapidly estimate the seismic risk of existing buildings.

FEMA P-154 Rapid Visual Screening Method: Developed by the Applied Technology Council (ATC), this method aims to determine the seismic risk potential of buildings through exterior and interior visual observations. During the assessment, parameters such as structural type, construction year, number of stories, irregularities, and soil conditions are recorded using standardized forms. The collected data are processed through weighted factors to compute an “S” score, which is then used to classify the building’s risk level.

Canadian Seismic Screening Method: Developed by the National Research Council of Canada (NRCC), this method establishes a Seismic Performance Index (SPI) that accounts for both structural and non-structural components. The Structural Index (SI) is obtained by multiplying coefficients A (seismicity), B (soil conditions), C (structural system), D (irregularities), and E (building importance). The Non-Structural Index (NSI) is defined based on coefficients B, E, and F (non-structural hazards). The sum of SI and NSI yields the SPI value, which is used to prioritize buildings for further evaluation.

P-25 Method: Developed in Turkey as an improved version of previous models such as P5 and P24, this method incorporates a broad set of parameters, including infill wall characteristics, column and shear wall stiffness, structural irregularities, number of

stories, soil type, and material properties. The total score, referred to as the “P” score, is used to classify buildings into low, moderate, or high seismic risk categories.

Pera2019 Method: Initially proposed by Ilki et al. (2014) and later updated under the name PERA2019, this method was developed for the rapid seismic performance evaluation of reinforced concrete buildings. The approach considers, based on field data, the shear force (V_e), shear capacity (V_r), moment capacity (M_r), and axial loads of columns and shear walls. The computed demand-to-capacity ratios ($r_1 = V_e/V_r$ and $r_2 = M_e/M_r$) and interstory drift ratios serve as the primary indicators for assessing element damage levels. In addition, reduction coefficients are introduced to account for structural irregularities, allowing for a more realistic risk classification. In this framework, if any column or shear wall at the ground floor exhibits a high-risk condition, the building is classified as “high risk.”

In this thesis, these methods are also compared with the Rapid Seismic Risk Assessment (RSRA) approach to identify the aspects in which RSRA provides a more detailed and distinctive evaluation framework.

2. RAPID SEISMIC RISK ASSESSMENT METHODS: APPLICATIONS IN THE LITERATURE AND A COMPARISON-BASED APPROACH

In countries exposed to significant earthquake risk, conducting detailed engineering analyses for every structure within the building stock is practically unrealistic due to the sheer number of buildings. This has necessitated the development of rapid assessment methods, which allow for the prioritization of vulnerable buildings and the efficient allocation of resources. Rapid screening methods aim to provide an evaluation that is based on field observations, can be completed within a short time, and can be applied by an experienced engineer in field conditions.

In this chapter, the FEMA P-154, Canadian Seismic Screening Method, P-25, and Pera2019 approaches—widely used in the international literature and developed for the building typologies of different countries—are examined systematically. Each of these methods aims to estimate the seismic risk level of buildings by taking into account different geographical, structural, and regulatory conditions. Accordingly, they are comparable in terms of their assessment principles and scoring frameworks.

The rapid assessment methods presented in the following subsections are analyzed with respect to their data collection procedures, field application principles, and computational approaches. The structure offered by these methods constituted both the conceptual and practical foundation for the development of the RSRA method and forms the basis of the comparative analyses discussed in later chapters.

2.1 Fema P-154 Rapid Screening Method

The FEMA P-154 Rapid Screening Method is a screening procedure that can be considered a preliminary evaluation stage for identifying potentially hazardous buildings using on-site observational data. The method enables an initial judgment regarding the expected seismic performance of a structure without performing a detailed analysis. Developed in the United States by the Applied Technology Council

(ATC), it was last updated in 2015 and is still widely used internationally as a primary screening tool for various building inventories.

The key advantage of the method is that it allows a building to be screened in approximately 15–20 minutes and provides a generalized performance indicator (score). This is particularly beneficial for prioritizing large building stocks by saving both time and resources.

The FEMA P-154 method consists of two main stages:

- Level 1 Screening: Collection of building data through rapid on-site visual inspection and scoring via a standardized form.
- Level 2 Screening: Collection of more detailed observational data and refinement of the score in cases where the Level 1 score is insufficient or uncertainty remains.

Through these two stages, an initial evaluation of whether the building is considered seismically at risk can be made. The application principles of the FEMA P-154 method, along with details of each stage, are presented in the following sections.

2.1.1 Overview of the Screening Procedure

The FEMA P-154 rapid screening method is a field-based, quick, and low-cost approach that facilitates the identification of potentially risky buildings within a building stock. It is used as a pre-screening tool prior to detailed structural analysis, ensuring that only structures with real seismic risk are subjected to more comprehensive evaluation.

The procedure consists of two stages. The first stage involves a rapid field inspection lasting approximately 15 minutes; the second stage consists of a more detailed field investigation and scoring process, which can only be conducted by qualified engineers.

In the first stage, fundamental data such as the structural system type, building use, height, and year of construction are collected. These data are entered into one of the five different data collection forms prepared in accordance with the seismic region in which the building is located. During the inspection, particular attention is given to horizontal and vertical irregularities, separation distance from adjacent buildings, safety of exterior nonstructural components, and site characteristics.

No structural calculations are required during the screening process. Based purely on observational data, a score called the Final Level 1 Score (SL1) is obtained. This score is evaluated by comparing it with the threshold values recommended by FEMA. If $SL1 \geq 2.0$, the building is considered seismically acceptable; otherwise, a more detailed investigation or a performance analysis is recommended.

The second stage is optional and applied only when additional information is required or when a clear judgment cannot be made based on the first-stage observations. In this stage, more comprehensive score modifiers and nonstructural hazards are evaluated. The Level 2 form also includes additional risk factors such as short columns, soft stories, story-level discontinuities, additions, and plan irregularities.

The data collection forms used in this process vary according to structural type and seismic region, and each includes score modification coefficients specific to certain building characteristics. The content of the forms and the detailed application principles are explained in later sections.

2.1.2 Stage One: Screening Steps

The rapid visual screening process carried out within the scope of FEMA P-154 begins with the first-stage screening conducted in the field. This stage aims to inspect the building on site within a short period of time and to rapidly evaluate its seismic performance based on its fundamental structural characteristics. The inspection typically takes about 15–30 minutes per building. The evaluation is performed mainly through observations of the exterior façades; however, where possible, the interior of the building is also examined.

The screening steps can be summarized as follows:

- **Pre-Field Planning:** Preliminary information such as the building's location, year of construction, and intended use is collected. The seismicity level of the region, site class, and geological hazards are investigated. This information ensures that the correct version of the Level 1 Data Collection Form is selected (see **Figure 2.1**).
- **Completion of the Data Collection Form:** The sections of the standard forms provided by FEMA P-154 are filled in sequentially. These sections include

building identification data, number of stories, structural type, construction material, site characteristics, structural irregularities, and observed damage.

Rapid Visual Screening of Buildings for Potential Seismic Hazards
FEMA P-154 Data Collection Form

Level 1
HIGH Seismicity

PHOTOGRAPH

SKETCH

Address: _____ Zip: _____

Other Identifiers: _____

Building Name: _____

Use: _____

Latitude: _____ Longitude: _____

S₁: _____ S₂: _____

Screener(s): _____ Date/Time: _____

No. Stories: Above Grade: _____ Below Grade: _____ Year Built: ☐ EST

Total Floor Area (sq. ft.): _____ Code Year: _____

Additions: ☐ None ☐ Yes, Year(s) Built: _____

Occupancy: Assembly ☐ Commercial ☐ Other Services ☐ Historic ☐ Shelter
Industrial ☐ Office ☐ School ☐ Government
Utility ☐ Warehouse ☐ Residential, #Units: _____

Soil Type: ☐ A Hard Rock ☐ B Avg Rock ☐ C Dense Soil ☐ D Stiff Soil ☐ E Soft Soil ☐ F Poor Soil ☐ DNK #DNK, assume Type D.

Geologic Hazards: Liquefaction: Yes/No/DNK Landslide: Yes/No/DNK Surf. Rupt.: Yes/No/DNK

Adjacency: ☐ Pounding ☐ Falling Hazards from Taller Adjacent Building

Irregularities: ☐ Vertical (type/severity) _____
☐ Plan (type) _____

Exterior Falling Hazards: ☐ Unbraced Chimneys ☐ Heavy Cladding or Heavy Veneer
☐ Parapets ☐ Appendages
☐ Other: _____

COMMENTS:

☐ Additional sketches or comments on separate page

BASIC SCORE, MODIFIERS, AND FINAL LEVEL 1 SCORE, S _{L1}																		
FEMA BUILDING TYPE	Do Not Know	W1	W1A	W2	S1 (MRF)	S2 (BF)	S3 (LM)	S4 (RC SN)	S5 (URM RF)	C1 (MRF)	C2 (GW)	C3 (URM IN)	PC1 (TU)	PC2	RM1 (FD)	RM2 (RD)	URM	MH
Basic Score		3.6	3.2	2.9	2.1	2.0	2.6	2.0	1.7	1.5	2.0	1.2	1.6	1.4	1.7	1.7	1.0	1.5
Severe Vertical Irregularity, V _{s1}		-1.2	-1.2	-1.2	-1.0	-1.0	-1.1	-1.0	-0.8	-0.9	-1.0	-0.7	-1.0	-0.9	-0.9	-0.9	-0.7	NA
Moderate Vertical Irregularity, V _{s2}		-0.7	-0.7	-0.7	-0.6	-0.6	-0.7	-0.6	-0.5	-0.5	-0.6	-0.4	-0.6	-0.5	-0.5	-0.5	-0.4	NA
Plan Irregularity, P _{s1}		-1.1	-1.0	-1.0	-0.8	-0.7	-0.9	-0.7	-0.6	-0.6	-0.8	-0.5	-0.7	-0.6	-0.7	-0.7	-0.4	NA
Pre-Code		-1.1	-1.0	-0.9	-0.6	-0.6	-0.8	-0.6	-0.2	-0.4	-0.7	-0.1	-0.5	-0.3	-0.5	-0.5	0.0	-0.1
Post-Benchmark		1.6	1.9	2.2	1.4	1.4	1.1	1.9	NA	1.9	2.1	NA	2.0	2.4	2.1	2.1	NA	1.2
Soil Type A or B		0.1	0.3	0.5	0.4	0.6	0.1	0.6	0.5	0.4	0.5	0.3	0.6	0.4	0.5	0.5	0.3	0.3
Soil Type E (1-3 stories)		0.2	0.2	0.1	-0.2	-0.4	0.2	-0.1	-0.4	0.0	0.0	-0.2	-0.3	-0.1	-0.1	-0.1	-0.2	-0.4
Soil Type E (> 3 stories)		-0.3	-0.6	-0.9	-0.6	-0.6	NA	-0.6	-0.4	-0.5	-0.7	-0.3	NA	-0.4	-0.5	-0.6	-0.2	NA
Minimum Score, S _{MIN}		1.1	0.9	0.7	0.5	0.5	0.6	0.5	0.5	0.3	0.3	0.3	0.2	0.2	0.3	0.3	0.2	1.0

FINAL LEVEL 1 SCORE, S_{L1} ≥ S_{MIN}

EXTENT OF REVIEW

Exterior: ☐ Partial ☐ All Sides ☐ Aerial

Interior: ☐ None ☐ Visible ☐ Entered

Drawings Reviewed: ☐ Yes ☐ No

Soil Type Source: _____

Geologic Hazards Source: _____

Contact Person: _____

LEVEL 2 SCREENING PERFORMED?

☐ Yes, Final Level 2 Score, S_{L2} _____ ☐ No

Nonstructural hazards? ☐ Yes ☐ No

OTHER HAZARDS

Are There Hazards That Trigger A Detailed Structural Evaluation?

☐ Pounding potential (unless S_{L2} > cut-off, if known)

☐ Falling hazards from taller adjacent building

☐ Geologic hazards or Soil Type F

☐ Significant damage/deterioration to the structural system

ACTION REQUIRED

Detailed Structural Evaluation Required?

☐ Yes, unknown FEMA building type or other building

☐ Yes, score less than cut-off

☐ Yes, other hazards present

☐ No

Detailed Nonstructural Evaluation Recommended? (check one)

☐ Yes, nonstructural hazards identified that should be evaluated

☐ No, nonstructural hazards exist that may require mitigation, but a detailed evaluation is not necessary

☐ No, no nonstructural hazards identified ☐ DNK

Where information cannot be verified, screener shall note the following: EST = Estimated or unreliable data OR DNK = Do Not Know

Legend: MRF = Moment-resisting frame RC = Reinforced concrete URM MRF = Unreinforced masonry mRF MH = Manufactured Housing FD = Flexible diaphragm
BR = Braced frame SW = Shear wall TU = Tilt up LM = Light metal RD = Rigid diaphragm

Figure 1-1 RVS Level 1 Data Collection Form for High seismicity region.

Figure 2.1: Level 1 data form for the rapid screening method.

- **Identification of Building Type:** The evaluator selects the most appropriate type among the 17 building categories defined by FEMA for the structure being assessed (for example, S1 – Steel Moment Frame, C1 – Concrete Moment Frame, URM – Unreinforced Masonry, etc.). This selection is critically important for the calculation of the basic score (see **Figure 2.1**).
- **Determination of the Basic Score:** Based on the selected building type and the seismicity of the region in which the structure is located, the Basic Score defined in FEMA P-154 is determined. This value represents the initial risk level of the building.
- **Application of Score Modifiers:** The score is increased or decreased according to various parameters such as the structural system type, year of construction, site class, and vertical or plan irregularities. The score modifiers are determined using the tables provided in FEMA P-154.
- **Calculation and Interpretation of the Final Score:** The sum of the Basic Score and the score modifiers constitutes the final Level 1 score for the building. If this score is below the specified threshold, the structure is considered to have high seismic risk and is referred to Level 2 evaluation.

2.1.2.1 Building Identification Information

One of the initial steps of the rapid visual screening process is to collect the basic identification information of the building under examination. Located in the upper-right portion of the FEMA P-154 Data Collection Form, this information contributes both to maintaining orderly records in the field and to the subsequent reporting process after analysis. It also facilitates the re-identification of the building in any future detailed evaluations (**Figure 2.2**).

The principal information requested on the form is as follows:

- **Address and Building Name:** The full address of the building under evaluation and its specific name, if any, are recorded. This information is particularly important in large-scale screening campaigns involving many buildings, as it helps prevent confusion.
- **Latitude and Longitude:** The geographic coordinates of the building are determined using a GPS device or mapping services and entered on the form.

These data allow correlation with seismic hazard maps and enable an accurate definition of the seismic hazard level at the building site.

- **Screener and Date:** The name of the person conducting the assessment and the date of the screening are recorded to ensure transparency and traceability of the evaluation process.

Address: _____	
_____	Zip: _____
Other Identifiers: _____	
Building Name: _____	
Use: _____	
Latitude: _____	Longitude: _____
Ss: _____	Sr: _____
Screener(s): _____	Date/Time: _____

Figure 2.2: Building identification information.

2.1.2.2 Number of Stories, Year of Construction, and Height

Building height, number of stories, and year of construction are among the fundamental inputs in the FEMA P-154 screening form that influence the seismic performance potential of a structure. These parameters reflect not only the geometry of the building but also the engineering design criteria and the level of compliance with the codes in force at the time of construction (**Figure 2.3**).

- **Number of Stories:** When determining the number of stories, the topographic conditions of the site must be taken into account. On sloping ground, all levels from the lowest point of the building to the highest point of the roof should be counted. Basement levels or underground parking areas must also be included in the assessment.
- **Year of Construction:** The design or construction year of the building provides insight into the building codes that were in effect at that time. For structures designed according to earlier codes, it is likely that earthquake-resistant design criteria were inadequate or absent. Therefore, the year of construction is particularly important for classification as “pre-code” or “post-code”.

- **Height:** The total height of the building is another important factor affecting its behavior during an earthquake. Taller structures generally have longer fundamental periods, which is directly related to spectral acceleration. Building height is evaluated together with the number of stories and design year in the FEMA P-154 scoring system and can directly influence the risk score.

No. Stories:	Above Grade: _____	Below Grade: _____	Year Built: _____	☐
Total Floor Area (sq. ft.): _____			Code Year: _____	
Additions:	☐ None	☐ Yes, Year(s) Built: _____		

Figure 2.3: Building characteristics.

2.1.2.3 Photographing the Building

In the rapid screening process conducted within the framework of FEMA P-154, photographing the building is important not only for visual record-keeping but also from a structural evaluation standpoint. Building photographs help document potential irregularities and damage risks while also contributing to the visual completeness of the form (**Figure 2.4**).

The following criteria should be observed when taking photographs:

- **Façade Photographs:** At least one photograph should be taken from each side of the building (front, rear, right, and left façades). This allows the structural arrangement and architectural features of all façades to be analyzed.
- **Adjacency Conditions:** The position of the building relative to neighboring structures (attached, detached, block-type configurations, etc.) should be clearly shown. This information is important for assessing the risk of pounding.
- **Vertical and Horizontal Discontinuities:** Potential irregularities such as large openings at the ground floor, cantilevered projections, and mezzanine levels should be explicitly documented.
- **Nonstructural Elements:** The condition of nonstructural components such as chimneys, parapet walls, exterior cladding, and external mechanical units should be clearly recorded.
- **Interior Views:** Where access is possible, photographs should be taken of the interior structural system, including the layout of columns, shear walls, and beams. In reinforced concrete buildings, particular emphasis should be placed

on column–beam joints, short columns, and regions where soft-story behavior may occur.

- Sketch and Location: A sketch placed adjacent to the photograph area, showing the building’s position and orientation within the street, increases the reliability and clarity of the assessment.



Figure 2.4: Illustrates an example layout of suitable photographs taken for a typical building.

2.1.2.4 Occupancy and Use Information

The FEMA P-154 rapid screening method incorporates not only structural characteristics but also the occupancy type and occupancy level of the building into the evaluation process. The use of a structure is directly related to the consequences of an earthquake, and this information is especially critical for buildings that must be prioritized in terms of life safety.

On the data collection form, the occupancy type of the building is marked using one of the following classifications:

- Residential
- Commercial
- Educational
- Emergency Services
- Industrial
- Office
- Assembly (such as auditoriums, cinemas, and other gathering spaces)
- Warehouse

- Public Utility

In addition, some buildings may be classified as “Critical Facility”. This group includes structures such as hospitals, fire stations, police stations, and disaster coordination centers, which are required to remain operational even in the event of damage. For such buildings, it is assumed that they must maintain functionality under all circumstances (**Figure 2.5**).

The occupancy condition of the building is evaluated according to whether it is actively inhabited or used. The following questions are particularly important:

- Is the building currently in use or abandoned?
- What is the occupancy level? (fully occupied, partially occupied, vacant)
- Is the use temporary or permanent?
- Are people present inside the building for most of the day?

Occupancy:	Assembly	Commercial	Emer. Services	☐ Historic	☐ Shelter
	Industrial	Office	School	☐ Government	
	Utility	Warehouse	Residential, # Units: _____		

Figure 2.5: Presents in detail the occupancy and use classification employed within the FEMA P-154 framework.

2.1.2.5 Evaluation of Site Class and Geological Hazards

In assessing the effects of earthquakes on structures, not only the characteristics of the building itself but also the properties of the supporting soil are decisive. For this reason, the evaluation of site class and potential geological hazards during pre-field preparations and on-site investigations is of great importance within the FEMA P-154 method (**Figure 2.6**).

Selection of Site Class

In FEMA P-154, if the site class is not known, Site Class D is assumed by default. However, this assumption may be revised based on local geotechnical investigation reports, geological maps, and data from local authorities for the region where the building is located. The site class is one of the score modifiers that directly affects the performance score of the building.

The general site class categories are as follows:

- A: Hard rock
- B: Rock
- C: Very dense soil and soft rock
- D: Stiff soil profile (soft soil)
- E: Soft soil profile (very soft soil)
- F: Special soils (e.g., liquefiable soils, peat, etc.)

Soil Type:	☐ A Hard Rock	☐ B Avg Rock	☐ C Dense Soil	☐ D Stiff Soil	☐ E Soft Soil	☐ F Poor Soil	DNK <i>If DNK, assume Type D.</i>
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Figure 2.6: Typical site classes and their levels of influence.

Geological Hazards

In addition to site class, certain distinct geological risks can threaten structural safety in some regions (**Figure 2.7**). FEMA P-154 considers the following hazards:

- Liquefaction Potential: If the soil is saturated and consists of loose granular material, it may lose its bearing capacity during shaking.
- Landslide Risk: On sloping terrain, ground movement may threaten the stability of the structure.
- Surface Fault Rupture: The case in which the building is located directly on an active fault trace.
- Settlement / Ground Subsidence: A risk particularly associated with loose fill or compressible soils.

The presence of these hazards, where observed in the field, should be marked under the “Other Hazards” section of the data collection form. Identification of such hazards may trigger a recommendation for detailed structural evaluation of the building.

Geologic Hazards: Liquefaction: Yes/No/DNK Landslide: Yes/No/DNK Surf. Rupt.: Yes/No/DNK

Figure 2.7 : Geological hazards defined in FEMA and criteria for field identification.

2.1.2.6 Identification of Structural Irregularities

One of the most important parameters directly affecting the performance of buildings under earthquake loading is the regularity and continuity of the structural system. The

FEMA P-154 rapid screening method places substantial emphasis on the identification of structural irregularities based on field observations. It is well known that vertical and horizontal discontinuities may cause damage to concentrate in certain regions of a building and, in some cases, lead to partial or total collapse. For this reason, irregularities that can be visually identified are systematically examined in the screening form and a score modifier is applied for each.

The principal types of structural irregularities considered in the FEMA P-154 procedure are presented below, and each is discussed in detail under its own subheading.

Pounding

Pounding refers to damage caused by the impact of adjacent buildings during an earthquake. When buildings with different dynamic properties oscillate out of phase during seismic shaking, serious structural damage can occur, particularly if the separation distance between them is insufficient. If pounding occurs at approximately mid-height of a column, it introduces an additional risk. Such interactions may lead to cracking, crushing, localized collapse, and even complete failure of load-bearing elements.

Within the FEMA P-154 screening method, the relative positions, height differences, and separation distances of neighboring buildings are visually examined. If there is insufficient separation between two structures and one or more of the following criteria are met, pounding risk is considered present and the relevant box on the screening form is checked:

- Misalignment of floor levels: If the floor levels between adjacent buildings are different and the mass distribution is not aligned (for example, one structure is four stories and the other six stories),
- Proximity of buildings with different heights: A taller building may overturn or impact a shorter neighboring structure,
- Location at the end of a building row: In a continuous row of attached buildings, those located at the ends of the row are generally more vulnerable.

FEMA defines recommended minimum separation distances per story according to the level of seismicity, as presented in **Table 2.1**.

Table 2.1: Minimum separation distance by seismicity level.

Seismicity Level	Recommended Minimum Separation (inches per story)
Very High	2.0 inches
High	1.5 inches
Moderate	1.0 inch
Low	0.5 inch

For buildings where pounding risk is identified, the relevant section of the FEMA form is marked, an explanatory note is added, and the “Detailed Structural Evaluation Required” box is checked. Furthermore, if there are any preliminary signs of impact damage on the façades or columns where pounding is expected, these should be noted separately and documented with photographs.

Vertical Irregularities

Vertical irregularities arise when the mass, stiffness, or load-carrying capacity of a building changes abruptly along its height. Under seismic loading, this leads to disproportionately large displacements and stresses at certain stories compared to others, thereby increasing the likelihood of severe damage or collapse at those levels (**Figure 2.8**).

The FEMA P-154 screening system classifies vertical irregularities into two categories according to severity:

- Severe Vertical Irregularities: Conditions that significantly affect structural performance, such as ground stories with substantial stiffness reductions or excessive story height differences.
- Moderate Vertical Irregularities: Less pronounced but still important height differences or intermediate stiffness transitions that require attention.

Common Types of Vertical Irregularities:

- Ground floor left open for commercial use while upper floors are residential: On the ground floor, the number of load-bearing elements or partition walls may be reduced, and when combined with soft/weak-story effects, this leads to vertical irregularity.

- Significant differences in story height: For example, a 5 m ground story with 3 m upper stories can adversely affect vertical load transfer and the building's response to lateral seismic forces.
- Sudden changes in stiffness: When exterior walls are removed at the ground floor but retained at upper levels, or when the cross-sectional dimensions of columns and shear walls are significantly reduced at a higher story, large stiffness variations between stories occur.

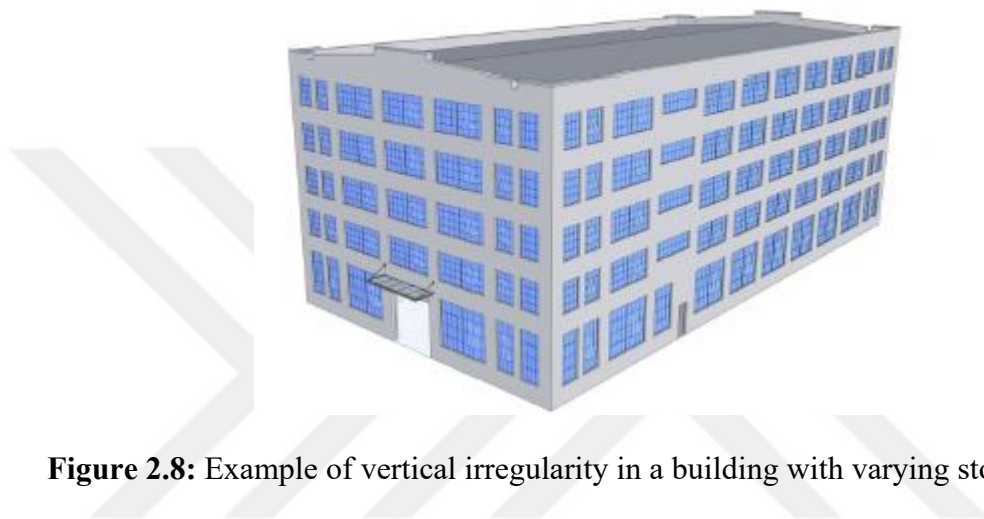


Figure 2.8: Example of vertical irregularity in a building with varying story heights.

Vertical Irregularity Check in FEMA P-154

During the screening, the following questions are assessed visually:

- Are story heights similar across all levels?
- Do vertical load-bearing elements (columns, shear walls) remain continuous through all stories?
- Are there significant openings or stiffness reductions at the ground floor?

If these questions are answered negatively, the “Vertical Irregularities” box on the form is checked and the severity (Severe / Moderate) is specified. Relevant photographic documentation and observation notes are also attached to the form.

Sloped Site

Buildings located on sloping ground exhibit much more complex dynamic interaction with the supporting soil than those on level terrain. In particular, where there are no stiff basement walls and one side of the building is almost completely exposed,

significant stiffness imbalances can arise in certain regions of the structure. Within FEMA P-154, such conditions are treated as a type of vertical irregularity (**Figure 2.9**).



Figure 2.9: Building located on a sloped site.

Risks Associated with Sloped Sites:

- When significant elevation differences exist between story levels, stress concentrations may occur at transitions between stiff and soft zones.
- During an earthquake, the portion of the building on the uphill side generally behaves more stiffly, while the downhill side is relatively more flexible and subjected to larger displacements.
- The risk of short-column behavior is elevated because some columns may become effectively shorter and stiffer than others.

Acceptable Slope Limits According to FEMA

According to FEMA P-154:

- For W1 (light wood-frame) buildings, if the elevation difference across the length of the building reaches one full story, the condition should be classified as a severe vertical irregularity.
- For other structural types, if such slopes exist but stiff basement walls are present, the condition may be classified as a moderate vertical irregularity.

Screening Form Actions:

- If a sloped site is identified, this condition is indicated in the “Vertical Irregularities” section by checking the appropriate box.

- It is also recommended to record the condition under “Other Hazards”, accompanied by a brief note and photographs.

Weak and/or Soft Story

During earthquakes, structural damage often concentrates at stories that are weak or flexible in terms of strength and stiffness. This situation is especially common at ground floors and significantly increases the risk of collapse. FEMA P-154 defines these behaviors as “weak story” and “soft story” irregularities (**Figure 2.10**).

Definitions:

Weak Story: A story where the load-bearing elements have lower strength compared to those of the story above.

Soft Story: A story where the lateral stiffness of the structural system is lower than that of the story above. This condition is common at ground floors used as commercial spaces or open parking areas.

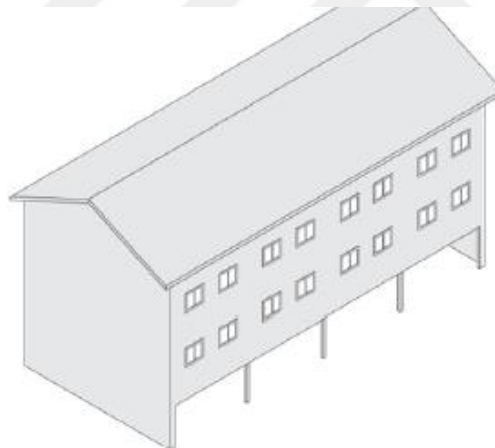


Figure 2.10: Example of a building with a soft-story irregularity.

Screening in FEMA P-154:

- During observation, if the ground story contains large openings, lacks load-bearing walls, includes wide garage openings, or has extensive glass façades, the story may be considered weak and/or soft.
- Significant differences in story height, flat-slab systems without beams, or wide open-shopfronts at the ground floor also fall into this category.

FEMA Evaluation:

- For light wood-frame structures (W1), this irregularity is considered particularly hazardous.
- If the ground story serves as an occupied level and carries the full superstructure above, the condition should be classified as a “severe vertical irregularity”.

Screening Form Notes:

- The relevant boxes under “Vertical Irregularities” are checked.
- Observation notes and explanations are recorded in the “Comments” section.

Out-of-Plane Setback

This type of irregularity occurs when vertical load-bearing elements (columns, shear walls, walls) in the upper stories are not aligned in the same vertical plane with those in the story below. Architectural features such as set-back upper stories or cantilevered floor slabs can cause this condition (**Figure 2.11**).

Definition and Causes:

- When continuity of vertical load-bearing elements at lower levels is not maintained, irregularities in load transfer occur during an earthquake.
- High displacement and rotation demands can lead to the accumulation of shear and flexural damage in certain structural components.
- If the upper floor slab projects beyond or is recessed relative to the wall or column below, the slab may act as a cantilever.

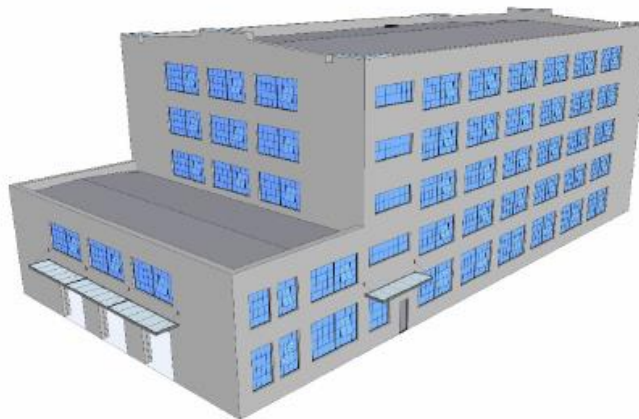


Figure 2.11: Example of a building with an out-of-plane setback at upper stories.

Application in FEMA P-154:

- If vertical structural elements are misaligned in elevation, this irregularity is classified as a severe vertical irregularity.
- Such conditions are considered a significant performance deficiency in FEMA P-154 and lead to a reduction of the score.

Observation Criteria:

- From visual inspection of the building's façades, it should be evaluated whether cantilevered slabs, balconies, or projections disrupt the continuity of the load-bearing system.
- Changes in use, particularly at commercial stories, may also introduce this irregularity.

Screening Form Notes:

- Observations are recorded by checking the "Out-of-Plane Setback" box under "Vertical Irregularities".
- Any additional details are documented in the comments section.

In-Plane Setback

In-plane setback refers to the lack of continuity of load-bearing elements in the plan, where vertical elements in upper stories (such as shear walls or columns) are not aligned with those in the stories below. This disrupts the continuity of the structural system and adversely affects the uniform transfer of lateral loads (**Figure 2.12**).

Typical Conditions:

- Load-bearing elements in upper stories may have been removed or relocated at the ground floor.
- Corner shear walls may exist in upper stories but be absent at the ground level.
- Such modifications, often made for architectural, commercial, or openness requirements, can disturb diaphragm action.

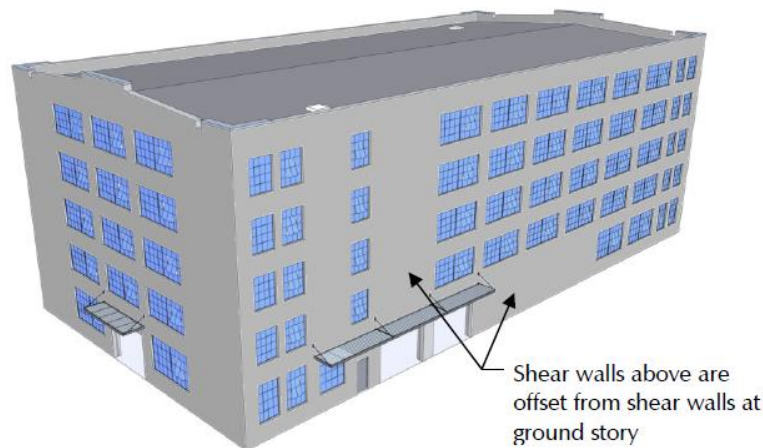


Figure 2.12: Example of an in-plane setback.

Evaluation in FEMA P-154:

- In-plane setbacks are classified as moderate vertical irregularities.
- They are treated as negative score modifiers within the scoring system.
- The irregularity is particularly critical in W1, C1, and C2 type buildings.

Observation Clues:

- When examining floor plans, it should be checked whether upper-story shear walls or columns continue down to the ground floor.
- Visible gaps or discontinuities in the structural system, especially at the front or side façades, may indicate this condition.

Screening Form Marking:

- The “In-Plane Setback” option is checked on the form.
- If necessary, further explanation is provided in the comments section.

Short Column Effect

The short column effect is a critical type of irregularity that occurs when the effective height of a column is reduced by adjacent elements, causing it to behave as if it were shorter than its actual height. During an earthquake, such columns are subjected to higher shear forces because of their reduced height and may suffer severe damage. For example, as illustrated in **Figure 2.13**, if 2 out of 8 columns on a façade are identified as short columns, then the short-column ratio on that façade reaches 25%. According to FEMA P-154, if 20% or more of the vertical load-bearing elements on a given

façade behave as short columns, the building is considered to have a short-column irregularity. In Level 2 evaluation, this condition is treated as a negative score modifier and typically results in an approximate reduction of 0.7 in the score.

Formation of the Short Column Effect:

- If a portion of a column is partially infilled by walls, windows, or parapets, its effective length is reduced.
- Only the exposed portion of the column is then free to deform laterally.
- Mezzanine floors, partial infill walls, or intermediate slab levels often trigger the short column effect.

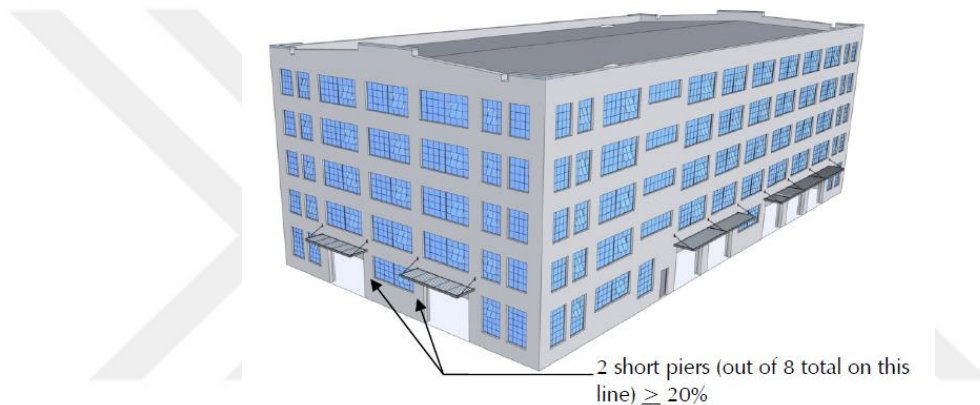


Figure 2.13: Examples of conditions that lead to the short column effect.

Evaluation in FEMA P-154:

- The short column effect is classified as a severe vertical irregularity.
- When visually detected, negative score modifiers are applied.
- It is particularly critical for reinforced concrete frame systems such as C1, C2, PC1, and PC2.

Observation Criteria:

- Special attention should be paid to regions where column height is significantly reduced (for example, below window openings).
- If part of a column is covered by a slab edge or an infill wall, the presence of the short column effect should be assumed.
- Diagonal cracking observed in columns of earthquake-damaged buildings is a typical sign of the short column effect.

Screening Form Marking:

- The “Short Column Effect” box is checked on the form.
- All observed short columns are photographed and recorded in the “Comments” section.

Split Level

A split-level condition arises when different parts of a building have floors at different elevations. This disrupts the continuity of the floor diaphragm and can hinder the proper transfer of seismic forces (**Figure 2.14**).

Definition and Importance:

- The presence of multiple floor levels at different elevations within the same structure introduces discontinuities in lateral load paths.
- The effective transfer of lateral forces to the foundation through shear walls or columns is impeded.
- This condition leads to additional demands on systems that rely on diaphragm behavior and causes abrupt changes in stiffness.
- It can also contribute to the development of short-column effects in some structural members.

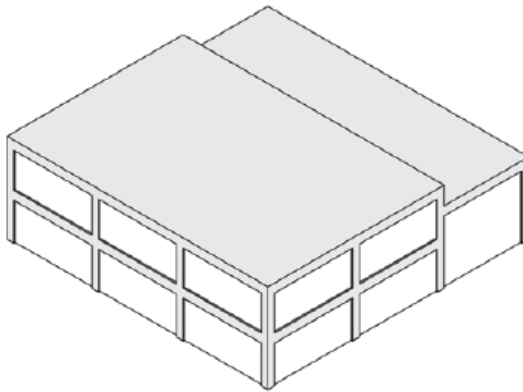


Figure 2.14: Example of a building with a split-level configuration.

Evaluation in FEMA P-154:

- Split-level configuration is treated as a vertical irregularity.
- It is considered to have a moderately adverse influence and leads to a reduction in the building’s score.

- According to FEMA P-154, light wood-frame (W1) structures are particularly vulnerable to this irregularity.

Observation Criteria:

- Differences in floor elevation levels should be carefully checked.
- The presence of internal areas connected by short ramps or stairways indicates an increased likelihood of split-level conditions.
- If the continuity of the diaphragm is interrupted, load transfer paths are disturbed and the seismic performance of the structure is reduced.

Screening Form Marking:

- The “Split Level” box is checked on the screening form.
- The split-level condition is explained and documented with photographs.

Other Vertical Irregularities

Beyond the standard vertical irregularities defined in FEMA P-154, there are certain special conditions that can adversely affect structural performance. Although such irregularities do not occur in every building, they must be carefully evaluated when observed.

Definition and Importance:

- Discontinuities in the vertical load-bearing system, abrupt changes in stiffness distribution, mass irregularities, or unexpected structural details are examined under this heading.
- These irregularities can cause seismic energy to concentrate in localized regions, leading to local damage or even collapse.
- They are often introduced by architectural preferences or subsequent renovations.

FEMA P-154 Approach:

- These irregularities are recorded under the heading “Other Vertical Irregularities” on the screening form.
- The evaluating engineer notes any features that may affect performance but are not explicitly included in the standard irregularity categories.

- This section particularly relies on the judgment of an experienced engineer, as FEMA does not define a fixed list for such conditions.

Example Cases:

- Overloaded individual columns,
- Asymmetric roof masses,
- Partition walls added in a manner inconsistent with the load-bearing system,
- Sudden increases in story height or large openings concentrated at a single level.

Screening Form Marking:

- Any vertical irregularities observed in the field that are not explicitly listed in FEMA's classification are documented in this section.
- They are supported with brief explanations and, where possible, photographs or sketches.

Note: This heading provides a flexible means of incorporating experience-based engineering judgment into the FEMA scoring system. The observed condition and its expected impact on building performance should be clearly described.

2.1.2.7 Plan Irregularities

Plan irregularities arise when a building is not designed in a symmetric and regular manner in the horizontal plane. Such irregularities may cause torsional response of the structure or concentration of stresses in certain regions under lateral loading. FEMA P-154 defines several basic criteria and visual indicators for the identification of these irregularities. Buildings with E-, L-, T-, U-, or plus-shaped plans, excessive openings, or non-orthogonal load-bearing systems are generally classified as having plan irregularities.

Torsional Irregularity

Torsional irregularity occurs when load-bearing elements (such as reinforced concrete shear walls or frames) are not symmetrically arranged with respect to the center of mass of the building. This asymmetric configuration introduces torsional effects during an earthquake, causing some columns and beams to experience greater demands than expected (**Figure 2.15**).

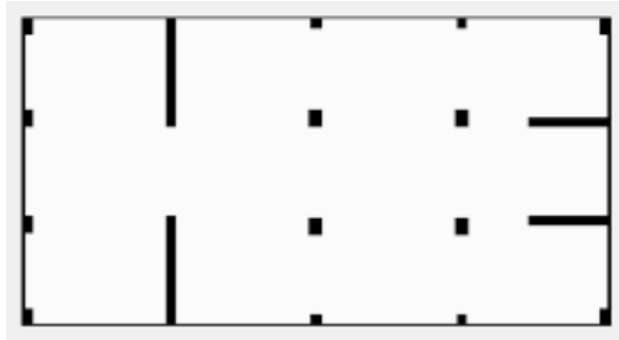


Figure 2.15: Example of torsional irregularity caused by asymmetrically placed shear walls.

Non-Orthogonal Systems

FEMA P-154 defines systems in which load-bearing elements do not intersect at right angles as “non-orthogonal systems”. When the connection angles between structural elements in plan deviate from 90 degrees, uncertainties in load transfer and torsional effects are amplified (**Figure 2.16**).



Figure 2.16: Example of a non-orthogonal building layout.

Non-Uniform Floor Plans / Re-entrant Corners

When a floor plan has E, L, T, U, or plus shapes and the re-entrant projections at interior corners exceed 6 m (20 feet), stress concentrations and localized torsional effects frequently occur. Such buildings can exhibit both plan and vertical irregularities (**Figure 2.17**).

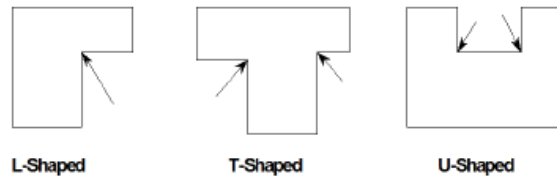


Figure 2.17: L- and T-shaped building plans with re-entrant corners.

Diaphragm Discontinuity

If there are large openings or weak connection regions within floor diaphragms, the ability of the diaphragm to transfer lateral loads is reduced. Large atrium openings or vertical shafts within the floor are typical causes of diaphragm discontinuity (**Figure 2.18**).

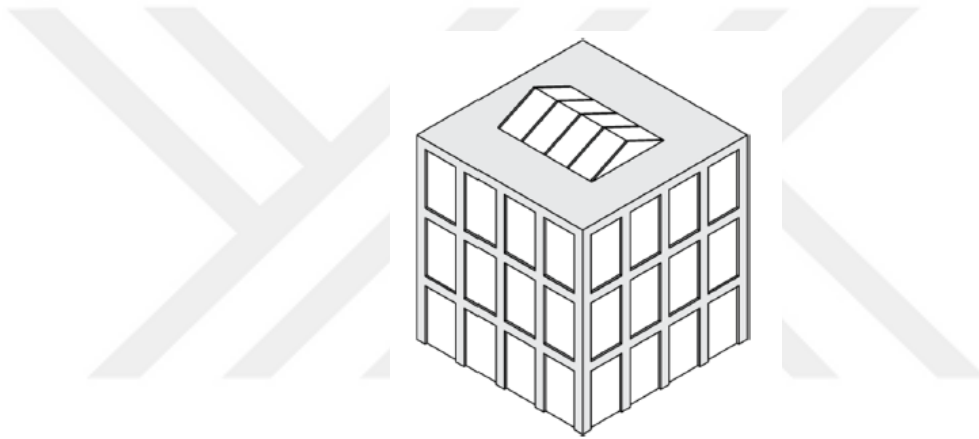


Figure 2.18: Example of a building with diaphragm discontinuity.

Out-of-Plane Offset of Columns and Beams

When beams in plan are not aligned with the axes of columns, abrupt shear forces may develop at beam–column joints. This increases the risk of cracking or local failure in the joint region (**Figure 2.19**).

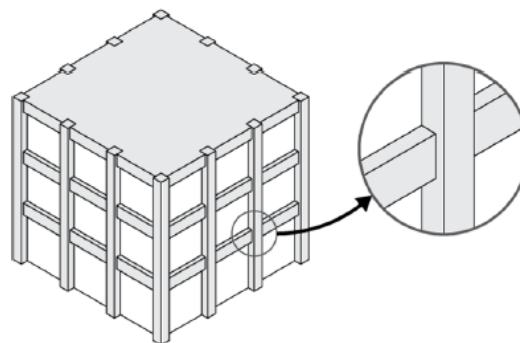


Figure 2.19: Example of beam–column misalignment.

2.1.2.8 Unusual Conditions and Hazards

Within the FEMA P-154 method, attention must be paid not only to the structural system of buildings but also to conditions related to the building environment and its history. Certain hazards may pose risks to life safety and necessitate detailed evaluation, even if they do not directly affect the structural response.

The situations evaluated under this heading are generally listed near the bottom of the Level 1 screening form and are recorded based on the observations of the evaluator.

Risk of Falling or Overturning Nonstructural Elements

Nonstructural components such as chimneys, parapet walls, exterior cladding, and large billboards may easily overturn during an earthquake. If these elements are loose, cracked, or inadequately anchored, they can cause serious casualties (**Figure 2.20**).



Figure 2.20: Solar panels with a risk of falling during an earthquake.

Damage from Previous Earthquakes

If signs of previous earthquake damage are observed—such as column cracking, exposed reinforcement, deflection or crushing in vertical load-bearing members—the performance of the building must be questioned and a detailed assessment should be recommended.

Abandoned or Poorly Maintained Buildings

Buildings that have been unused for a long time or have not been adequately maintained may exhibit deterioration such as concrete carbonation, reinforcement corrosion, moisture damage, or loss of cross-section in reinforcement. These conditions directly reduce the structural capacity.

Foundation Distress and Ground Settlement

If ground settlement, separation of the foundation from the soil, or evidence of slope instability is observed around the building, such conditions may lead to unstable behavior under seismic loading.

Fire or Impact Damage

In buildings that have experienced fire or impact to the structural system (for example, vehicle collision), microscopic or visible damage to the concrete and reinforcement may exist. These defects reduce the seismic capacity of the structure.

Geological Risks

If hazards related to the site, such as landslides, liquefaction, or surface faulting, are observed, the building is directly exposed to significant risk. In such cases, the relevant box on the form should be checked and a detailed analysis recommended (**Figure 2.21**).

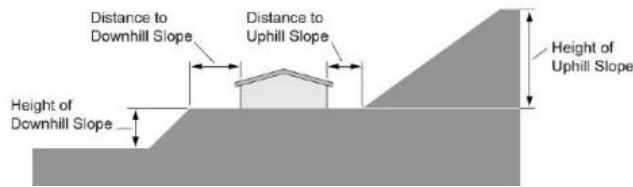


Figure 2.21: Building located on a slope under landslide risk.

2.1.2.9 Identification of Structural System and Material Type

In the FEMA P-154 rapid screening method, the structural system type and the material from which this system is constructed are among the most critical factors determining the seismic performance of a building. At this stage of the Level 1 screening, the evaluator is responsible for classifying the building according to both its structural system and material type.

FEMA Building Types and Classification Criteria

In FEMA P-154, buildings are grouped into 17 main categories based on structural system and construction material. This classification provides a general indication of the expected performance and is used in the calculation of the Basic Score (see **Table 2.2**).

Table 2.2: Classification and abbreviations of FEMA P-154 building types.

No	FEMA Type Code	Description
1	W1	Light wood-frame building (single- or multi-family, low-rise)
2	W1A	Multi-story, multi-unit light wood-frame building
3	W2	Heavy wood-frame commercial/industrial structures (>5000 ft ²)
4	S1	Steel moment frame
5	S2	Steel braced frame
6	S3	Light metal structure
7	S4	Steel frame with cast-in-place reinforced concrete shear walls
8	S5	Steel frame with unreinforced masonry infill walls
9	C1	Reinforced concrete moment frame
10	C2	Reinforced concrete shear wall building
11	C3	Reinforced concrete frame with unreinforced masonry infill walls
12	PC1	Tilt-up concrete building
13	PC2	Precast concrete frame
14	RM1	Reinforced masonry with rigid diaphragms
15	RM2	Reinforced masonry with flexible diaphragms
16	URM	Unreinforced masonry bearing wall buildings
17	MH	Manufactured housing

For more detailed classification, see FEMA P-154, Table 3-1.

Identification of Structural System

Accurate identification of the structural system is very important for determining the Basic Score. The system may be one of the following:

- Moment frame
- Shear wall system
- Steel braced frame
- Dual or mixed system

This classification is made using architectural plans, field observations, and, where available, original design documents.

Identification of Construction Material

Material identification is based on visual observations. Reinforced concrete, steel, timber, and masonry structures have different material strengths and characteristics, which directly influence the risk score. The following types are the most common:

- Reinforced concrete: Columns, beams, and slabs consist of concrete with internal reinforcement.
- Steel: The load-bearing system consists of structural steel members.
- Timber: Typically observed in low-rise residential buildings.
- Masonry: Load-bearing walls constructed of brick or stone.

Field Observation Clues

When identifying the structural system and material type, the following clues should be considered:

- Presence or absence of column traces beneath exterior cladding,
- Beam depths and spans,
- Regularity or irregularity of the structural system,
- Thickness and bonding pattern of brick or stone walls.

2.1.2.10 Identification of Score Modifiers

In the FEMA P-154 method, the Basic Score of a building is a fixed value determined according to its structural type and the seismicity level of the region. However, each building may possess specific favorable or unfavorable characteristics. For this reason,

the Basic Score must be adjusted to reflect certain building-specific features. These features are referred to as score modifiers in the FEMA literature (see **Table 2.3**).

Purpose of Score Modifiers

Score modifiers reflect the presence of conditions that may positively or negatively affect the expected seismic performance of the building. Through these modifiers, each structure is subjected to a more refined risk evaluation that considers local site conditions and its particular structural characteristics.

Table 2.3: FEMA score modifiers and their effects.

Category	Type of Effect	Description
Vertical irregularities	Negative	If the building exhibits severe or moderate vertical discontinuities, a negative modifier is applied.
Plan irregularities	Negative	Torsion, L-shaped plans, etc., are penalized with negative modifiers.
Pre-code design	Negative	A negative modifier is applied to buildings designed before the enforcement of modern seismic codes.
Post-code design	Positive	A positive modifier may be assigned to buildings designed under modern codes.
Site class (A, B, E)	$\pm$	Positive modifiers are applied for rock or stiff soil (A–B), while negative modifiers are applied for soft soils (e.g., E).

Application Procedure

1. The Basic Score is determined according to the FEMA building type.
2. The evaluator records all irregularities and special conditions observed in the field.
3. The appropriate score modifiers are selected (circled) for each observation.

4. The sum of all score modifiers is added to or subtracted from the Basic Score.
5. The result is the Final Level 1 Score (SL1).

Notes:

- If multiple irregularities of the same type are present, only the most unfavorable one is considered.
- Some score modifiers may overlap; in such cases, only one is applied.
- If the total score becomes negative, a minimum score (SMIN) is used.

2.1.2.11 Calculation of Final Level 1 Score (SL1)

Within FEMA P-154, the Final Level 1 Score (SL1) obtained from rapid screening of a building is calculated by adding the Basic Score and the total of all score modifiers. However, this calculation must follow certain rules and bounds.

Calculation of the Basic Score

The Basic Score defined by FEMA is determined based on the structural system and material type of the building, taking into account the seismicity level of the evaluation region. Each structural type and region is associated with a specific Basic Score. (see **Table 2.4**).

Table 2.4: Basic Scores by FEMA building type.

Building Type Code	Description	Basic Score
W1	Light wood-frame building	3.6
W1A	Multi-unit light wood-frame building	3.2
W2	Heavy wood-frame building	2.9
S1	Steel moment frame (MRF)	2.1
S2	Steel braced frame (BR)	2.0

Table 2.4: Continued.

Building Type Code	Description	Basic Score
S3	Light metal building (LM)	2.6
S4	Steel frame with reinforced concrete shear walls (RC SW)	2.0
S5	Steel frame with unreinforced masonry infill (URM INF)	1.7
C1	Reinforced concrete moment frame (MRF)	1.5
C2	Reinforced concrete shear wall building (SW)	2.0
C3	Reinforced concrete frame with unreinforced masonry infill (URM INF)	1.2
PC1	Tilt-up concrete building (TU)	1.6
PC2	Precast concrete frame	1.4
RM1	Reinforced masonry with rigid diaphragms (FD)	1.7
RM2	Reinforced masonry with flexible diaphragms (RD)	1.7
URM	Unreinforced masonry building	1.0
MH	Manufactured housing	1.5

Adding Score Modifiers

Score modifiers determined from field observations are added to or subtracted from the Basic Score. These modifiers account for vertical or plan irregularities, site class, code level, and similar factors (**Figure 2.22**).

The calculation can be expressed in equation (2.1) as:

$$SL1 = \text{Basic Score} + \Sigma(\text{Score Modifiers}) \quad (2.1)$$

BASIC SCORE, MODIFIERS, AND FINAL LEVEL 1 SCORE, S_{L1}																		
FEMA BUILDING TYPE	Do Not Know	W1	W1A	W2	S1 (URF)	S2 (BR)	S3 (LM)	S4 (RC SW)	S5 (URM INF)	C1 (URF)	C2 (SW)	C3 (URM INF)	PC1 (TU)	PC2	RM1 (FD)	RM2 (RD)	URM	MH
Basic Score		3.6	3.2	2.9	2.1	2.0	2.6	2.0	1.7	1.5	2.0	1.2	1.6	1.4	1.7	1.7	1.0	1.5
Severe Vertical Irregularity, V_{L1}		-1.2	-1.2	-1.2	-1.0	-1.0	-1.1	-1.0	-0.8	-0.9	-1.0	-0.7	-1.0	-0.9	-0.9	-0.9	-0.7	NA
Moderate Vertical Irregularity, V_{L1}		-0.7	-0.7	-0.7	-0.6	-0.6	-0.7	-0.6	-0.5	-0.5	-0.6	-0.4	-0.6	-0.5	-0.5	-0.5	-0.4	NA
Plan Irregularity, P_{L1}		-1.1	-1.0	-1.0	-0.8	-0.7	-0.9	-0.7	-0.6	-0.6	-0.8	-0.5	-0.7	-0.6	-0.7	-0.7	-0.4	NA
Pre-Code		-1.1	-1.0	-0.9	-0.6	-0.6	-0.8	-0.6	-0.2	-0.4	-0.7	-0.1	-0.5	-0.3	-0.5	-0.5	0.0	-0.1
Post-Benchmark		1.6	1.9	2.2	1.4	1.4	1.1	1.9	NA	1.9	2.1	NA	2.0	2.4	2.1	2.1	NA	1.2
Soil Type A or B		0.1	0.3	0.5	0.4	0.6	0.1	0.6	0.5	0.4	0.5	0.3	0.6	0.4	0.5	0.5	0.3	0.3
Soil Type E (1-3 stories)		0.2	0.2	0.1	-0.2	-0.4	0.2	-0.1	-0.4	0.0	0.0	-0.2	-0.3	-0.1	-0.1	-0.1	-0.2	-0.4
Soil Type E (> 3 stories)		-0.3	-0.6	-0.9	-0.6	-0.6	NA	-0.6	-0.4	-0.5	-0.7	-0.3	NA	-0.4	-0.5	-0.6	-0.2	NA
Minimum Score, S_{MIN}		1.1	0.9	0.7	0.5	0.5	0.6	0.5	0.5	0.3	0.3	0.3	0.2	0.2	0.3	0.3	0.2	1.0
FINAL LEVEL 1 SCORE, $S_{L1} \geq S_{MIN}$																		

Figure 2.22: Basic Scores and score modifiers by building type.

However, an important lower bound must be respected: the minimum score (SMIN).

Minimum Score Limit (SMIN)

Certain characteristics of a building may result in several negative modifiers and cause the $SL1$ score to fall to zero or even negative values. Such an outcome would be illogical and overly pessimistic. Therefore, FEMA defines a Minimum Score (SMIN) for each building type (**Table 2.5**).

Table 2.5: Application of minimum scores (as shown on the form).

Building Type Code	Minimum Score (SMIN)
W1	1.1
W1A	0.9
W2	0.7
S1 (MRF)	0.5
S2 (BR)	0.6
S3 (LM)	0.6
S4 (RC SW)	0.5
S5 (URM INF)	0.5
C1 (MRF)	0.5
C2 (SW)	0.3
C3 (URM INF)	0.3
PC1 (TU)	0.3
PC2	0.2
RM1 (FD)	0.3
RM2 (RD)	0.3
URM	0.2
MH	1.0

Rule:

- If $SL1 < SMIN$, then $SL1$ is set equal to $SMIN$.

Interpretation of $SL1$

The computed $SL1$ score is compared against the threshold score defined by FEMA, which is 2.0.

- $SL1 \geq 2.0 \rightarrow$ The building is considered capable of providing acceptable seismic performance.
- $SL1 < 2.0 \rightarrow$ The building is considered at risk and Level 2 screening or detailed analysis is recommended.

2.1.2.12 Summary and Evaluation

The Level 1 rapid visual screening process conducted under FEMA P-154 is a low-cost preliminary evaluation method that can be completed in a short time. Within this method, numerous parameters such as architectural and structural characteristics, site conditions, irregularities, and material conditions are taken into account to calculate the Final Level 1 Score ($SL1$). This score provides a quick and practical indication of the probable seismic behavior of the building.

The evaluation results can be summarized as follows:

- $SL1 > \text{cut-off (threshold value)}$: The building is considered relatively safe from a seismic standpoint. Detailed analysis is not mandatory but may be carried out if requested.
- $SL1 \leq \text{cut-off}$: The building is considered seismically vulnerable and requires detailed structural performance assessment.

Factors that significantly influence the $SL1$ score (such as weak or soft stories, short-column effects, plan irregularities, structural type, and site class) must be evaluated with great care. Misclassification or neglect of critical conditions during the assessment may directly compromise the reliability of the results.

The outcomes obtained at this stage are used solely for prioritization purposes. The rapid procedure provided by FEMA P-154 serves as a filtering tool for large building stocks containing thousands of structures and plays a crucial role in deciding which buildings should be subjected to detailed engineering analysis.

2.1.3 Second Stage: Level 2 Screening Method

For buildings that obtain scores close to the threshold values within the scope of Level 1, or that draw attention due to their structural irregularities, a more detailed evaluation may be required. In such cases, the Level 2 Screening Method defined by FEMA P-154 is employed. Level 2 should only be carried out by trained engineers who are experienced in seismic risk assessment. This stage has been developed to collect more in-depth information about the building and to define the risks, including nonstructural components, in a more precise manner. Although the inspection process takes more time, the data obtained provide a much higher level of accuracy in terms of building safety. Within this context, a detailed analysis will be presented under the headings of definition of the Level 2 form, computation of the adjusted baseline score, new score modifiers, and evaluation of nonstructural hazards.

2.1.3.1 Definition of the Level 2 Form

The Level 2 evaluation form defined in FEMA P-154 is intended to enable a more detailed investigation of buildings that, during the first-stage rapid visual screening process, were found to have insufficient scores or to be potentially at risk. This form may only be completed by engineers who are experts in seismic evaluation or building design. The Level 2 form goes beyond the scoring data obtained in Level 1 and allows for a detailed analysis of the behavior of the load-bearing system, the severity of irregularities, possible past strengthening interventions, and nonstructural hazards.

In the upper part of the form, the building name, the Level 1 score, and any previously applied score modifiers (such as plan or vertical irregularities) are indicated. Based on this information, a series of observations are made about the building in Level 2, and, where necessary, corrective score modifiers (VL2, PL2 and M) are added to deepen the evaluation.

The Level 2 form provides a comprehensive preliminary analysis by considering not only the vertical load-bearing system of the structures, but also their nonstructural elements. In this way, the final decision of “Detailed Structural Evaluation Required / Not Required” is placed on a much sounder technical basis. An example illustration of the FEMA P-154 Level 2 evaluation form is presented below (**Figure 2.23**).

Rapid Visual Screening of Buildings for Potential Seismic Hazards

FEMA P-154 Data Collection Form

Optional Level 2 data collection to be performed by a civil or structural engineering professional, architect, or graduate student with background in seismic evaluation or design of buildings.

Level 2 (Optional)

HIGH Seismicity

Bldg Name: _____

Scriber: _____

Date/Time: _____

Final Level 1 Score: $S_{L1} =$ _____ (do not consider S_{L1})

Level 1 Irregularity Modifiers: Vertical Irregularity, $V_{L1} =$ _____ Plan Irregularity, $P_{L1} =$ _____

ADJUSTED BASELINE SCORE: $S' = (S_{L1} - V_{L1} - P_{L1}) =$ _____

STRUCTURAL MODIFIERS TO ADD TO ADJUSTED BASELINE SCORE					
Topic	Statement (if statement is true, circle the "Yes" modifier; otherwise, cross out the modifier.)	Yes	No	Subtotal	
Vertical Irregularity, V_{L1}	Slipping	W1 building: There is at least a full story grade change from one side of the building to the other.	-1.2		
	Site	Non-W1 building: There is at least a full story grade change from one side of the building to the other.	-0.3		
	Weak and/or Soft Story (circle one maximum)	W1 building cripple wall: An unbraced cripple wall is visible in the crawl space.	-0.6		
		W1 house over garage: Underneath an occupied story, there is a garage opening without a steel moment frame, and there is less than 8' of wall on the same line (for multiple occupied floors above, use 15' of wall minimum).	-1.2		
		WTA building open front: There are openings at the ground story (such as for parking) over at least 50% of the length of the building.	-1.2		
	Setback	Non-W1 building: Length of lateral system at any story is less than 50% of that at story above or height of any story is more than 2.0 times the height of the story above.	-0.9		
		Non-W1 building: Length of lateral system at any story is between 50% and 75% of that at story above or height of any story is between 1.3 and 2.0 times the height of the story above.	-0.5		
		Vertical elements of the lateral system at an upper story are outboard of those at the story below causing the diaphragm to cantilever at the offset.	-1.0		
		Vertical elements of the lateral system at upper stories are inboard of those at lower stories.	-0.5		
		There is an in-plane offset of the lateral elements that is greater than the length of the elements.	-0.3		
Short Column/ Pier	C1, C2, C3, PC1, PC2, RM1, RM2: At least 20% of columns (or piers) along a column line in the lateral system have height/depth ratios less than 50% of the nominal height/depth ratio at that level.	-0.5			
	C1, C2, C3, PC1, PC2, RM1, RM2: The column depth (or pier width) is less than one half of the depth of the spandrel, or there are infill walls or adjacent floors that shorten the column.	-0.5			
	Split Level	There is a split level at one of the floor levels or at the roof.	-0.5		
Plan Irregularity, P_{L1}	Other	There is another observable severe vertical irregularity that obviously affects the building's seismic performance.	-1.0	$V_{L1} =$ _____	
	Irregularity	There is another observable moderate vertical irregularity that may affect the building's seismic performance.	-0.5	(Cap at -1.2)	
	Torsional irregularity: Lateral system does not appear relatively well distributed in plan in either or both directions. (Do not include the WTA open front irregularity listed above.)	-0.7			
	Non-parallel system: There are one or more major vertical elements of the lateral system that are not orthogonal to each other.	-0.4			
	Reentrant corner: Both projections from an interior corner exceed 25% of the overall plan dimension in that direction.	-0.4			
	Diaphragm opening: There is an opening in the diaphragm with a width over 50% of the total diaphragm width at that level.	-0.2			
	C1, C2 building out-of-plane offset: The exterior beams do not align with the columns in plan.	-0.4			
	Other irregularity: There is another observable plan irregularity that obviously affects the building's seismic performance.	-0.7	$P_{L1} =$ _____		
	Redundancy	The building has at least two bays of lateral elements on each side of the building in each direction.	+0.3	(Cap at -1.2)	
	Pounding	Building is separated from an adjacent structure by less than 1% of the height of the shorter of the building and adjacent structure and:			
S2 Building	X: bracing geometry is visible.	-1.0			
	PC1/RM1 Bldg	Flat plate serves as the beam in the moment frame.	-0.4		
	PC1/RM1 Bldg	There are roof-to-wall ties that are visible or known from drawings that do not rely on cross-grain bending. (Do not combine with post-benchmark or retrofit modifier.)	+0.3		
	URM	The building has closely spaced, full height interior walls (rather than an interior space with few walls such as in a warehouse).	+0.3		
	URM	Gable walls are present.	-0.4		
	Mt	There is a supplemental seismic bracing system provided between the carriage and the ground.	+1.2		
	Retrofit	Comprehensive seismic retrofit is visible or known from drawings.	+1.4	$M =$ _____	
	FINAL LEVEL 2 SCORE: $S_{L2} = (S' + V_{L1} + P_{L1} + M) \geq S_{BAC}$ (Transfer to Level 1 form)				
	There is observable damage or deterioration or another condition that negatively affects the building's seismic performance: ☐ Yes ☐ No				
	If yes, describe the condition in the comment box below and indicate on the Level 1 form that detailed evaluation is required independent of the building's score.				

OBSERVABLE NONSTRUCTURAL HAZARDS				
Location	Statement (Check "Yes" or "No")	Yes	No	Comment
Exterior	There is an unbraced unreinforced masonry parapet or unbraced unreinforced masonry chimney.			
	There is heavy cladding or heavy veneer.			
	There is a heavy canopy over exit doors or pedestrian walkways that appears inadequately supported.			
	There is an unreinforced masonry appendage over exit doors or pedestrian walkways.			
	There is a sign posted on the building that indicates hazardous materials are present.			
	There is a taller adjacent building with an unanchored URM wall or unbraced URM parapet or chimney.			
Interior	Other observed exterior nonstructural falling hazard:			
	There are hollow clay tile or brick partitions at any stair or exit corridor.			
Other observed interior nonstructural falling hazard:				
Estimated Nonstructural Seismic Performance (Check appropriate box and transfer to Level 1 form conclusions) ☐ Potential nonstructural hazards with significant threat to occupant life safety → Detailed Nonstructural Evaluation recommended ☐ Nonstructural hazards identified with significant threat to occupant life safety → But no Detailed Nonstructural Evaluation required ☐ Low or no nonstructural hazard threat to occupant life safety → No Detailed Nonstructural Evaluation required				
Comments: _____				

Figure 2.23: Data form for stage 2 of the rapid screening method.

2.1.3.2 Definition of the Level 2 Form

The first step carried out within the scope of the Level 2 evaluation is to obtain the adjusted baseline score S' by removing certain score modifiers from the score calculated on the Level 1 form. This is done by subtracting the sum of the vertical irregularity score modifier ($VL1$) and the plan irregularity score modifier ($PL1$), which were applied in Level 1, from the Level 1 final score ($SL1$). The new score obtained is taken as the starting point of the more detailed Level 2 evaluation. Thus, the same irregularities are prevented from being penalized a second time within Level 2. This calculation is expressed by the following equation (2.2):

$$S' = SL1 - (VL1 + PL1) \quad (2.2)$$

This adjusted baseline score S' forms the basis for the other structural and nonstructural score modifiers (such as $VL2$, $PL2$ and M) that will be added during the Level 2

evaluation. In this way, the information obtained in the previous assessment process is preserved, while the influence of the detailed analyses carried out in the second stage can be monitored more clearly. The S' score constitutes the starting point of the Level 2 analyses in order to obtain a more accurate and balanced evaluation of the building.

2.1.3.3 Vertical Irregularities (VL2)

In the FEMA P-154 Level 2 screening form, under the heading Vertical Irregularities (VL2), distinct deviations in the distribution of stiffness and mass between stories of the building are evaluated. These deviations may cause unexpected local stress concentrations and damage accumulation in the structural response. While the irregularities observed in the Level 1 form are transferred to Level 2, in this section further classifications are made based on more advanced engineering observations, and corresponding score modifiers are applied for each case (**Figure 2.24**).

These irregularities are generally evaluated under the following sub-headings:

1. Sloped Site

Definition: Exists when there is a difference in elevation of one story or more between one side of the building and the other.

Evaluation: Especially for W1 type buildings, structures located on sloping ground may exhibit short-column effects on the lower side and stiffness weakness on the upper side.

Application: If the basement walls are not rigid, a negative score modifier is applied.

2. Weak or Soft Story

Definition: A significant difference in stiffness between the ground story and the upper stories, frequently observed in buildings with open commercial frontages or parking at the ground level.

Evaluation: Inadequate lateral load-bearing elements at the lower story lead to unbalanced displacements along the total height of the building.

Application:

- If both weak and soft story conditions exist, the more unfavorable one is taken as the basis.
- If only one is present, the score modifier appropriate to that condition is selected.

3.Unbraced Cripple Walls (only for W1)

Definition: Presence of unsupported walls between the foundation and the floor at ground level, especially in timber structures.

Evaluation: This may create a collapse risk despite the low building height.

Application: If there is no bracing, a negative score is assigned.

4.Short Column Effect

Definition: Increase in shear forces arising from window openings, parapets, or infill walls that reduce the clear height of the column.

Evaluation: This condition may cause shear failures particularly at the lower stories.

Application: If it is at a critical level, a penalty score is applied.

5.Split Level

Definition: Existence of level differences within the same floor plane due to floors or ground entrances located at different elevations.

Evaluation: Adversely affects diaphragm continuity and load transfer.

Application: A moderate irregularity score is assigned.

6.Other Vertical Irregularities

Definition: Additional deviations that do not fall under the categories above but may affect performance.

Application: Explained in the comments box and an appropriate penalty score is marked.

STRUCTURAL MODIFIERS TO ADD TO ADJUSTED BASELINE SCORE					
Topic	Statement (If statement is true, circle the "Yes" modifier; otherwise cross out the modifier.)			Yes	Subtotals
Vertical Irregularity, V_{L2}	Sloping Site	W1 building: There is at least a full story grade change from one side of the building to the other.		-1.2	
		Non-W1 building: There is at least a full story grade change from one side of the building to the other.		-0.3	
	Weak and/or Soft Story (circle one maximum)	W1 building cripple wall: An unbraced cripple wall is visible in the crawl space.		-0.6	
		W1 house over garage: Underneath an occupied story, there is a garage opening without a steel moment frame, and there is less than 8' of wall on the same line (for multiple occupied floors above, use 16' of wall minimum).		-1.2	
		W1A building open front: There are openings at the ground story (such as for parking) over at least 50% of the length of the building.		-1.2	
		Non-W1 building: Length of lateral system at any story is less than 50% of that at story above or height of any story is more than 2.0 times the height of the story above.		-0.9	
		Non-W1 building: Length of lateral system at any story is between 50% and 75% of that at story above or height of any story is between 1.3 and 2.0 times the height of the story above.		-0.5	
	Setback	Vertical elements of the lateral system at an upper story are outboard of those at the story below causing the diaphragm to cantilever at the offset.		-1.0	
		Vertical elements of the lateral system at upper stories are inboard of those at lower stories.		-0.5	
		There is an in-plane offset of the lateral elements that is greater than the length of the elements.		-0.3	
	Short Column/ Pier	C1,C2,C3,PC1,PC2,RM1,RM2: At least 20% of columns (or piers) along a column line in the lateral system have height/depth ratios less than 50% of the nominal height/depth ratio at that level.		-0.5	
		C1,C2,C3,PC1,PC2,RM1,RM2: The column depth (or pier width) is less than one half of the depth of the spandrel, or there are infill walls or adjacent floors that shorten the column.		-0.5	
	Split Level	There is a split level at one of the floor levels or at the roof.		-0.5	
	Other Irregularity	There is another observable severe vertical irregularity that obviously affects the building's seismic performance.		-1.0	
		There is another observable moderate vertical irregularity that may affect the building's seismic performance.		-0.5	
				$V_{L2} =$ (Cap at -1.2)	

Figure 2.24: Vertical irregularities and related score modifiers.

2.1.3.4 Plan Irregularities (PL2)

In the Level 2 screening stage, the arrangement of the load-bearing system in plan and the layout of its elements are considered as critical factors that directly affect the behavior of the structure under lateral loads. Within the scope of FEMA P-154, when determining plan irregularities (PL2), factors such as symmetry, stiffness distribution, openings, and the locations of centers of mass are taken into account (**Figure 2.25**).

Plan irregularities are examined in detail in the Level 2 form and are classified under the following sub-headings:

1. Torsional Irregularity

- **Definition:** Concentration of rigid elements (e.g., reinforced concrete shear walls) in one direction of the building and their being located far from the center of mass, leading to torsional response.
- **Evaluation:** Asymmetric load distribution during an earthquake results in rotational effects.
- **Application:** A penalty score is applied for severe torsional irregularities.

Note: If there are large openings at the ground story in W1A type buildings, this condition may receive a penalty in both vertical and plan irregularity categories.

2. Non-Orthogonal Systems

- **Definition:** Occurs when the axes of the load-bearing system are not perpendicular to each other.

- Evaluation: Transfer of lateral loads becomes complex.
- Application: A moderate penalty score is applied.

3.Re-Entrant Corners

- Definition: E, L, T, U, and + shaped plans carry risk in terms of stress concentration.
- Evaluation: Local damage is frequently observed particularly at corner regions.
- Application: If the projection length exceeds 25% of the building dimension in that direction, the condition is accepted as an irregularity.

4.Diaphragm Discontinuities

- Definition: Presence of openings larger than 50% of the diaphragm (floor).
- Evaluation: Transfer of lateral loads to vertical elements is weakened.
- Application: A penalty score is applied and the system is assumed to have limited diaphragm action.

5.Out-of-Plane Offset Between Beams and Columns

- Definition: Misalignment between the axes of columns and beams on exterior frames.
- Evaluation: Moment transfer and shear safety at beam-column joints are impaired.
- Application: For this condition, frequently observed in older reinforced concrete buildings, a penalty is applied within the scope of PL2.

6.Other Plan Irregularities

- Definition: Asymmetry in plan or eccentric mass distributions that are observed in the plan but do not fall under the categories above.
- Application: A detailed explanation is written in the observation and comments section and the penalty score is applied together with this comment.

Plan Irregularity, $P_{2.2}$	Torsional irregularity: Lateral system does not appear relatively well distributed in plan in either or both directions. <i>(Do not include the W/A open front irregularity listed above.)</i>	-0.7	$P_{2.2} =$ (Cap at -1.1)
	Non-parallel system: There are one or more major vertical elements of the lateral system that are not orthogonal to each other.	-0.4	
	Reentrant corner: Both projections from an interior corner exceed 25% of the overall plan dimension in that direction.	-0.4	
	Diaphragm opening: There is an opening in the diaphragm with a width over 50% of the total diaphragm width at that level.	-0.2	
	C1, C2 building out-of-plane offset: The exterior beams do not align with the columns in plan.	-0.4	
	Other irregularity: There is another observable plan irregularity that obviously affects the building's seismic performance.	-0.7	

Figure 2.25: Plan irregularities and score modifiers.

2.1.3.5 Other Score Modifiers (M)-1

During the Level 2 screening process, some buildings are subjected to additional score modifiers due to various conditions that are not covered by vertical and plan irregularities but may still influence seismic performance. Such situations are listed in the “Other Structural Modifiers” section of the FEMA P-154 Level 2 Form and are evaluated under the parameter M (**Figure 2.26**).

The main headings related to these score modifiers are explained below:

1.Pounding Potential

- Definition: When there is insufficient separation between two buildings, a risk of collision arises during an earthquake.
- Criterion: Evaluated if SL2 is less than the pounding cut-off value.
- Effect: If there is a severe pounding risk, a significant score reduction is applied.

2.Falling Hazard from a Taller Adjacent Building

- Definition: Presence of chimneys, parapets, or cladding materials on a taller neighboring building that may fall onto the subject building.
- Effect: While such falling hazards affect environmental safety rather than the performance of the building itself, they may still result in a penalty added to SL2.

3.Geological Hazards / Site Class F

- Definition: Natural hazards such as liquefaction, landslides, and surface fault rupture, or the presence of an F-class site.
- Effect: If such conditions exist, the building’s score is reduced and a detailed structural evaluation is recommended.

4.Significant Damage / Deterioration in the Building

- Definition: Visual findings such as corrosion, spalling of concrete cover, cracks, fire damage, or punching shear indications.
- Effect: If such damage points to a systematic deterioration, the building is not considered to be on the safe side.

5.Previous Strengthening Application

- Definition: Visually identifiable interventions such as added shear walls, steel bracings, or carbon fiber wrapping.
- Effect: If these interventions cover the building as a whole, they are taken as a positive score modifier.

6. Building-Type-Specific Conditions

- Examples:
 - S2 type buildings: Penalty if K-bracing is present.
 - C1 type buildings: Penalty due to punching shear risk if flat slab systems are present.
 - URM buildings: Score reduction if gable walls are present due to out-of-plane overturning risk.

Redundancy	The building has at least two bays of lateral elements on each side of the building in each direction.		+0.3	
Pounding	Building is separated from an adjacent structure by less than 1% of the height of the shorter of the building and adjacent structure and:	The floors do not align vertically within 2 feet.	(Cap total	-1.0
		One building is 2 or more stories taller than the other.	pounding	-1.0
		The building is at the end of the block.	modifiers at -1.2)	-0.5
S2 Building	"K" bracing geometry is visible.			-1.0
C1 Building	Flat plate serves as the beam in the moment frame.			-0.4
PC1/RM1 Bldg	There are roof-to-wall ties that are visible or known from drawings that do not rely on cross-grain bending. (Do not combine with post-benchmark or retrofit modifier.)			+0.3
PC1/RM1 Bldg	The building has closely spaced, full height interior walls (rather than an interior space with few walls such as in a warehouse).			+0.3
URM	Gable walls are present.			-0.4
MH	There is a supplemental seismic bracing system provided between the carriage and the ground.			+1.2
Retrofit	Comprehensive seismic retrofit is visible or known from drawings.			+1.4

M = _____

Figure 2.26: Other score modifiers (M) and their effects.

2.1.3.6 Other Score Modifiers (M)-2

Once the Level 2 screening is completed, the final score of the building is calculated as SL2 (Score Level 2). This score is obtained using the adjusted baseline score (S') and the sum of the three main groups of score modifiers identified within the scope of Level 2 in equation (2.3)

$$SL2 = S' + VL2 + PL2 + M \quad (2.3)$$

Where:

- S': The adjusted baseline score obtained by subtracting VL1 and PL1 from the Level 1 score.
- VL2: The score effect of vertical irregularities observed in Level 2.
- PL2: The score effect corresponding to plan irregularities.
- M: The total of modifiers arising from other structural and nonstructural hazards.

Lower and Upper Bounds:

- If SL2 is less than the predefined SMIN value, then SL2 is taken as $SL2 = SMIN$.
- The SMIN value has been determined according to the combination of the most unfavorable building characteristics.

Interpretation and Action Decision:

- If $SL2 \geq$ threshold score, the building is considered acceptable.
- If $SL2 <$ threshold score, a detailed structural analysis is recommended for the building.
- In some cases, the SL2 score may be higher than the SL1 score. This indicates that the detailed evaluation yielded a more favorable result than the initial impression.

2.1.3.7 Other Score Modifiers (M)-3

In the lower part of the FEMA P-154 Level 2 screening form, there is a dedicated section for identifying nonstructural hazards. Although this section does not directly affect the seismic performance of the building, it is of great importance in terms of life safety. Nonstructural hazards may cause injuries, obstruct escape routes, or render the building unusable, especially during major earthquakes (**Figure 2.27**).

The evaluation in this section is generally grouped under the following two main headings:

1.Exterior Falling Hazards

Within this scope, the following elements are evaluated:

- Parapet walls: Stone or reinforced concrete parapets may overturn during an earthquake and fall onto sidewalks or roads.
- Masonry chimneys: Frequently seen in masonry buildings and have a high risk of falling.
- Heavy exterior claddings or plasters: Claddings that are not adequately anchored or that rely on aged adhesives may detach and fall under seismic effects.

- Façade ornaments, railings, sunshades: These elements, generally constructed for decorative purposes, may create hazards due to their non-rigid connections.
- Presence of hazardous materials in the building: If chemical, flammable, or explosive materials are stored in the building, this must be indicated.



Figure 2.27: Examples of exterior nonstructural falling hazards.

2. Interior Falling Hazards

In this part, potential hazards that may directly affect the occupants inside the building are evaluated (**Figure 2.28**). These include:

- Suspended ceiling systems: Ceiling panels or supporting systems with loose or insufficient connections.
- Lighting fixtures and HVAC systems: Heavy fixtures or equipment fixed to the ceiling that may fall.
- Heavy cabinets and shelving systems: Unanchored cabinets that may topple during an earthquake.
- Obstacles on emergency escape routes: Items that may obstruct access to fire stairs or exit doors.



Figure 2.28: Examples of nonstructural hazards observed in interior spaces.

The purpose of these evaluations is to identify nonstructural components that may constitute life-threatening hazards even in a building with high seismic performance. FEMA P-154 emphasizes that these hazards should not be overlooked and recommends that, where necessary, they also be reported and included in the comprehensive evaluation.

2.1.3.8 Final Comments and Documentation

In the Level 2 screening form, there is a dedicated section where the evaluating engineer can provide final comments regarding their observations and assessments. This section is reserved for elaborating special conditions encountered in the field or findings that do not fully correspond to the predefined items on the form.

In this area, it is particularly expected that the following be explained:

- Structural irregularities that have been observed but are not defined as score modifiers,
- Suspected but unverified risks (such as possible strengthening, weak details),
- Architectural or functional characteristics of the building that may affect seismic performance,
- Any finding that may lead to the need for a more advanced structural analysis.

The explanations made in this section not only support the general evaluation regarding the building, but also provide justification for carrying out a detailed analysis when necessary. In particular, cases such as the building having a unique load-bearing

system layout, having been strengthened in the past, or containing hazardous elements outside the load-bearing system should be described in this part.

2.2 Rapid Seismic Screening Method In Canada

The Rapid Seismic Screening Method of Canada has been developed as a flexible and practical assessment approach that is open to regional adaptation, owing to the fact that the country's seismic hazard potential varies across different regions (**Figure 2.29**). This method aims to evaluate the seismic risk of buildings in a rapid and systematic manner. Inspired by U.S.-based procedures such as FEMA P-154, the system has been customized by considering building typologies, material usage, climatic conditions, and the historical development of seismic codes specific to Canada.

The method has been put into use to contribute to the risk analysis of building stock in different provinces, particularly in British Columbia and Quebec. Across Canada's vast geography, seismic hazard constitutes a serious reality along the corridor extending from Vancouver Island to the Ottawa Valley. Within this framework, the screening procedure has been developed with the primary objective of assessing public buildings, educational facilities, and critical infrastructure structures as a priority.

The Canadian Rapid Seismic Screening Method stands out as a fast and cost-effective procedure that can also be applied by non-engineering field personnel. However, it also incorporates a multi-level evaluation system in which detailed engineering analyses are recommended for buildings identified as being at risk. In general terms, the method produces a risk score by taking into account the structural system type, irregularities, building height, construction year, and soil conditions. The score obtained is then compared with defined threshold values to provide an initial insight into the seismic performance of the building.

In this section, the fundamental principles, development process, form structure, and implementation steps of the Canadian Rapid Seismic Screening Method will be discussed in detail, and attention will also be drawn to its similarities and differences with methods such as FEMA.

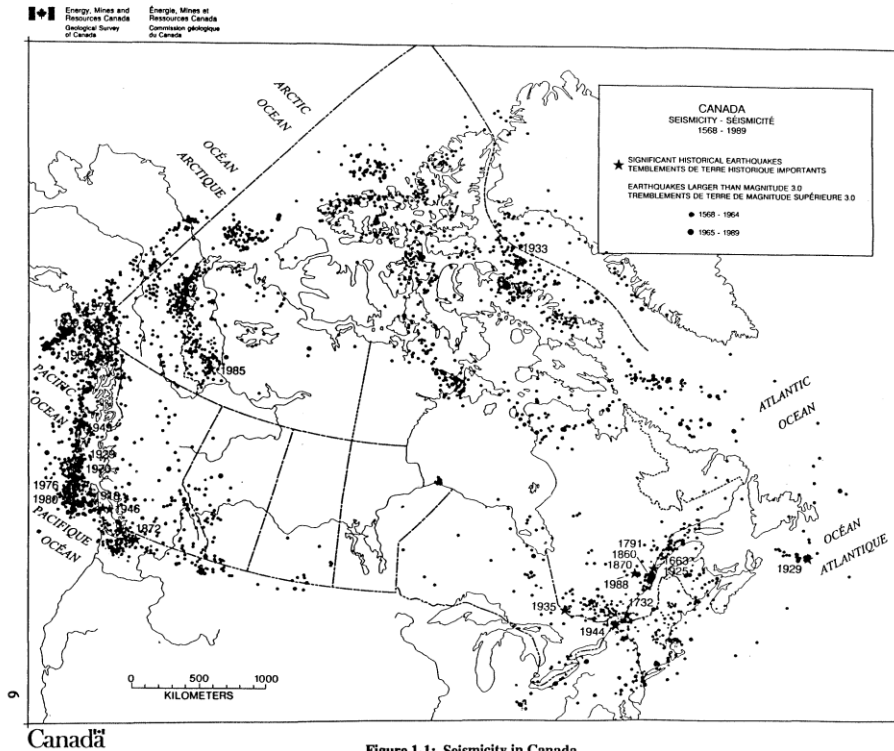


Figure 1-1: Seismicity in Canada

Figure 2.29: Major seismic risk regions in Canada and risk distribution map based on past earthquakes.

2.2.1 Development and Objectives of the Method

Rapid screening methods aimed at evaluating seismic hazard in Canada began to be developed in the 1990s. The main objective of this process has been to ensure that the building stock can be screened in a rapid and economical manner, in a way that can also be implemented by non-engineering personnel, and to identify buildings that require priority intervention. Field studies conducted in different seismic regions of Canada and analyses of past earthquakes have made it necessary for this method to be continuously updated and adapted to regional characteristics.

The Canadian Rapid Seismic Screening Method (Rapid Seismic Screening of Buildings in Canada), although inspired by international procedures such as FEMA P-154, has been structured by taking into account Canadian-specific building types, material properties, construction eras, and the historical evolution of seismic codes. In particular, in versions developed after 2000, the building stock designed and

constructed in accordance with local codes has been considered, thereby improving the accuracy of the method.

The fundamental objective of the method is to rapidly distinguish the seismically vulnerable portion of the existing building stock and to direct these buildings to more detailed evaluation processes. In this way, it becomes possible to use limited resources more effectively. At the same time, the method contributes to emergency planning, prioritization of strengthening interventions, and decision-making processes related to the use of buildings.

2.2.2 General Structure of the Method

The Canadian Rapid Seismic Screening Method is designed to provide a systematic and scientific approach to the seismic assessment of building stock. This method incorporates a multi-layered evaluation model that can be implemented quickly in the field, yet supports the decision-making process with sound engineering principles. Thus, the goal is not only to identify high-risk buildings as a priority, but also to clearly define strengthening and detailed evaluation priorities.

The structure of the method is divided into three main stages: pre-evaluation, collection of field data, and operation of the final decision mechanism. Each stage is designed such that it can be customized according to building typology, soil conditions, level of seismicity, and characteristics of the structural system. This design provides an algorithm that guides not only the evaluation of building performance but also the decision-making process of the practitioner in the field.

All of these processes are represented visually in a flowchart included in the Canadian guideline, which clearly shows that the logic of the assessment is based on sequential steps. Each box in this diagram corresponds to specific parts of the method that will be explained in detail in later sections. In this way, the entire process can be followed at both conceptual and operational levels (**Figure 2.30**).

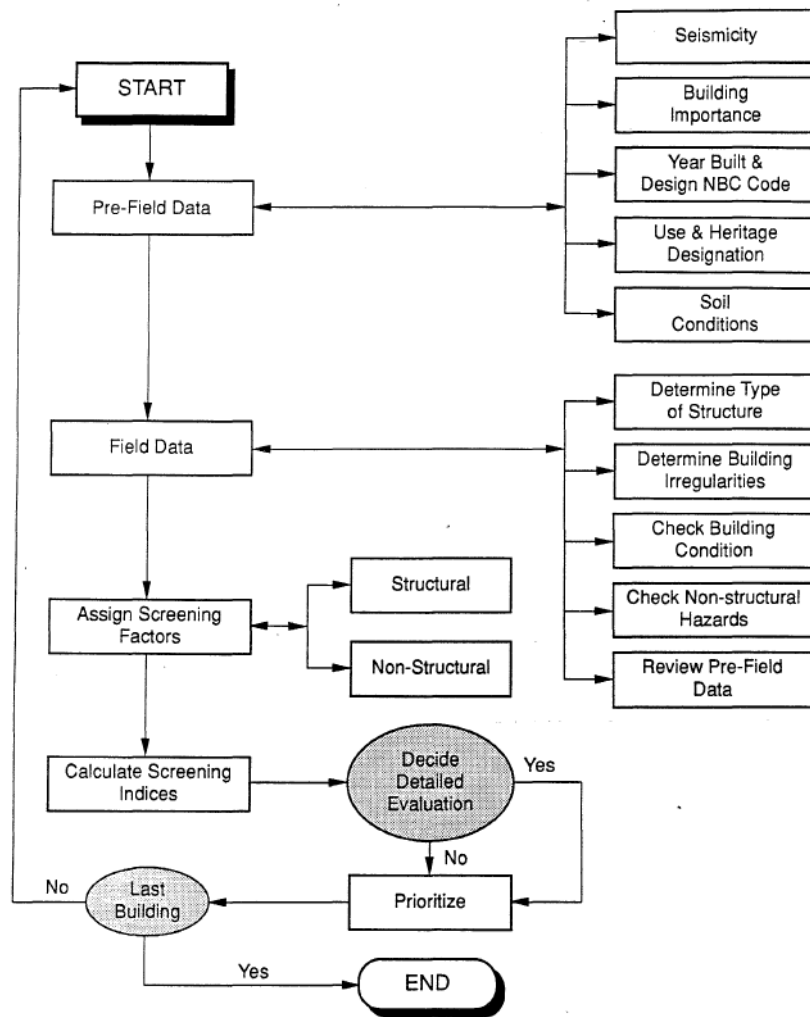


Figure 2.30: Schematic diagram illustrating the general operation of the Canadian rapid seismic screening method.

To enable a systematic implementation of the screening process, a two-page data collection form has been developed within the scope of the Canadian Rapid Seismic Screening Method. On the first page of this form, basic descriptive information regarding the building, the structural type, and observed structural irregularities are recorded; the second page includes numerical assessment fields based on a scoring system. The form is arranged so as to correspond directly to the steps indicated in the flowchart, with the aim of standardizing the screening process and ensuring consistency among practitioners in the field. In this respect, the form enhances the ease of application of the method and allows field observations to be documented in a systematic manner (**Figure 2.31, Figure 2.32**).

SEISMIC SCREENING FORM				p. 1 of 2	ITEM No.:
Address:			Postal Code:		Bldg. Name:
No. of storeys:	Total Floor Area:		m ²	Year Built:	Design NBC:
Primary use (see list on p. 2):				Heritage Designation:	
Inspector:		Date:		Checked by:	
Sketch			Photo		
TYPE OF STRUCTURE (circle appropriate descriptors) see 4.3.2			BM	BUILDING IRREGULARITIES (circle appropriate descriptors) see 4.3.3	
Wood	WLF WPB	Wood Light Frame Wood, Post and Beam	90	1. Vertical Irregularity	Abrupt changes in plan dimensions over height (e.g. setback or building on hill)
Steel	SMF SBF SLF SCW SIW	Steel Moment Frame Steel Braced Frame Steel Light Frame Steel Frame with Concrete Shear Walls Steel Frame with Infill Masonry Shear Walls	90	2. Horizontal Irregularity (Torsion)	Irregular building shapes such as "L", "V", "E", "T", eccentric stiffness in plan (e.g. shear wall on only one side of building)
Concrete	CMF CSW CIW PCF PCW	Concrete Moment Frame Concrete Shear Walls Concrete Frame with Infill Masonry Shear Walls Precast Concrete Frame Precast Concrete Walls	85	3. Short Concrete Columns	Short columns restrained by partial storey height walls (structural or infill) or deep spandrels
Masonry	RML RMC URM	Reinforced Masonry Bearing Walls with Wood or Metal Deck Floors or Roofs Reinforced Masonry Bearing Walls with Concrete Diaphragms Unreinforced Masonry Bearing Wall Building	90	4. Soft Storey	Severe reduction of stiffness caused by discontinuous shear walls, openings, etc.
				5. Pounding	Separation between buildings less than 20 Z _v x no. of storeys (in mm)
				6. Major Modifi- cations	Any change in function, use or addition which results in significant increase in loading or weight
				7. Deterior- ation	Structural elements are damaged, poor condition of building is apparent (corroded reinforcement or steel, rotted wood, poor concrete or masonry)
				8. None	None of the irregularities listed above is present.
NON - STRUCTURAL HAZARDS (Circle appropriate descriptors) see 4.3.4					
F₁ Falling Hazards to Life: Exterior: Masonry chimneys, parapets, veneer or stone / precast panels, non-safety glass, or canopies over exits and walkways Interior: Heavy components; masonry partitions; non-safety glass in egress areas; storage shelves which may collapse onto areas of human occupancy F₂ Hazards to Continuous Operation of Special Buildings: Equipment or lifelines required for continuous operation of special facilities. The owner or authority should provide a list of critical items needed for continuing operations.					

From: Manual for Screening of Existing Buildings for Seismic Investigation, IRC / NRC, Canada, Ottawa, September 1992

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Figure 2.31: Data collection form (Page 1).

SEISMIC SCREENING FORM															p. 2 of 2		ITEM No.:	
SEISMIC PRIORITY INDEX: Circle appropriate value and enter each result on right side. Use asterisk (*) with uncertain values																		
A	Seismicity	Design NBC	Effective Seismic Zone (Z_v or $Z_v + 1$ if $Z_d > Z_v$)												A =			
			2	3	4	5	6											
		Pre - 65	1.0	1.5	2.0	3.0	4.0											
		65 - 84	1.0	1.0	1.3	1.5	2.0											
Post - 85	1.0	1.0	1.0	1.0	1.0													
B	Soil Conditions	Design NBC	Soil Category												B =			
			Rock or Stiff Soil	Stiff Soil > 50 m	Soft Soil > 15 m	Very Soft or Liquefiable Soil	Unknown Soil											
		Pre - 65	1.0	1.3	1.5	2.0	1.5											
		Post - 65	1.0	1.0	1.0	1.5	1.5											
C	Type of Structure (BM = Benchmark year, see p.1)	Design NBC	Construction Type and Symbol (see p. 1)												C =			
			Wood	Steel	Concrete	Precast	Masonry Infill	Masonry										
		WLF	WPB	SLF	SMF	SBF	SCW	CMF	CSW	PCF	PCW	SIW, CIW	RML, RMC	URM				
		Pre - 70	1.2	2.0	1.0	1.2	1.5	2.0	2.5	2.0	2.5	2.0	3.0	2.5		3.5		
70 - BM	1.2	2.0	1.0	1.2	1.5	1.5	1.5	1.5	1.8	1.5	2.0	1.5	3.5					
Post - BM	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	—					
D	Building Irregularities	Design NBC	1. Vertical	2. Horiz.	3. Short Concrete Columns	4. Soft Storey	5. Pounding	6. Modification	7. Deterioration	8. None	D = product of circled Numbers (Max of 4.0) =							
			Pre - 70	1.3	1.5	1.5	2.0	1.3	1.3	1.3		1.0						
		Post - 70	1.3	1.5	1.5	1.5	1.3	1.0	1.3	1.0								
		Pre - 70	1.3	1.5	1.5	2.0	1.3	1.3	1.3	1.0								
E	Building Importance	Design NBC	Low Occupancy N < 10	Normal Occupancy N = 10 - 300	School, or High Occupancy N = 301 - 3000	Post Disaster, or Very High Occup. N > 3000	Special Operational Requirements	E =										
			Pre - 70	0.7	1.0	1.5	2.0		3.0									
		Post - 70	0.7	1.0	1.2	1.5	2.0											
		Pre - 70	0.7	1.0	1.2	1.5	2.0											
N = Occupied Area x Occupancy Density x Duration Factor* = ... X X = Primary Use: Occupancy Density Average Weekly Hours Persons / m ² of Human Occupancy Assembly 1 5 - 50 Mercantile, Personal service 0.2 50 - 80 Offices, Institutional, Manufacturing 0.1 50 - 60 Residential 0.05 100 Storage 0.01 - 0.02 100 * Duration Factor is equal to the average weekly hours of human occupancy divided by 100, not greater than 1.0																		
SI STRUCTURAL INDEX = A • B • C • D • E = SI =																		
F	NON - STRUCTURAL HAZARDS		Description (see p. 1)		None		Yes		Yes *		F = max (F ₁ , F ₂) =							
	F ₁	Falling Hazards to Life			Pre - 70 NBC	1.0	3.0	6.0										
	F ₂	Hazards to Vital Operations			Post - 70 NBC	1.0	2.0	3.0										
					Any Year	1.0	3.0	6.0										
* applies only if one or more of the following descriptors on page 1 are circled: SMF, CMF, soft storey, torsion																		
NSI NON - STRUCTURAL INDEX = B • E • F = NSI =																		
SPI SEISMIC PRIORITY INDEX = SI + NSI = SPI =																		
Comments:																		

From: Manual for Screening of Existing Buildings for Seismic Investigation, IRC / NRC, Canada, Ottawa, September 1992

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Figure 2.32: Data collection form (Page 2).

2.2.3 Pre-Field Data Collection

One of the first and most critical stages of the seismic screening process is the pre-field data collection stage, which involves compiling all available information and documents about the building at the desk. The data obtained at this stage form the basis for subsequent steps of the screening and allow for a preliminary evaluation of the building's seismic risk.

The pre-field data collection process covers parameters that directly affect structural behavior, such as the location of the building, its importance, intended use, year of construction, and the soil conditions at the site. The information shown in the flowchart in **Figure 2.33**, which pertains to this data collection process, is typically obtained from municipal archives, building permits, architectural or structural drawings, land registry records, or previously prepared investigation reports. The accuracy and currency of this information directly influence the reliability of the final assessment.

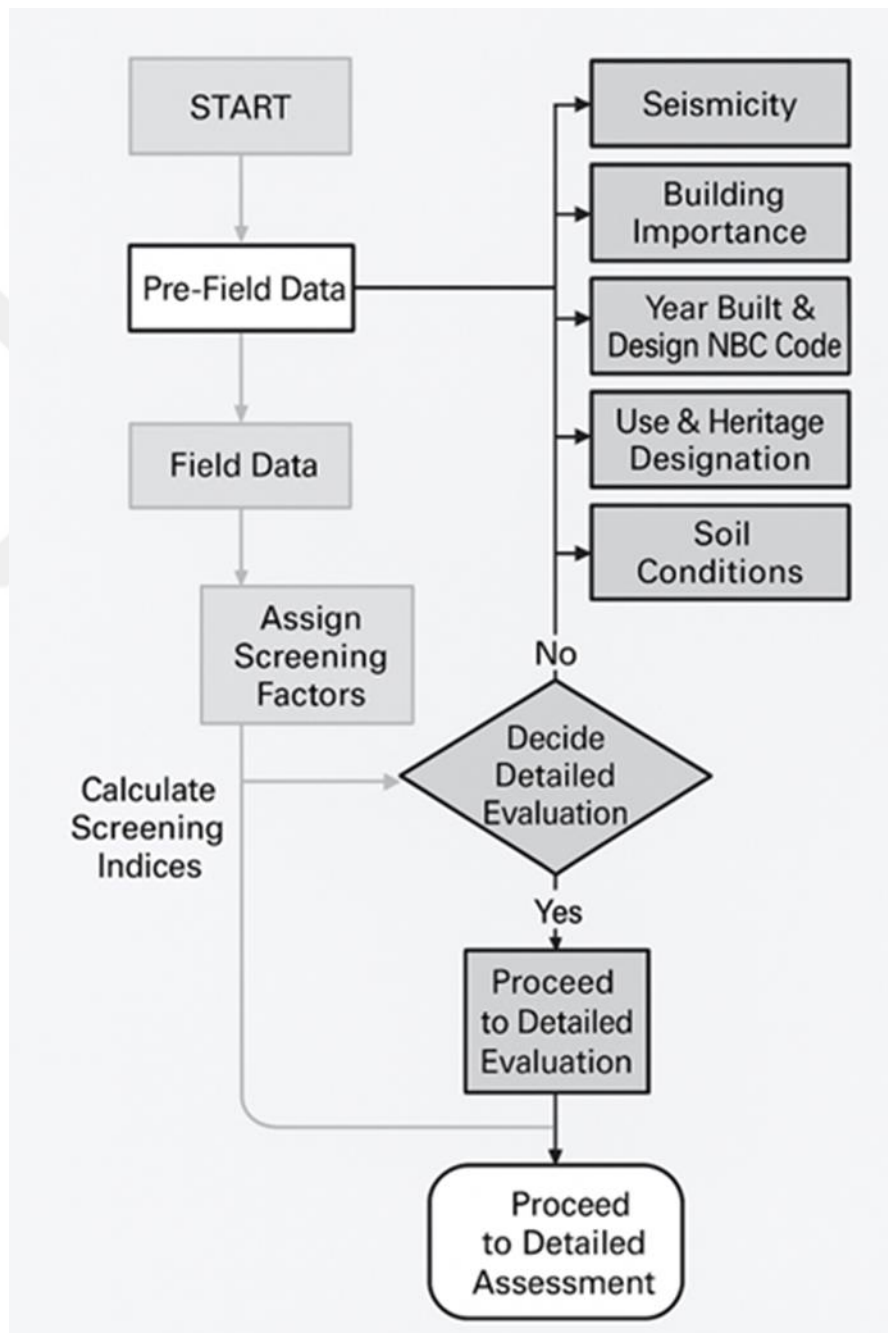


Figure 2.33: Flowchart illustrating the information to be collected during the Pre-Field Data stage.

As a result of the pre-field data collection efforts, critical information is obtained regarding the seismicity region in which the building is located, the NBC (National Building Code of Canada) edition to which it is subjected based on its year of construction, the importance factor, and the use & occupancy class. In addition, whether the building has heritage status and its soil class are determined at this stage. This information is recorded in the relevant section of the data collection form and made ready to be evaluated together with the observations to be made in the field.

2.2.3.1 Number of Stories, Year of Construction, and Height

The seismic activity of the region in which the building is located is evaluated according to the Effective Seismic Zone defined in the Canadian Seismic Hazard Maps. This value is obtained by interpreting the parameters Z_a (acceleration), Z_v (velocity), and Z_t (duration), which are determined based on the location of the building. Within the scope of the Canadian Rapid Seismic Screening Method, the following rule is used when determining the regional seismic zone:

- If $Z_a > Z_v$, the effective seismic zone for that region is taken as $Z_v + 1$.
- If $Z_v \geq Z_a$, the seismic zone of the region is determined directly according to the value of Z_v .

The table in the Seismic Screening Form shown below presents the regional seismic hazard defined according to the zone value in question, as well as a screening score defined as a coefficient A, depending on the NBC era in which the building was designed (**Table 2.6**):

Table 2.6: Screening score table for the seismicity parameter.

A	Seismicity	Design NBC	Effective Seismic Zone (Z_v , or $Z_v + 1$ if $Z_a > Z_v$)					A =
			2	3	4	5	6	
		Pre - 65	1.0	1.5	2.0	3.0	4.0	
		65 - 84	1.0	1.0	1.3	1.5	2.0	
		Post - 85	1.0	1.0	1.0	1.0	1.0	

The A values in this table indicate:

- Values around 1.0: regions where seismic effects are relatively low,
- Values around 4.0: regions where seismic hazard is severe.

In addition, the code era (NBC) under which the building was designed (for example, Pre-1965, 1965–1984, or Post-1985) directly influences this scoring. In this regard, the “Seismicity” parameter is an important indicator that simultaneously evaluates both the regional hazard level and the adequacy of the building’s seismic design.

2.2.3.2 Number of Stories, Year of Construction, and Height

In the seismic performance assessment of a building, not only its physical characteristics but also its intended use and its importance to society must be taken into account. In this context, the Canadian Rapid Seismic Screening Method includes a classification system that defines the functional importance of buildings. This classification is used to determine the coefficients that influence the evaluation scores based on how critical the building is under risk (**Table 2.7**).

Buildings are generally grouped under the following four main categories:

- Category 1 – Low-Importance Buildings: Small warehouses, simple shelters, or buildings that do not provide essential functions for society.
- Category 2 – Normal-Importance Buildings: Most residential buildings and standard office buildings fall into this group. These are structures that meet the basic needs of society but do not provide critical services.
- Category 3 – High-Importance Buildings: Buildings that are expected to remain operational after an earthquake such as schools, hospitals, police stations, and fire stations.
- Category 4 – Post-Disaster (Very High-Importance) Buildings: Special classes of structures such as nuclear power plants, major infrastructure facilities, or industrial buildings with very high risk.

Within the screening method, building importance is integrated into the numerical scoring system that governs prioritization and strengthening decisions. Threshold values are kept lower for buildings of critical importance; thus, potential risks can be identified more sensitively.

Table 2.7: Code eras by construction year and seismic design approaches.

Importance Category	Definition	Example Buildings
Low	Buildings with limited impact on the functioning of society	Warehouses, single-story workshops, agricultural buildings
Normal	Buildings in general use with limited life-safety risk in case of collapse	Residential buildings, small commercial buildings, school buildings
High	Buildings with critical functions whose collapse may cause significant loss of life or disruption of services	Hospitals, fire stations, police buildings, energy centers
Post-Disaster	Buildings that are required to remain functional after an earthquake	Emergency response centers, disaster coordination centers

2.2.3.3 Year Built & NBC

The year of construction of a building has a directly decisive effect on its seismic performance assessment. The main reason is that the engineering code under which the building was designed varies in line with the evolving state of knowledge and code standards over time. In this context, the main reference document in Canada is the National Building Code of Canada (NBCC).

The NBCC has undergone significant changes over the years with respect to seismic design criteria. For example, buildings constructed before 1970 were designed with seismic effects virtually ignored, whereas in the versions of the code issued after 1995, consideration of seismic forces became mandatory. Therefore, the construction year of a building indirectly reflects the level of engineering practice applied.

Below is a brief summary of some critical periods and the code changes made during these periods:

- < 1970: Seismic design was essentially neglected.
- 1970–1985: Initial seismic evaluation parameters were incorporated into the codes, but practical implementation remained limited.
- 1985–2005: Concepts such as spectral acceleration and soil effects were integrated into the NBCC.
- >2005: Probabilistic seismic hazard analyses and structural performance criteria were introduced in a more detailed manner.

This classification is reflected in the scoring system that affects the evaluation coefficient of the building on the screening form. In particular, buildings constructed according to older codes receive higher risk scores based on the assumption that they may exhibit lower seismic performance (**Table 2.8**).

Table 2.8: Code eras by construction year and seismic design approaches.

Construction Year Range	Relevant NBCC Version	Seismic Design Status
< 1970	NBCC 1965 and earlier	No seismic design / negligible level
1970–1985	NBCC 1970–1985	Initial seismic coefficients / limited application
1985–2005	NBCC 1985–2005	Soil class and spectral acceleration concepts
> 2005	NBCC 2005–2020+	Advanced analysis / performance-based approach

2.2.3.4 Use & Heritage Designation

The intended use of buildings and any heritage designation they may have are among the factors that directly influence the assessment of seismic risk. The function for

which a building is used is important in terms of population density during a possible earthquake, continuity of operation, and economic/psychological impacts. In particular, facilities that serve as emergency infrastructure, such as hospitals, schools, and fire stations, have a high level of importance, whereas buildings with heritage status, such as museums, historic churches, or listed residential structures, require a different evaluation perspective (**Table 2.9**).

Table 2.9: Effects of different uses and heritage status on the evaluation.

Intended Use	Effect on Seismic Risk Assessment	Explanation
Residential	Medium	Standard population density; structural performance is expected to be moderate.
Educational Building	High	Evacuation and safety of students are a priority.
Medical (Hospital)	Very High	Continuous service is required; both structural and nonstructural elements are critical.
Commercial Building	Medium–High	Potential for high economic loss.
Emergency Services	Very High	Post-earthquake functionality is essential.
Heritage	High (in terms of aesthetic and historical losses)	Strengthening may be limited; conservation criteria must be carefully considered.

Points to Consider for Heritage Structures:

- Interventions may be restricted due to listing or protection status.
- The requirement to preserve original materials and details limits strengthening options.

- During evaluation of the structural system, any past restoration works must be taken into account.

This heading aims to clarify the intended use of the building and, where applicable, to anticipate the impact of special statuses (e.g., “protected structure”) on the evaluation, within the scope of pre-field data collection. Determining which group the building belongs to in this classification directly affects both the preparations for field observations and the coefficients to be used in the final risk scoring.

2.2.3.5 Soil Conditions

The seismic performance of a building depends not only on the strength of its load-bearing system, but also on the characteristics of the soil layer on which it is founded. Within the scope of the Canadian Rapid Seismic Screening Method, soil conditions are evaluated as a critical parameter that directly influences the expected earthquake acceleration at the site and its amplification.

Soil classification is typically carried out based on data obtained from sources such as local geological maps, geotechnical reports, or municipal records. This classification, which is carried out without detailed site-specific geotechnical investigations, provides a sufficient level of insight for a rapid screening method. The evaluation is made under the heading “Soil Category” in Section B of the Seismic Screening Form, and the classes defined therein (such as Rock/Stiff Soil, Soft Soil > 15 m, Very Soft or Liquefiable Soil) represent differences in seismic soil behavior among buildings.

The following aspects are taken into consideration when evaluating soil conditions:

- Geological history (e.g., former riverbed, alluvial fill)
- Previous soil investigations in the area
- Foundation system of the building and settlement history
- Observational information about the soil of adjacent buildings

The evaluation under this heading leads to a decision on which soil class the building belongs to and enables the determination of the soil effect coefficient (B) to be used in the final seismic risk scoring. This coefficient varies depending on the construction year and is tabulated as a function of soil type and the design code (**Table 2.10**).

Table 2.10: Coefficients used according to soil class and year of construction.

B	Soil Conditions	Design NBC	Soil Category					B =
			Rock or Stiff Soil	Stiff Soil > 50 m	Soft Soil > 15 m	Very Soft or Liquefiable Soil	Unknown Soil	
		Pre - 65 Post - 65	1.0 1.0	1.3 1.0	1.5 1.0	2.0 1.5	1.5 1.5	

2.2.4 Field Observations

This section describes the field observations and the process of collecting critical data on the building within the scope of the Canadian Rapid Seismic Screening Method. Observations made in the field allow the identification of various structural and nonstructural characteristics that may directly affect the seismic performance of the building. This information forms the fundamental input for the building's risk assessment.

The basic information collected during fieldwork generally includes:

- General geometric characteristics of the building (height, number of stories, plan irregularity, etc.)
- Structural system type and condition (reinforced concrete, steel, masonry, etc.)
- Presence of structural irregularities (soft story, short columns, torsional irregularity, etc.)
- Identification of existing structural damage or deterioration
- Intended use and occupancy level of the building
- Condition and anchorage of critical equipment or services within the building
- The surroundings of the building and its relationship with neighboring structures

This data is entered directly into the Canadian Seismic Screening Form (Field Screening Form), and a comprehensive profile of the building is generated. The information collected must be recorded in a consistent and systematic manner during field screening activities.

The field observations section of the Canadian screening form is standardized so as to ensure consistency and comparability of observations. A detailed explanation of this form and its use will be provided in later sections of the study.

2.2.4.1 Type of Structure

Within the scope of the Canadian Rapid Seismic Screening Method, the first step in collecting building information is to determine the type of structure. The structural type is essential information for evaluating seismic performance. Once this information is accurately established in the field, it serves as a reference for subsequent steps in the screening.

In the Canadian Seismic Screening Form, the structural type is generally defined by classifying the building into the following categories:

- Reinforced Concrete Frame Structures
- Reinforced Concrete Shear Wall Structures
- Steel Structures
- Masonry Structures
- Wood Structures
- Mixed Structures

Structural types are expressed by different coefficients depending on the year of construction of the building and the type of structural system used. The classification of structural types according to the Canadian Rapid Seismic Screening Method and the coefficients associated with each structural type are given in the **Table 2.11** below.

Table 2.11: Structural types and coefficients related to structural systems (adapted from the Canadian seismic screening form).

C	Type of Structure (BM = Benchmark year, see p.1)	Design NBC	Construction Type and Symbol (see p. 1)													C =
			Wood		Steel				Concrete		Precast		Masonry Infill	Masonry		
			WLF	WPB	SLF	SMF	SBF	SCW	CMF	CSW	PCF	PCW	SIW, CIW	RML, RMC	URM	
		Pre - 70	1.2	2.0	1.0	1.2	1.5	2.0	2.5	2.0	2.5	2.0	3.0	2.5	3.5	
	70 - BM	1.2	2.0	1.0	1.2	1.5	1.5	1.5	1.8	1.5	2.0	1.5	3.5			
	Post - BM	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	—		

This table serves as a reference for field personnel in determining the structural type and is used to achieve a more accurate assessment of the structural condition of the building.

2.2.4.2 Building Irregularities

This section covers the identification of structural irregularities that may affect the structural performance of the building within the scope of the Canadian Rapid Seismic Screening Method. Structural irregularities significantly influence the damageability and behavior of the building during earthquakes. Therefore, their accurate identification during field observations is of critical importance.

Structural irregularities are generally examined under the following categories:

- Vertical Irregularities
 - Soft story
 - Weak story
 - Vertical discontinuities
- Horizontal (Plan) Irregularities
 - Torsional irregularity
 - Re-entrant corners in plan
 - Diaphragm discontinuities
- Specific Structural Irregularities
 - Short columns
 - Heavy cantilevers
 - Complex structural systems

During field screening, the presence or absence of the irregularities listed above is carefully observed and recorded on the Canadian Seismic Screening Form. Accurate and consistent characterization of these irregularities enhances the reliability of the assessments related to the seismic safety of the building.

2.2.4.3 Building Condition

An important step in field observations conducted within the scope of the Canadian Rapid Seismic Screening Method is the assessment of the existing condition of the building. The existing condition of the building encompasses damage, deterioration, or material weaknesses that have developed over time and that may directly affect its seismic performance.

In evaluating the existing condition of the building, the following issues are particularly considered in the field:

- **Structural Damage:** Cracks, breaks, or deformations in structural elements such as columns, beams, slabs, and shear walls are evaluated.
- **Material Deterioration:** Corrosion, rusting, or spalling in reinforced concrete members; degree of corrosion in steel elements; cracks or separations in masonry components are observed.
- **Past Damage and Repairs:** Previous damage, performed repairs, or strengthening works and their current state are examined. The quality and effectiveness of these repairs are evaluated.
- **Maintenance and Use Condition:** The general maintenance condition of the building, use-related deterioration, or modifications incompatible with the intended use of the structure are identified.
- **Environmental Effects:** The exposure of the building to environmental effects such as moisture and freeze–thaw cycles is examined.
- **Foundation and Soil Problems:** Observable settlement, sliding, or tilting of the building's foundation or soil are identified.

During fieldwork, the observations related to the existing condition of the building are systematically recorded on the Canadian Seismic Screening Form. This information is of critical importance for making accurate and up-to-date evaluations of the structural performance of the building.

2.2.4.4 Non-Structural Hazards

An important aspect of building evaluation in the Canadian Rapid Seismic Screening Method is the identification of risks arising from nonstructural elements. Nonstructural elements comprise components outside the building's load-bearing system and may pose serious safety risks to occupants during an earthquake. Therefore, they must be carefully examined during field screening.

Nonstructural hazards are generally evaluated under the following categories:

- **Façade Elements:** The anchorage conditions and falling risks of glass façades, exterior cladding, panels, or decorative façade elements are evaluated.

- **Interior Elements:** Anchorage conditions and overturning risks of suspended ceilings, interior partitions, shelves, cabinets, and furniture are examined.
- **Mechanical and Electrical Equipment:** The anchorage conditions and movement risks of critical equipment such as air conditioning units, ventilation systems, heating systems, generators, boilers, and electrical panels are evaluated.
- **Fire Safety Components:** The condition and accessibility of fire suppression systems, fire alarms, emergency exits, and directional signage are checked.
- **Hazardous Materials and Storage Areas:** The safe storage conditions and risks of overturning or spillage of chemical substances and explosive or flammable materials are identified.

During fieldwork, observations regarding nonstructural hazards are recorded on the Canadian Seismic Screening Form in a standardized manner, thereby providing comprehensive and reliable information about the safety of building use.

2.2.4.5 Review of Pre-Field Data

Within the framework of the Canadian Rapid Seismic Screening Method, pre-field data previously compiled are reviewed and verified on site in order to enhance the quality and reliability of the data collected during fieldwork. In this way, the accuracy and consistency of the dataset created at the preliminary evaluation stage are checked, and more reliable results are obtained.

During the review of pre-field data in the field, the following points are particularly taken into account:

- **Seismicity and Seismic Hazard Data:** The seismic hazard level and site acceleration values determined during preliminary evaluation are compared with field conditions and verified.
- **Building Importance:** The importance category assigned to the building beforehand is compared with field observations and updated where necessary.
- **Year Built and Applicable Code (Year Built & NBC):** The year of construction and the building code in force at that time are checked for consistency with the structural characteristics observed in the field.

- **Use and Heritage Designation:** Pre-field data regarding the intended use of the building and any heritage protection status are verified through field observations and corrected if necessary.
- **Soil Conditions:** The soil class and soil characteristics determined during the pre-field stage are compared with field observations and confirmed.

This stage is of critical importance for increasing the reliability of pre-field data and ensuring that the Canadian Seismic Screening Form is completed in a comprehensive and accurate manner.

2.2.5 Final Evaluation

In the Canadian Rapid Seismic Screening Method, the final evaluation is carried out by combining the pre-field data with the building characteristics collected on site and converting them into a numerical score through certain coefficients. Although this is a rapid screening method, it aims to express the relative seismic risk level of the building in a systematic and comparable way.

At the end of the evaluation process, a “Final Score” is calculated for each building. This score is a critical indicator of the relative seismic safety of the building and is used as the basis for risk classification.

2.2.5.1 Assigning Screening Factors

The evaluation stage of the Canadian Rapid Seismic Screening Method is based on three fundamental pillars:

1. Collection of pre-field data (Pre-Field Data)
2. Recording of observational data in the field (Field Data)
3. Conversion of the parameters obtained from these two stages into numerical scores (Assign Screening Factors)

This third stage constitutes the component that forms the final decision mechanism of the method. In this section, all data obtained before and during fieldwork are systematically scored on the Canadian Seismic Screening Form. The scores are expressed by certain coefficients, each reflecting the relative influence of a given parameter on the seismic performance of the building.

Structural Components

The structural scoring (Structural Index – SI), which evaluates the risks related to the load-bearing system of the building, is performed based on five main parameters:

- A: Seismicity
- B: Soil Conditions
- C: Type of Structure
- D: Building Irregularities
- E: Building Importance

Values corresponding to these headings are selected using the relevant tables in the screening form, and the following formula is applied for each building in equation (2.4)

$$SI = A * B * C * D * E \quad (2.4)$$

(Maximum SI value: 40)

This formula yields the structural risk index obtained by the combined evaluation of the five fundamental factors affecting the performance of the structural system.

Non-Structural Components

Risks arising from systems that are integrated into the building but are non-load-bearing are evaluated using the Non-Structural Index (NSI). This evaluation includes three parameters:

- B: Soil Conditions
- E: Building Importance
- F: Non-Structural Hazards

The coefficients obtained from these three parameters are multiplied as follows to compute the NSI value in equation (2.5).

$$NSI = B * E * F \quad (2.5)$$

(Here F is taken as the greater of F₁ and F₂.)

The parameter F represents hazards related to overturning, falling, or operational systems within the building, and the maximum of F₁ (falling hazards to life) and F₂ (hazards to vital operations) is used.

Summary Structure

These two axes of evaluation (structural and nonstructural) constitute the basic scoring system of the Canadian method (**Table 2.12**) The following summary clarifies this distinction:

Table 2.12: Summary table of score.

Category	Parameters	Description
Structural (SI)	A, B, C, D, E	Load-bearing system and building characteristics
Non-Structural (NSI)	B, E, F	Soil, building use, and nonstructural element risk

The sum of these two components (SI and NSI) gives the overall seismic priority score of the building. This score is used to evaluate the relative risk levels of buildings in a comparative manner and to determine intervention priorities in equation (2.6).

$$SPI = SI + NSI \quad (2.6)$$

Determination of the Limit Score

The seismic index score (SPI) is used to prioritize buildings according to their levels of seismic performance. Based on this score, buildings that may require comprehensive engineering evaluation can be identified.

The following priority classification according to SPI scores is proposed in line with field applications in the literature and the evaluation logic of the method:

$SPI < 10$: Low-priority buildings

$SPI = 10-20$: Medium-priority buildings

$SPI > 20$: High-priority buildings that may require detailed investigation

$SPI > 30$: Buildings that may be considered potentially hazardous

Below, **Figure 2.34** and **Figure 2.35** an example evaluation form filled out within the scope of the Canadian Rapid Seismic Screening Method is presented. In this form, the

coefficients corresponding to each parameter are marked and the final SPI score is calculated. This form is included to illustrate the application process of the method.

No. of Storeys: 4		Total Floor Area: 480 m ²		Year Built: 1940		Design No.: —	
Primary use (see list on p. 2): Residential (Apartments)				Heritage Designation: —			
Inspector: MJ		Date: 92-02-05		Checked by: HR			

Sketch

not available

Photo

TYPE OF STRUCTURE (circle appropriate descriptors) see 4.3.2			BM	BUILDING IRREGULARITIES (circle appropriate descriptors) see 4.3.3	
Wood	<u>WLF</u> WPB	Wood Light Frame Wood, Post and Beam	90	1. Vertical Irregularity	Abrupt changes in plan dimensions over height (e.g. setback or building on hill)
Steel	SMF SBF SLF SCW SIW	Steel Moment Frame Steel Braced Frame Steel Light Frame Steel Frame with Concrete Shear Walls Steel Frame with Infill Masonry Shear Walls	90	2. Horizontal Irregularity (Torsion)	Irregular building shapes such as "L", "V", "E", "T", eccentric stiffness in plan (e.g. shear wall on only one side of building)
Concrete	CMF CSW CIW PCF PCW	Concrete Moment Frame Concrete Shear Walls Concrete Frame with Infill Masonry Shear Walls Precast Concrete Frame Precast Concrete Walls	85	3. Short Concrete Columns	Short columns restrained by partial storey height walls (structural or infill) or deep spandrels
Masonry	RML	Reinforced Masonry Bearing Walls with Wood or Metal Deck Floors or Roofs	90	4. <u>Soft Storey</u>	Severe reduction of stiffness caused by discontinuous shear walls, openings, etc.
	RMC	Reinforced Masonry Bearing Walls with Concrete Diaphragms		5. <u>Pounding</u>	Separation between buildings less than 20 Z _v x no. of storeys (in mm)
	URM	Unreinforced Masonry Bearing Wall Building		6. Major Modifications	Any change in function, use or addition which results in significant increase in loading or weight
				7. <u>Deterioration</u>	Structural elements are damaged, poor condition of building is apparent (corroded reinforcement or steel, rotted wood, poor concrete or masonry)
				8. None	None of the irregularities listed above is present.

NON - STRUCTURAL HAZARDS (Circle appropriate descriptors) see 4.3.4

F₁ Falling Hazards to Life:
 Exterior: Masonry chimneys, parapets, veneer or stone / precast panels, non-safety glass, or canopies over exits and walkways
 Interior: Heavy components; masonry partitions; non-safety glass in egress areas; storage shelves which may collapse onto areas of human occupancy

F₂ Hazards to Continuous Operation of Special Buildings: Equipment or lifelines required for continuous operation of special facilities. The owner or authority should provide a list of critical items needed for continuing operations.

80

From: Manual for Screening of Existing Buildings for Seismic Investigation, IRC / NRC Ottawa, September 1992

Figure 2.34: Example of score-1.

SEISMIC SCREENING FORM												p. 2 of 2		ITEM No.: 1	
SEISMIC PRIORITY INDEX: Circle appropriate value and enter each result on right side. Use asterisk (*) with uncertain values															
A	Seismicity	Design NBC	Effective Seismic Zone (Z_v , or $Z_v + 1$ if $Z_v > Z_d$)										A = 3.0		
			2	3	4	5	6								
		Pre - 65	1.0	1.5	2.0	3.0	4.0								
		65 - 84	1.0	1.0	1.3	1.5	2.0								
		Post - 85	1.0	1.0	1.0	1.0	1.0								
B	Soil Conditions	Design NBC	Soil Category										B = 2.0		
			Rock or Stiff Soil	Stiff Soil > 50 m	Soft Soil > 15 m	Very Soft or Liquefiable Soil	Unknown Soil								
		Pre - 65	1.0	1.3	1.5	2.0	1.5								
		Post - 65	1.0	1.0	1.0	1.5	1.5								
C	Type of Structure (BM = Benchmark year, see p.1)	Design NBC	Construction Type and Symbol (see p. 1)												C = 1.2
			Wood		Steel		Concrete		Precast	Masonry Infill	Masonry				
		WLF	WPB	SLF	SMF	SBF	SCW	CMF	CSW	PCF	PCW	SIW, CIW	RML, RMC	URM	
		Pre - 70	1.2	2.0	1.0	1.2	1.5	2.0	2.5	2.0	2.5	2.0	3.0	2.5	
		70 - BM	1.2	2.0	1.0	1.2	1.5	1.5	1.5	1.5	1.8	1.5	2.0	1.5	3.5
		Post - BM	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	—	
D	Building Irregularities	Design NBC	1. Vertical	2. Horiz.	3. Short Concrete Columns	4. Soft Storey	5. Pounding	6. Modification	7. Deterioration	8. None	D = product of circled Numbers (Max of 4.0) = 3.7				
		Pre - 70	1.3	1.5	1.5	2.0	1.3	1.3	1.3	1.0					
		Post - 70	1.3	1.5	1.5	1.5	1.3	1.0	1.3	1.0					
E	Building Importance	Design NBC	Low Occupancy N < 10	Normal Occupancy N = 10 - 300	School, or High Occupancy N = 301 - 3000	Post Disaster, or Very High Occup. N > 3000	Special Operational Requirements	E = 1.0							
		Pre - 70	0.7	1.0	1.5	2.0	3.0								
		Post - 70	0.7	1.0	1.2	1.5	2.0								
N = Occupied Area x Occupancy Density x Duration Factor* = ... 360... x 0.05... x 1.0... = 18 Primary Use: Occupancy Density Average Weekly Hours Persons / m ² of Human Occupancy Assembly 1 5 - 50 Mercantile, Personal service 0.2 50 - 80 Offices, Institutional, Manufacturing 0.1 50 - 60 Residential 0.05 100 Storage 0.01 - 0.02 100											* Duration Factor is equal to the average weekly hours of human occupancy divided by 100, not greater than 1.0				
SI STRUCTURAL INDEX = A · B · C · D · E =											SI = 24.5				
F	NON - STRUCTURAL HAZARDS		Description (see p. 1)		None		Yes		Yes *		F = max (F ₁ , F ₂) = 6.0				
	F ₁	Falling Hazards to Life			Pre - 70 NBC	1.0	3.0	6.0							
					Post - 70 NBC	1.0	2.0	3.0							
	F ₂	Hazards to Vital Operations			Any Year	1.0	3.0	6.0							
* applies only if one or more of the following descriptors on page 1 are circled: SMF, CMF, soft storey, torsion															
NSI NON - STRUCTURAL INDEX = B · E · F = 2.0 x 3 =											NSI = 12.0				
SPI SEISMIC PRIORITY INDEX = SI + NSI = 24.5 + 12.0 =											SPI = 36.5				

Figure 2.35: Example of score-2.

2.3 P-25 Rapid Screening Method

The P-25 Rapid Screening Method is an observational preliminary assessment approach developed to enable a fast and practical evaluation of the seismic performance of the reinforced-concrete building stock in Türkiye. The method focuses on identifying structural vulnerabilities frequently observed in buildings constructed prior to the year 2000 and has been simplified so that it may be applied in field conditions by a team with engineering knowledge.

The primary objective of the method is to classify buildings on a prioritization basis with respect to seismic safety and to rapidly identify those requiring detailed engineering analysis. This approach offers significant advantages to local authorities and disaster management units operating under time and resource constraints.

2.3.1 Assessment Approach and Parameters Used

The P-25 Rapid Screening Method was developed to evaluate the seismic performance of buildings rapidly and comparatively based on structural and environmental data collected through field observations. The method focuses on seven fundamental indicators of weakness, each affecting structural response at varying levels. These indicators encompass different engineering parameters, ranging from the characteristics of the load-bearing system to soil conditions. Each parameter is assigned a specific score according to field observations, and these scores constitute the basis for the final evaluation.

The parameters used are summarized below:

- P1 – Fundamental Structural Score: A basic performance score derived from the type of structural system, ductility level, and the effects of structural irregularities.
- P2 – Short Column Score: Determined according to the presence of architectural elements such as windows, walls, or parapets that reduce column height.
- P3 – Soft-Story Score: Evaluates vertical discontinuities arising from insufficient stiffness on lower floors, open commercial fronts, or inadequate infill walls.

- P4 – Projection and Discontinuity Score: Calculated based on façade projections and vertical discontinuities in the load-bearing system.
- P5 – Pounding Score: Assesses the risk of collision between adjacent structures based on separation distance, floor-level compatibility, and period differences.
- P6 – Liquefaction Risk Score: Indicates the level of risk associated with the liquefaction potential of the soil beneath the building.
- P7 – Ground Movement Score: Evaluates whether the ground on which the building is located is susceptible to settlement, sliding, or loose-fill deformation.

The fundamental assumption of the method is that the weighted average of the scores obtained from these seven parameters yields a composite value that meaningfully reflects the seismic behavior of the structure. The evaluation process is based directly on field observations and, where available, technical documentation associated with the building.

2.3.2 Parameter Definitions

The P-25 Rapid Screening Method provides a scoring-based preliminary assessment framework aimed at identifying both structural and environmental weaknesses that may affect a building's seismic performance. The seven fundamental parameters used in this method exhibit considerable similarity to those included in internationally recognized procedures such as FEMA 154 and the Canadian Rapid Seismic Screening Method. This similarity indicates that the same core risk factors are generally considered in seismic vulnerability assessments. Nevertheless, the scoring logic, coefficient definitions, and weighting approaches differ across methods, reflecting distinct engineering assumptions and implementation priorities. These differences show that such rapid screening approaches are inherently adaptable and open to further refinement.

In the P-25 method, each parameter is assigned a score based on specific observational criteria, and these scores are weighted using predefined coefficients to determine the overall risk level of the structure. The seven core parameters used in the method are presented below with their technical content.

2.3.2.1 P1: Fundamental Structural Score

In the P-25 Method, the Fundamental Structural Score (P_1) is a multi-component indicator that takes into account the characteristics of the building's load-bearing system, its geometric configuration, and material performance. The value forming the basis of this score, denoted as P_0 , is defined as the structural capacity determined by the cross-sectional area and moment of inertia of the load-bearing elements on the critical story (**Table 2.13**).

P_0 is a composite indicator obtained by comparing the effective areas and effective moments of inertia of the columns, shear walls, and infill walls on the critical story with the building's footprint area.

This process is carried out through two main components:

(a) Area Index (C_A)

The total cross-sectional area of load-bearing elements on the critical story in the x- and y-directions is normalized by the effective floor area in equation (2.7 - 2.20).

$$C_{A_x} = 2 \times 10^5 * \left(\frac{A_{ef,x}}{A_e} \right) \quad (2.7)$$

$$C_{A_y} = 2 \times 10^5 * \left(\frac{A_{ef,y}}{A_e} \right) \quad (2.8)$$

The terms $A_{ef,x}$ and $A_{ef,y}$ represent the sum of cross-sectional areas of columns, shear walls, and infill walls on the critical story:

$$A_{ef,x} = A_c + A_{sx} + \left(\frac{E_m}{E_c} \right) * A_{wx} \quad (2.9)$$

$$A_{ef,y} = A_c + A_{sy} + \left(\frac{E_m}{E_c} \right) * A_{wy} \quad (2.10)$$

Here:

- A_c : total cross-sectional area of columns on the critical story
- A_{sx} : total cross-sectional area of shear walls in the x-direction
- A_{wx} : total cross-sectional area of infill walls in the x-direction
- $E_m / E_c \approx 0.15$ (ratio of infill wall to concrete modulus of elasticity)

C_{Ax} and C_{Ay} are taken as minimum and maximum components, and the combined area index is computed as:

$$C_A = \text{sqrt} \left((0.87 * C_{A,min})^2 + (0.50 * C_{A,max})^2 \right) \quad (2.11)$$

(b) Moment of Inertia Index (C_I)

Using effective moments of inertia of columns, shear walls, and infill walls:

$$C_{I_x} = 2 \times 10^5 * \left(\frac{I_{ef,x}}{I_x} \right)^{0.2} \quad (2.12)$$

$$C_{I_y} = 2 \times 10^5 * \left(\frac{I_{ef,y}}{I_y} \right)^{0.2} \quad (2.13)$$

Where:

$$I_{ef,x} = I_{c,x} + I_{s,x} + \left(\frac{E_m}{E_c} \right) * I_{w,x} \quad (2.14)$$

$$I_{ef,y} = I_{c,y} + I_{s,y} + \left(\frac{E_m}{E_c} \right) * I_{w,y} \quad (2.15)$$

$I_{c,x}$ and $I_{c,y}$ denote the total moments of inertia of the columns at the critical story with respect to the x and y directions, respectively,

$I_{s,x}$ and $I_{s,y}$ denote the total moments of inertia of the shear walls at the critical story with respect to the x and y directions, respectively,

$I_{w,x}$ and $I_{w,y}$ denote the total moments of inertia of the infill walls at the critical story with respect to the x and y directions, respectively,

E_m / E_c represents the ratio of the modulus of elasticity of the infill walls to that of the reinforced concrete structural elements, which is approximately taken as 0.15.

The inertia moment indices $C_{I,x}$ and $C_{I,y}$ calculated according to Equations (2.12) and (2.13) are considered as the minimum and maximum components ($C_{I,min}$ and $C_{I,max}$), respectively, and substituted into Equation (2.16). Accordingly, the resultant inertia moment index is obtained as follows:

$$C_I = \text{sqrt} \left((0.87 * C_{I,min})^2 + (0.50 * C_{I,max})^2 \right) \quad (2.16)$$

(c) Computation of P_0

P_0 is calculated by normalizing the sum of the two indices by a height-correction factor:

$$P_0 = \frac{C_A + C_I}{h_0} \quad (2.17)$$

$$h_0 = -0.6H^2 + 39.6H - 13.4 \quad (2.18)$$

where H is the total building height (m).

This empirical expression provides height-based calibration enabling comparison among buildings with varying story counts.

(d) Computation of P_1

P_1 is obtained by multiplying P_0 with fourteen modifiers (f_i) accounting for torsional irregularity, story discontinuities, corrosion, soil class, foundation type, and other factors:

$$P_1 = P_0 * (f_1 * f_2 * \dots * f_{14}) \quad (2.19)$$

or equivalently:

$$P_1 = P_0 \times \prod_{i=1}^{14} f_i \quad (2.20)$$

Each modifier ranges between 0.65 and 1.00; lower values indicate more structural deficiencies, increasing P_1 accordingly.

Since P_1 incorporates the widest set of information, it generally serves as the strongest indicator of a building's overall structural adequacy and carries significant weight in the final scoring.

Table 2.13: Fundamental structural score.

Katsayı	Tanım	Risk Seviyesi		
		Yüksek	Az	Yok
f_1	Burulma Düzensizliği	0.90	0.95	1.00
f_2	Döşeme Süreksizliği	0.90	0.95	1.00
f_3	Düşey Doğrultuda Süreksizlik	0.65-0.75	0.90	1.00
f_4	Kütle Düzensizliği	0.90	0.95	1.00
f_5	Korozyon Mevcudiyeti	0.90	0.95	1.00
f_6	Ağır Cephe Elemanları	0.90	0.95	1.00
f_7	Asma Kat Mevcudiyeti (γ =Asma kat / Kat alanı)	0.90 $\gamma \geq 0.25$	0.95 $0 < \gamma < 0.25$	1.00 $\gamma = 0$
f_8	Katlarda Seviye Farkı veya Kısmi Bodrum	0.80	0.90	1.00
f_9	Beton Kalitesi ⁽¹⁾	$f_9 = (f_c / 20)^{0.5}$		
f_{10}	Zayıf Kolon-Kuvvetli Kiriş ⁽²⁾	$f_{10} = [(I_x + I_y) / 2 I_b]^{0.15} \leq 1.0$		
f_{11}	Etriye Sıklığı ⁽³⁾	$f_{11} = 0.60 \leq (10 / s)^{0.25} \leq 1.0$		
f_{12}	Zemin Sınıfı	0.90 (Z4 için)	0.95 (Z3 için)	1.00 (Z2, Z1 için)
f_{13}	Temel Tipi	0.80 - 0.90 (Tekil temel)	0.95 (Sürekli temel)	1.00
f_{14}	Temel Derinliği	0.90 (1m'den az)	0.95 (1 - 4m arası)	1.00 (4m'den fazla)

⁽¹⁾ f_c , binanın MPa cinsinden beton kalitesidir.

⁽²⁾ I_x , I_y değerleri, kritik kat kolonlarının ortalama boyutlarından elde edilen temsili kolonun atalet momenti, I_b değeri ise, yine kritik katta en çok tekrar eden kirişin atalet momentidir.

⁽³⁾ s , cm cinsinden sarılma bölgesindeki etriye aralığıdır.

2.3.2.2 P2: Short Column Condition

Short columns—which arise due to architectural constraints such as partial infill walls, parapets, or mezzanine elements limiting free column height—constitute one of the most critical irregularities under seismic action (**Table 2.14**). They lead to severe shear concentrations and accelerated local damage. In the P-25 Method, the presence and distribution of short columns determine the penalty score P₂.

Two variables govern the scoring:

- The ratio of short columns to total columns (%), and
- The ratio of short-column height to story height (h_s/h).

The combination of these governs the P_2 score, with higher ratios and shorter column heights yielding more severe penalties.

Table 2.14: Short column scoring table (preserved exactly as in the original).

Percentage of Short Columns Present	(0.75–1.00)h	(0.40–0.75)h	(0.15–0.40)h	(0.00–0.15)h
Low (less than 5%)	70	64	57	50
Moderate (5%–15%)	60	50	44	37
High (15%–30%)	50	40	30	24
Very High (greater than 30%)	40	30	20	10

2.3.2.3 P3: Soft-Story and Weak-Story Score

Soft-story behavior arises when the ground floor has significantly lower stiffness than the upper floors due to increased height, reduced infill walls, or open commercial fronts. Lower drift capacity makes this story a critical location during earthquakes, often resulting in concentrated damage. The severity of this weakness is quantified by the P_3 score in equations (2.21-2.23)

$$P_3 = 100 * \left[r_a * r_r * \left(\frac{h_{i+1}}{h_i} \right)^3 \right]^{0.60} \quad (2.21)$$

where:

- h_i = height of the critical story
- $h_{\{i+1\}}$ = height of the story above
- r_a = ratio of effective cross-sectional areas
- r_r = ratio of effective moments of inertia

Definitions:

$$r_a = \frac{A_{ef,x}}{A_{ef,x+1}} \leq 1 \quad (2.22)$$

$$r_r = \frac{I_{ef,x}}{I_{ef,x+1}} \leq 1 \quad (2.23)$$

The exponent 0.60 is an empirically calibrated fitting coefficient.

P_3 ranges between 0 and 100; higher values indicate adequate stiffness and continuity.

2.3.2.4 P4: Projection and Vertical Discontinuity Score

Architectural projections and vertical discontinuities in the load-bearing system negatively influence seismic behavior by disrupting load paths and creating localized demand concentrations (**Table 2.15**). P_4 is determined by evaluating projections on all four façades together with vertical alignment irregularities of beams and columns.

Table 2.15: Projection and discontinuity scoring table.

Overhang Condition	Single Facade	Two Facades	Three–Four Facades
Frame Beam Present	90	80	70
Frame Beam Absent	70	60	50

2.3.2.5 P5: Pounding Score

Pounding between adjacent buildings during an earthquake constitutes a significant risk factor that may lead to local collapses in the structural system, brittle failures at beam and column ends, and even partial or total story collapses. Within the scope of the P-25 method, the inadequacy of separation distance between neighboring buildings, incompatibility of story levels, and differences in the dynamic characteristics of adjacent structures are evaluated through the P-25 pounding score. The following factors are considered in the assessment of pounding risk (**Table 2.16**):

- Separation distance between adjacent buildings:

- If buildings are fully adjacent, the pounding risk is at its maximum level.
 - If the available separation gap is insufficient to accommodate lateral displacements during an earthquake, contact between structures becomes inevitable.
- Story level compatibility:
 - If adjacent buildings have different story heights or slab elevations, pounding becomes more destructive not only due to inertial effects but also because of structural incompatibility.
- Differences in dynamic properties:
 - Buildings with different stiffness and mass characteristics possess different natural periods; therefore, they oscillate independently during seismic excitation, amplifying pounding effects.
- Central and eccentric pounding:
 - If the line connecting the centers of mass of two buildings passes through the midpoint of the impact line, the interaction is defined as central pounding.
 - If this line lies outside the impact line, the interaction is classified as eccentric pounding, which increases the likelihood of torsional effects.
- End buildings in an attached building row:
 - In building blocks, structures located at the ends of an attached sequence are considered more critical in terms of pounding risk, as they do not receive lateral restraint from both sides and must directly resist impact forces rather than dissipating them.

Table 2.16: Overhang and discontinuity pounding factors.

Pounding Type	Central Pounding (Same Slab Level)	Central Pounding (Different Slab Levels)	Eccentric Pounding (Same Slab Level)	Eccentric Pounding (Different Slab Levels)
End building in attached block	35	15	20	10
One building stiffer and/or heavier than the other	40	25	30	20
Low-rise building adjacent to high-rise building	50	30	30	20
Buildings of equal height	70	60	60	50

2.3.2.6 P6: Liquefaction Risk Score

Liquefaction is the phenomenon in which saturated, loose sandy soils temporarily lose their bearing capacity under seismic excitation. This condition may cause severe structural problems such as ground settlement, structural tilting, or sudden collapse of the load-bearing system. Therefore, liquefaction risk is considered a critical parameter within the P-25 Rapid Screening Method (**Table 2.17**).

Liquefaction risk assessment is performed based on the geological formations of the construction site, groundwater table depth, soil type, and historical earthquake records. Using available geotechnical reports, soil investigation data, or municipal

infrastructure archives, the likelihood of liquefaction is determined and scored according to the classification given below.

Table 2.17: Liquefaction potential scores.

Groundwater Table Depth (GWT)	Low	Moderate	High
> 10 m	60	45	30
2.0 m – 10.0 m	45	33	20
< 2.0 m	30	20	10

This scoring may be updated depending on the level of available soil information. If detailed geotechnical investigations include liquefaction analyses, the results are directly reflected in the scoring. In cases of insufficient data, engineering judgment, regional observations, and general geological characteristics are considered.

2.3.2.7 P₇: Ground Movement Score

Ground movements include natural phenomena such as settlement, slope failure, landslides, and mass movements occurring at the building site. Since these movements may cause severe damage to foundations and structural systems, they are evaluated using the P₇ – Ground Movement Score within the P-25 Rapid Screening Method.

The scoring is based on field observations, geological maps, landslide inventories, and documented past ground movement events. Accordingly, the P₇ score is defined as follows (**Table 2.18**):

Table 2.18: Ground movement scores.

Soil Class	GWT (m)	P ₇ Score
Z1, Z2	–	100
Z3	GWT ≤ 5.0	25
Z3	GWT > 5.0	35

Z4	GWT $\leq$ 5.0	10
Z4	GWT $>$ 5.0	20

This approach allows evaluation of a structure's sensitivity to potential sudden deformations or long-term ground settlements. A lower P_7 score indicates a higher vulnerability to ground movement effects.

α – Correction Factor

In the P-25 method, the minimum score among the different collapse-related criteria represents the critical performance indicator of the building. However, evaluating this minimum score alone may neglect environmental conditions such as topography and seismicity. Therefore, the minimum score is modified by multiplying it with the α correction factor.

The correction factor α is calculated using the site topographic slope (t), the effective seismic acceleration coefficient (A_0), the importance factor (I), and the live load participation factor (n), as given by equation (2.24)

$$\alpha = \left(\frac{1}{I}\right) * (1.4 - A_0) * \left(\frac{1}{0.4n + 0.88}\right) * t \quad (2.24)$$

In this equation, the topographic slope coefficient is generally taken as $t = 1$. The value of A_0 can be obtained from the Turkish Seismic Hazard Map. Considering the spectral differences between TEC-2007 and TBEC-2018, the S_{DS} value obtained according to TBEC-2018 may be divided by 2.5 and used in place of A_0 in Equation (2.24). The purpose of this formulation is to correctly reflect the relative risk of buildings under different topographic conditions.

β – Correction Factor

After determining the seven building parameters (P_1 – P_7), the parameter with the lowest score is multiplied by the highest weighting coefficient ($w = 4$) given in **Table 2.19**. The remaining scores are also multiplied by their corresponding weighting coefficients. The weighted average score P_w is then calculated using Equation (2.25):

$$P_w = \frac{\sum w_i P_i}{\sum w_i} \quad (2.25)$$

Table 2.19: Weighting coefficients.

Weight	P ₁	P ₂	P ₃	P ₄	P ₅	P ₆	P ₇	P _{min}
w	4	1	3	2	1	3	2	4

The calculated weighted average score P_w is then used to determine the β correction factor using the graph provided in **Figure 2.36**.

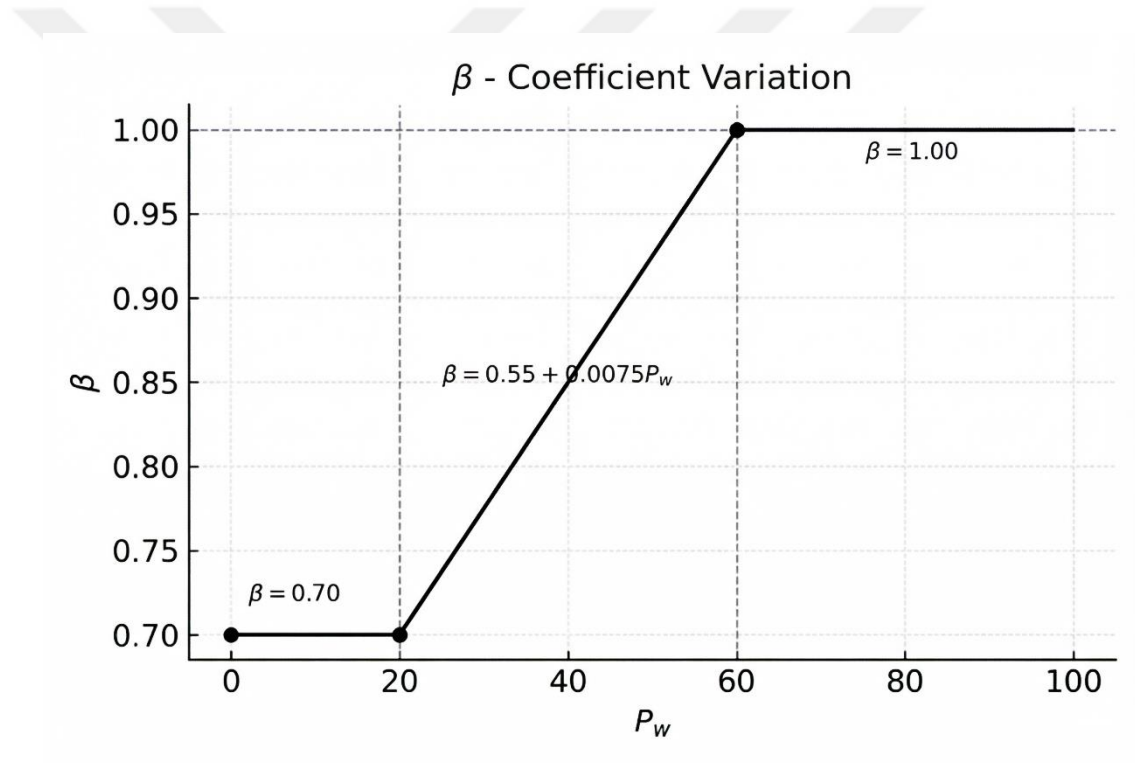


Figure 2.36: β – correction factor.

This approach allows a more favorable assessment of buildings with higher average scores, while enforcing conservative evaluation for low-score cases.

2.3.2.8 Calculation of the Final Score

In the P-25 method, the final performance score (P) for each building is obtained by multiplying the previously defined α and β correction factors with the minimum evaluation score (P_{min}) in equation (2.26).

$$P = \alpha * \beta * P_{min} \quad (2.26)$$

Where:

β is the correction factor derived from the relative weighting of structural parameters,

α is the correction factor accounting for topography, seismicity, building importance, and load participation,

P_{min} is the minimum value among the scores P_1 – P_7 .

This formulation yields a numerical value representing the overall seismic risk level of the building. As the value of P decreases, the building is considered more seismically vulnerable.

The final score obtained using the P-25 method provides a relative numerical scale for seismic performance comparison among buildings. However, the original source of the method does not explicitly define an absolute “safe–unsafe” threshold. The P-25 method was primarily developed to identify relatively high-risk buildings within a building stock and to support intervention prioritization. Nevertheless, comparisons with field observations and comprehensive performance analyses (Yakut, 2004; Al-Nimry et al., 2015) suggest that certain score ranges may be indicative of risk levels, although these ranges vary depending on building characteristics such as height and irregularity conditions.

2.4 PERA2019 Method

2.4.1 Data Collection and Preliminary Assumptions

The first stage of the calculations within the PERA method is the collection of basic building data. During the field survey, the following parameters are determined:

- Number of stories
- Floor plan dimensions
- Story heights
- Cross-sectional dimensions and locations of columns and shear walls at the critical story
- Slab type and typical beam cross-sectional dimensions
- Concrete compressive strength (determined by Schmidt hammer)
- Reinforcement type (plain/deformed), transverse reinforcement spacing, and longitudinal reinforcement ratio
- Plan and vertical irregularities
- Reinforcement diameter losses due to corrosion
- Seismic demand determined considering soil conditions

Under these preliminary assumptions, all structural parameters required for a rapid and reliable assessment of the building behavior are obtained. For example, for a 6-story building, the period is approximately assumed as quation (2.27).

$$T = 0.2 * (\text{Number of Stories}) \quad (2.27)$$

Accordingly, the period is taken as $T = 1.2$ s.

2.4.2 Calculation of Base Shear Force

Based on the collected data, the base shear force acting on the structure is calculated in the second stage. In this calculation, the total mass of the building and the spectral acceleration value are used. The structural behavior factor is taken as $R = 1$. The spectral acceleration is determined using the period of the building in equation (2.28).

$$V_{(t,E)(x)} = 0.85 * m_t * S_{(ae)}(T_p(x)) \geq 0.4 * m_t * I * S_{DS} * g \quad (2.28)$$

where

m_t denotes the total mass of the building,

$S_{a,e}(T_p(x))$ is the spectral acceleration obtained from the structural period,

I is the importance factor,

S_{DS} is the short-period design spectral acceleration coefficient, and

g is the gravitational acceleration.

2.4.3 Distribution of Base Shear Force to Vertical Structural Elements

Following the determination of the base shear force, the next step is to distribute this force among the vertical load-bearing elements of the structure (columns and shear walls). In the PERA2019 method, this distribution is performed only at the critical story. Although the critical story is typically the ground story, an intermediate story with irregularities such as a soft story or overhang may also be selected as the critical story.

The total base shear force V_t calculated for the critical story is distributed in proportion to the stiffness of columns and shear walls. This distribution is performed using Equation (2.29):

$$V_i = V_t * \frac{\frac{I}{L_n^3}}{\sum \left(\frac{I}{L_n^3} \right)} \quad (2.29)$$

where

V_i is the shear force transferred to the i -th vertical element,

V_t is the total base shear force at the critical story,

I is the moment of inertia of the column or shear wall cross section, and

L_n is the net height of the column.

This approach is based on the flexural stiffness of elements (EI / L^3). Elements with higher stiffness attract a larger portion of the seismic shear force, while less stiff elements carry a smaller portion. Thus, the total base shear force is distributed among vertical structural elements using a realistic stiffness-based approach.

2.4.4 Calculation of Column and Shear Wall Moments (Muto and Smith Methods)

Based on the earthquake-induced shear forces transferred to the vertical elements at the critical story, column and shear wall end moments are calculated using either the Muto or Smith methods. Both approaches are fundamentally based on the following equation (2.30):

$$M_e = V_{e,i} * L_n * y \quad (2.30)$$

where

$V_{e,i}$ is the shear force transferred to the i-th element,

L_n is the net story height at the critical story (free height of the column or wall), and

y is the moment arm coefficient.

The value of y is selected from **Table 2.20** depending on the number of stories and the relative stiffness ratio. In the calculations, typical beams are assumed to have a span of 5 m and cross-sectional dimensions of 30×60 cm.

Table 2.20: y -coefficient for triangular distribution of lateral loads.

Number of Stories	0.10	0.20	0.30	0.40	0.50	0.60	0.70	0.80	0.90	1.00	2.00	3.00	4.00	5.00
1	0.80	0.70	0.70	0.65	0.65	0.60	0.60	0.70	0.70	0.60	0.55	0.55	0.55	0.55
2	0.95	0.80	0.75	0.70	0.65	0.65	0.65	0.60	0.60	0.60	0.55	0.55	0.55	0.55
3	1.00	0.85	0.80	0.75	0.70	0.70	0.70	0.65	0.65	0.60	0.55	0.55	0.55	0.55
4	1.10	0.90	0.80	0.75	0.70	0.70	0.70	0.65	0.65	0.60	0.55	0.55	0.55	0.55
5	1.20	0.95	0.80	0.75	0.70	0.70	0.70	0.65	0.65	0.65	0.55	0.55	0.55	0.55
6	1.20	0.95	0.85	0.80	0.75	0.70	0.70	0.65	0.65	0.65	0.55	0.55	0.55	0.55
7	1.20	0.95	0.85	0.80	0.70	0.70	0.70	0.65	0.65	0.65	0.55	0.55	0.55	0.55
8	1.20	1.00	0.85	0.80	0.70	0.70	0.70	0.65	0.65	0.65	0.55	0.55	0.55	0.55
9	1.20	1.00	0.85	0.80	0.75	0.70	0.70	0.65	0.65	0.65	0.55	0.55	0.55	0.55

2.4.4.1 Muto Method

In the Muto approach, the moment arm coefficient y is determined based on the inverted triangular lateral load distribution along the building height, the number of stories, and the relative stiffness of the element in plan. The relative stiffness ratio $\bar{k}$ is defined using the stiffness of adjacent frames (k_1, k_2) and the stiffness of the column or shear wall (k_c):

For an interior column in equation (2.31):

$$\bar{k} = \frac{k_1 + k_2}{k_c} \quad (2.31)$$

For an edge or corner column in equation (2.32):

$$\bar{k} = \frac{k_{adjacent}}{k_c} \quad (2.32)$$

Based on the selected $\bar{k}$ value and number of stories, the y coefficient is obtained and the moments are calculated using equation (2.33)

$$M_e = V_{e,i} * L_n * y \quad (2.33)$$

2.4.4.2 Smith Method

In the Smith approach, the moment arm coefficient is taken as a constant. For inverted triangular lateral load distribution, the moment arm for columns or shear walls at the critical story is approximately assumed as half of the story height in equation (2.34).

$$y \approx 0.50 \rightarrow M_e = V_{e,i} * L_n * 0.50 \quad (2.34)$$

This method is faster than the Muto approach; however, it does not explicitly account for plan stiffness distribution or variations due to the number of stories. Therefore, it is preferred for rapid screening of large building inventories.

2.4.4.3 Application Steps (Summary)

1. The shear forces $V_{e,i}$ distributed to the elements at the critical story are determined using Equation (2.31).
2. The net story height L_n at the critical story is identified.

3. The relative stiffness ratio $\bar{k}$ is calculated (for Muto):
 - a. Interior columns: $(k_1 + k_2) / k_c$
 - b. Edge/corner columns: $k_{\text{adjacent}} / k_c$
4. The y value is selected according to the number of stories and $\bar{k}$ (Muto), or $y = 0.50$ is used (Smith).
5. Column moments are calculated using $M_e = V_{e,i} \cdot L_n \cdot y$.

The calculated M_e values are subsequently used in Section 2.4.4.1 for the determination of tensile and compressive axial forces in exterior columns due to the base overturning moment (TDM).

2.4.5 Calculation of Axial Forces Due to Vertical Loads

In the PERA method, axial forces in columns resulting from vertical loads are distributed proportionally according to weighting coefficients based on the column's position in the plan, as illustrated in **Figure 2.37** and **Figure 2.38**. The column labeling is defined as follows:

Columns on the first/last grid line in the X direction: x_1, x_2

Columns on the first/last grid line in the Y direction: y_1, y_2

Interior (middle) columns: O

Exterior corner columns at intersections, e.g., x_1y_1, x_2y_1 , etc.

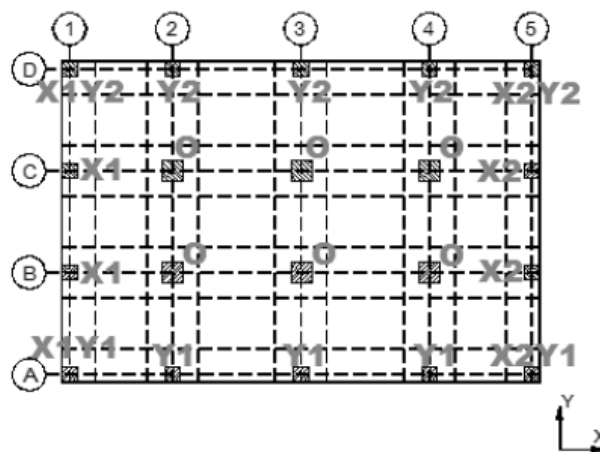


Figure 2.37: Naming of columns according to their position within the building in the PERA method

The position-based weight (distribution) coefficients (s) are defined as follows:

- Corner columns: $s = 1$
- Edge columns: $s = 2$
- Interior (middle) columns: $s = 4$

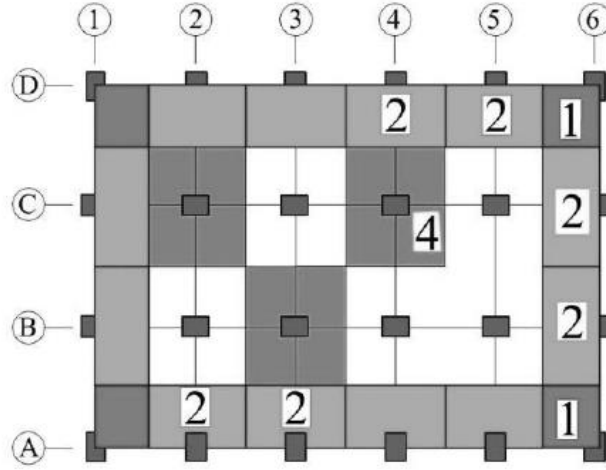


Figure 2.38: Coding of columns according to their position within the building in the PERA method

After obtaining the total building vertical load W (dead load + reduced live load), the axial force N_{G+Q} induced by the vertical load assigned to each column is calculated by Equation (2.35):

$$N_{G+Q} = \left(\frac{s}{\sum s} \right) * W \quad (2.35)$$

Here:

- N_{G+Q} : axial force induced by vertical loads assigned to the relevant column
- s : distribution coefficient of the relevant column (1/2/4)
- $\sum s$: sum of coefficients for all columns at the critical story
- W : total building vertical load (in practice, PERA assumes $G + nQ \approx 12 \text{ kN/m}^2$ and determines W based on the story area)

Application steps

1. Naming and classification of columns: According to **Figure 2.37**, the codes x_1 , x_2 , y_1 , y_2 , and O are assigned; according to **Figure 2.38**, each column is assigned the coefficient s (1/2/4).

2. Total vertical load W : Calculated using story areas and the assumed unit load (in PERA field applications, the unit load is taken as 12 kN/m^2).
3. Calculation of Σs : The s values of all columns at the critical story are summed.
4. Axial force per column: N_{G+Q} is obtained using Equation (9).

This stage covers only the axial forces induced by vertical loads. In the next stage, additional tensile/compressive axial forces that develop in exterior columns due to the base overturning moment (TDM) caused by horizontal earthquake effects will be calculated and considered in addition to the N_{G+Q} values obtained here (interior columns are outside the scope of the TDM effect calculation).

2.4.5.1 Effect of Base Overturning Moment on Column Axial Forces

In addition to axial forces induced by vertical loads, horizontal forces originating from the earthquake also lead to additional axial forces in columns. Horizontal earthquake forces exhibit an inverted triangular distribution along the building height, as shown in **Figure 2.39**, and the resultant of these forces acts approximately at the height of $(2/3 H_n)$. Consequently, the horizontal loads acting on the structure generate a base overturning moment (TDM) that tends to overturn the lateral load-resisting system.

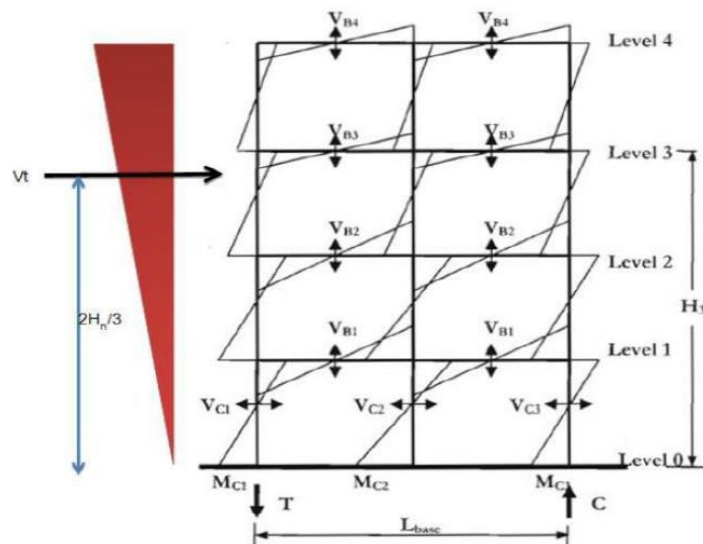


Figure 2.39: Distribution of TDM-induced axial forces in columns.

As a result of this overturning effect, tensile (T) and compressive (C) forces develop in columns located on the exterior grid lines, while the axial forces in interior columns are neglected. These additional axial forces are added to the axial forces induced by vertical loads calculated previously, thereby yielding the total column axial forces.

In the PERA method, the calculation of earthquake-induced axial forces in exterior columns is performed through TDM using Equation (2.36) and Equation (2.37):

$$TDM = V_t * \left(\frac{2}{3} * H_n \right) \quad (2.36)$$

$$T = -C = \frac{TDM - \Sigma M_{e,i}}{L_{taban}} \quad (2.37)$$

Here:

- V_t : total base shear force at the critical story
- H_n : total height of the building
- ΣM_{ei} : sum of column/shear wall moments (see Section 2.X.4)
- L_{taban} : distance between exterior grid lines at the building base
- T : tensile force developed in exterior columns
- C : compressive force developed in exterior columns

In **Figure 2.39**, a schematic representation of the tensile and compressive forces generated in exterior columns by the overturning moment is provided. As can be seen from this schematic, due to the TDM effect, columns on one side are subjected to tension while columns on the opposite side are subjected to compression.

2.4.6 Calculation of Shear Force Capacity in Columns

In calculations related to the shear force capacity of columns, it is necessary to compute the moment capacities (M_{ri} and M_{rj}) of the beams connected to the relevant column. For interior columns, the column end moments are determined according to Equation (2.38) by taking the average of the moment capacities of the beams connected to the column in two directions (depending on the earthquake direction).

$$M_{uc (alt-ust)} = \frac{M_{rj} + M_{ri}}{2} \quad (2.38)$$

By dividing the sum of the moments at the bottom and top ends by the net column height, the shear force (V_e) is obtained according to Equation (2.39).

$$V_e = \frac{M_{ust} + M_{alt}}{L_n} \quad (2.39)$$

For corner columns, since the column is connected to beams only in one direction, half of the moment capacity of the connected beam (depending on the direction of earthquake action) is considered as the column bottom and top end moments.

$$M_{uc (alt-ust)} = \frac{M_{rj}}{2} \quad (2.40)$$

The sum of the obtained end moments is divided by the net column height according to Equation (2.39) to compute V_e . If the column is directly supported by the foundation, the bottom-end moment is determined by calculating the column's moment capacity.

In the PERA2019 method, the moment capacities of beams (M_{ri} , M_{rj}) are determined based on cross-sectional dimensions and reinforcement class. In this context, beams are typically assumed to have a 5 m span and cross-sectional dimensions of 300×600 mm. If the reinforcement class is S220, the positive and negative moment capacities are taken as 120 kNm and 160 kNm, respectively; if the reinforcement class is S420, these capacities are taken as 210 kNm and 290 kNm. Depending on whether the earthquake acts from the left or the right, these moment capacities are considered accordingly.

The calculated V_e value is compared with V_r , which is computed using the relations defined in TS500 Section 8.1. If $V_e / V_r > 1$ (i.e., $r_1 > 1$), the column is evaluated as being in the collapse region. If $V_e / V_r < 1$ ($r_1 < 1$), then the damage region is determined depending on the demand/capacity ratio and the confinement condition. According to TS500, shear capacity of reinforced concrete elements is calculated considering both concrete and transverse reinforcement contributions. By summing 80% of the concrete shear capacity and the contribution of stirrups, V_r (i.e., shear capacity) is obtained. Equation (2.41-2.43)

$$V_r = 0.8V_c + V_w \quad (2.41)$$

$$V_c = 0.65 * f_{ctd} * b_w * d * \left(1 + \gamma * \left(\frac{N_d}{A_c} \right) \right) \quad (2.42)$$

$$V_w = \left(\frac{A_s}{s} \right) * f_{ywm} * d \quad (2.43)$$

Here:

- f_{ctd} : axial tensile strength of concrete
- b_w : column width
- d : effective depth of the column
- γ : coefficient (0.07 under axial compression, and -0.3 under axial tension)
- N_d : axial force (shall be taken as positive in both tension and compression)
- A_c : cross-sectional area
- A_s : total cross-sectional area of shear reinforcement
- s : stirrup spacing
- f_{ywm} : yield strength of transverse reinforcement

2.4.7 Calculation of Column Moment Capacity

In the PERA2019 method, the column moment capacity (M_r) is determined depending on the assumed longitudinal reinforcement distribution in the column cross section. As a result of investigations conducted by İlki et al. (2014) on a total of 149 buildings and 912 column sections in Kocaeli, Van, and Istanbul, the reinforcement layout used in column cross sections was statistically identified, and the reinforcement distribution shown in **Figure 2.40** was proposed. According to this arrangement, 40% of the longitudinal reinforcement is placed in the bottom and top regions of the column, and 20% is placed in the middle region.

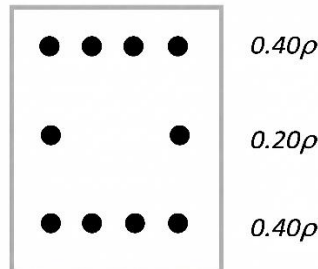


Figure 2.40: Longitudinal reinforcement distribution adopted in the PERA method.

During the calculation process, the axial force value determined within the scope of Section 2.X.5 is used to determine the following points for the column with the relevant reinforcement arrangement:

- Pure axial load capacity $\rightarrow$ Point 1,
- Moment capacity at the balanced condition $\rightarrow$ Point 2,
- Moment capacity under pure flexural effect $\rightarrow$ Point 3,
- Axial load capacity under tensile force $\rightarrow$ Point 4.

These values are marked on the axial force–moment interaction diagram. Based on the values calculated for the four conditions mentioned above (with intermediate values assumed to vary linearly), a simplified axial force–moment interaction diagram as shown in **Figure 2.41** is constructed.

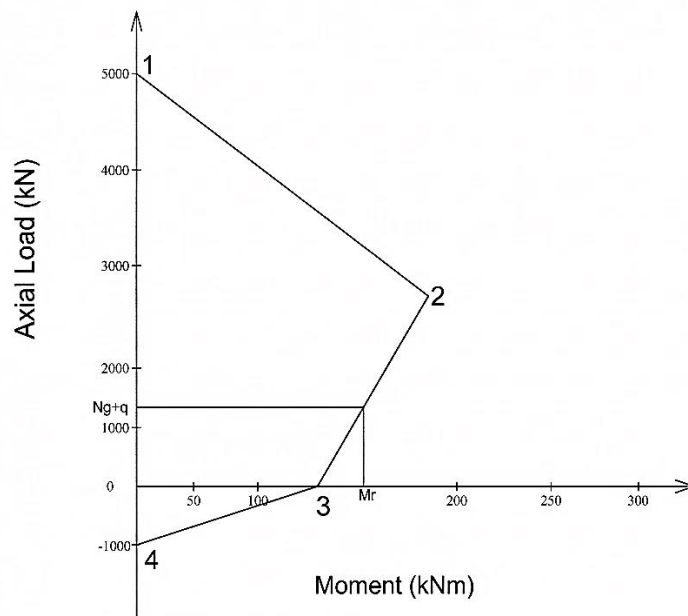


Figure 2.41: N–M interaction diagram.

After the axial force calculated according to Section 2.X.5 is marked on the vertical axis of the constructed diagram, a line parallel to the horizontal axis is drawn until it reaches the curve shown in **Figure 2.41**. From the intersection point obtained, a line is dropped downward, i.e., perpendicular to the horizontal axis. Thus, the M_r value is read on the horizontal axis.

With this method, the realistic moment capacity of the column is determined by considering the section geometry, reinforcement ratio, and material properties. Within

the scope of the PERA2019 method, the behavior of column and shear wall elements under earthquake effects is evaluated in an integrated manner by considering shear force, moment capacity, and axial loads. In this context, the base shear force is first calculated and distributed to the vertical load-bearing elements at the critical story. Subsequently, the moments, shear forces, and load-carrying capacities in columns and shear walls are determined.

The shear and moment capacities of columns are evaluated based on TS500 provisions using interaction diagrams. In addition, the limit values used in determining the damage levels of columns are given in **Table 2.21**. In determining the damage levels of columns, the demand/capacity ratios ($r_1 = V_e/V_r$, $r_2 = M_e/M_r$) are used to establish safety limits. With this approach, both shear and flexural behaviors are considered, and a comprehensive assessment of the earthquake performance of columns is achieved.

Table 2.21: Demand/capacity ratio limits for reinforced concrete columns.

N_k / A_{efcm}	Confined	V_e / bdf_{cm}	SH	KH	GÖ
≤ 0.1	Yes	≤ 0.65	3	6	8
≤ 0.1	Yes	≥ 1.35	2.5	5	6
≥ 0.4 and ≤ 0.7	Yes	≤ 0.65	2	4	6
≥ 0.4 and ≤ 0.7	Yes	≥ 1.35	1.5	2.5	3.5
≤ 0.1	No	≤ 0.65	2	3.5	5
≤ 0.1	No	≥ 1.35	1.5	2.5	3.5
≥ 0.4 and ≤ 0.7	No	≤ 0.65	1.5	2	3
≥ 0.4 and ≤ 0.7	No	≥ 1.35	1	1.5	2
> 0.7	-	-	1	1	1

2.4.8 Calculation of Relative Interstory Drifts

In the PERA method, the calculation of relative interstory drifts is performed in parallel with TBEC 2018 principles. The relative interstory drift at the ground story is calculated using Equation (2.44), and the obtained values are compared with the code-defined limits to determine the damage states of the elements.

$$\delta = V_t * \frac{L^2}{12 * E_c * (\sum D_i)} \quad (2.44)$$

Here, δ/L denotes the drift ratio, E_c denotes the modulus of elasticity of concrete, and D_i is calculated using Equation (2.45) and represents the stiffness-related parameter associated with column stiffness. The stiffness parameter is defined for fixed-base columns according to the Muto method using Equation (2.46):

$$D_i = a * k_c \quad (2.45)$$

$$a = \frac{0.5 + \bar{k}}{2 + \bar{k}} \quad (2.46)$$

The modulus of elasticity of concrete is determined by the relation given in TS500:

$$E_c = 14000 + 3250 * \text{sqrt}(f_{ck,j}) \quad (2.47)$$

The calculated drift ratio is compared with the limit values given in **Table 2.22**, and the damage state of the structure is determined depending on these values.

Table 2.22: Interstory drift limit values for reinforced concrete structures.

LD	CD	CP
0.01	0.03	0.04

During the evaluation of drifts, the existing irregularities in the structure are also considered. In the PERA method, reduction coefficients are used for irregularity types A1–A3 and B1–B3 defined in TBEC 2018 (**Table 2.23**). If more than one irregularity exists in the structure, the coefficients are multiplied to obtain the total reduction coefficient.

For example, if a structure exhibits both B2 (Soft Story) and A3 (Plan Projections) irregularities, the coefficients of both irregularities are multiplied to calculate the total reduction coefficient:

$$0.85 \times 0.95 = 0.81$$

In this case, the calculated moment and shear capacities of columns are reduced by multiplying them by the obtained coefficient of 0.81. Thus, the adverse effects of irregularities on structural performance are taken into account.

Table 2.23: Reduction coefficients for structural irregularities.

Irregularities	Reduction Coefficient
A1: Torsional Irregularity	0.85
A2: Diaphragm Discontinuity	0.95
A3: Presence of Plan Projections	0.95
B1: Weak Story	0.95
B2: Soft Story	0.85
B3: Vertical Element Discontinuity	0.95

The PERA2019 method enables a holistic consideration of shear forces, moment capacities, axial loads, and relative interstory drifts in order to rapidly and reliably evaluate the earthquake performance of reinforced concrete buildings. Within the method, the base shear force is first determined and distributed to the vertical load-bearing elements at the critical story. Then, the moments, shear forces, and axial loads occurring in columns and shear walls are calculated and compared with the corresponding capacity values. In this context, shear and moment capacities are evaluated by considering TS500 provisions, and the damage levels of elements are determined based on demand/capacity ratios ($r_1 = V_e/V_r$, $r_2 = M_e/M_r$) and relative interstory drifts.

As a result of these evaluations, the damage regions of elements can also be defined through the internal force–deformation relationship. As shown in **Figure 2.42**, structural elements sequentially reach the regions of Limited Damage (LD), Controlled Damage (CD), and Pre-Collapse (CP); beyond these limits, the collapse region is reached. This approach is consistent with the capacity-based assessment used in the PERA2019 method and allows the performance of elements under earthquake effects to be interpreted both visually and analytically.

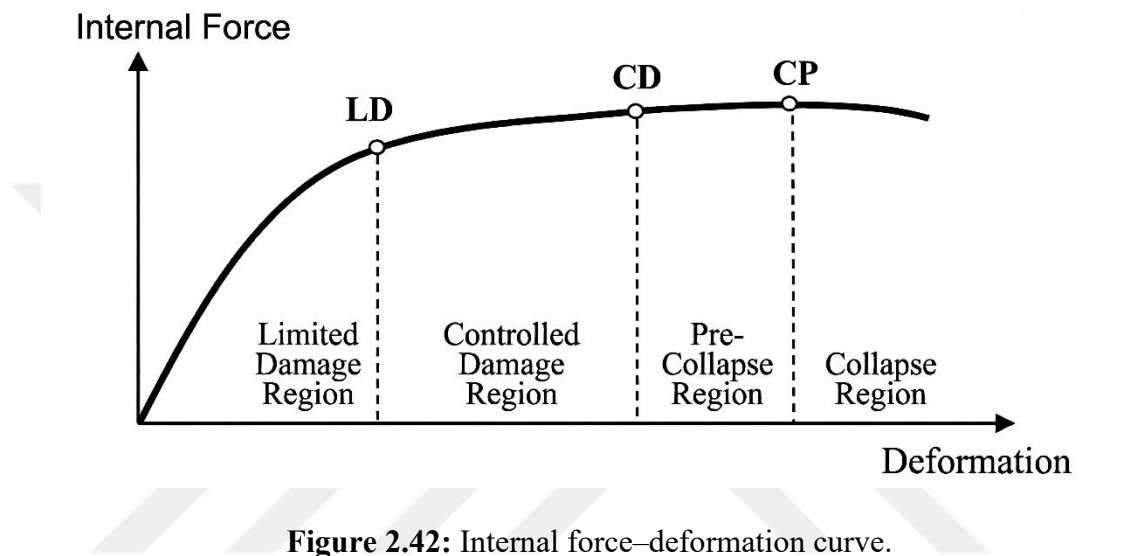


Figure 2.42: Internal force–deformation curve.

Consequently, in the PERA2019 method, the performance of column and shear wall elements is not based on a single parameter; rather, it is established through the combined evaluation of shear, moment, and drift demands. Thus, if high risk is observed in one or more elements at the critical story, the entire structure is classified in the high-risk category. This approach is important particularly for the screening of large building stocks due to its rapid applicability and its ability to produce results grounded in engineering realism.

3 NEWLY DEVELOPED RAPID SEISMIC RISK ASSESSMENT METHOD

3.1 Definition of the RSRA Method

The Rapid Seismic Risk Assessment (RSRA) method has been developed to assess the seismic safety of the existing building stock in a fast, practical, and reliable manner. In particular, the method enables institutions with large building inventories to prioritize buildings under time and resource constraints, and it aims to provide a preliminary evaluation of the performance of the structural system based on field observations and limited laboratory data.

The parameters considered within the scope of the RSRA method are similar to the criteria used in comparable approaches reported in the literature (e.g., FEMA P-154, the Canadian Seismic Screening method). Each parameter is evaluated within a predefined score range according to its impact on building performance; these scores are then combined through weighting coefficients to obtain a final risk score for the building. Accordingly, it becomes possible to perform a relative ranking among a large number of buildings in terms of risk level.

3.1.1. Development Process of the RSRA Method

The Rapid Seismic Risk Assessment (RSRA) method is a practical, field-data-driven tool for risk identification and intervention prioritization, developed for circumstances in which a large number of buildings must be evaluated with limited resources. It was initially designed for institutional buildings with extensive inventories, such as bank buildings, and over time it has been strengthened scientifically through field experience and has undergone many updates.

While the method initially performed risk classification using only superficial information such as building age and occupancy, it later evolved into a multidimensional evaluation system extending from material properties to an equivalent system model. Each new version was shaped by technical needs encountered in the field, and the method has been continuously updated.

When updating the method, new features were introduced in a manner that would not alter the comparability criteria of the RSRA method. To this end, the score of the category to which a new feature was added was either reduced by multiplying by a coefficient or limited by upper bounds. For example, when the development length/hook detail was added under the materials heading, the resulting category score was reduced by a factor of 0.75 so that the total contribution of the materials category would not become disproportionately high and overshadow other features.

RSRA Version (**Table 3.1**) Developments:

- V1 (2020): Provided a basic classification based only on building age, number of storeys, and occupancy type. It began as an office-based pre-evaluation system that did not require any field inspection.
- V2 (Late 2020): The first observation-based field forms were developed to identify the structural system. Reinforced concrete, masonry, and mixed systems began to be distinguished. The short-period design spectral acceleration coefficient (SDS), depending on soil type, was also considered.
- V3 (2021): The building period was computed using the empirical relationships in Turkish Building Earthquake Code (TBEC 2018), and the spectral acceleration value (S_{ae}) was added to the scoring. In this way, resonance behavior of buildings with different heights was considered. In addition, a Seismicity-Soil Type score was defined based on the distance of the period from the peak of the spectrum.
- V4 (Mid 2021): Because carbonation risk increases in older buildings located in hot and arid climates and the Schmidt hammer was not considered sufficient, core samples began to be taken to determine concrete strength more reliably.
- V5 (2023): Following the 6 February earthquakes, the significant effects of vertical acceleration were clearly observed; in this context, load combinations in column capacity calculations were revised to account for the multiplier of vertical acceleration on dead load (details are explained in Section F).
- V6 (Late 2023): Extremely large and irregular aggregates identified in the field (6-9 cm in diameter) became a direct reason to classify the building as risky due to bond problems. In such cases, the building was abandoned by assigning a score of 900+ without considering any other criteria. In addition, the upper

limit of 6 in the materials score was removed and irregularity criteria were expanded (e.g., slabs at different elevations, excessively sloped ground).

- V7 (Early 2024): Element-based demand-to-capacity ratios obtained from gravity-load analyses on an equivalent system model generated from field-average data began to be compared with the damage limits in TBEC 2018.
- V8 (Late 2024 - Current): Linear performance analyses were carried out on the equivalent system model. From these analyses, A1 (torsional) and B2 (soft-storey) irregularities were explicitly identified, shear force effects in columns were evaluated, and the performance level of the model was classified according to TBEC 2018. Thus, a direct relationship between the RSRA score and equivalent structural behavior was established.

Table 3.1: Development stages of the RSRA method by versions.

Version	Year / Period	Development Implemented / Feature Added
V1	2020	Basic classification based only on building age, number of storeys, and occupancy type
V2	Late 2020	Structural system definition; creation of the first forms through field observation
V3	2021	Seismic demand score with empirical period + Sae; seismicity-soil score based on distance from spectrum peak
V4	Mid 2021	Addition of core sampling due to carbonation risk
V5	2023	Inclusion of vertical acceleration effects in load combinations after the 6 February earthquake
V6	Late 2023	Direct classification as risky for concrete with highly irregular aggregate; removal of upper limit on materials score; addition of new irregularity criteria

Table 3.1 (continued): Development stages of the RSRA method by versions.

Version	Year / Period	Development Implemented / Feature Added
V7	Early 2024	Comparison of demand/capacity ratios in the equivalent system model with damage limits
V8	Late 2024	Linear performance analysis on the equivalent model; identification of A1-B2 irregularities and performance level

3.2 Differences of the RSRA Method from Existing Methods

Compared with other rapid assessment approaches such as FEMA P-154, the Canadian Seismic Screening method, and the P-25 method, the RSRA method provides a more comprehensive procedure based on structural performance. While existing methods typically operate with scoring systems based on field observations focusing on basic parameters such as exterior appearance, occupancy, and number of storeys, the RSRA method aims to deliver a more reliable risk evaluation based on numerical computations by enabling an approximate structural model using data collected from the building.

Within the scope of the RSRA method presented in this thesis, the following procedures are carried out differently from other methods:

- Building modeling: A “simplified model” is prepared, limited by the building footprint area and height, having an axis spacing close to the average spacing of the building, and having member dimensions/reinforcement layouts consistent with samples obtained from the field. The OpenSees-based simplified model is analyzed to obtain element demand/capacity ratios and material deformations, which are used in various evaluations.

- Static analysis: Using the mathematical model, the modal parameters of the structure, flexural/axial/shear capacities are computed. Demand/capacity ratios are determined for each element from the obtained results.
- Performance analysis: A linear analysis method provided in TBEC 2018 is used to evaluate performance.
- Structural irregularities: In addition to visually identified irregularities, interstory drift and torsional irregularities are also checked numerically using the simplified model.
- Material tests: Concrete strength is determined by extracting core samples; reinforcement is investigated via X-ray scanning and cover removal tests.
- Reinforcement analysis: Stirrup diameters and spacing, development lengths, and reinforcement ratios are checked.
- Determination of seismic demand: After analyzing the OpenSees-based simplified model, the period of the building is computed. Spectral acceleration coefficients corresponding to the computed period are obtained and the lateral seismic forces/base shear acting on the building are determined.

3.3 Comparison of Rapid Assessment Methods

In the literature, various rapid screening methods have been developed to evaluate earthquake-prone buildings in a short time. Some of these methods rely solely on visual evaluation, whereas others include limited sampling from the building. However, each method differs in scope, data collection procedures, and sensitivity in risk identification.

The Rapid Seismic Risk Assessment (RSRA) method proposed in this study provides a more comprehensive approach based on material properties and structural system components. Because the quality of materials and workmanship varies significantly in the building stock of Türkiye, the developed RSRA method examines buildings in more detail than other methods particularly in terms of material and workmanship. Data obtained from field tests and observations are converted into quantitative measures and used to compute risk scores.

In the table below, the RSRA method is compared with FEMA P-154, the Canadian Seismic Screening method, the P-25 method, and the Pera2019 method. **Table 3.2**

summarizes the purpose, data sources, scope, advantages, and disadvantages of each method and highlights the prominent aspects of RSRA.

Table 3.2: Comparative overview of RSRA, FEMA P-154, Canadian Seismic Screening, P-25, and Pera2019

Comparison on Criterion	RSRA Method	FEMA P- 154	Canadian Seismic Screening	P-25 Method	Pera2019
Objective	Rapid prioritization and field-based risk score	Preliminary screening through visual evaluation	Prioritization and risk ranking of buildings	Rapid risk identification (especially for public buildings)	Risk assessment based on shear, moment, and drift behavior of columns and walls under earthquake effects
Data Source	On-site core tests, Schmidt hammer, cover removal tests, reinforcement observation, simple measurements	Exterior appearance, number of storeys, plan configuration, structural type	Plan information, building age, structural system type	Observational data plus some engineering estimates	On-site measured section dimensions, reinforcement detection, Schmidt test-based concrete strength, and code-based limits

Table 3.2: Continued.

Scope	Material data such as concrete quality, reinforcement detailing, stirrup spacing	Building geometry and basic structural system observations	Number of storeys, structural type, irregularities, general building type	Structural system type, number of storeys, presence of irregularities	Moment, shear, and drift capacities of column and wall elements
Time / Cost	Medium - requires field inspection and sampling	Very rapid - observation only	Rapid - observation and information form	Very rapid - questionnaire or short field form	Medium - requires field data, but calculations are more systematic and faster as no nomograms
Scoring System	Detailed, numerical, and weighted parameter scores	Simple 'count the points' approach	Prioritization using scoring tables	Parameter-based scoring system	determined from limit tables using demand/capacity ratios and drift ratio
Risk Classification	Score range 0-900+ -> 6 risk levels	Pre-screening result -> risky/safe distinction	Priority level based on score range	Risk score (high, medium, low)	Risk level according to SH, KH, GO limits

Table 3.2: Continued.

Required Field Expertise	Basic laboratory devices (Schmidt, core drill, etc.)	Observation only	Observation and technical knowledge	Observational and engineering knowledge	Reinforcement detection device, field measurement, concrete strength test (Schmidt)
Advantage	More realistic scoring that considers material performance	Very rapid and simple	Extensive application history; simplified form	Widely used practical system	More reliable risk identification based on material and section geometry
Disadvantage	Requires field equipment and software know-how for material testing and analysis; requires practitioner capability to build a model	Provides no information on material quality	Material quality is not considered	Provides limited information on structural behavior	Requires field data and engineering expertise

3.4 Implementation Steps of the RSRA Method

In this section, the field implementation of the Rapid Seismic Risk Assessment (RSRA) method is described step by step. The method has been developed to assess earthquake risk of buildings rapidly and reliably, and it consists of six systematically defined main stages (A, B, C, D, E, F). These stages enable a holistic evaluation of observational and experimental data obtained from the building. In each step, numerical scoring is carried out based on specific parameters, allowing an objective determination of the building's risk condition.

Each main-stage score is obtained by summing its subcategory scores. The main-stage scores are then multiplied with each other to obtain the RSRA score. Because high scores in different categories increase building risk multiplicatively, category scores are multiplied, while subcategory scores are summed within their respective categories.

The hierarchical structure of the RSRA method is provided in **Figure 3.1**.

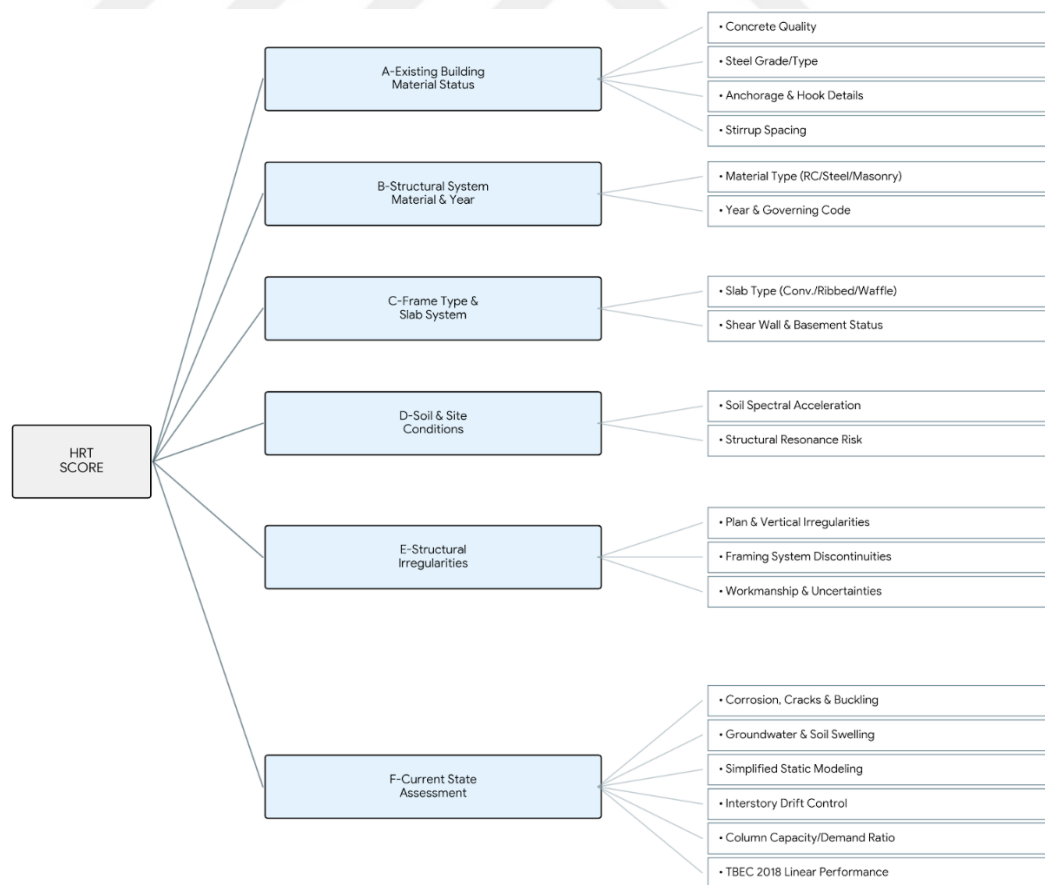


Figure 3.1: Hierarchical structure of the RSRA method.

In the following subsections, these six stages that constitute the application procedure of the RSRA method are presented in detail, and the workflow, tools, and evaluations used in each stage are supported with examples.

3.4.1 Existing Building Material Condition (Section A)

The objective of this section is to evaluate the current condition of the structural system and the materials used in the building on site. Within this scope, the building is first inspected with a general overview; preliminary information is collected regarding the structural system type, number of storeys, building geometry, and potential irregularities. These observations conducted by the responsible engineer in the field are recorded in a standardized field inspection form (Figure 3.2).

Şube İsmi	Rehberlik Kurulu	Yapım Yılı	2018
Kat sayısı(cepho foto)	0+2+6	Bodrum Kat Sayısı	1
Donatı okuması 1(foto)	9 cm	Donatı okuması 2(foto)	11 cm
Boyuna çap(kumpas foto)	BC 11 - 014	Etriye(kumpas aralık foto)	BC 11 08 / 15
Korozyon(foto)	Yok	Kanca(foto)	Var
Bindirme 60cm(foto)	Var	Şişen elm(önce-sonra foto)	Yok
Kademeli yapı(m. foto)	Yok	Döşeme kalınlığı	15 cm
Güçlendirme(m. foto)	Yok	Kısa kolon(foto)	Yok
Zayıf/Nor kat yk.(foto)	Var	Çekiçleme etkisi(foto)	Yok
Çerçeve-Perde (Tek yön, burulma, alansal oran)	Konansiyondan sonra tek yön perde girer	Kiriş Ebatı(foto)	25x50
Düsey süreklilik(m. foto)	Sürekli	Planda Düzensizlik	Yok
Schmidt Okumaları (karot/ çekir foto)	36 32 32 32 36 35 35 38 35 34 32 36	Topografya () Vadi altı () Ova (X) Tepelik	Z

E	Düseyde			Planda			Çekiçleme	Kısa kolon	Saplamalı kırış	Güçlü kolon
	Zayıf kat	Zayıf kat (H>5m)	Düsey elem. sür. (B3)	Burulma (A1)	Düsey sür. (A2)	Çekim (A3)				
Var	0	2,5	1,2	1,3	1,3	1,3	1,3	1,3	1,3	1,3
Yok	0,1	0,1	0,1	0,1	0,1	0,1	0,1	0,1	0,1	0,1

"Donatı pürsantajı <1% / "farklı kotta diyafram" / "fazla eğimli arazi" / "işçilik" / "ağır çıkma" / "tek yön taşıyıcı" / "yüksek-dar yapı" / "yapıya zaman içinde yapılan eklemeler yüzünden çalışma prensibinin belirsiz"

Kolon etkili alanı (ÖNEMLİ NOT: KOLON YÖNLERİ DOĞRU GİRİLMELİ)	
Y: 424	Y: 635
X: 413	X: 581
80x25	40x40
X: 422	X: 0
Y: 345	Y: 407

Yaklaşık statik sistem bina ebadı: 7000x1070 X ve Y aks sayısı: 6 x 2 Mimar: Var

Notlar:	✓	Perde Oranı	kirişli plak	kirişsiz plak	tek yönlü asmolen	kaset
Bodrumsuz	0	2	4	3,6	2	
Bodrumlu	<1/3	0,7	1,4	1,3	0,7	
Bodrumsuz	<1/3	1,4	2,8	2,5	1,4	
Bodrumlu	>1/3	0,5	1	0,9	0,5	
Bodrumsuz	>1/3	1	2	1,8	1	
Tünel Kalıp	0,2	0,4	0,4	0,2		

Notlar: Bodrum ve zemin kat 01molen, 25/150 kırış. Proje 2007 yönceliğine göre bindirme katı kırışında. 25cm radyo temel 014/10. Bodrum cephe girme süren oldegi işi girme kabul yapıldı. Tavan süren tek yön.

Saha Formu Rev.8

Figure 3.2: Field inspection form.

During the inspection, structural elements are observed to identify common damage traces, possible corrosion indicators, short column effects, and the formation of weak/soft storeys. These observations are summarized in the general evaluation section on the first page of the method (**Figure 3.3**).



Figure 3.3: Possible corrosion indicators and short column effects.

Another step to understand material quality and condition is to remove the concrete cover of structural members and examine reinforcement detailing. During this process, reinforcement diameter, spacing, stirrup arrangement, reinforcement grade (BÇI/BÇIII), and corrosion condition are observed. These data provide important indicators regarding adequacy of reinforcement layout and ductility. In some cases, reinforcement detection devices are used to scan reinforcement configuration in columns without cover removal and to perform verification (**Figure 3.4**).



Figure 3.4: Investigation of material strength parameters of the building.

All these data are scored based on four main parameters under the main heading of Section A:

- a. Concrete Strength
- b. Reinforcement Class
- c. Anchorage (Development) Length
- d. Stirrup Spacing

Each parameter is evaluated within a predefined score range according to predetermined criteria. These scores are summed to calculate the Existing Building Material Condition Score, and this total is multiplied by a coefficient of 0.75 to obtain the final score of Section A.

As a result of this stage, the material quality and detailing of the structural system are converted into quantitative data. This data forms the basis for the overall risk assessment carried out in subsequent stages.

3.4.1.1 Concrete Compressive Strength

Under this subsection, the concrete compressive strength obtained from core samples and non-destructive tests conducted using the Schmidt hammer is evaluated. Concrete strength, which is one of the most critical data for determining the material performance of the building, provides direct information on the load-carrying capacity of the existing building and is also used as a fundamental parameter in computing the risk score.

Core tests are performed preferably on samples taken from one or two columns located at the ground floor, and the results are reported by an accredited laboratory. These values are compared with Schmidt hammer measurements performed in the field. Consistency between the two methods provides preliminary information on overall concrete quality and carbonation condition. In some cases, due to owners not allowing core extraction, only non-destructive test results are used (see **Figure 3.5**).



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YAPI MALZEMESİ LABORATUVARI
34469 MASLAK / İSTANBUL TEL: (0212) 285 3757-58 FAX: (0212) 2856587

SERTLEŞMİŞ BETONDAN ALINAN NUMUNENİN BASINÇ MUKAVEMET (KAROT) RAPORU

Başvuru No/ Tarihi	1523369/18.02.2025	Rapor No/Tarih	97/26.02.2025
Alınış Tarihi	-	Bayındırlık Kayıt No.	-
Deney Tarihi	25.02.2025	Lab. Kod No.	-

FİRMA BİLGİLERİ

Proje Adı	Ziraat Bankası Şubeleri
Müteahhit Firma	Milenyum Proje Danışmanlık San. Ve Tic. Ltd. Şti.
Şantiye Adresi	-
Ada No. :-	Pafta:- Parsel:-

NUMUNE BİLGİLERİ

Numunenin Temsil Ettiği Kütle	-	Üretici Firma	-
Adet/Tip	11/ Karot	Beton Dök. Tarihi	-
Boyutlar (mm)	*	Hedeflenen Beton Sınıfı	-

UYGULANAN STANDARTLAR: TS 12390-3

DENEY SONUCU BULUNAN DEĞERLER

Numune No	Numune Alınan Eleman	Yaşı	Çap (mm) (*)	Numune Yüksekliği (mm) (*)	Kırılma Yüklü (kN)	Basınç Dayanımı (N/mm ²)
1	Muğla Ortaca SB01	-	74	79	119,0	27,7
2	Muğla Kavaklıdere SB01	-	"	80	40,0	9,3
3	Muğla Dalaman SZ01	-	"	77	42,0	9,8
4	Isparta Yenişar Bademli SZ01	-	"	80	94,2	21,9
5	Aydın Didim SB01	-	"	80	37,9	8,8
6	Bodrum Ortakent PB01	-	"	79	76,0	17,7
7	Muğla Ula SZ01	-	"	80	121,8	28,3
8	Aydın Efeler SB01	-	"	78	104,5	24,3
9	Isparta Sütçüler SB01	-	"	78	83,1	19,3
10	Marmaris Uzuncalı SB01	-	"	79	118,4	27,5
11	Isparta Gelendost S101	-	"	80	118,2	27,5

- Numuneler yerinden başvuru sahibi firma tarafından alınmış olup, laboratuvarında teknisyen tarafından teslim alınmıştır. Deney sonuçları dışındaki bilgiler (inşaat adresi, karot numarası, numune alınan eleman vb bilgiler) firmanın başvuru dilekçesindeki beyanına göre yazılmıştır. Numunelerin söz konusu projeye ait olup olmadığı konusunda imza sahipleri sorumluluk kabul etmez.

Figure 3.5: Report of core test results.

Within RSRA, the target best concrete quality is 30 MPa. Because the method is designed primarily to evaluate low- to mid-rise buildings, this strength level is assigned the lowest risk score. For 30 MPa and higher concrete quality, the risk score is taken as “1”.

A gradually increasing risk score is defined as concrete quality decreases. The formula of this risk score is given in equation (3.1).

$$SCORE = 0.0042fcd^2 - 0.2542fcd + 4.87 \quad (3.1)$$

According to this scoring system, the variation of the score with concrete quality is shown in **Figure 3.6**.

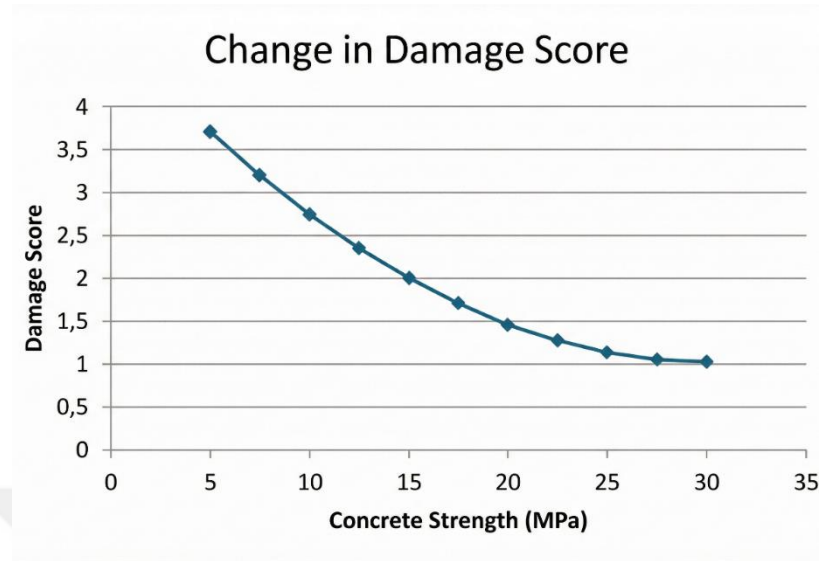


Figure 3.6: Variation of damage score with concrete compressive strength.

With this scoring, the risk score increases at an accelerating rate as concrete quality decreases. In addition, if a specific problem is observed in the concrete during field inspections, the score of this item is increased more than that indicated by the formula (**Figure 3.7**).



Figure 3.7: Concrete samples with irregular aggregate distribution.

3.4.1.2 Reinforcement Class

Reinforcement grade is of critical importance in terms of the ductility level and load-carrying capacity of a structure. This parameter directly affects the structural performance, particularly under earthquake effects, by governing key behaviors of reinforcement such as yield strength and deformation capacity. Within the scope of the RSRA method, the reinforcement grade is determined by identifying the diameters and types of the reinforcement exposed by cover removal from selected columns during field inspections.

Reinforcement grade is generally classified into two basic categories, namely S220 and S420. These classes primarily indicate the yield strength level of the reinforcement. While S220-class bars have lower yield strength, S420-class bars provide higher strength. In older buildings, S220-class reinforcement is encountered more frequently, whereas S420-class reinforcement is common in newer buildings. Determination of reinforcement grade provides critical data for evaluating the overall strength capacity of the building's structural system.

Reinforcement grade not only defines yield strength, but also indicates the production method of reinforcement. Plain (non-deformed) bars may be fabricated under workshop conditions using scrap steel; in contrast, deformed (ribbed) bars are typically factory-produced. For this reason, the risk score associated with plain bars is higher.

Within RSRA, fixed scores are assigned according to reinforcement grade, as shown in **Table 3.3**. If reinforcement in the building is of grade S220, a score of 2 is assigned; if it is of grade S420, a score of 1 is assigned. In some reinforced concrete buildings constructed in the 1990s, longitudinal bars are S420 while stirrups are S220. For such cases, the score is taken as 1.5. These scores are weighted to reflect the influence of material behavior on building safety. The resulting score is summed with the scores from other sub-parameters and contributes to the total score of Stage 1.

Table 3.3: Fixed scores assigned based on reinforcement grade.

Reinforcement Steel	Score
S220	2
S420	1
S220–S420	1.5

Reinforcement grade can be determined either by direct observation through cover removal on site or, if necessary, indirectly with reinforcement detection devices. This assessment provides an important indication regarding stiffness, ductility, and consequently seismic performance.

3.4.1.3 Anchorage (Development) Length and Hook Angle

Adequate anchorage of reinforcement bars by embedding them into concrete over a sufficient length at column ends is of great importance for the strength of the structural system. This requirement is critical for effective force transfer by reinforcement and for preventing brittle failures. Reinforcement with insufficient development length may experience unexpected bond loss during earthquakes, leading to sudden capacity drops.

Similarly, if stirrup and cross-tie hooks are not bent at 135° , confinement may open up and the section may behave in a brittle manner. As a result, the section cannot dissipate the expected energy and may be damaged under cyclic loading.

Deficiencies such as development length and hook angle, as well as similar detailing deficiencies, are considered within a subcategory and scored between 1 and 1.5 as shown in **Table 3.4**. If cover removal is carried out such that the lap splice/development region is expected but the lap length is not observed, or if a 90° hook is observed at the corresponding stirrup corner, the risk score is taken as 1.5.

Table 3.4: Scores assigned according to development length and hook detailing.

Anchorage (Development) Length of Reinforcement	Score
Observed	1
Not observed	1.5

Because development length is a fundamental detailing component that ensures integration of reinforcement with the structural system, deficiencies in this parameter may adversely affect the overall seismic performance of the building. Therefore, its influence on the scoring system has been determined by considering both material safety and strength continuity.

3.4.1.4 Stirrup Spacing

For structural members to exhibit ductile behavior under earthquake effects, an adequate stirrup arrangement is required. Sufficiently tight stirrup spacing increases the deformation capacity of longitudinal reinforcement before buckling and enhances the deformability of concrete, thereby directly contributing to the overall performance of the structural system. For this reason, stirrup spacing is treated as an independent evaluation parameter within RSRA.

During field inspections, stirrup diameter, spacing, and the presence of 135° hooks are observed on the surfaces of stripped columns or beams. Thus, direct information is obtained regarding detailing quality of the structural system. A stirrup spacing around 10 cm is considered ideal, whereas spacing of 20 cm and above is evaluated unfavorably in terms of ductility.

This scoring shown in **Table 3.5** represents the level of detailing of the structural system and, consequently, the expected behavioral pattern. The evaluation of stirrup spacing enables interpretation of the ductility capacity of the building. Moreover, the score obtained from this parameter is directly used in determining the total score under the heading of Existing Building Material Condition.

Table 3.5: Scores assigned according to stirrup spacing.

Stirrup Spacing	Score
25 cm and above	2.5
20–25 cm	2
15–20 cm	1.5
10–15 cm	1
Below 10 cm	0.5

3.4.2 Evaluation Based on Construction Year and Structural System Type (Section B)

In Section B, which constitutes the second stage of the RSRA method, the design quality and code compliance of buildings are evaluated based on their construction year. Because engineering knowledge, building materials, analysis methods, and legal regulations have evolved over time, the construction year of a building is a significant criterion. Therefore, a building constructed in 1975 and a building constructed after 2007 are generally not expected to have the same seismic resistance.

In this section, considering only the construction year, the applicable earthquake code is identified and scoring is performed accordingly. Construction year information is typically obtained from the institution's real estate inventory, building personnel, or the relevant municipality.

In addition, the structural system type (reinforced concrete, steel, timber, masonry) is also considered to determine the final score. For reinforced concrete systems, a more detailed and higher-risk scoring is applied, whereas steel, timber, and masonry systems are evaluated with different coefficients **Table 3.6**.

Table 3.6: Scoring based on construction year and structural system type.

Code Year	Reinforced Concrete	Steel	Timber	Masonry
Before 1975	4	1.2	1.2	3
1975–1998	3	1.2	1.2	2
1998–2007	2	1.2	1.2	1.5
2007 and later	1	1	1	1

Particularly in rural regions, it has been observed that building construction standards are adopted with approximately a 10-year delay compared to large cities. Accordingly, if a building does not reflect the technical characteristics of its construction year, an older construction year may be adopted in risk scoring. For example, continued use of cast-in-place concrete in years when ready-mixed concrete became widespread, or the preference of BÇI reinforcement in a period when BÇIII reinforcement had become standard, can be considered as such cases. These situations typically arise when construction is carried out long after the design date or when construction remains incomplete for a period.

This scoring represents the engineering and regulatory level of the period in which the building was designed. The obtained score contributes directly to the RSRA structural risk score.

3.4.3 Formation of Floor Diaphragm (Section C)

In Section C, which constitutes the third application step of the RSRA method, the structural behavior is analyzed by jointly evaluating the floor system and the basement layout. Within this scope, the formation of rigid diaphragm action and the ability to effectively transfer lateral forces are considered. Since basement shear walls distribute loads to the foundation system and provide favorable seismic behavior, the risk score of buildings with basement shear walls is reduced by half (**Figure 3.8**).

The expressions “With Basement” and “Without Basement” in **Table 3.7** indicate whether the basement storey is surrounded by rigid reinforced concrete walls on at least three sides. If this condition is not satisfied, or if the basement exterior walls consist of weak materials such as brick/stone, the building is evaluated as “Without Basement.” This criterion is an important parameter affecting structural stiffness.

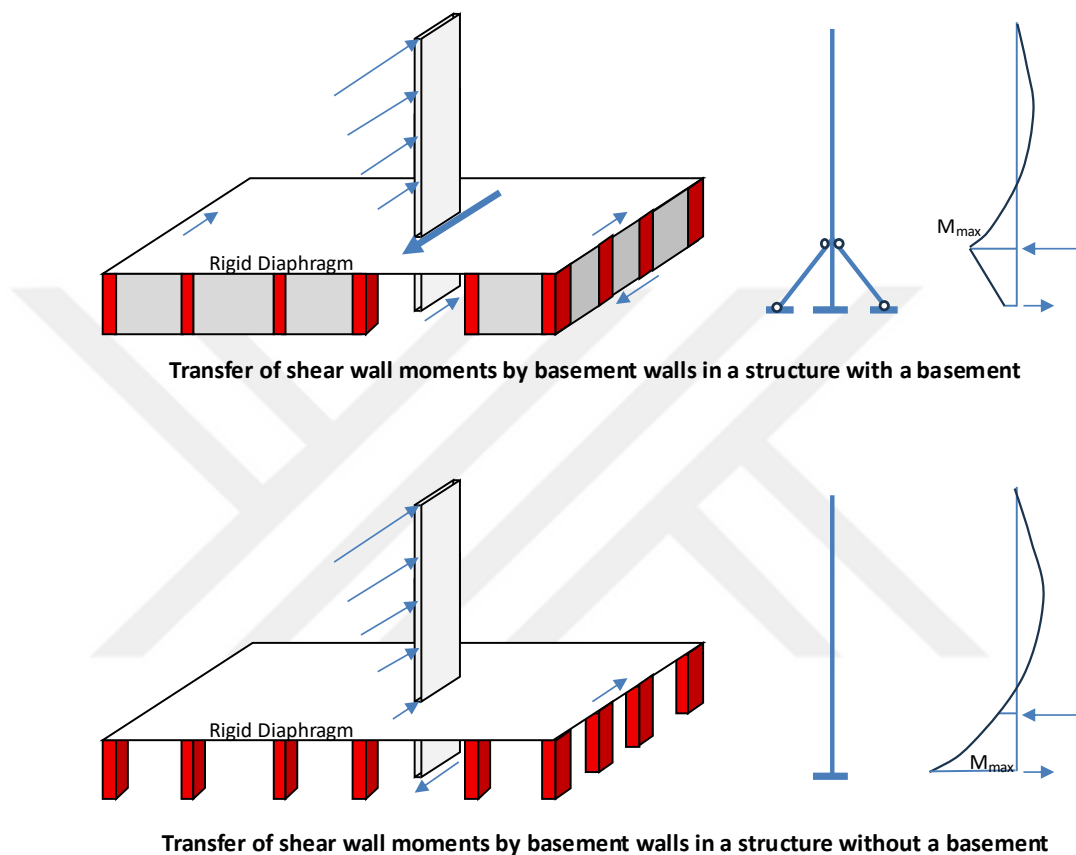


Figure 3.8: Shear wall moments.

Another important parameter, the “Shear Wall Ratio,” indicates the extent to which reinforced concrete shear walls are present in the structural system. Frame-only buildings with no shear walls are accepted as “Shear Wall Ratio = 0,” whereas buildings with shear walls only at the stair or elevator core are considered as “Shear Wall Ratio < 1/3.” If shear walls are used widely throughout the structural system, the ratio is evaluated as “Shear Wall Ratio > 1/3.” This interpretation is based on the field observations of the inspecting engineer, project information, and engineering judgment.

Table 3.7: Scoring based on floor system and shear wall ratio.

	Beam-and-Slab	Flat Slab	One-Way Hollow Block (Asmolen)	Waffle (Coffered)
With Basement Shear Wall Ratio = 0	1	2	1.8	1
Without Basement Shear Wall Ratio = 0	2	4	3.6	2
With Basement Shear Wall Ratio < 1/3	0.7	1.4	1.3	0.7
Without Basement Shear Wall Ratio < 1/3	1.4	2.8	2.5	1.4
With Basement Shear Wall Ratio > 1/3	0.5	1	0.9	0.5
Without Basement Shear Wall Ratio > 1/3	1	2	1.8	1
Tunnel Form	0.2	0.4	0.4	0.2

The floor system plays a critical role in structural behavior under earthquake effects. Beam-and-slab floors provide a better rigid diaphragm effect than other systems and are advantageous in lateral load distribution. In contrast, systems such as hollow block slabs and waffle slabs may exhibit weaker diaphragm action, which may adversely affect lateral load transfer. Flat slab systems are rated as the riskiest floor system because punching shear problems may arise and frame deficiencies may occur due to insufficient effective connectivity between columns; moreover, sensitivity to workmanship errors further increases their risk. Therefore, when evaluating floor type, engineering judgment should be applied by considering not only geometry but also structural effects and strength.

The score obtained by jointly evaluating floor type and shear wall ratio represents an important input for the behavior of the building under lateral loads and affects the RSRA risk score.

3.4.4 Seismicity and Soil Type – Scoring Based on Spectral Acceleration (Section D)

In earlier versions of the RSRA method, the building period was approximately computed using the empirical relationship defined in TBEC 2018 Article 4.7.3.4, and the corresponding spectral acceleration value (S_{ae}) was obtained from a graph. However, the method has evolved over time, and starting with Version 5, a more scientific and computation-based approach was adopted. Accordingly, the engineering calculations required for Stages D, E, and F are performed using an OpenSees-based software developed for this purpose. In this version of RSRA, the approximate periods of buildings are obtained through an equivalent system model created in this software, and S_{ae} is computed more accurately. Using the same model, key parameters such as modal mass participation ratios, natural vibration frequency, and base shear force are also determined. The computed period is used to determine the spectral acceleration coefficient and to calculate the corresponding risk score.

The operating principle of the software-based equivalent system model will be presented in detail under Section 3.6.

The spectral acceleration value $S_{ae}(T)$ is defined as a function of period within four intervals, as given by Equations (3.2)–(3.5):

$$S_{ae}(T) = \left(0.4 + 0.6 \frac{T}{T_A}\right) S_{DS} \quad (0 \leq T \leq T_A) \quad (3.2)$$

$$S_{ae}(T) = S_{DS} \quad (T_A \leq T \leq T_B) \quad (3.3)$$

$$S_{ae}(T) = \frac{S_{D1}}{T} \quad (T_B \leq T \leq T_L) \quad (3.4)$$

$$S_{ae}(T) = \frac{S_{D1} \cdot T_L}{T^2} \quad (T_L \leq T) \quad (3.5)$$

Determination of soil class plays a critical role in defining earthquake effects. In this study, the soil class of some buildings was determined directly by seismic measurements using MASW (Multichannel Analysis of Surface Waves); for some others, USGS (United States Geological Survey) data were used, as shown in **Figure 3.9**.

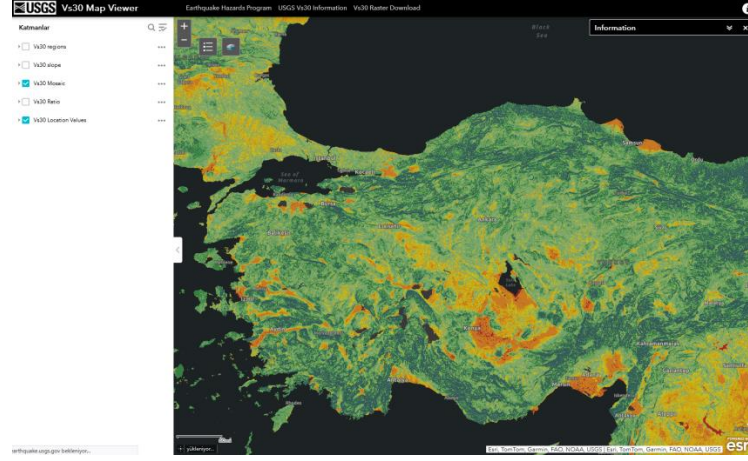


Figure 3.9: Soil class map obtained from USGS.

For the remaining buildings, soil class ZC was assumed based on field observations. For each building, SDS and SD1 values were obtained from the Türkiye Earthquake Hazard Map according to the determined soil class, and these parameters were used directly in computing the elastic design spectrum $S(T)$. After $S(T)$ is computed, the risk score is determined according to Equation (3.6):

$$PUAN: \begin{cases} S(T) \leq 0.5 & \rightarrow 1 \\ 0.5 < S(T) < 1.5 & \rightarrow 1 + 5.0 \times [S(T) - 0.5] \\ S(T) \geq 1.5 & \rightarrow 6 \end{cases} \quad (3.6)$$

Saha Bilgileri		Yapı Davranışı	
Zemin	ZC	Kat Sayısı	3
Sds	0,865	Bodrum	1
Sd1	0,269	Tx	0,089
Ta	0,062	Ty	0,075
Tb	0,311	S(0,08)	0,865
Tl	6	Puan(0,865)	2,83
Tpa	0,428	Puan(0,865)	2,83
Ct-Çerçeve: 0.1		Puan	2,83

Figure 3.10: Spectral acceleration-based scoring formula.

In selecting the period value, the direction in which the structural behavior is more critically demanded (shear force or displacement) was considered; in general, the period corresponding to the more severe risk was adopted. Within RSRA, in order to remain on the safe side, the period corresponding to the larger spectral acceleration value was preferred (see **Figure 3.10**).

3.4.5 Structural Irregularities (Section E)

In the fifth stage of the RSRA method, the objective is to evaluate structural irregularities in the vertical and horizontal directions. In this stage, certain structural irregularities defined in TBEC 2018 are considered, and a presence/absence-based scoring is performed for each irregularity type. These scores reflect potential weaknesses in terms of stiffness, continuity, and behavioral reliability. The main irregularities considered within RSRA, shown in **Table 3.8**, are grouped under two main headings.

Table 3.8: Structural irregularities.

E	Düşeyde			Planda			Çekişleme	Kısa kolon	Saplama kiris	Güçlü Kolon
	Zayıf Kat	(H>5m) Kat	Düşey elem. sifr. (B3)	Burulma (A1)	Düşeme sifr. (A2)	Çıkıntı (A3)				
Var	2	2,5	1,2	1,3	1,3	1,3	1,3	1,3	1,3	1,3
Yok	0,1		0,1	0,1	0,1	0,1	0,1	0,1	0,1	0,1
NOT: “Donatı porsantaj <%1” / “farklı kotta diyafram” / “fazla eğimli arazi” / “işçilik” / “ağır çıkma” / “tek yön taşıyıcı” / “yüksek-dar yapı” / “ortogonal olmayan yapı” / “yapıya zaman içinde yapılan eklemeler yüzünden çalışma prensibinin belirsiz”										

- Vertical Irregularities:
 - Weak storey
 - High weak storey ($h > 5$ m)
 - Vertical element discontinuity (B3)
- Plan Irregularities:
 - Torsion (A1)
 - Diaphragm discontinuity (A2)
 - Plan projections (A3)
 - Short column
 - Stub beam
 - Frame discontinuity
 - Pounding (coexistence/overlap of multiple structural system behaviors)

For each of these irregularities, a fixed score is assigned if present. These scores are generally defined in the range of 1.2–2.5. If an irregularity is absent, 0.1 is assigned for all subcategories; thus, the total score of this section becomes 0.9, resulting in a reduction in the Structural Index Score.

Identification of irregularities is based on analyses of the equivalent system model created using the OpenSees-based software, as well as review of architectural/structural drawings and field observations. While scoring can be performed for all irregularities based on field observations, calculation-based evaluation methods are also applied for certain special irregularities such as “relative interstory drift,” “soft storey,” and “torsional irregularity,” as explained in detail in Section 3.4.6.4.

Special Cases and Notes

In addition to code-defined irregularities, certain special cases that affect structural behavior but are not explicitly defined in the code are also evaluated under this category. For example:

- Elements with reinforcement ratio below 1%
- Floor systems at different elevations
- Buildings founded on excessively sloped ground
- Very tall but narrow buildings
- Structures in which the load-carrying system works predominantly in one direction
- Excessive cantilevers, workmanship errors, or unengineered interventions made later

Such cases are included in the irregularity category, and the relevant explanations are written into the blank columns in the form, increasing the risk score. The irregularity scores obtained at the end of this stage directly affect the overall RSRA risk score. Since a substantial part of these checks is performed through the software-based equivalent system model, a holistic and consistent analysis is targeted.

3.4.6 Condition of the Building at the Time of Assessment (Section F)

In this section, both observational damage findings for the investigated building and analytical results obtained from the equivalent system model are evaluated together. Performance analysis is conducted based on the linear method defined in TBEC 2018 Chapter 15, and damage states of members at different storeys are reported in detail.

The resulting numerical data provide a technical basis for the RSRA risk scoring and enable holistic interpretation of field observations and numerical analysis findings.

In this section, first, specific scores are defined for each of the following irregularity and damage criteria:

- Corrosion condition is classified according to the following three options:
 - None: 0.3
 - Less than 30%: 1.0
 - More than 30%: 1.5
- Water emergence from beneath the foundation, ground heave, or liquefiable soil type:
 - No soil problem observed: 0.3
 - Problem observed: 1.5
- Significant stiffness difference in the weak direction of the building:
 - No: 0.3
 - Yes: 2.0
- Member capacity checks:
 - The columns that carry the largest span and have the smallest section are identified, and the vertical compression load acting on these columns is compared with column capacity (details in Section 3.4.6.1).
 - Based on determined damage ratios, the required spectrum scaling factor for the building to achieve controlled damage is determined (details in Section 3.4.6.2).
 - Demand/capacity (E/C) ratios are determined in columns using strength-based design criteria (details in Section 3.4.6.3).
 - Story drift values are compared (details in Section 3.4.6.4).
 - Damage ratios are determined in columns and beams using deformation-based design criteria (details in Section 3.4.6.5).

The evaluations performed under the “Member Capacity Checks” item are explained below.

3.4.6.1 Scoring Based on Axial Stresses (Maximum Compression Acting on Column / Column Section Area)

In this step, the stress caused by gravity loads in columns is evaluated both approximately using calculation sheets and separately using the equivalent system model. Typically, at the ground storey, two columns are selected that may experience the maximum axial load from the most unfavorable gravity load combination (1.4G + 1.6Q)—these columns are generally located near the middle of the plan and have relatively small cross-sections. These columns are evaluated using the “tributary area method,” in which the approximate contribution of vertical load from surrounding floor areas is calculated. The vertical load is divided by the column cross-sectional area to compute the average compressive stress in concrete.

This stress value is compared with 90% of the concrete compressive strength in accordance with TS500 Article 7.4.1. For example, based on a concrete strength of 14.16 MPa obtained from core tests, the limit is determined as:

$$0.9 \times f_{ck} = 0.9 \times 14.16 = 12.74 \text{ MPa}$$

If the vertical load acting on the first column is approximately calculated as 1399.91 kN and the column cross-sectional area is 4900 cm², the average compressive stress in concrete is:

$$\sigma = 1399.91 / 4900 = 0.286 \text{ kN/cm}^2 \rightarrow 2.86 \text{ MPa}$$

Similarly, if the vertical load for the other column is calculated as 468.27 kN and its section area is 2100 cm², the stress is:

$$\sigma = 468.27 / 2100 = 0.223 \text{ kN/cm}^2 \rightarrow 2.23 \text{ MPa}$$

When each stress value is divided by the limit value, the E/C ratios are obtained as 0.202 and 0.157, respectively. This value represents the first E/C ratio, i.e., the ratio of concrete compressive stress demand to concrete compressive strength capacity.

To account for different loading cases and perform a more detailed analysis, the simplified system model is analyzed under eight different load combinations. The most unfavorable axial force value among these combinations is considered. In this example, if the maximum axial force is 1333.8 kN in a different column and the column cross-sectional area is 1500 cm², the average compressive stress is:

$$\sigma = 1333.8 / 1500 = 0.8892 \text{ kN/cm}^2 \rightarrow 8.892 \text{ MPa}$$

The E/C ratio obtained from the analysis of the equivalent model is computed as $8.892 / 14.16 = 0.628$.

Comparing the three E/C ratios computed above, the largest value in this example is obtained from the equivalent model. Therefore, in RSRA scoring, this higher value (0.628), representing the most critical scenario, is adopted.

The main reasons for using both hand calculations and model-based data in this step are as follows:

- Hand calculations provide a rapid preliminary evaluation in the field and enable verification of the accuracy of the equivalent model.
- The equivalent system model provides more accurate results by accounting for hyperstatic behavior and load distribution; it is more reliable than hand calculations particularly for identifying the column where the maximum axial force occurs.
- The equivalent model is analyzed under eight different load combinations; thus, the most unfavorable scenario is safely identified.

This two-stage control procedure demonstrates that RSRA is an engineering-based approach and enables more rational scoring.

3.4.6.2 Linear Performance Analysis of the Equivalent Model and Scoring Based on Spectral Acceleration Coefficient (Spectrum Scaling Factor for Significant Damage)

In this part, the second E/C ratio in the RSRA evaluation logic is obtained by examining the behavior of the structure at a certain spectral acceleration level through linear analysis on the equivalent system model. The objective is to identify the performance level of the structure against seismic hazard and to numerically determine at which demand level the target performance can be achieved by reducing (or, in some cases, increasing) the spectral demand.

First, the design spectrum is constructed depending on the region, soil class, and dominant period of the building. This spectrum is determined using the SDS and SD1 parameters through the relationships defined in TBEC 2018 Article 2.3.4.1. The

earthquake effect corresponding to $S_{ae}(T)$ is applied to the equivalent system model generated in the OpenSees-based software, and linear analysis is performed. If the building satisfies the target performance level, i.e., Controlled Damage (KH), under the initial analysis, no reduction is applied to the spectrum and the structure is considered adequate; the corresponding score is recorded as “0.” However, if the analysis result corresponds to Collapse Prevention or a lower performance level, it indicates that the building cannot meet the demand. To identify the demand level that can be met, the design spectrum is gradually reduced and analyses are repeated.

The information obtained through this procedure is the linear-method-based performance per TBEC 2018 Chapter 15 without using any ductility reduction factor. Each reduction applied until the target performance is achieved is expressed as the “spectrum scaling factor.” The smaller this factor, the weaker the building is against the existing demands. Taking the inverse of this factor yields a value that is directly reflected in the RSRA scoring and is evaluated as an E/C ratio.

For example, suppose that Controlled Damage cannot be achieved under a design spectral acceleration of 0.8g; however, when this value is multiplied by a factor of 0.2 and reduced to 0.16g (i.e., reduced by a factor of 5), the target performance is achieved. In the RSRA evaluation table, 0.2 is recorded as the “Spectrum Scaling Factor,” and the E/C ratio is computed as $1 / 0.2 = 5$. For this building the spectral acceleration had to be reduced by a factor of 5 to reach Controlled Damage; therefore, $E/C = 5$ is recorded.

With this method, RSRA becomes a hybrid approach supported not only by visual/field-based observations, but also by engineering calculations and strength limits of the building. Thus, RSRA can be used not only as a screening tool but also as a prioritization and risk analysis tool supported by numerical data.

3.4.6.3 Scoring Based on Column Performances in the Equivalent Model Using E/C Ratios (Reinforced Concrete Column E/C Ratio)

In this stage, the behavior of column members under both flexural and shear effects is analyzed using the equivalent system model. Internal force values (M_2 , M_3 , V_2 , V_3) are obtained for each column under eight different load combinations.

For the flexural evaluation, the load combination producing the most unfavorable moment effect is identified for each column; the M2 and M3 values from this combination are evaluated together with the axial load (N) to generate the moment–axial load interaction diagram. The internal force components are plotted as points on this diagram; points outside the capacity curve indicate that flexural capacity has been exceeded. Using these diagrams, an E/C ratio related to flexure–axial load interaction is computed for each column.

For the shear evaluation, the V2 and V3 directions are analyzed separately; for each direction, the most unfavorable load combination is identified. In accordance with TS500, shear capacities (V_r) are computed using concrete strength, stirrup spacing, and axial load information obtained from the field. The following formulas are used in these computations:

- Diagonal cracking strength: $V_{k_s} = 0.65 \cdot f_{ctd} \cdot b \cdot d \cdot (1 + (N \cdot d) / A_c)$
- Stirrup contribution: $V_s = (A_{sw} / s) \cdot f_{yd} \cdot z$
- Total capacity: $V_r = 0.8 \cdot (V_{k_s} + V_s)$

For each column, the obtained shear demand V_e is compared with the capacity V_r . If $V_e > V_r$, the member is considered to have brittle damage risk, and a shear-based E/C ratio is computed.

After flexural and shear analyses, the computed E/C ratios for each column are compared, and the largest value is assigned to that column. Among all columns, the maximum E/C ratio is selected and recorded in the evaluation table as one of the candidate parameters for the B_0 coefficient used in stress scoring. Within this evaluation, the ductility level determined for the equivalent model generally leads to selecting R and D coefficients from the “Limited Ductility” part of TBEC 2018, because field observations indicated that ductility requirements were not satisfied in the investigated building inventory.

As an example, **Figure 3.11** shows that the axial load and biaxial moment effects for a column obtained from the analysis fall outside the interaction diagram. Because the internal force components exceed this limit, the demand is larger than the column capacity. In this case, the E/C ratio is computed as 1.70 and reflected in RSRA stress scoring.

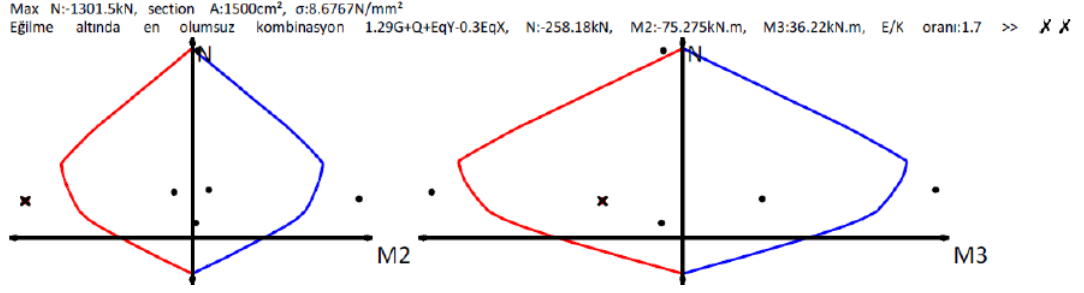


Figure 3.11: Interaction diagram for axial load and biaxial moment effect.

This approach enables a more comprehensive and representative evaluation of the overall safety level of the structure by identifying the weakest behavior mode within the structural system. This value is the third E/C ratio obtained for capacity checks.

3.4.6.4 Identification of Structural Irregularities Using the Equivalent Model and Scoring with E/C Ratio (Irregularities E/C Ratio)

In this stage, potential irregularities in the building are identified based on both field observations and the equivalent system model generated using the developed software. In particular, torsional irregularity (A1) and soft-storey irregularity (B2) related to relative interstorey drifts are evaluated numerically in accordance with relevant provisions of TBEC 2018. For the remaining irregularities, architectural and structural drawings as well as field observations are utilized.

For torsional behavior evaluation, lateral displacements at four different corner points on each storey diaphragm (δ_1 , δ_2 , δ_3 , δ_4) are determined. Using these values, the maximum displacement ($\delta_{\max}$), minimum displacement ($\delta_{\min}$), and average displacement (δ_{avg}) are computed. The torsional ratio defined in TBEC 2018 Article 4.7.4 ($\eta_b = \delta_{\max} / \delta_{\text{avg}}$) is computed, and the A1 torsional irregularity check is performed for each storey. If $\eta_b > 1.2$, torsional irregularity is considered present at that storey; sample results are shown in **Table 3.9** and **Table 3.10**.

Table 3.9: Torsional irregularity check in X direction.

Story	δ_1 [mm]	δ_2	δ_3	δ_4	δ_{max}	δ_{min}	δ_{avg}	η_{bx}	A1	4.7.4	Dbi	e%	DyfGen [m]
Bas_1	0.37123	0.37123	0.37123	0.37123	0.37123	0.37123	0.37123	1	No	✓	1	5	
Ground	39.217	39.217	39.217	39.217	39.217	39.217	39.217	1	No	✓	1	5	
Story_1	93.126	93.126	93.126	93.126	93.126	93.126	93.126	1	No	✓	1	5	
Story_2	142.61	142.61	142.61	142.61	142.61	142.61	142.61	1	No	✓	1	5	
Story_3	180.34	180.34	180.34	180.34	180.34	180.34	180.34	1	No	✓	1	5	
Story_4	204.21	204.21	204.21	204.21	204.21	204.21	204.21	1	No	✓	1	5	

Table 3.10: Torsional irregularity check in Y direction.

Story	δ_1 [mm]	δ_2	δ_3	δ_4	δ_{max}	δ_{min}	δ_{avg}	η_{by}	A1	4.7.4	Dbi	e%	DyfGen [m]
Bas_1	0.3712	0.3712	0.37123	0.3712	0.3712	0.37123	0.3712	1	No	✓	1	5	
Ground	39.217	39.217	39.217	39.217	39.217	39.217	39.217	1	No	✓	1	5	
Story_1	93.126	93.126	93.126	93.126	93.126	93.126	93.126	1	No	✓	1	5	
Story_2	142.61	142.61	142.61	142.61	142.61	142.61	142.61	1	No	✓	1	5	
Story_3	180.34	180.34	180.34	180.34	180.34	180.34	180.34	1	No	✓	1	5	
Story_4	204.21	204.21	204.21	204.21	204.21	204.21	204.21	1	No	✓	1	5	

For the evaluation of relative interstory drifts, the displacement at each storey (δ_i) is divided by the corresponding storey height (h_i) to compute the drift ratio (δ_i/h_i). These values are compared with the limit value $\lambda \cdot (\delta/h)$ defined in TBEC 2018 Article 4.9.1. For reinforced concrete buildings with brittle and adjacent infill walls, this limit is generally taken as 0.008. Using these ratios, an E/C ratio is obtained for each storey as $(\delta_i/h_i)/0.008$.

Table 3.11: Soft-storey irregularity check in X direction.

Story	DD2 Lower disp. [mm]	DD2 Upper disp.	Δ	h	δ	$\lambda\delta/h$	Limit	E/C	✓/✗
Bas_1	0	0.37123	0.37123	4150	1.4849	0.00012242	0.008	0.015303	✓
Ground	0.37123	39.217	38.846	4150	155.38	0.01281	0.008	1.6013	✗✗
Story_1	39.217	93.126	53.909	4150	215.64	0.017778	0.008	2.2223	✗✗
Story_2	93.126	142.61	49.48	4150	197.92	0.016317	0.008	2.0397	✗✗
Story_3	142.61	180.34	37.733	4150	150.93	0.012443	0.008	1.5554	✗✗
Story_4	180.34	204.21	23.869	4150	95.477	0.0078715	0.008	0.98394	✓

B2: Stiffness Irregularity Between Adjacent Stories (Soft Story): The largest difference is between “Story_3” and “Story_4,” with $\eta_{ki} = 0.0124 / 0.0079 = 1.55 \leq 2$ (1 basement storey) (no storey setback) (Table 3.11).

Table 3.12: Soft-storey irregularity check in Y direction

Story	DD2 Lower disp. [mm]	DD2 Upper disp.	Δ	h	δ	$\lambda\delta/h$	Limit	E/C	✓/✗
Bas_1	0	0.37123	0.37123	4150	1.4849	0.00012242	0.008	0.015303	✓
Ground	0.37123	39.217	38.846	4150	155.38	0.01281	0.008	1.6013	✗✗
Story_1	39.217	93.126	53.909	4150	215.64	0.017778	0.008	2.2223	✗✗
Story_2	93.126	142.61	49.48	4150	197.92	0.016317	0.008	2.0397	✗✗
Story_3	142.61	180.34	37.733	4150	150.93	0.012443	0.008	1.5554	✗✗
Story_4	180.34	204.21	23.869	4150	95.477	0.0078715	0.008	0.98394	✓

B2: Stiffness Irregularity Between Adjacent Stories (Soft Story): The largest difference is between “Story_3” and “Story_4,” with $\eta_{ki} = 0.0124 / 0.0079 = 1.55 \leq 2$ (1 basement storey) (no storey setback) (Table 3.12).

Based on both analyses, irregularities emerging in the structural system are identified through numerical methods. A1 torsional irregularity and B2 soft-storey irregularity are evaluated both within code limits and in terms of their impact on scoring under the heading “Irregularities D/C Ratio.” All storey-level D/C ratios are examined, and the

maximum ratio in the building is recorded in the evaluation table as the basis for scoring. This value is the fourth D/C ratio.

The four D/C ratios (a, b, c, d) obtained by the four methods are tabulated, and a risk score is obtained (see **Table 3.13**) according to the following criterion:

Table 3.13: Summary of E/C ratios and input values (example).

Item	Value	D/C Ratio / Notes
(Maximum compression on column / column area)	$\sigma = 8.892 \text{ MPa}$	D/C = 0.628
(Spectrum scaling factor for significant damage)	Spectrum factor = 0.2	D/C = $1/0.2 = 5.000$
(Reinforced concrete column D/C ratio)	1st D/C	1.700
(Irregularities D/C ratio)	2nd D/C	1.55
		(Max D/C = 5.000)

E/C Score : 21,5

Among these E/C ratios, the largest value is defined as the demand-to-capacity ratio. Depending on whether the E/C ratio is smaller or larger than 0.8, equations are used to compute the E/C score.

$$\frac{D}{C} SCORE: \begin{cases} \frac{D}{C} < 0.8 & \rightarrow 0 \\ \frac{D}{C} \geq 0.8 & \rightarrow 0.5 + (BO - 0.8) \times 5 \end{cases}$$

The obtained E/C score is the total score of Section F. It has been observed that it may reach very large values particularly for buildings that are weak and located in highly seismic regions. Therefore, the Section F score is capped at a maximum of “5.”

3.4.6.5 Determination of Performance Level Based on Deformation Limits in Columns and Beams

The equivalent system model is evaluated using the linear analysis method according to TBEC 2018 Section 15.5. Accordingly, internal forces occurring in each structural member under the applied earthquake loads are determined, and damage states are

identified based on these forces. As defined in TBEC 2018 Article 15.8, members are classified into the damage levels of Limited Damage (SH), Significant Damage (BH), Advanced Damage (İH), and Collapse (GÖ).

Using the performance table generated from the analysis, the total shear forces for each storey, the distribution of these forces among structural members, and member-level damage states can be observed in detail. Below, an example analysis output for the +X direction is presented in **Table 3.14** and **Table 3.15**.

Table 3.14: Performance level result for +X direction.

Tablo: Kat_1 katı +X yönü için kesme kuvveti tablosu (Toplam: 37699 kN)

Mem	V[kN]	%	Perf	Mem	V[kN]	%	Perf	Mem	V[kN]	%	Perf
Col0.0 Sto1 Kat 1	68.761	0.18	SH	Col0.0 Sto1 Kat 1	67.717	0.18	SH	Col0.2 Sto1 Kat 1	70.301	0.19	SH
Col0.3 Sto1 Kat 1	66.968	0.18	SH	Col1.0 Sto1 Kat 1	53.294	0.14	SH	Col1.1 Sto1 Kat 1	53.741	0.14	Gö
Col0.2 Sto1 Kat 1	59.217	0.16	SH	Col2.1 Sto1 Kat 1	61.434	0.16	SH	Col2.0 Sto1 Kat 1	51.405	0.14	SH
Col1.2 Sto1 Kat 1	47.791	0.13	Gö	Col2.2 Sto1 Kat 1	34.032	0.09	BH	Col2.3 Sto1 Kat 1	51.124	0.14	SH
Col3.0 Sto1 Kat 1	51.495	0.14	SH	Col3.1 Sto1 Kat 1	56.162	0.15	Gö	Col3.2 Sto1 Kat 1	70.936	0.19	SH
Col3.3 Sto1 Kat 1	49.688	0.13	SH	Col4.0 Sto1 Kat 1	53.528	0.14	SH	Col4.1 Sto1 Kat 1	58.073	0.15	Gö
Col4.2 Sto1 Kat 1	82.017	0.22	SH	Col4.3 Sto1 Kat 1	58.435	0.15	SH	Col5.0 Sto1 Kat 1	69.308	0.18	SH
Col5.1 Sto1 Kat 1	65.019	0.17	SH	Col5.2 Sto1 Kat 1	58.429	0.15	SH	Col5.3 Sto1 Kat 1	70.031	0.19	SH
WIL1 Sto1 Kat 1	7012.2	18.6	Gö	WIL2 Sto1 Kat 1	35.064	0.09	BH	WIL3 Sto1 Kat 1	40.819	0.11	BH
WIL4 Sto1 Kat 1	2915.3	7.73	SH	WIL5 Sto1 Kat 1	3965.2	10.52	SH	WIL6 Sto1 Kat 1	2117.6	5.62	SH
WIL7 Sto1 Kat 1	2481.2	6.58	SH	WIL8 Sto1 Kat 1	2881	7.64	SH	WIL9 Sto1 Kat 1	3294.8	8.74	SH
WIL10 Sto1 Kat 1	2161.6	5.73	SH	WIL11 Sto1 Kat 1	2692.4	7.14	SH	WIL12 Sto1 Kat 1	2742.5	7.27	SH
WIL13 Sto1 Kat 1	3715.6	9.86	SH	WIL14 Sto1 Kat 1	49.169	0.13	SH	WIL15 Sto1 Kat 1	48.393	0.13	SH
WIL16 Sto1 Kat 1	21.165	0.06	SH	WIL17 Sto1 Kat 1	18.931	0.05	SH	WIL18 Sto1 Kat 1	39.894	0.11	SH
WIL19 Sto1 Kat 1	37.334	0.1	SH								

Hasar tespit katı:1
Göçme bölgesinde eleman bulunmaktadır, Sınırlı Hasar X
Göçme bölgesinde kolon bulunmamaktadır, Göçme Öncesi X
Deprem +X yönü yapı performansı: Göçme

Table 3.15: Distribution of member damage for +X direction.

K	Sınırlı				Belirgin				İleri				Göçme				A&Ü	TOP	Kat
	Kiriş	Kolon	Vkol[kN]	%Vkol	Kiriş	Kolon	Vkol[kN]	%Vkol	Kiriş	Kolon	Vkol[kN]	%Vkol	Kiriş	Kolon	Vkol[kN]	%Vkol	%>BH	Vkol[kN]	Hasarı
1	38/38	35/43	30361	80.5	-	3/43	109.91	0.3	-	-	0	0	-	5/43	7228	19.2	0	37699	G
2	38/38	5/27	1108.7	6.4	-	0	0	0	-	-	0	0	-	22/27	16342	93.6	84.6	17451	G
3	38/38	5/27	1097.6	7.2	-	0	0	0	-	1/27	147.86	1	-	21/27	13916	91.8	92.8	15161	G
4	38/38	5/27	1066.4	9.3	-	3/27	459.3	4	-	3/27	477.87	4.2	-	16/27	9494.1	82.6	84.1	11498	G
5	38/38	24/27	5919.3	87	-	0	0	0	-	-	0	0	-	3/27	886.67	13	13	6806	G

Notlar:

A&Ü: Alt & Üst bölgesi

SH: Sınırlı Hasar

G: Göçme

İH: İleri Hasar

Gö: Göçmenin Önlenmesi

In **Table 3.14**, each structural member is presented together with both the share (%) of the storey shear it carries and its performance level. Thus, not only the count of

members reaching a certain damage level but also their influence on global structural response becomes apparent.

In **Table 3.15**, the distribution of members into damage classes is given for each storey. For example, it is observed that on Story 1, 3 out of 43 columns are in Significant Damage; the total lateral force of 109.91 kN carried by these three columns corresponds to only 0.3% of the total storey shear. Thus, the evaluation is performed by jointly considering both quantitative and performance-related measures.

Moreover, the “A&Ü” column enables separately tracking the damage classes at the lower (A) and upper (Ü) ends of column members. This allows for a more practical implementation of damage assessment required by TBEC 2018 Articles 15.8.4(c) and 15.8.5(b).

3.5 Results and Discussion – Applicability of the RSRA Method and Structural Index Calculation

In this study, the Rapid Seismic Risk Assessment (RSRA) method, which integrates field-based observations with engineering calculations and has high practical applicability, was introduced and exemplary applications were performed on buildings. The RSRA method consists of six categories: existing building material condition, structural system characteristics and construction year, floor diaphragm behavior, soil and seismic demand parameters, irregularities, and the condition of the building at the time of assessment. Scores obtained from subheadings of each category are integrated into a structural index using a multiplier system.

The total risk score for each building is determined with the Structural Index (SI) formula in equation (3.7).

$$SI = A * B * C * D * E \quad (3.7)$$

Within this formula, each multiplier is evaluated according to engineering data obtained from the building and classified systematically (see **Figure 3.12**).

For example, if a building is evaluated with the following high risk scores:

- A. Existing material condition = 6.120
- B. Structural system type and year = 2.000

- C. Floor diaphragm formation = 0.7
- D. Seismicity and soil effect = 5.690
- E. Irregularities = 3.300
- F. Condition of the building at the time of assessment = 5.000

then the total SI is computed as $SI = 804.41$. The prioritization scale defined by RSRA for this example building is interpreted as follows:

RESULT: STRUCTURAL INDEX CALCULATION

When calculating the structural index, each penalty score category is multiplied to score the structure.

$$SI = A \times B \times C \times D \times E \times F$$

SI: Structural Index

Building Structural Index Score: 804.41			
Priority Status	Score Limit	Sub-categories	
Very low risk	0-200	Existing material condition (a)	6.120
Low risk	201-350	Structural system type and year (b)	2.000
Lower priority	351-500	Floor diaphragm formation(c)	0.7
Non-urgent intervention	501-700	Seismicity and soil effect (d)	5.690
Search for alternatives	701-900	Irregularities(e)	3.300
Priority intervention	>900	Condition of the building at the time of assessment (f)	5.000

Figure 3.12: Structural Index Calculation

- $SI > 701-900 \rightarrow$ Falls into the “Search for Alternatives” category. This indicates that long-term use of the building is not considered safe; therefore, replacement with a new building or planning of comprehensive intervention is required. Such buildings should also be evaluated economically and addressed through alternative solutions.

With this evaluation model, the RSRA method provides a structural risk ranking that is both rapid and technically grounded, offering decision-makers concrete, data-driven guidance. Detailed explanations for the building conditions corresponding to these score intervals are provided below in **Table 3.16**.

Table 3.16: Score intervals and explanation.

SI Range	Priority Status	Explanation
SI > 900	Priority intervention	Structures in this range have very high structural risk; serious engineering deficiencies are considered present. Interventions such as strengthening or decommissioning should be prioritized.
701–900	Search for alternatives	Long-term use is not considered reliable; replacement with a new building or comprehensive intervention should be planned. Economic evaluation and alternative solutions are recommended.
501–700	Non-urgent intervention	The building has certain engineering deficiencies, but there is no extremely severe risk for current use. Strengthening, maintenance, or monitoring is recommended in the medium term.
351–500	Lower priority	Buildings in this range are in a gray/intermediate zone; they may have some weaknesses but may not show clear inadequacy. Detailed evaluation and, if necessary, comprehensive analyses are recommended.
201–350	Low risk	The building is generally acceptable in terms of engineering, material, and system behavior. However, because RSRA is a preliminary method, building-specific conditions should be considered especially for values near the boundaries.
0–200	Very low risk	According to current observations and calculations, no significant engineering weakness is identified. Comprehensive intervention is not required; nevertheless, this does not imply zero risk, and basic maintenance and monitoring are recommended.

In general, the following actions are expected based on the scoring results:

- 0–350 points: Continued use of the building without the need for a more detailed assessment.
- 351–700 points: If the building is intended to remain in use, more detailed data should be collected through performance analysis.
- 701 points and above: Discontinuation of building use without requiring more detailed assessment. If no alternative facility exists, a detailed performance analysis should be conducted.

3.6 Equivalent System Model and Software Application

With Version 5 of the RSRA method, an equivalent system model began to be used to place the risk scoring on a more realistic and scientifically grounded basis, and static solutions of this system were performed using OpenSees or an internal solver. Compared to the empirical period estimations used in earlier versions, this model enables more accurate determination of modal characteristics based on the actual condition of the structure. The modeling process is conducted based on average values of axis spacing, column-beam section dimensions, and storey heights obtained from detailed field inspections.

The prepared three-dimensional model is analyzed using OpenSees or an internal solver to obtain dynamic characteristics. From these analyses, period values (T_x , T_y), vibration modes, mass participation ratios, base shear forces, and irregularity conditions ($A1$, $B2$, etc.) are obtained and used both as inputs to the RSRA scoring and as indicators of the general condition of the structure.

Live loads in the model are defined in accordance with TS498, while dead loads are identified in the field. Lateral seismic forces are distributed to storey levels in proportion to storey masses and heights. These forces are distributed to reinforced concrete members in proportion to their stiffness. Using internal forces obtained from the analysis, demand/capacity ratios and damage conditions are determined for columns and beams; these outputs also provide insight into whether the building requires prioritized intervention.

In the modeling, stiffness multipliers are applied in accordance with TBEC 2018 Table 4.2 (“Effective Section Stiffness Multipliers for Reinforced Concrete Structural

Members”). In addition, the building information level coefficient is taken as 0.75 or 1 depending on whether there are drawings and whether the building was constructed under controlled conditions. Stirrup strength adjustments per TBEC 2018 Article 15.7.1.3 are applied according to stirrup hook angle. If plain reinforcement is used in the building, seismic demand is increased in accordance with TBEC 2018 Article 15.7.1.2.

With this modeling approach, a simplified system model is obtained that is compatible with field data and represents the general geometry. Integrated with Version 5 of the RSRA method, this system provides technical depth to the scoring system and offers decision-makers a more robust, engineering-based evaluation. In determining dynamic parameters of the equivalent system model integrated with the current version of RSRA, a linear analysis approach is preferred. However, within the analysis, the limiting criteria for the linear analysis method defined in TBEC 2018 Article 15.5.3.1 (e.g., $BYS < 5$, B3 irregularity, etc.) were not separately checked for each building; the modeling process was carried out under the same approach for all buildings. This preference is a deliberate simplification made to preserve the comparative analysis content of the study and to provide a technical basis for the scoring system.

4. COMPARISON OF RSRA METHOD WITH NONLINEAR PERFORMANCE ANALYSIS

When structural behaviour is investigated, the more a given method rests on simplifying assumptions, the further its results may deviate from the actual behaviour. For this reason, compared to linear elastic methods, nonlinear and deformation-based analysis procedures have a greater potential to represent the seismic behaviour of structures in a realistic manner. In particular, since existing buildings and poorly designed structures are expected to exhibit nonlinear response, nonlinear analysis methods are of critical importance for engineering practice when assessing such buildings.

Within the scope of the Turkish Building Earthquake Code (TBEC-2018), nonlinear performance analyses may be conducted in three different ways: single-mode pushover analysis with invariant loading, multi-mode pushover analysis, and nonlinear time-history analysis. Although time-history analysis allows detailed modelling of structural response as a function of time, it requires extensive ground-motion records, advanced modelling capability, and significant computational effort. Multi-mode pushover analysis, on the other hand, can simultaneously account for the response of the structure in several vibration modes, but is limited in practice by its modelling complexity and sensitivity. These factors restrict the widespread use of both methods in applications where a large number of buildings must be analysed in a short period of time.

In this study, among nonlinear analysis procedures, the invariant single-mode pushover analysis method has been adopted. This choice does not merely reflect a boundary condition imposed by the code; it is also a deliberate methodological decision within the framework of this study. Invariant single-mode pushover analysis provides a suitable solution for the present work due to its ease of application, low computational demand, applicability to a wide building stock, and its ability to generate comparative results that can be matched with the RSRA method. Furthermore, since this method applies lateral loads corresponding to the first mode shape to the structure and performs the pushover in that manner, it produces response outputs that are easier to interpret in the analysis process. This provides clarity in the

engineering assessment of the structure and the understanding of its overall behaviour. Accordingly, the analysis approach adopted in this thesis simultaneously considers the nonlinear behaviour of the buildings and enables analytical comparisons with the RSRA scoring system.

4.1 Nonlinear Performance Assessment of Existing Buildings

In TBEC-2018, the nonlinear behaviour of existing buildings in seismic performance evaluation is taken into account directly through structural analysis and by obtaining the corresponding deformations from this analysis. In this context, the code defines three main nonlinear analysis procedures for determining building performance. Each procedure differs in its intended use, modelling requirements, and level of realism with respect to actual behaviour. These methods are summarised below.

4.1.1 Single-Mode Pushover Analysis (Invariant Single-Mode Pushover Method)

This method aims to represent the behaviour of a multi-degree-of-freedom system through an equivalent single-degree-of-freedom (SDOF) system reduced to the first vibration mode. Lateral loads applied to the structure are distributed according to the first mode shape, and a nonlinear pushover analysis is performed in that direction. The resulting capacity curve is then transformed into modal coordinates, compared with the spectral demand, and the performance point is determined. Although the method is computationally efficient and consistent with engineering intuition, TBEC-2018 Article 5.6.2.2 explicitly states that it cannot be used for all building types. Annex 5B of TBEC-2018 provides, in detail, the transformation equations and procedures required for the implementation of the method and for the calculation of the performance point.

4.1.2 Multi-Mode Pushover Method

The multi-mode pushover method aims to account for the contribution of several modes of vibration, rather than reducing the building behaviour solely to the first mode. During the pushover analysis, the contribution of each mode is calculated

separately, and then the responses associated with these modes are combined using an appropriate combination rule (e.g., SRSS or CQC). The combination procedure also includes the construction of the overall pushover curve of the structure by superimposing the modal pushover curves. This method yields more realistic results, especially in the assessment of irregular, tall, and torsion-sensitive buildings. However, the computational process is more complex and requires separate modelling and analysis for each mode.

4.1.3 Nonlinear Time-History Analysis Method

In this method, the ground motion is defined directly in the time domain and the structural response is calculated at each time step. This approach, which is based on solving the dynamic equilibrium equations step-by-step in time, is the most realistic yet also the most complex method. Time-history analysis is preferred for buildings that are required to remain operational after earthquakes, for historic structures, or for systems with major irregularities. Moreover, when the distance between the building and the causative fault is less than 15 km, time-history analysis is also recommended. In this method, records of past earthquakes generated by faults with characteristics similar to those of the target fault are used; thus, the realism of the method is quite high. Parameters such as analysis duration and the choice of damping model are of critical importance in this approach.

4.2 Calculation of Demand Displacement by Single-Mode Pushover Analysis and Determination of Building Performance

According to TBEC-2018, invariant single-mode pushover analysis cannot be applied to every type of structure. To employ this method within the limits of the code, the torsional irregularity index must satisfy $\eta_{bi} < 1.4$ and the modal mass participation factor of the first mode must not be less than 0.70. In the present study, however, nonlinear pushover analysis has been adopted for 28 different buildings with the specific intention of enabling a comparative evaluation with the risk scores previously obtained using the RSRA method. The primary objective in applying the method is to obtain capacity curves that represent the lateral load-carrying capacity of the buildings under seismic loading, and then to determine the performance points by comparing these curves with the spectral demand curve. In this way, the performance levels

obtained for each building can be compared with the RSRA scores, and the consistency of the method relative to pushover analysis can be evaluated. Within this framework, regardless of the restrictive conditions prescribed in the code, nonlinear pushover analysis has been performed for each building, and the results have been used solely for the purpose of quantifying performance levels.

4.2.1 Implementation Steps of the Pushover Analysis Procedure

In this study, nonlinear pushover analyses were carried out in accordance with the invariant single-mode pushover method defined in Annex 5B of TBEC-2018. The implementation process follows the flow chart illustrated in **Figure 4.1**.

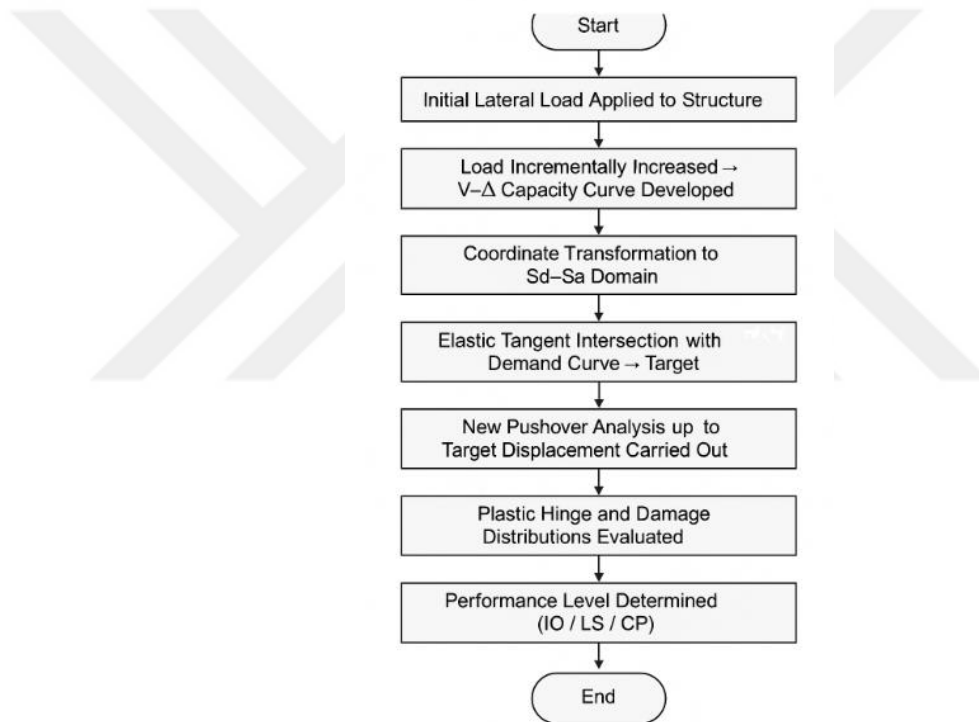


Figure 4.1: Flowchart of the pushover analysis procedure.

According to the flowchart:

1. Construction of the Capacity Curve
 - a. Lateral loads distributed according to the first mode shape are applied to the building.
 - b. These loads are incrementally increased until the structure reaches a collapse mechanism.

- c. At each load step, the roof displacement (Δ) and the base shear force (V) are calculated.
 - d. Using these values, the capacity curve of the building ($V-\Delta$) is obtained.
2. Construction of the Demand Curve
- a. The capacity curve is transformed into the spectral displacement–spectral acceleration (S_d-S_a) space.
 - b. This transformation is carried out according to TBEC-2018 using the properties of the structure (effective mass, natural period, etc.).
 - c. In the same space, the elastic spectrum corresponding to the seismic hazard at the building site is plotted.
 - d. The elastic spectrum is reduced according to the damping ratio of the system to obtain the inelastic demand curve.
3. Determination of the Demand Point
- a. A bilinear idealisation of the capacity curve is constructed by matching the initial elastic slope.
 - b. The elastic spectral displacement is converted to inelastic spectral displacement through the displacement modification factor C_R . If $T_1 > T_B$, then $C_R = 1$. Otherwise, C_R is calculated iteratively using TBEC-2018 Equation 5B.17b.
4. Determination of Performance Level
- a. The building is pushed again up to the target displacement, and the final analysis is performed at this deformation level.
 - b. At this stage, the locations of plastic hinges, deformation demands, and storey drifts are evaluated.
 - c. The damage levels of elements are compared with the performance limits defined in TBEC-2018 (IO, LS, CP).
 - d. Based on these results, it is determined whether the structure satisfies the target performance level.

4.2.1.1 Lateral Loading and Construction of the Capacity Curve

In the nonlinear pushover analyses conducted in this study, the lateral load pattern was determined according to the site coordinates and soil class of each building. In

accordance with TBEC-2018 Section 2.3, the elastic design spectrum corresponding to the local soil class and coordinates was constructed for each structure, and the spectral acceleration values S_{ae} were defined. Under the calculated lateral loads, nonlinear pushover analyses were performed, and the capacity curves shown schematically in **Figure 4.2** were obtained. These curves represent the relationship between the base shear force and the horizontal displacement at the monitored control node (typically the roof). The capacity curves are independent of earthquake records; instead, they reflect the geometry of the buildings, their lateral load-carrying capacity, ductility, material properties, and overall behavioural characteristics.

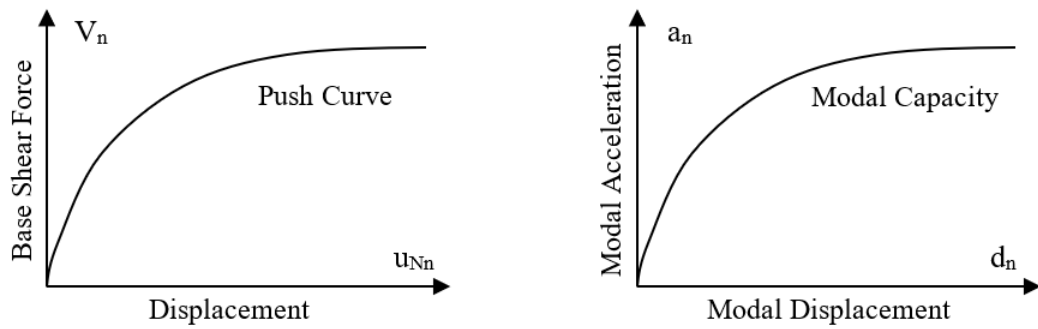


Figure 4.2: Typical capacity curve.

In this figure, V_n denotes the base shear corresponding to a given analysis step, and u_{nN} represents the horizontal displacement at the control node for that step. Along the curve, it is possible to track the sequence of plastic hinge formation in the members and to assess the system behaviour. To evaluate the pushover curve together with the demand curve, an axis transformation is applied using the relations given in Equation (4.1) and Equation (4.2). The base shear V_n is converted to modal acceleration a_n , and the roof displacement u_{nN} is converted to modal displacement d_n :

$$a_n = \frac{V_n}{\bar{M}_n} \quad (4.1)$$

$$d_n = \frac{u_{nN}}{\varphi_{nN}\Gamma_n} \quad (4.2)$$

4.2.1.2 Earthquake Demand and Construction of the Demand Curve

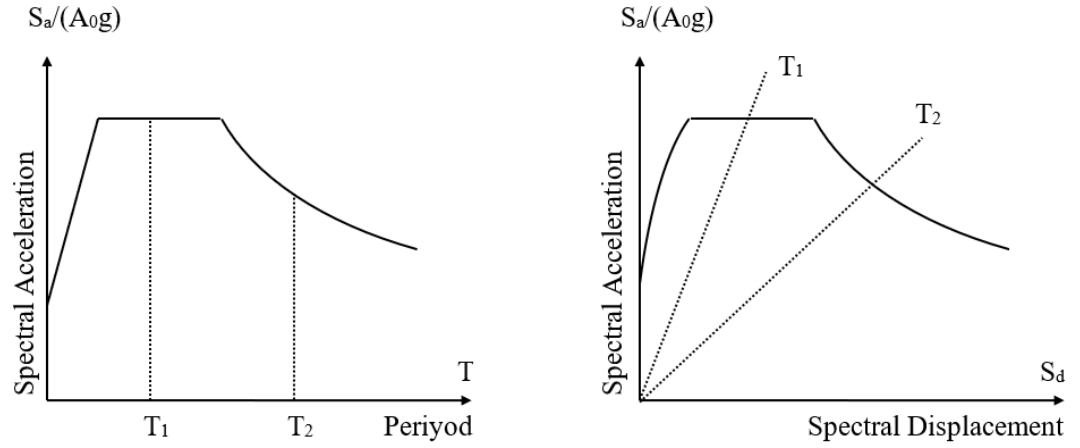


Figure 4.3: Spectral axis transformation.

The quantities related to the design earthquake are defined through spectral curves in seismic codes. As shown in **Figure 4.3**, the spectrum is described in the S_a – T space. In order to compare this spectrum with the pushover curve, it must be transformed into the S_a – S_d space. This transformation can be performed using Equation (4.3):

$$S_d = \left(\frac{T}{2\pi} \right)^2 S_a \quad (4.3)$$

4.2.1.3 Determination of Earthquake Demand Displacement

Once both curves are expressed in the same space, the displacement demand of the system under the earthquake action can be determined. The capacity of the system has been calculated on the basis of nonlinear behaviour. However, since the earthquake spectrum is elastic, both curves must be expressed in a compatible manner. The ATC-40 guideline proposes converting the elastic response of the earthquake to the nonlinear behaviour of the system. Since this transformation depends on nonlinear response, an iterative approach is required to locate the performance point.

In TBEC-2018 Annex 5B, by contrast, the capacity curve is converted to an equivalent elastic representation similar to the demand curve. For this purpose, the initial tangent of the capacity curve is intersected with the spectrum curve in the S_a – S_d plane, and the elastic displacement of the equivalent system against the earthquake demand (d_{max}^e) is obtained. Subsequently, using the equal-displacement rule, the elasto-plastic displacement of the system (d_{max}^e) is determined:

$$d_{\max(n)}^{ep} = C_{R(n)} d_{\max(n)}^e \quad (4.4)$$

In this expression, n denotes the relevant vibration mode. The transformation factor $C_{R(n)}$ is computed as equation (4.5) and (4.6):

$$T_n \leq T_B C_{R(n)} = \frac{1}{R_{y(n)}} \left[1 + (R_{y(n)} - 1) \frac{T_B}{T_n} \right] \quad (4.5)$$

$$T_n > T_B C_{R(n)} = 1 \quad (4.6)$$

The strength reduction factor $R_{y(n)}$ is determined by equation (4.7).

$$R_{y(n)} = \frac{S_{ae(n)}}{a_{y(n)}} \quad (4.7)$$

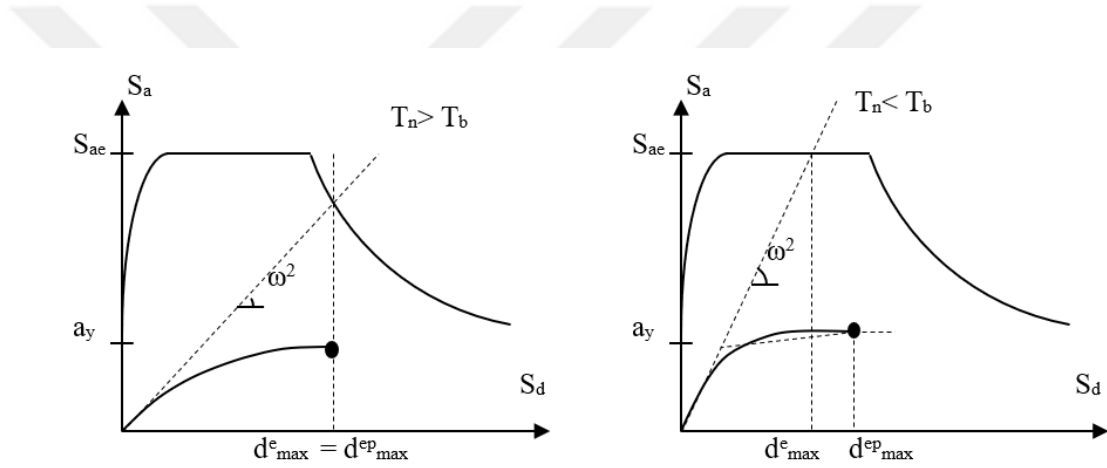


Figure 4.4: Determination of elasto-plastic displacement using the elastic spectrum and capacity curve.

As illustrated in **Figure 4.4**, a_y is obtained from a new bilinear representation of the capacity curve having equal area with the original curve. Since the performance point of the capacity curve is not known at the outset, this process is carried out through a few trial steps. If the capacity curve cannot reach the performance point, it means that the structure collapses before meeting the earthquake demand. As plastic hinges form and the system reaches a mechanism state, the structure may fail prior to attaining the performance point.

4.2.1.4 Determination of Structural Performance

Using the deformation values obtained from the pushover analysis up to the performance point, member damage states and the overall performance level of the

structure are determined in accordance with TBEC-2018 Section 15. For ductile elements, three damage states and associated damage limits are defined at section level: Limited Damage (SH), Controlled Damage (KH), and Near-Collapse Damage (GÖ). Limited Damage corresponds to a small amount of inelastic behaviour at the section; Controlled Damage corresponds to inelastic behaviour at which section strength can still be safely maintained; and Near-Collapse damage corresponds to advanced inelastic response at the section. This classification does not apply to elements failing in a brittle manner.

The purpose of nonlinear analysis methods used for determining the seismic performance of existing or strengthened buildings and for strengthening design is to compute plastic deformation and plastic rotation demands associated with ductile behaviour, as well as internal force demands associated with brittle behaviour, for a given earthquake. These demand quantities are then compared against deformation and internal force capacities; thus, structural performance assessment is carried out at both section and building level.

4.3 Nonlinear Modelling Approach

In this study, the performance analysis of each investigated building was carried out using the ETABS v21.3.0 software. In the modelling process, the original structural drawings of each building were strictly followed; cross-section dimensions of structural elements, span lengths, storey heights, and slab thicknesses were taken directly from the original design drawings. Material properties of the buildings were determined from field data. For each building, concrete strength was obtained through core tests; bar diameters and spacing were determined through reinforcement exposure and rebar scanning (x-ray) techniques. In this way, building-specific material definitions were assigned in the analyses. In accordance with TBEC-2018 Article 15.2.3.1, the knowledge level factor and reinforcement reliability factor were considered in these definitions; the measured concrete and reinforcement strengths were reduced appropriately and reflected in the analytical model.

For beam elements, plastic hinges were defined based on moment–curvature relationships and the rotation limits determined by TBEC-2018 Equations (5.6) and (5.7b). For column elements, the confined concrete behaviour was modelled using the

Mander model as illustrated in **Figure 4.5**, and distributed plasticity was implemented through fibre models.

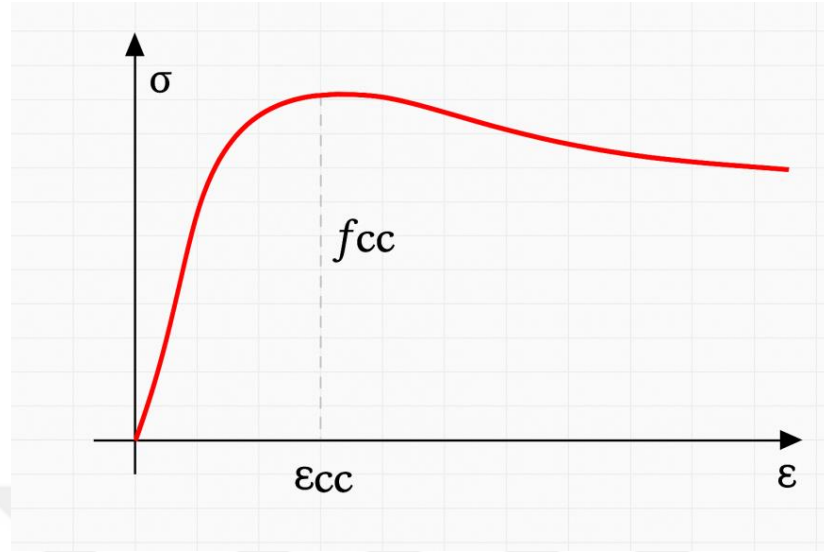


Figure 4.5: Example stress–strain (σ – ε) curve of the confined concrete material model used in the analyses.

For wall elements, the MVLEM element was employed, which is capable of capturing distributed plasticity. The Collapse Prevention and Controlled Damage limits were determined based on the strain limits defined in TBEC-2018; the strain values specified by Equation (5.4a) for concrete and Equation (5.7a) for steel were used as damage control thresholds in the fibre models.

To properly capture the structural behaviour, plastic regions were defined in the load-bearing members. Distributed plasticity was adopted for columns and lumped plasticity for beams. All plasticity limits were defined in accordance with TBEC-2018 Articles 5.4 and 15.4.

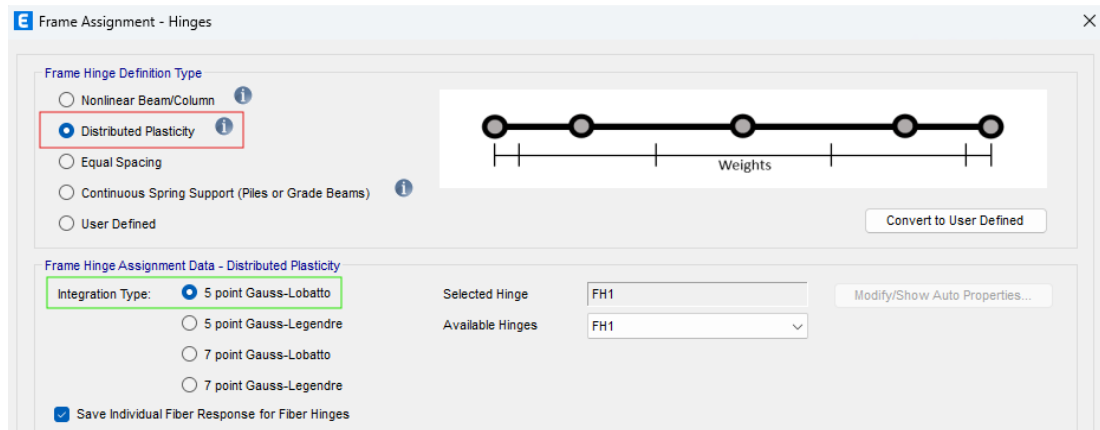


Figure 4.6: Definition of distributed plasticity in columns and selection of Gauss–Lobatto integration points (ETABS interface).

Since the nonlinear modelling of beams was carried out under the plastic hinge hypothesis, only the limited regions near beam–column joints exhibit nonlinear behaviour, while the intermediate beam segment remains elastic (**Figure 4.6**). As a consequence, the behaviour of these elastic segments near the limit of member strength must also be represented properly. Therefore, effective section stiffnesses of beams were calculated using TBEC-2018 Article 5.4.5.2 and Equation (5.2). In this relation, the effective yield moment (M_y) and yield rotation (θ_y) of the plastic hinge region are considered, and stiffnesses are determined directly from member dimensions and reinforcement ratios on a building-specific basis.

For columns and walls, since distributed plasticity was adopted, all points along the member are affected by increasing loads and spreading nonlinearity, and there is no need to impose a priori assumptions about when the member reaches its strength. This state is automatically computed in subsequent analysis steps by the nature of the adopted method. Because assigning additional effective stiffness reduction factors from outside would be inappropriate for column and wall elements analysed with fibre models, no extra stiffness modification was applied, in line with the note by CSI: “When using fiber hinges, you are already modeling the reduced stiffness due to cracking, and should not apply further stiffness modifiers.” (CSI, Analysis Reference Manual, Computers and Structures Inc., 2021). For slabs and basement walls, effective section stiffness factors were taken from TBEC-2018 Table 4.2.

In some of the analysed buildings, the slab systems were modelled with rigid diaphragm assumptions; in others, due to slab discontinuities or waffle slabs, a “semi-rigid” diaphragm definition was preferred (**Figure 4.7**).

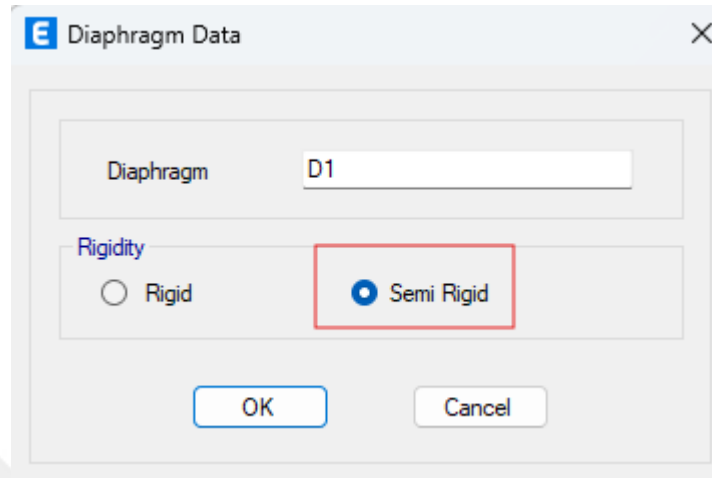


Figure 4.7: Example of a ‘Semi-Rigid’ diaphragm definition in buildings with slab discontinuities (ETABS interface).

In defining seismic effects, the determination of soil class played a critical role. In some buildings, the soil class was obtained directly through MASW (Multichannel Analysis of Surface Waves) seismic measurements. For others, the soil class was assessed using USGS (United States Geological Survey) data, while for the remaining buildings, soil class ZC was adopted based on field observations. Depending on the soil class definitions, spectral acceleration coefficients S_{Ds} and S_{D1} were determined, and seismic loads were generated in accordance with TBEC-2018 Section 2.3.4.1. Gravity loads were defined in accordance with TS 498 (**Figure 4.8** and **Figure 4.9**).

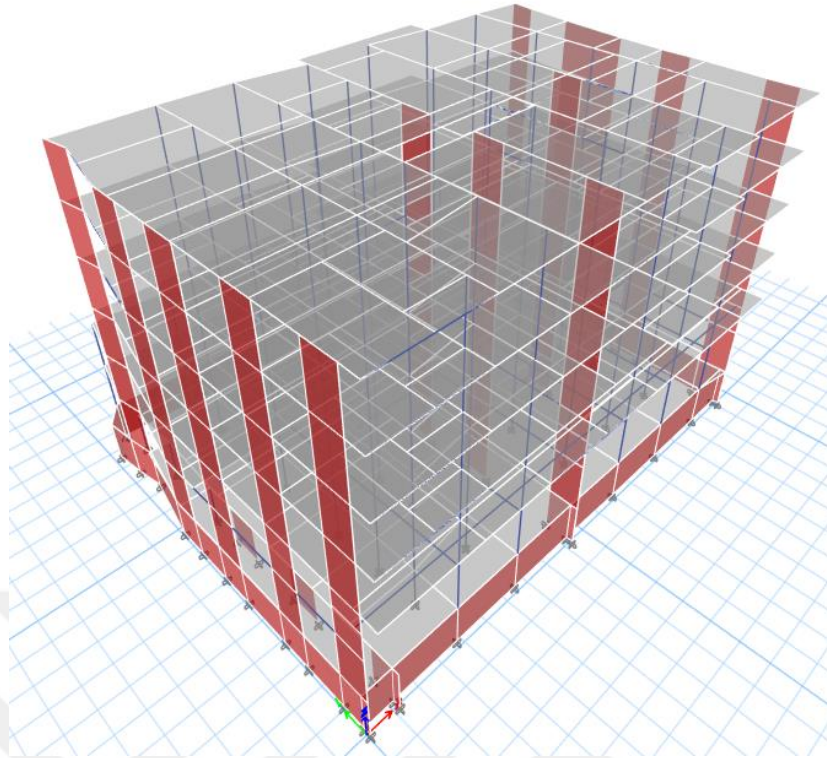


Figure 4.8: Three-dimensional view of the structural system modelled in ETABS.

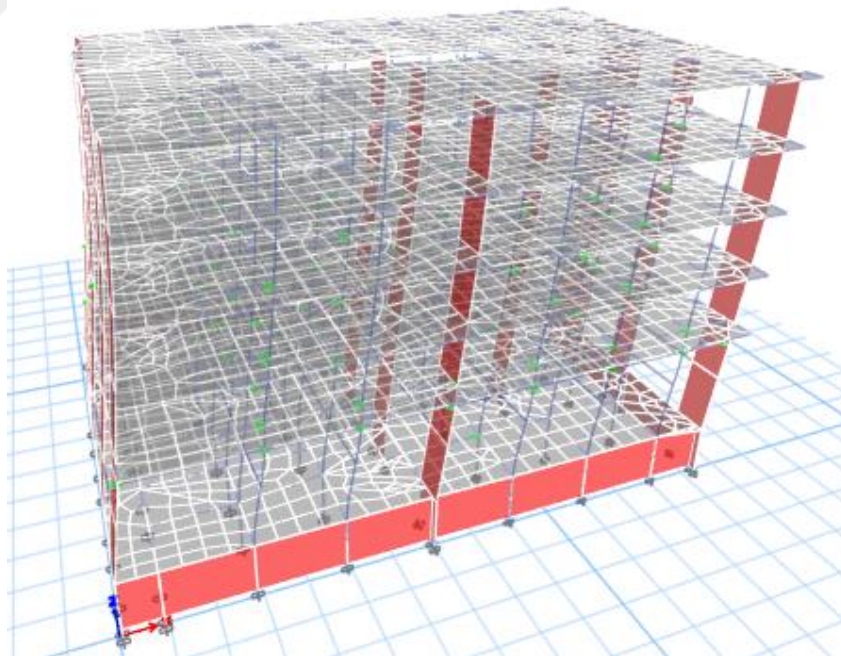


Figure 4.9: Distribution of distributed and lumped plastic hinges in the building.

4.4 Evaluation of Results Obtained with the RSRA Method

The Rapid Seismic Risk Assessment (RSRA) method is a practical approach that aims to quickly evaluate the seismic safety of buildings using field-collected data and engineering calculations. However, in order to assess the reliability of this method, it must be compared with performance analysis methods that are analytically more complex and provide higher accuracy. In this context, the results obtained with the RSRA method have been compared with the nonlinear pushover analysis results for the same buildings.

In this chapter, nonlinear analyses were carried out in ETABS for a total of 28 reinforced concrete buildings located in different regions of Turkey, using their original design drawings and field data. In these analyses, each structure was modelled in accordance with its existing static project; no template or “typical” model was used for any of the buildings. Material properties were defined directly based on experimental field data. Core strengths, bar diameters and layouts from rebar exposure and x-ray scanning, stirrup spacing and reinforcement ratios were defined for each model to reflect the in-situ conditions. In this way, an analysis environment capable of representing the detailed performance of each structure was established.

In modelling mechanical behaviour, distributed plasticity was adopted for columns, lumped plasticity for beams, and the MVLEM model for walls. All plasticity limits were set in accordance with the performance limit values defined in TBEC-2018 Sections 5 and 15. As a result of the pushover analyses, the performance levels of each building were classified under the categories Collapse, Collapse Prevention, Controlled Damage, and Limited Damage in accordance with TBEC-2018. These data were then compared with the previously calculated RSRA risk scores for the same buildings, and the predictive performance of the RSRA method in the field was examined. In addition, a numerical relationship was established between the RSRA score of each building and the spectral acceleration factor obtained from the nonlinear analyses; the consistency of the RSRA method was investigated on the basis of this relationship.

Through this comparison, the building groups for which the RSRA method provides higher accuracy, the conditions under which significant deviations occur, and the overall level of agreement were evaluated statistically. Thus, the validity, reliability,

and applicability of the RSRA method were tested within a scientific framework, and its engineering performance was supported by explicit analysis outputs. The types, geographical distribution, and basic structural characteristics of the buildings used in this comparison are described in Section 4.6, while inconsistencies and deviations observed in the relationship between RSRA scores and spectral factors are discussed in detail in Section 4.7.

4.5 Determination of Strength Limits Using the Spectral Factor

After completing the nonlinear analyses of the buildings, it was first evaluated whether each building satisfied the “Controlled Damage” performance level. For those buildings that did not meet the required performance at the initial analysis, the spectral acceleration value S_{ae} was gradually reduced, as illustrated in **Figure 4.10**; at each reduction step, a new performance analysis was carried out, and the minimum spectral demand under which the building could satisfy the Controlled Damage level was determined.

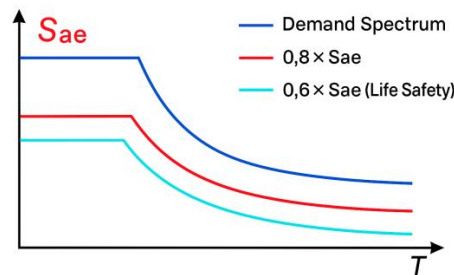


Figure 4.10: Stepwise reduction of spectral acceleration.

Conversely, for buildings that initially satisfied the Controlled Damage level, the S_{ae} value was gradually increased (e.g., by factors of 1.2, 1.4, 1.6), as shown in **Figure 4.11**, and the analyses were repeated until the collapse limit was reached.

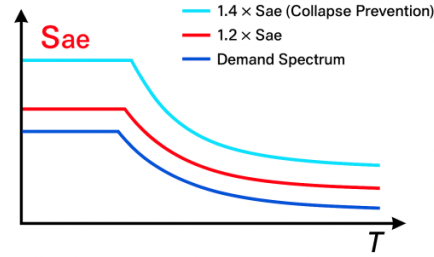


Figure 4.11: Stepwise increase of spectral acceleration.

In this way, for each building, the maximum or minimum spectral acceleration value that can be accepted as its performance limit was determined, and these values were interpreted in relation to the RSRA scores. Throughout this process, the deformation-based assessment criteria defined in TBEC-2018 were adopted, and the damage levels of elements were determined in accordance with the definitions given in the code.

4.6 General Characteristics of the Analysed Buildings

Within the scope of this study, a total of 28 reinforced concrete buildings located in various cities of Turkey were investigated. The buildings analysed are located in the following cities:

- Adana
- Adiyaman
- Amasya
- Antalya
- Erzincan
- Giresun
- İstanbul
- Mersin
- Rize
- Samsun
- Tokat
- Trabzon
- Zonguldak

The number of storeys of the buildings ranges from a minimum of 2 to a maximum of 9, with the majority consisting of 5- or 6-storey buildings. The main target of the RSRA method is to evaluate the seismic risk of such buildings, which constitute a large proportion of the building stock in Turkey. The construction years of the analysed buildings range between 1965 and 2013. This allowed the performance of both pre-2010 and post-2010 buildings to be evaluated and compared. During field studies, the following engineering data were obtained for each building:

- Concrete strength measured by core tests
- Reinforcement layouts determined through bar exposure and x-ray scanning
- Stirrup spacing and diameter measurements
- Architectural and structural drawings
- Geometric data such as storey heights, span lengths, and column and beam dimensions
- Soil class and spectral acceleration coefficients (S_{DS} , S_{D1})

4.6.1 Damage Distributions and Plastic Hinge Observations

During the pushover analyses, damage distributions and plastic hinge formations in the buildings were examined in detail. The analysis outputs, evaluated both at member and storey levels, provided important insights into the structural behaviour. In line with the plasticity regions defined in the models, the behaviour of distributed and lumped plastic hinges was assessed using the colour-coded output generated by the ETABS software. Damage distribution and plastic hinges in beam and column elements were expressed symbolically with the colours shown in **Figure 4.12**, consistent with the limit values defined in TBEC-2018:

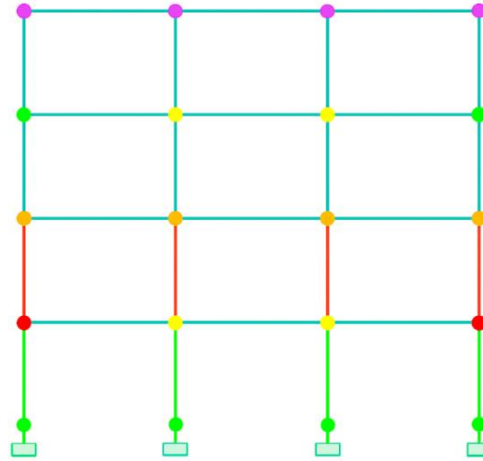


Figure 4.12: Symbolic representation of damage distribution and plastic hinge formation.

- Green: Elastic behaviour
- Yellow: Limited Damage
- Orange: Significant Damage
- Red: Collapse region

For each building, the sequence of plastic hinge formation in the X and Y directions was examined step-by-step; the timing and spread of hinges, as well as their locations on the members, were taken into account. Early plasticity in columns, in particular, provided important indications regarding potential collapse mechanisms. In some buildings, plastic hinges were concentrated mostly in beams while columns remained within elastic limits. In contrast, in other examples, column plasticity occurred earlier and was associated with differences in stiffness between storeys, strength irregularities, and overall structural irregularities.

In systems composed only of moment frames, located in regions with high spectral acceleration values, it was observed that storey drifts, especially at the ground floor, increased significantly and that plastic hinges were predominantly concentrated in the ground-floor columns, as shown in **Figure 4.13**.

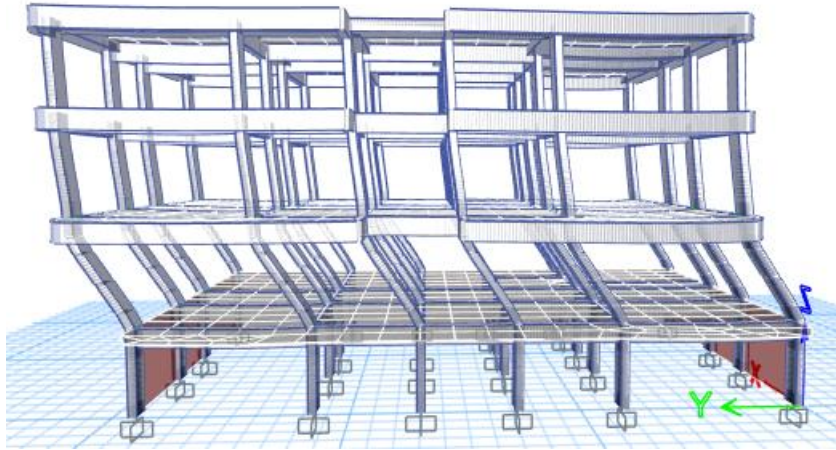


Figure 4.13: Deformation distribution in a frame-type building obtained from pushover analysis.

The high storey drifts observed at the ground floor led to concentrated damage in this region, as illustrated in **Figure 4.14**.

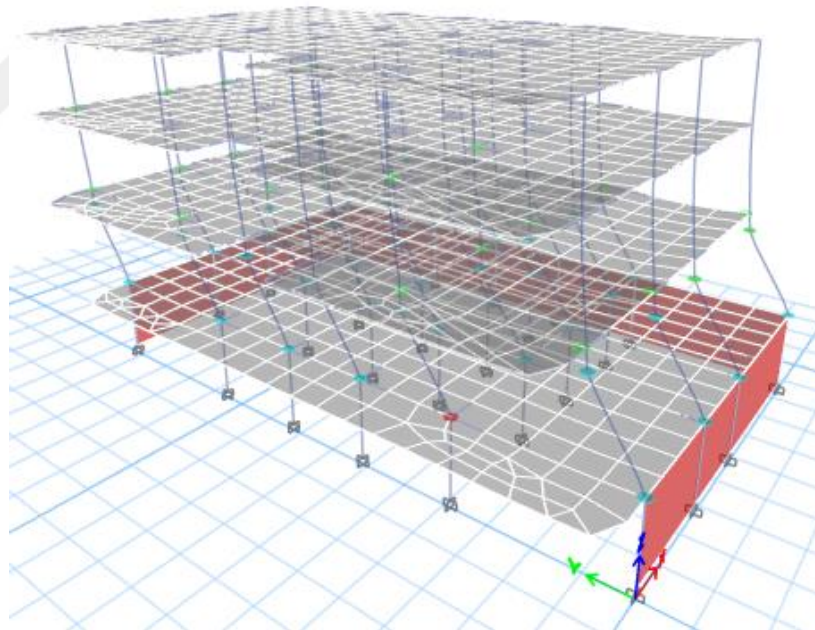


Figure 4.14: Concentration of plastic hinges particularly in ground-floor columns.

This is a clear indication of the weak-storey effect. Compared with these buildings, systems with reinforced concrete walls exhibited more uniform damage distribution and plasticity extending into upper storeys. **Figure 4.15** presents the deformed shape of a wall-frame building after analysis, where a more balanced drift distribution

between storeys can be observed. This highlights the contribution of walls to stiffness and lateral load-carrying capacity.

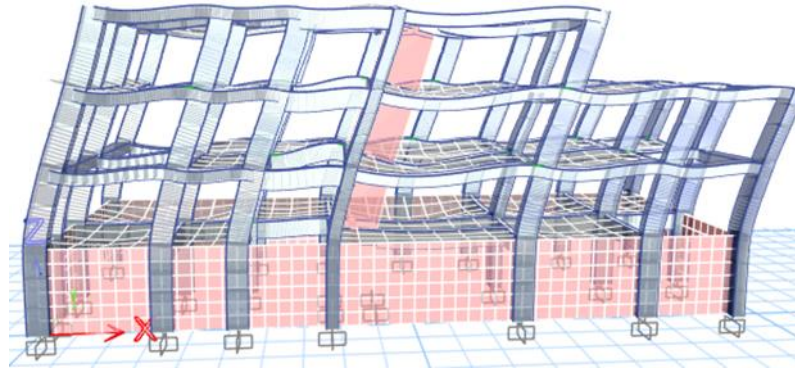


Figure 4.15: Deformation pattern in a wall-frame building.

A comparison of plastic hinge distributions is shown in **Figure 4.16**. In the wall-frame system, plastic hinges are mostly concentrated in beams, while columns sustain comparatively less damage. This confirms the protective role of walls for columns.

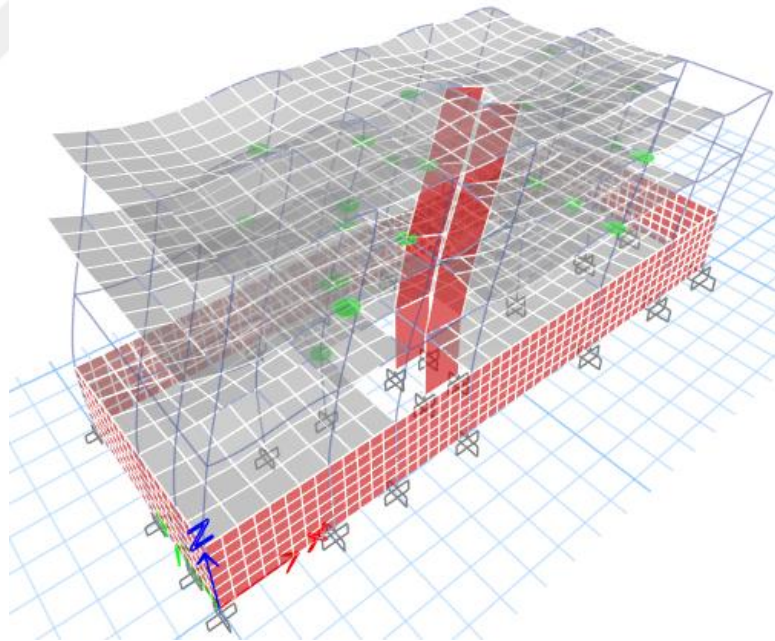


Figure 4.16: Homogeneous plastic hinge distribution in a wall-frame building.

The interstorey drifts of the two systems are compared in **Figure 4.17**. The pronounced deformation at the ground floor of the frame-only structure is identified as one of the most significant parameters triggering the formation of a weak storey.

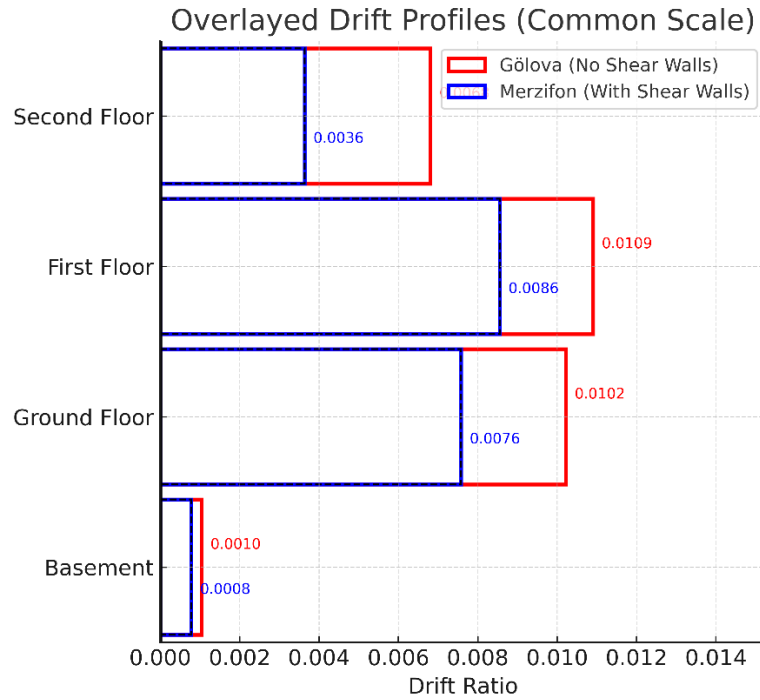


Figure 4.17: Comparison of interstorey drifts in a wall-frame and a frame-only.

To demonstrate the contribution of walls to structural behaviour more concretely, a comparative study was carried out on two different buildings with similar number of storeys and total mass. As presented in **Table 4.1**, a building located in the Merzifon district of Amasya, with walls, solid slabs, rigid basement walls on all four sides, high concrete strength, and S420 reinforcement, was compared with a building in the Gölova district of Sivas, which has no walls, waffle (asmolen) slabs, basement walls on only three sides, relatively low concrete strength, and S220 reinforcement. Both buildings have the same number of storeys (B+G+2) and similar mass.

In order to avoid the influence of different seismic hazard levels on the comparison, the building in Gölova was analysed under the seismic hazard parameters of Merzifon. Thus, the behaviour of the two buildings under the same seismic conditions was investigated. According to the analysis results, the interstorey drifts in the walled building in Merzifon remained at lower levels, whereas those in the wall-less building in Gölova increased significantly, showing a higher tendency toward weak-storey formation.

Table 4.1: Comparison of two buildings located in Gölova and Merzifon

Property	Merzifon (Amasya)	Gölova (Sivas)	Comment
Number of storeys	B+G+2	B+G+2	Same
Building mass	1000 t	1250 t	Suitable for comparison
Slab system	Solid slab	Waffle (asmolen) slab	Solid slabs are stiffer and reduce drift
Basement walls	Rigid on all four sides	Rigid on three sides	Difference in basement rigidity is critical
Shear walls	Present	Absent	Advantageous in terms of collapse mechanism
Concrete strength	High	Low	Advantage in material strength
Reinforcement grade	S420	S220	Advantage in material strength
Drift ratio	Low	High	Consistent with RSRA scoring
Ground-floor storey height H_n	260 cm	350 cm	Higher storey height at ground level reduces lateral stiffness and increases interstorey drifts; this effect is pronounced in the Gölova building and leads to concentrated plasticity at ground floor.
Typical storey height H_n	260 cm	290 cm	With increased displacement demand, drift ratios rise in upper storeys of the Gölova building as well.
RSRA score	88	2280	RSRA scores consistently reflect the differences in material quality and stiffness between the two buildings.

Within the RSRA method, parameters such as the presence of walls and the weak-storey effect are considered among critical factors and are explicitly included in the

scoring. In this respect, a certain consistency is observed between rapid field assessments and detailed structural performance analyses. These findings confirm that the weighting assigned to parameters such as presence of walls, basement rigidity, slab type, concrete strength, and reinforcement grade in the RSRA method is appropriate. In other words, the inclusion of these criteria in RSRA scoring is based not only on field observations but also on nonlinear analysis results, demonstrating that the method rests on a scientifically sound approach.

4.7 Cases Outside the Scope of the RSRA Method

The RSRA method is a screening approach that aims to identify the relative risk levels of buildings, rather than classifying them directly as “safe” or “unsafe”. In this context, the scores produced by the method are derived by considering the engineering quality, material condition, construction defects, and structural system characteristics, and they provide a means of ranking buildings according to their relative risk. However, this relative assessment system may not always correspond one-to-one with deterministic methods such as performance analyses.

This discrepancy becomes particularly evident in the score range of 351–500, which can be regarded as an “outside the main scope” region of the RSRA method. In some buildings within this interval, performance analyses indicate that the Controlled Damage level is satisfied, while the RSRA method yields a relatively high risk score. The main reason is that RSRA does not consider only the seismic response under design loads; it also takes into account engineering quality, design integrity, and field observations. As a consequence, especially in regions with low seismic demand, a building may exhibit adequate performance under pushover analysis, whereas poor construction quality drives the RSRA score into a higher risk category.

For example, short-column irregularities, heavy cantilevers, an abundance of stub beams, insufficient stirrup spacing, or poor concrete quality are factors that increase the RSRA score even if the building remains within Controlled Damage under moderate seismic demands. Since such design deficiencies are considered to undermine the overall engineering safety of the building, the RSRA method reflects these buildings as relatively more risky. Therefore, a building that satisfies the pushover performance requirements but receives a high RSRA score in the “grey zone”

does not represent a methodological contradiction; rather, it is a consequence of the multi-dimensional evaluation approach of RSRA.

For this reason, special care is required when assessing buildings that fall within this outside-scope interval. The uncertainty of RSRA in this score range naturally leads to a region where the method's capacity to provide a definitive judgement is limited. In this region, instead of simply stating that a building is “unsafe”, RSRA indicates that “it may be more risky compared with other buildings”. Thus, RSRA should be regarded not as a deterministic performance analysis method, but as a field-based, rapid, and prioritisation-oriented filtering system.

In order to reach a sound final decision on such buildings, those within the outside-scope interval must be evaluated together with supporting performance analyses. Furthermore, field data—such as concrete strength, aggregate homogeneity, column stirrup spacing, and corrosion condition—must be interpreted in conjunction with structural behaviour and incorporated into engineering judgement.

In conclusion, the RSRA method is not merely a numerical scoring tool; it is also a multi-dimensional evaluation mechanism that can intuitively analyse structural behaviour, design practice, and construction quality. In this respect, it serves as a powerful preliminary assessment tool for engineering-level screening of large building stocks.

4.8 An Example of an Outside-Scope Case: A Reinforced Concrete Building in Sivas City Centre

The building located in the Kümbet district of Sivas was evaluated with the RSRA method and received a score of 410, placing it within the “outside-scope region”. In the performance analysis, the building initially satisfied the Controlled Damage level; however, when the spectral acceleration factor was gradually increased, the structure reached the collapse state only at a factor of 2.0. Although this indicates the presence of some reserve capacity, the discrepancy between the RSRA score and the performance analysis result can be explained by the deficiencies in engineering and construction quality (**Figure 4.18**).



Figure 4.18: The building located in the Kümbet district of Sivas city centre.

The concrete strength of the building is 15.67 MPa, which is below the desired compressive strength. The reinforcement used is of grade S220 and plain (non-ribbed). It was observed that stirrup spacing locally exceeds 20 cm and that no confinement zones are provided. The building has no shear walls; as shown in **Figure 4.19**, it has an irregular frame system with stub beams. Openings in the architectural layout lead to short-column formation, heavy cantilevers on the façade, and asymmetrical planning that induces torsional irregularities. There are stiffness irregularities at the ground floor, and there is a mismatch in behaviour between the ground floor and the upper storeys.

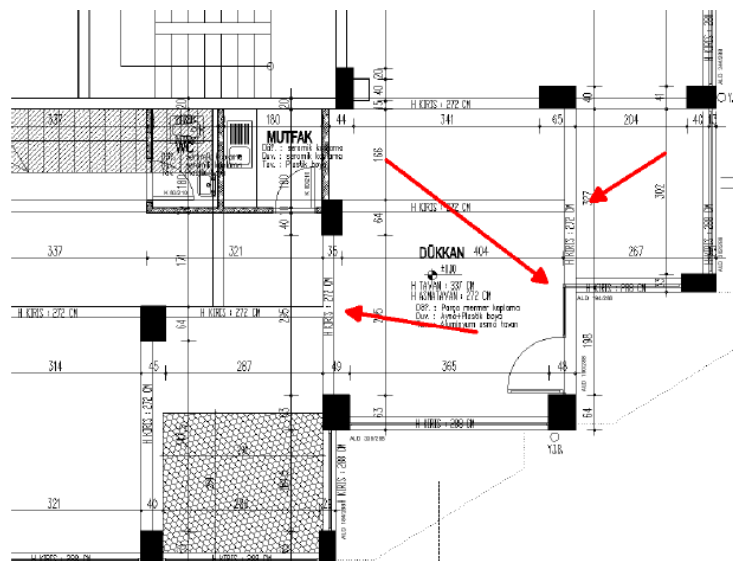


Figure 4.19: An irregular frame system with stub beams.

In addition, according to the structural drawings obtained from the municipality, the building was designed as B+G+7 storeys, whereas site investigations revealed that it was actually constructed as B+G+4 storeys. This discrepancy shows that assessments based solely on design documents may be misleading and underscores once again the importance of field-based methods such as RSRA. Therefore, in this study, the analytical model was revised according to the number of storeys actually constructed, and information from the municipal project was used only selectively where appropriate.

This example illustrates that performance analysis tests the structural behaviour under specific loading scenarios, while the RSRA method provides a multi-dimensional assessment based on field observations and engineering attributes. It is clear that buildings in the outside-scope region must be examined not only in terms of analysis results but also in terms of construction quality and design integrity.

5. RESULTS AND DISCUSSION

In this chapter, the consistency between the developed Rapid Seismic Risk Assessment (RSRA) method and nonlinear performance analysis results is evaluated using statistical indicators and comparative results. Initially, sensitivity, specificity, accuracy, and area under the curve (AUC) values are presented to quantify the predictive performance of the RSRA scoring system. Subsequently, the numerical relationship between RSRA scores and seismic parameters, particularly the spectral multiplier (SPEC), is examined. Furthermore, the average seismic risk values of buildings are analyzed based on construction year, providing insights into the temporal evolution of seismic safety within Turkey's existing building stock. When appropriate, regional comparative evaluations are also incorporated.

5.1 Statistical Evaluations

The Rapid Seismic Risk Assessment (RSRA) method developed within the scope of this thesis aims to evaluate and prioritize buildings according to seismic risk through an engineering-based, field-observation-driven, and scoring-oriented framework. Although conventional performance analysis approaches can reveal the seismic behavior of buildings in considerable detail, they remain inadequate for screening large building stocks due to the long analysis duration, the difficulty of obtaining the required data, and the need for specialized expertise. Moreover, they often classify buildings simply as adequate/inadequate without indicating which structures are comparatively more critical. For these reasons, the need for a faster and more practical approach necessitated the development of the RSRA method.

A wide range of rapid screening methods are used internationally, including FEMA 154, ATC-14, JBDPA (Japan), and the Canadian Seismic Screening Method. A common feature of these methods is that they typically attempt to determine building risk based on visual inspection or a limited set of simplified calculations. However, because these systems frequently do not explicitly consider key structural parameters—such as concrete compressive strength, reinforcement detailing, stirrup spacing, and vertical discontinuities—they may not yield sufficiently discriminative

results, particularly in countries with heterogeneous and complex building inventories such as Türkiye.

The RSRA method developed in this thesis establishes a quantitative scoring system that expresses earthquake risk by using parameters that can be directly identified in the field and that provide a high level of information on the engineering quality of the building. Accordingly, the method is structured not merely as a visual screening approach, but as an integrated system that holistically captures engineering vulnerabilities such as concrete strength, reinforcement layout, column stirrup spacing, short-column irregularity, overhang/projection conditions, and soft-story behavior.

With the latest version of the method, equivalent static models were created for selected buildings (those evaluated in the current version) using a dedicated OpenSees-based software interface. Engineering outputs obtained from linear analyses of these models—such as key dynamic parameters, irregularity types, and demand-to-capacity ratios—formed the numerical basis for several parameters used in the RSRA method. Consequently, RSRA evolved beyond a purely observation-based system and became a hybrid framework that also incorporates numerically supported engineering assessments.

Field photographs, three-dimensional ETABS model images, and floor plans for the 28 buildings subjected to performance analysis in this study are provided in Appendix 2. The performance levels obtained from nonlinear analyses conducted in ETABS were evaluated by comparison with the RSRA scores of each structure. For each building, where the controlled damage performance level could not be achieved, the spectral acceleration (S_a) was reduced incrementally; for buildings that satisfied the performance requirement, the acceleration coefficient was increased stepwise and the capacity was examined up to the collapse limit. As a result of these analyses, a strong inverse relationship was identified between the RSRA score and the spectral multiplier. This relationship was investigated first graphically and then through regression analyses. **Figure 5.1** presents a comparative depiction of the RSRA scores for 28 buildings and the minimum spectral multiplier values at which these buildings reached the controlled damage level. The graph indicates that as the RSRA score increases, the spectral multiplier decreases; that is, buildings classified as high-risk by the RSRA method reach the controlled damage threshold under lower seismic demands.

Regression analysis is one of the fundamental statistical approaches for revealing the relationship between an independent and a dependent variable. Within this thesis, the relationship between RSRA scores and the spectrum multipliers derived from performance analyses was tested using both linear and polynomial regression models. The findings indicate that the polynomial model provides a better fit than the linear model. The obtained value of R^2 of approximately 0.605 points to a strong level of association when interpreted against the thresholds suggested by **Cohen (1988)** and **Hair et al. (2010)**.

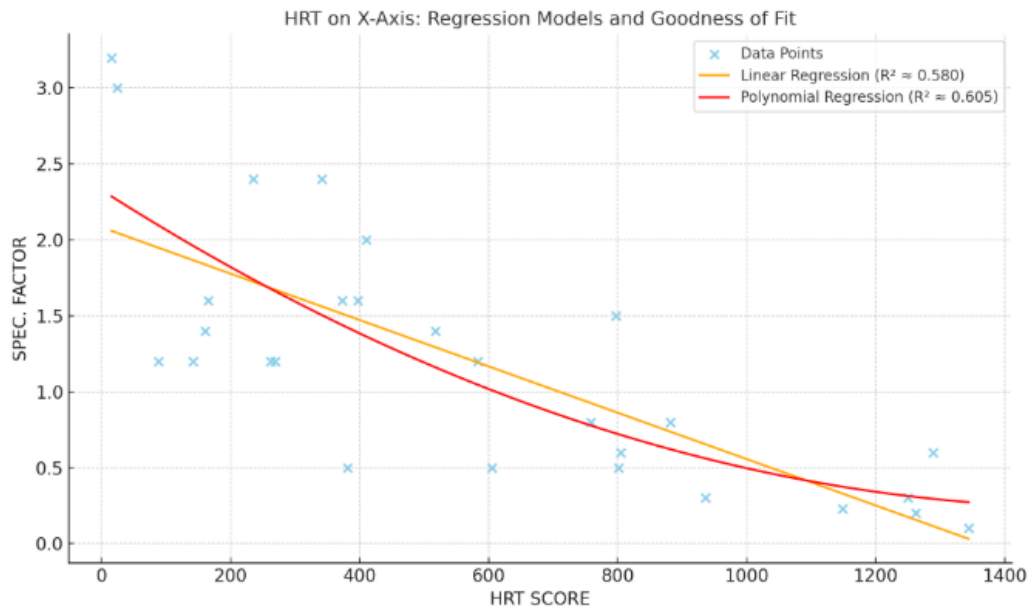


Figure 5.1: Relationship between RSRA score and spectral multiplier (with linear regression curve).

In the regression analysis, various model types were examined, and a second-degree polynomial model was observed to provide the best fit to the dataset. The coefficient of determination for this model was calculated as $R^2 \approx 0.605$, indicating a statistically meaningful and strong correlation between performance analysis results and RSRA scores. This inverse relationship between the RSRA score and the spectral multiplier demonstrates that the preliminary risk assessment provided by RSRA is consistent with structural performance.

In the subsequent stage, it was recommended that buildings with RSRA scores of 500 and above be evaluated under the qualification of "may require evacuation". This threshold was statistically tested based on both engineering-based field observations

and comprehensive performance analyses. The validity of the 500-point evacuation threshold was assessed using Receiver Operating Characteristic (ROC) analysis. ROC analysis is among the most widely used techniques for measuring the discriminative capability of binary classification models by comparing sensitivity and specificity values under different decision thresholds. The area under the ROC curve (AUC) is the key metric summarizing overall model performance. Buildings with a spectral multiplier below 1.0 were classified as "weak", and the discriminative capability of the RSRA score for this classification was evaluated accordingly. Based on the ROC curve presented in **Figure 5.2**, the AUC (Area Under the Curve) value was calculated as 0.964. This value indicates that RSRA scores perform with very high accuracy in identifying weak buildings.

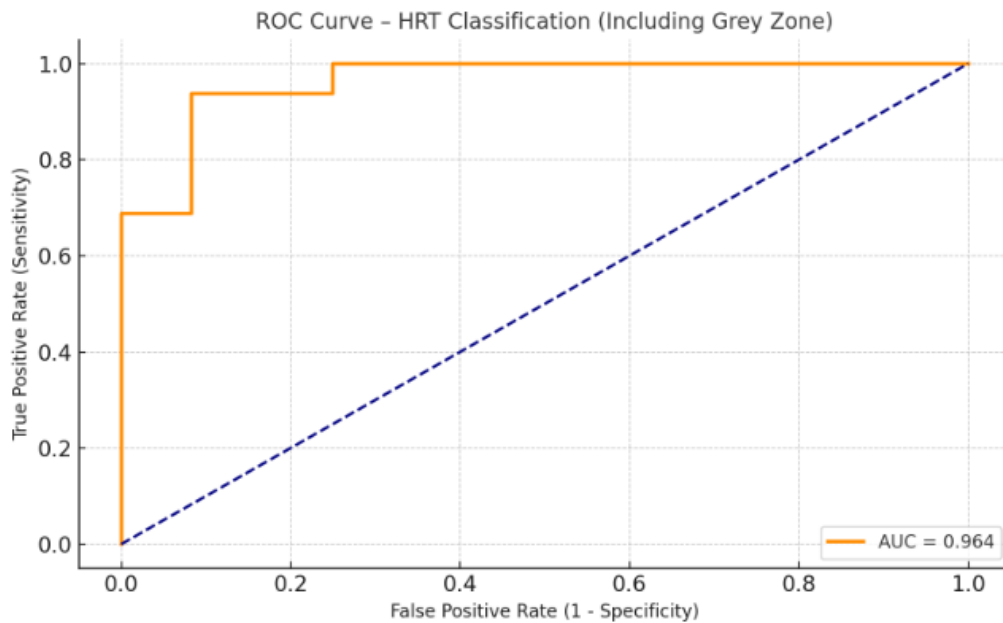


Figure 5.2: ROC curve: prediction of weak buildings using RSRA score.

Within this thesis, the classification based on the $RSRA \geq 500$ decision threshold was tested. The resulting statistics were obtained as follows: accuracy = 85.7%, sensitivity (correct identification rate of weak buildings) = 91.7%, specificity (correct retention rate of safe buildings) = 81.3%, and AUC = 0.964. These results correspond to the definition of "excellent discrimination" commonly used in the literature for $AUC \geq 0.90$ (Hinkle et al., 2003; Hair et al., 2010). Therefore, it is verified that the RSRA method is not merely an intuitive approach based on field observations but also a statistically validated filtering tool with a high level of reliability.

The statistical findings demonstrate that the predictive capability of the RSRA method for earthquake risk is considerably high. Accuracy above 85%, sensitivity above 90%, and specificity above 80% indicate that the method can successfully differentiate risky buildings while also preventing safe buildings from being unnecessarily categorized as risky. Considering the commonly accepted threshold values in the literature (Cohen, 1988; Hair et al., 2010), this performance level can be regarded not only as "acceptable" but also as "high reliability". As will also be seen in **Table 5.1**, this confirms that RSRA is a strong tool for rapid risk identification from both practical and scientific perspectives.

Table 5.1: Threshold values for statistical performance levels.

Analysis Metric	Source	Threshold Values	Interpretation
R ² (Coefficient of Determination)	Hair et al. (2010)	R ² ≥ 0.25: Acceptable R ² ≥ 0.50: Good R ² ≥ 0.75: Very good	R ² ≈ 0.60 in this study → Good explanatory power
ROC (AUC) Value	Hinkle et al. (2003); Cohen (1988)	AUC ≥ 0.90: Excellent 0.80–0.90: Good 0.70–0.80: Acceptable	AUC = 0.964 in this study → Excellent discriminative power
Sensitivity	—	≥ 90%: Very good 70–90%: Acceptable	91.7% in this study → Very good
Specificity	—	≥ 70%: Acceptable ≥ 85%: Very good	81.3% in this study → Acceptable
Accuracy	—	≥ 80%: Good	85.7% in this study → Good overall performance

Source: Prepared based on Hinkle et al. (2003), Cohen (1988), and Hair et al. (2010).

In conclusion, the RSRA method is proposed not only as a rapid approach but also as an engineering-validated, field-applicable, and scalable screening strategy. In earthquake-prone regions where large building inventories must be assessed quickly, it stands out as an effective and practicable tool for municipalities, public institutions, and disaster management authorities. The 500-point threshold proposed in this study

was found to be statistically meaningful and supported by engineering criteria. In this respect, RSRA can be regarded not merely as a scoring system, but as an assessment approach that interrogates the engineering design quality of buildings in a multi-dimensional manner.

5.2 Relationship Between Construction Year and RSRA Score

Within the scope of this thesis, RSRA assessments were carried out for a total of 513 buildings. However, 39 of these buildings lacked definitive information on construction year, and 47 were constructed in 2010 or later; therefore, they were not included in the period-based analyses. Under normal conditions, post-2010 buildings are not included in RSRA assessments because they fall under the building inspection system. Nevertheless, following the 6 February 2023 Kahramanmaraş Earthquakes, RSRA was exceptionally applied to certain buildings located in the earthquake-affected region that were constructed after 2010 but for which damage levels were uncertain. In addition, RSRA was conducted for buildings planned to be leased by the bank—particularly in small districts in Eastern and Southeastern Anatolia—where approved municipal projects or occupancy permits were missing. Accordingly, these 86 buildings were not a deviation from the method but rather special investigations required by field conditions. As the focus of the study is the evaluation of the pre-2010 building stock, these two groups were excluded, and 427 buildings with known construction years were included in the evaluation. This choice both emphasizes the structural breakpoint corresponding to the nationwide implementation of the building inspection system as of 2010 and ensures that the analyses are carried out on a more consistent basis.

The average RSRA scores obtained from calculations performed for 427 buildings are presented in **Figure 5.3**. The analysis results make it possible to evaluate the temporal development of seismic safety within Türkiye's building stock.

The number of buildings examined in the 1970–1979 period is relatively limited ($n = 53$). Nevertheless, the high average RSRA scores indicate that buildings constructed in this period are considerably risky. In the 1980–1989 ($n = 95$) and 1990–1999 ($n = 134$) periods, as the number of evaluated buildings increased, the average scores

remained at similarly high levels, suggesting that the expected improvement in construction quality was not achieved. In the 2000–2009 period, the largest number of buildings was examined ($n = 145$). The relatively lower average RSRA scores in this period indicate a partial improvement in seismic safety.

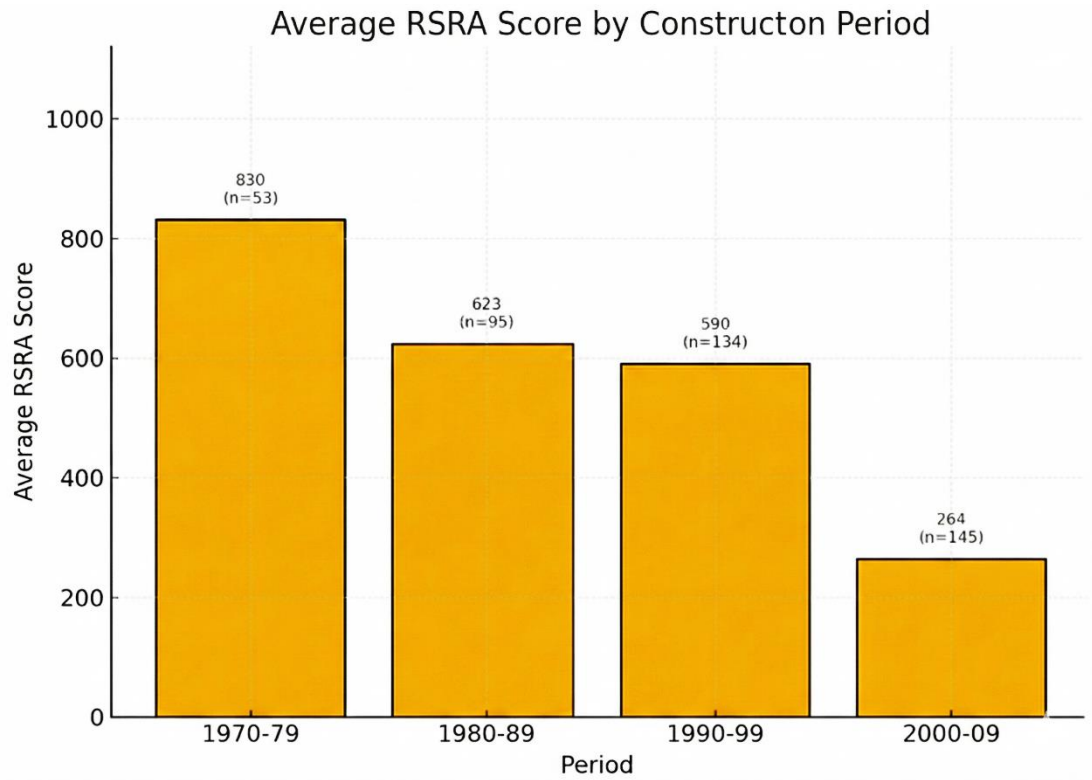


Figure 5.3: Distribution of RSRA scores by construction period.

A similar trend was observed in the official post-earthquake damage assessments conducted after the 6 February 2023 Kahramanmaraş Earthquakes. According to the Ministry of Environment, Urbanization and Climate Change, it was reported that 98% of the collapsed buildings were constructed prior to 1999, and only 2% belonged to the post-1999 period (Republic of Türkiye Presidency Directorate of Communications, Bulletin dated 15–16 February 2023). In the same assessments, it was determined that only 1.44% of new buildings subject to the post-2010 building inspection system collapsed. This outcome demonstrates the effectiveness of both the 1999 Earthquake Code and the post-2010 building inspection system and clearly reveals the progress achieved in seismic safety for newer buildings. In RSRA analyses, the rationale for excluding buildings constructed after 2010 is consistent with these findings (see **Table 5.2**).

Table 5.2: Damage distribution by building age in the 6 February earthquake (Old Buildings → Pre-1999).

As of February 14, 2023,
Collapsed Building Assessment Work

Province Name*	Number of Collapsed Buildings	Old Building**	New Building	Old Building (% percent)	New Building (% percent)
Adana	13	12	1	92,31	7,69
Adıyaman	1348	1315	33	97,55	2,45
Diyarbakır	52	52	0	100,00	0,00
Gaziantep	2648	2622	26	99,02	0,98
Hatay	1821	1780	41	97,75	2,25
Malatya	1124	1119	5	99,56	0,44
Kahramanmaraş	1816	1790	26	98,57	1,43
Şanlıurfa	30	30	0	100,00	0,00
Kilis	146	146	0	100,00	0,00
Osmaniye	148	148	0	100,00	0,00
TOTAL	9146	9014	132	98,56	1,44

Overall, the findings indicate that no notable improvement in seismic safety occurred in Türkiye from the 1970s to the 2000s, and that the primary breakpoint emerged in the post-2010 period. The strengthening of the building inspection system, the wider availability of engineering services, and the improvement of material quality contributed to a visible enhancement in building safety during this period.

5.3 Regional Evaluations

Within the scope of this thesis, RSRA evaluations conducted for 427 buildings constructed prior to 2010 and with known construction years were examined on the basis of Türkiye's seven geographical regions. Region-based proportions of safe (RSRA < 500) and risky (RSRA ≥ 500) buildings were determined, and the findings are summarized below (Figure 5.4).

- **Marmara Region (n = 194, 38.1% risky):** This region has the densest building stock in Türkiye. In absolute terms, the highest number of risky buildings was identified here (74 buildings). However, because a large number of safe buildings also exists, the proportion remained at approximately 38%. This result reflects the heterogeneous nature of the Marmara building stock, where older and newer buildings coexist.
- **Aegean Region (n = 77, 44.2% risky):** Low concrete strength, marine exposure effects, and insufficient inspection practices contributed to the high

proportion of risky buildings. In particular, the adverse effect of saline water on concrete quality in coastal provinces supports this finding.

- **Mediterranean Region (n = 51, 45.1% risky):** Rapid urbanization and irregular construction practices—especially in coastal areas—were influential. Additionally, field investigations after the 6 February 2023 earthquakes identified implementation deficiencies even in relatively new buildings, explaining the elevated risk ratio.
- **Central Anatolia Region (n = 34, 50.0% risky):** Although this high ratio may appear surprising at first, it can be explained by the proximity of the eastern parts of the region (e.g., Yozgat, Sivas, Çorum) to the North Anatolian Fault Zone. These provinces experience both higher seismic hazard and supervision deficiencies. Therefore, Central Anatolia is not necessarily as low-risk as commonly perceived.
- **Black Sea Region (n = 80, 36.2% risky):** Because seismic hazard is relatively lower in this region, the risky proportion is comparatively smaller. Nevertheless, certain irregularities associated with topography and site conditions exist in coastal cities. The relatively low proportion may also be influenced by the generally younger building stock and sample characteristics.
- **Eastern Anatolia Region (n = 35, 48.6% risky):** A substantial portion of the region is located in high seismic hazard zones. Weak material quality and limited inspection conditions were decisive in the high risky proportion.
- **Southeastern Anatolia Region (n = 3, 100% risky):** The sample size in this region is very small. Although all buildings were classified as risky, generalization is not methodologically appropriate. However, the limited available data indicate that construction quality and inspection deficiencies may also be a serious issue in this region.

Number of Buildings with RSRA by Region and Risk Rate (427 Records, Before 2010)

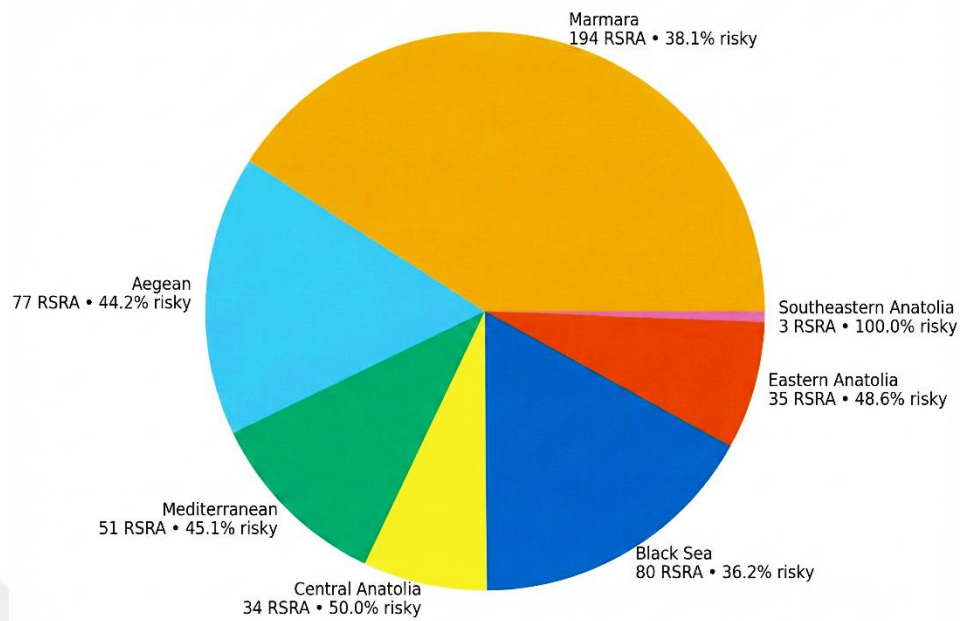


Figure 5.4: Regional distribution of risky buildings.

Figure 5.4 presents the regional distribution of RSRA evaluations for 427 buildings constructed before 2010 across Türkiye's seven geographical regions. Slice sizes represent the total number of buildings evaluated in each region, and the percentages shown on labels correspond to the proportion of risky buildings ($RSRA \geq 500$). The chart indicates that the proportions of risky buildings are relatively high in Marmara, Aegean, Mediterranean, and Eastern Anatolia; that the proportions are relatively low in the Black Sea region; and that Central Anatolia exhibits higher-than-expected values. The Southeastern Anatolia region cannot be generalized due to representation by only three buildings.

Overall, the findings indicate that Türkiye's building stock is not homogeneous in terms of seismic safety. While higher risky proportions were observed in Marmara, Aegean, Mediterranean, Central Anatolia, and Eastern Anatolia, lower proportions were identified in the Black Sea region. The fact that the results coincide with public perception in certain aspects (e.g., Marmara and Aegean being relatively risky) and diverge in others (e.g., the Black Sea being lower and Central Anatolia being higher) demonstrates that the RSRA method produces objective outcomes grounded in field data.

5.4 Overall Evaluation and Remarks

It is evident that the RSRA method developed in this study provides a comprehensive perspective. The findings obtained throughout the study reveal a high level of consistency between the method and earthquake performance analyses ($AUC = 0.964$, accuracy = 85.7%, sensitivity = 91.7%, specificity = 81.3%). These values demonstrate that the RSRA method is not only a theoretical model but also a field-applicable and reliable decision-support tool.

Comparisons by construction year indicate that, from the 1970s through the 2000s, the expected level of improvement in seismic safety in Türkiye was not achieved and that, in some periods, risk may even have deepened. In contrast, a visible improvement in construction quality was identified beginning in the second half of the 2000s and especially after 2010, coinciding with the introduction and strengthening of the building inspection system. This improvement is directly associated with the effectiveness of engineering services, advancements in material technologies, and the mandatory nature of supervision mechanisms.

The results also indicate that regional differences in the building stock remain pronounced. In particular, in Eastern and Southeastern Anatolia, questionable conditions were identified in certain buildings—including some newer structures—in terms of load-bearing system configuration, material quality, and construction practice; therefore, detailed evaluations were performed using the RSRA method. This finding shows that seismic safety is linked not only to legal regulations but also directly to the engineering culture and implementation quality in practice.

In general, the RSRA method goes beyond conventional rapid screening approaches by interrogating not only externally observable characteristics but also engineering parameters, material performance, and design quality. Accordingly, RSRA becomes not only a filtering tool that identifies risky buildings but also a strategic assessment mechanism capable of prioritizing interventions from the standpoint of seismic safety.

In conclusion, the RSRA method developed within the scope of this thesis offers important contributions toward a more systematic and reliable evaluation of Türkiye's existing building stock in terms of seismic safety. The high accuracy levels, statistical reliability, and field applicability of the method indicate significant potential for future academic research and public policy applications.

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BIOGRAPHY

Kadir AKMAN completed his entire formal education in Konya, Türkiye. His academic and professional interests primarily focus on the seismic performance assessment of existing structures, structural health monitoring systems, and structural retrofit materials together with intervention and strengthening techniques.

He has taken part in post-earthquake damage assessment campaigns conducted in the aftermath of the İzmir, Kahramanmaraş, Van, and Elazığ earthquakes. In addition, he has contributed to projects involving the installation, operation, and evaluation of structural health monitoring systems in both data center facilities and historical buildings. His professional experience also includes participation in multidisciplinary projects related to the ground improvement, reconstruction, and seismic retrofitting of heritage structures, through which he has gained practical field experience as well as analytical competence.

The author has an advanced command of English. His personal interests include traveling, listening to music, and swimming. He is married and the father of two children.

PUBLICATIONS AND PRESENTATIONS FROM THE THESIS

Akman, K., Tuğsal, Ü. M., Avcıoğlu, O., & Tokmak, A. (2025, October 13–15). *Verification of newly developed rapid earthquake risk identification studies with earthquake performance analysis* [Conference presentation]. 7th International Resilience Congress, Fırat University, Elazığ, Türkiye.



APPENDICES

APPENDIX-A: SAMPLE RSRA REPORT

In this section, a sample RSRA report is examined. Explanations related to the report are written in italic type within green-colored boxes.

Ziraat Bank Istanbul Guzelyali Branch (for lease) was visited on 18.04.2025 to collect preliminary information on seismic risks. The bank manager stated that the building was constructed in 2003. The building consists of Basement + Ground Floor + 1 typical storey.

Explanation: The inspection date of the building appears in the first part of the RSRA report. Key attributes such as the number of storeys and the construction material/system are also stated in this section.



Figure A.1: Photo from structure

Notes: The concrete quality of the building was evaluated as 15.74 MPa based on the rebound hammer result and 20.51 MPa based on the concrete core result, and the assessment was conducted accordingly. As frequently observed in many applications in the construction year (2003), reinforcement of the older designation BC-III

(equivalent to S420 in current practice) was used and ductility requirements were not satisfied (wide stirrup spacing, no crossties). In the available structural drawings, since details such as crossties are not shown and the stirrup spacing is 17 cm, and because the design year appears older to the extent it can be read, it is considered that the project/design was prepared prior to 1998. Accordingly, the building construction year was taken as pre-1998 for scoring purposes.

The building consists of a beam–slab floor system. The storey heights are 3.70 m, 3.70 m, and 3.00 m for typical storeys, respectively. A V-shaped shear wall exists near the stairwell; however, since it is small relative to the overall building, the structure was considered a frame system. Therefore, torsional irregularity scoring was not assigned for this shear wall.

For soil class ZD, the short-period design spectral acceleration coefficient is $SDS = 1.146$ for the site.

Core sampling was requested; however, the owner did not grant permission.

The typical span of the frame system is approximately 6.40 m, and the frame is formed with columns of 40×50 cm.

According to the approximate model established, the fundamental period of the building was calculated as $T_1 = 0.203$ s. For this model, the maximum E/K ratio among columns was determined as 0.62. In the calculations, material strengths were multiplied by 0.8.

Severe corrosion is observed in the reinforcement, and the corresponding score was assigned accordingly. It is recommended to clean and repair corrosion at the ground floor. Based on observational investigations, the building's Structural Index score was obtained as 269 (Low risk). If the building is to be used, corrosion cleaning should be carried out and preventive measures should be implemented.

Explanation: Observations obtained through engineering investigations of the building are stated in this section. Information is also provided regarding concrete quality, the reinforcement used, the structural system, seismicity, and the current condition of the building, and an overall evaluation is made in light of these data.



Figure A.2: Location

To establish the representative frame, the building plan dimensions were assumed as 30.00 m × 24.00 m in accordance with the available drawings. The adopted frame was generated based on typical beam and column dimensions.

Explanation: The selected span layout and the number of grid lines/axes used to establish the simplified structural system are specified in this section.

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SERTLEŞMİŞ BETONDAN ALINAN NUMUNENİN BASINÇ MUKAVEMET (KAROT) RAPORU

Başvuru No/ Tarihi	1567735/09.05.2025	Rapor No/Tarih	181/13.05.2025
Alınış Tarihi	-	Bayındırlık Kayıt No.	-
Deney Tarihi	13.05.2025	Lab. Kod No.	-

FİRMA BİLGİLERİ

Proje Adı	Ziraat Bankası Şubeleri
Müteahhit Firma	Milenyum Proje Danışmanlık San. Ve Tic. Ltd. Şti.
Şantiye Adresi	-
Ada No. :-	Pafta:-
	Parsel:-

NUMUNE BİLGİLERİ

Numunenin Temsil Ettiği Kütle	-	Üretici Firma	-
Adet/Tip	9/ Karot	Beton Dök. Tarihi	-
Boyutlar (mm)	*	Hedeflenen Beton Sınıfı	-

UYGULANAN STANDARTLAR: TS 12390-3
DENEY SONUCU BULUNAN DEĞERLER

Numune No	Numune Ait olan Eleman	Yapı	Çap (mm) (*)	Numune Yüksekliği (mm) (*)	Kırılma Yüklü (kN)	Basınç Dayanımı (N/mm²)
1	Giresun Esiye Şube SZ01	-	74	77	82,7	19,2
2	Giresun Esiye Lojman SZ01	-	"	76	155,9	36,3
3	Gümüşhane Kelkit Şube SB01	-	"	76	107,5	25,0
4	Erzurum Köprüköy Şube SZ01	-	"	77	75,7	17,6
5	Van Edremit Şube SZ01	-	"	77	94,4	22,0
6	Van Muradiye Şube SB01	-	"	78	86,2	20,0
7	Van Muradiye Lojman SZ01	-	"	76	65,6	15,3
8	Van Muradiye Lojman GPZ01	-	"	76	57,2	13,3
9	İstanbul Güzelialı Şube Kira SZ01	-	"	77	91,2	21,2

- Numuneler yerinden başvuru sahibi firma tarafından alınmış olup, laboratuvarda teknisyen tarafından teslim alınmıştır. Deney sonuçları dışındaki bilgiler (inşaat adresi, karot numarası, numune alınan eleman vb bilgiler) firmanın başvuru dilekçesindeki beyanına göre yazılmıştır. Numunelerin söz konusu projeye ait olup olmadığı konusunda imza sahipleri sorumluluk kabul etmez.

Mübin USLU
 İnş. Tek.

Prof. Dr. Hasan YILDIRIM
 Lab. Sorumlusu

Figure A.3: Material Test

EXISTING BUILDING MATERIAL CONDITION (A)

Explanation: This category evaluates fundamental material and section properties—such as concrete quality and reinforcement detailing—used in the structural members. Critical parameters including concrete strength obtained from core sampling and/or Schmidt rebound hammer tests, stirrup spacing and diameter, reinforcement grade (e.g., S220/S420; in older notation BC-I/BC-III), and development/anchorage length are compiled under this heading.

The obtained values are compared with the limits defined in TS 500 and TBEC 2018, and scoring is performed to represent the strength level of the building. This category enables the reflection of material-related weaknesses of the load-bearing system into the RSRA score and contributes to a capacity-based evaluation.

a) Concrete Quality

Concrete quality was investigated using the Schmidt rebound hammer test and a concrete core sample.

Table A.1: Schmidt Rebound Hammer Readings

Schmidt Rebound Hammer Readings					
Storey			Storey		
Basement			Basement		
Rebound R	Cube 150	Cylinder	Rebound R	Cube 150	Cylinder
28	21,41	17,13	28	21,41	17,13
26	18,33	14,67	29	22,98	18,39
28	21,41	17,13	30	24,58	19,66
27	19,86	15,89	32	27,85	22,28
27	19,86	15,89	30	24,58	19,66
28	21,41	17,13	28	21,41	17,13
28	21,41	17,13	32	27,85	22,28
29	22,98	18,39	30	24,58	19,66
28	21,41	17,13	30	24,58	19,66
28	21,41	17,13	31	26,20	20,96

Table A.2: Matreial Score

Minimum	14,67	17,13
Average Schmidt Value	16,76	19,68
Standard Deviation	1,02	1,82
Average – Standard Deviation	15,74	17,86
$0.85 \times \text{Average}$	14,24	16,73
Core (R = 74 mm)	21,2	
Core (R = 100 mm)	20,51	
Adopted (MPa—cylinder)	20,51	
Material Score	1,42	

Based on the rebound hammer test, the corrected concrete compressive strength was obtained as 20.51 N/mm².

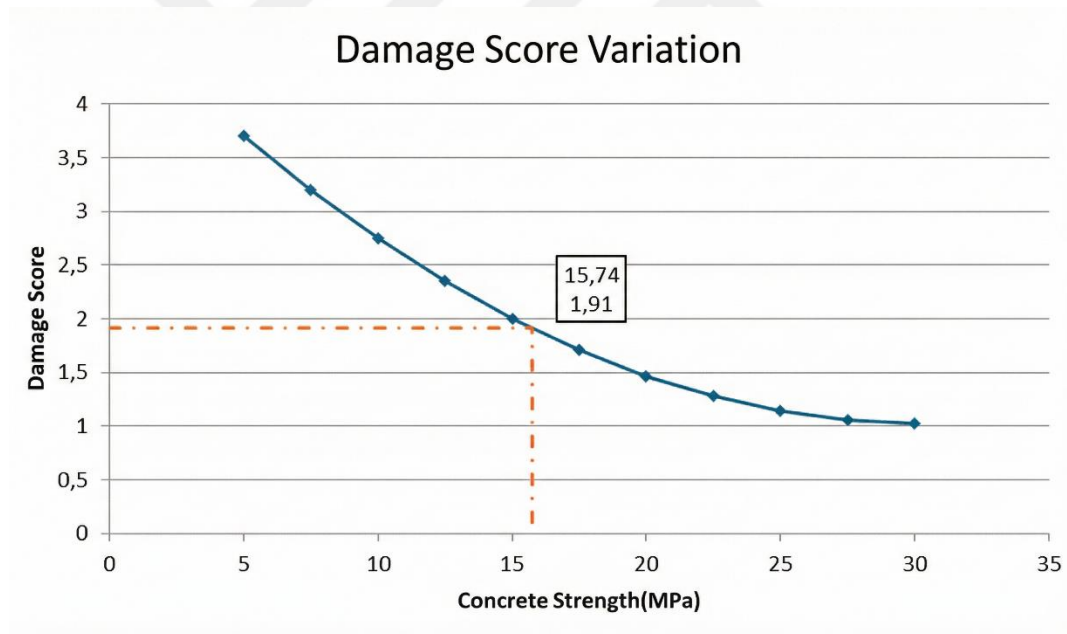


Figure A.4: Damage score variation

Formulation used to compute intermediate and outer values:

$$\text{SCORE} = 0.0042f_{cd}^2 - 0.2542f_{cd} + 4.87$$
$$a=1.42$$

b) S420 reinforcement steel was used for both longitudinal bars and stirrups.

Table A.3: Reinforcement steel score

Reinforcement Steel	Score
S220	2
S420	1✓
S220-S420	1.5

b=1.00 puan

c) Development/anchorage length

Table A.4: Development/anchorage length score

Rebar Development/Anchorage Length	Score
Observed	1
Not observed	1.5✓

c=1.50 puan

d) As a result of X-ray scanning and cover removal in the column, it was observed that Ø14 longitudinal bars are spaced at 11 cm and Ø8 stirrups are spaced at 25–27 cm; these values were used in the calculations.

Table A.5: Stirrup spacing score

Stirrup Spacing	Score
25 and above	2,5✓
20–25	2
15–20	1,5
10–15	1
below 10	0,5

d=2.50 puan

NOTE: The score obtained in this item will be multiplied by 0.75.

A (total) = 4.815 points.

RESISTING SYSTEM TYPE AND CODE/CONSTRUCTION PERIOD (B)

Explanation: This category evaluates the building by considering the type of the lateral-load resisting system and the earthquake code that was in effect during the construction/design period. The stiffness level of the structural system (e.g., moment-resisting frame, shear-wall system, mixed systems) and the applicable code level in the historical context are considered together.

Minimum performance requirements introduced by codes such as those of 1998, 2007, or 2018 are taken as a reference for system-level reliability. Accordingly, this category supports inference about the engineering environment in which the building was designed and the capacity of the structural system, and the scoring is carried out on that basis.

Table A.6: Structural system score

	CODE YEAR	STRUCTURAL SYSTEM TYPE			
		Reinforced Concrete	Steel	Timber	Masonry
B	Before 1975	4	1,2	1,2	3
	1975–1998	3✓	1,2	1,2	2
	1998–2007	2	1,2	1,2	1,5
	2007–2018	1	1	1	1

B = 3.00 points (the design may be non-ductile and may have been prepared earlier).

FLOOR DIAPHRAGM FORMATION (C)

Explanation: This category evaluates the presence and functionality of key structural components that govern the seismic response of the building. The floor system (e.g., slab, ribbed slab, hollow-block slab) is considered in terms of rigid diaphragm action enabling effective distribution of lateral loads.

In addition, the presence of stiff shear walls in basement storeys and the adequate use of reinforced concrete shear walls in upper storeys determine the building's resistance against uncontrolled drifts and torsional effects during earthquakes. This category contributes to scoring by assessing the integrity of the lateral load transfer path and whether load distribution is balanced.

Table A.7: Floor diaphragm score

			beam -slab	flat slab	one-way hollow-block slab	waffle slab
C	With basement	Shear -wall ratio = 0	1✓	2	1,8	1
	Without basement	Shear -wall ratio = 0	2	4	3,6	2
	With basement	Shear -wall ratio < 1/3	0,7	1,4	1,3	0,7
	Without basement	Shear -wall ratio < 1/3	1,4	2,8	2,5	1,4
	With basement	Shear -wall ratio > 1/3	0,5	1	0,9	0,5
	Without basement	Shear -wall ratio > 1/3	1	2	1,8	1
	Tunnel Form		0,2	0,4	0,4	0,2

C = 1.00 points.

SEISMICITY AND SOIL CLASS (D)

Explanation: This category includes a scoring procedure based on the spectral acceleration (S_a) value, which numerically represents the building's dynamic response under seismic action. The evaluation is computed as a function of the fundamental period (T) obtained from analyses performed on an equivalent static model developed for the building.

The dominant period of the building (T) is obtained from analysis, and the corresponding spectral acceleration is calculated using the design spectrum defined in TBEC 2018. Soil class is critical for obtaining the spectrum curve:

For some buildings, the soil class is determined using the MASW method;

for some, using USGS data;

and for the remaining buildings, it is assigned based on field observations (default ZC).

Periods are computed separately for the X and Y directions, and the shorter period (i.e., the larger Sa value) is taken as the basis for scoring.

For example:

Building periods: X direction = 0.203 s, Y direction = 0.164 s, Soil class: ZD

Spectral acceleration (Sa) value: 1.146 g = S(T), $0.5 < S(T) < 1.5$

Since this value falls within the range $0.5 < Sa < 1.5$, the score is computed as: Score = $1 + 5.0 \times (Sa - 0.5) = 4.23$

Scoring is performed according to the following thresholds:

$$\text{SCORE: } \begin{cases} S(T) \leq 0.5 & \rightarrow 1 \\ 0.5 < S(T) < 1.5 & \rightarrow 1 + 5.0 \times [S(T) - 0.5] \\ S(T) \geq 1.5 & \rightarrow 6 \end{cases}$$

This approach is the only parameter that directly links the magnitude of seismic demand with the building's risk score, and it constitutes an important quantitative assessment tool within the RSRA framework.

The building period is determined from the performed performance calculation or by an empirical expression.

4.7.3.4 – Ampirik hakim doğal titreşim periyodu Denk.(4.27) ile hesaplanacaktır:

$$T_{pA} = C_t H_N^{3/4} \quad (4.27)$$

(a) Taşıyıcı sistemi sadece betonarme çerçevelerden oluşan binalarda $C_t = 0.1$, çelik çerçevelerden veya çaprazlı çelik çerçevelerden oluşan binalarda $C_t = 0.08$, diğer tüm binalarda $C_t = 0.07$ alınacaktır.

Figure A.5: Empirical expression (TBEC 2018)

Using seismic measurements, the average shear-wave velocity in the upper 30 m (VS30) is obtained.

Using the Türkiye Earthquake Hazard Map, the spectrum is obtained based on the seismic hazard level (DD-2 used in this study) and the soil class.

2.3.4.1 – Gözönüne alınan herhangi bir deprem yer hareketi düzeyi için *yatay elastik tasarım ivme spektrumu*'nun ordinatları olan *yatay elastik tasarım spektral ivmeleri* $S_{ac}(T)$, doğal titreşim periyoduna bağlı olarak yerçekimi ivmesi [g] cinsinden **Denk.(2.2)**'de tanımlanmıştır (**Şekil 2.1**):

$$\begin{aligned} S_{ac}(T) &= \left(0.4 + 0.6 \frac{T}{T_A}\right) S_{DS} & (0 \leq T \leq T_A) \\ S_{ac}(T) &= S_{DS} & (T_A \leq T \leq T_B) \\ S_{ac}(T) &= \frac{S_{D1}}{T} & (T_B \leq T \leq T_L) \\ S_{ac}(T) &= \frac{S_{D1} T_L}{T^2} & (T_L \leq T) \end{aligned} \quad (2.2)$$

Figure A.6: Spectral values (TBEC 2018)

The spectral acceleration corresponding to the building's fundamental period, $S_{ac}(T_1)$, is obtained.

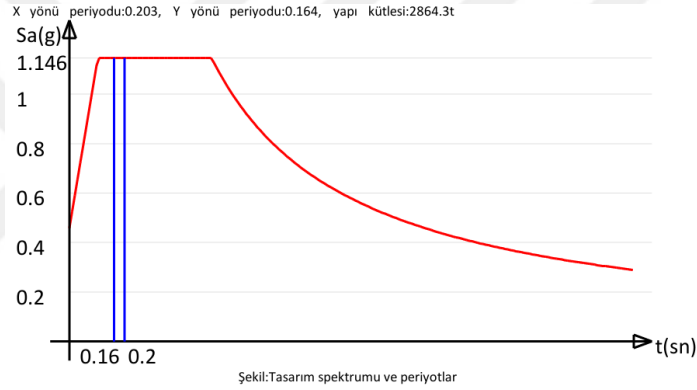


Figure A.7: Spectral acceleration

Points are calculated using the spectral acceleration $S_{ac}(T_1)$ together with the points corresponding to the short-period design spectral acceleration coefficient SDS.

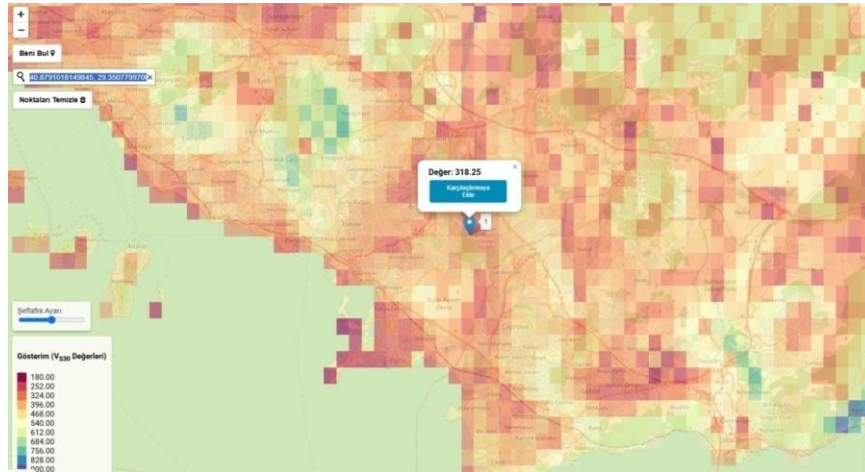


Figure A.8: Vs30 value for Ziraat Bank Istanbul Guzelyali Branch (for lease)

Table A.8: Spectral score

Saha Bilgileri			Yapı Davranışı	
Zemin	ZD	▼	Kat Sayısı	3
Sds	1,146		Bodrum	1
Sd1	0,599		Tx	0,203
Ta	0,104		Ty	0,164
Tb	0,522		S(0,16)	1,146
Tl	6		Puan(1,146)	4,23
Tpa	0,449		Puan(1,146)	4,23
Ct-Çerçeve: 0.1			▼ Puan	4,23

$$\text{SCORE: } \begin{cases} S(T) \leq 0.5 & \rightarrow 1 \\ 0.5 < S(T) < 1.5 & \rightarrow 1 + 5.0 \times [S(T) - 0.5] \\ S(T) \geq 1.5 & \rightarrow 6 \end{cases}$$

In scoring, the S(T1) score is considered such that the SDS-based score may be reduced by up to 30%.

D = 4.23 points.

Seismic information for the building:

Table A.9: Mode shapes and participation ratios

Mode shapes and participation ratios								
No	ω	T	rn-UX	rn-UY	rn-UZ	rn-RX	rn-RY	rn-RZ
1	31,002	0,203	0,558	0,021	0	0	0	0,035
2	38,26	0,164	0	0,377	0	0	0	0,241
3	48,986	0,128	0,063	0,238	0	0	0	0,338
4	138,853	0,045	0,135	0,003	0	0	0	0,003
5	162,935	0,039	0	0,099	0	0	0	0,035
6	173,936	0,036	0,003	0,038	0	0	0	0,062
7	277,037	0,023	0,229	0	0	0	0	0
8	304,79	0,021	0	0,211	0	0	0	0
9	421,559	0,015	0	0	0	0	0	0,285
Σ			0,988	0,988	0	0	0	1

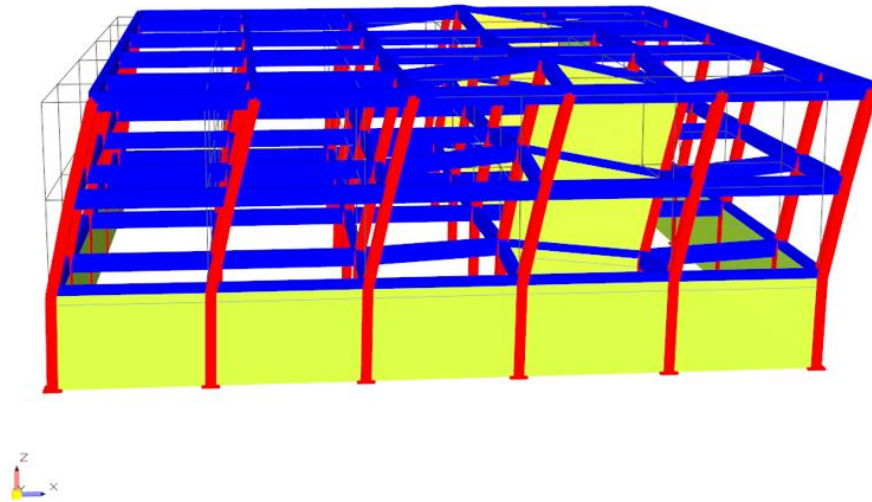


Figure A.9: Mode shapes-1

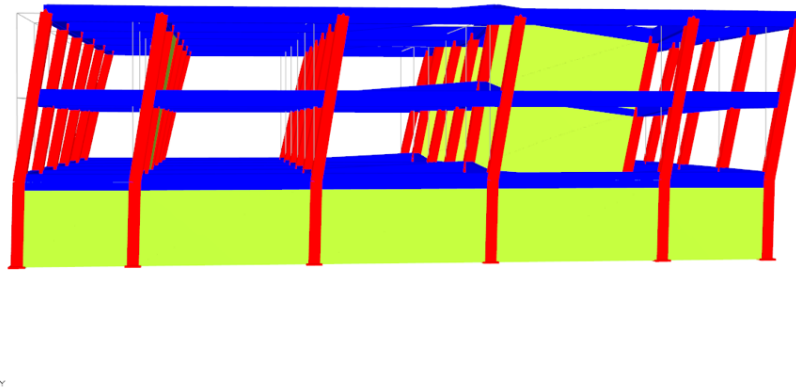


Figure A.10: Mode shapes-2

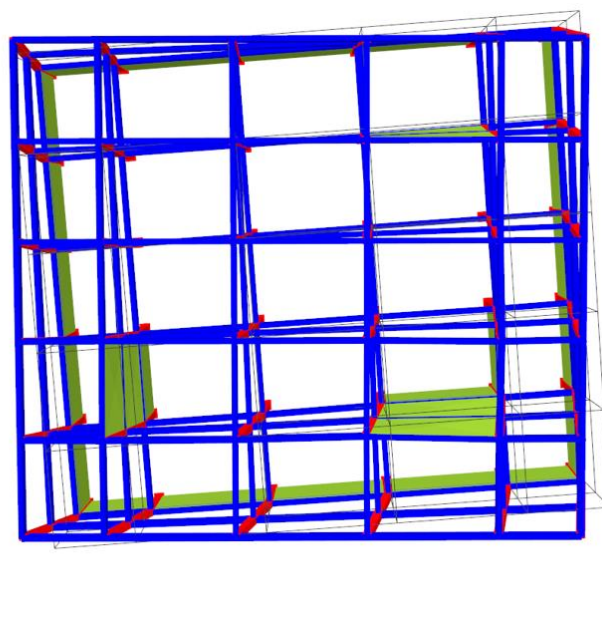


Figure A.11: Mode shapes-3

Table A.10: Base shear force

Base shear force (building mass: 2864.3 t)										
Dir	T	Sa	R	D	Tb	RaT	SaR/ g	mSR(Tp)[kN]	0.04mIS g	VtE(4.19) *
X	0,20 3	1,14 6	5, 6	2, 5	0,52 2	3,70 3	0,31	8695,9	1287,9	8695,9
Y	0,16 4	1,14 6	5, 6	2, 5	0,52 2	3,47 5	0,33	9266,9	1287,9	9266,9

STRUCTURAL IRREGULARITIES (E)

Explanation: This category evaluates structural irregularities observed in the load-bearing system both in plan and in elevation. The assessment is performed through review of structural/architectural drawings as well as field observations.

In-plan irregularities include:

Torsional irregularity (A1)

Diaphragm discontinuities

Heavy overhangs

Pounding risk

Stub beams

Slabs at different elevations

Settlement on steeply sloped terrain

Workmanship defects

Among vertical irregularities:

Soft-storey effect (B2)

Vertical discontinuity of load-carrying elements

Short-column formation

Strong-beam/weak-column adverse ratio

Each irregularity type is identified based on observations.

If present: the relevant score (typically 1.3) is reflected in the total.

If absent: it is represented within the system with 0.1 points.

If a soft storey exists and storey height > 5 m: score = 2.5 is used.

All irregularity scores are summed to obtain the total Category E score.

Subsequently, by reviewing the analysis outputs of the equivalent static model (OpenSees):

A1: Torsional irregularity

B2: Soft-storey check

Relative storey drift (δ/h)

irregularity checks are performed (e.g., torsion A1, soft storey B2, relative drift δ/h).

If there is consistency between field observations and analysis results, the scores are confirmed.

Because this category combines engineering observation with analysis-supported irregularity checks, it serves as a comprehensive structural risk assessment tool in RSRA scoring.

Table A.11: Irregularity check

E	In elevation			Planda			Pounding	Short column	Stub beam	Strong column
	Soft storey	Soft storey (H > 5 m)	Vertical discontinuity (B3)	(A1)	(A2)	Overhang (A3)				
Yes	2	2,5	1,2	1,3	1,3	1,3	1,3	1,3✓	1,3	1,3
No	0,1✓		0,1✓	0,1✓	0,1✓	0,1✓	0,1✓	0,1	0,1✓	0,1✓

NOTE: Issues such as "reinforcement ratio < 1%", "diaphragm at different elevations", "excessively sloped terrain", "workmanship defects", "heavy overhangs", and "uncertain structural behavior due to additions over time" may also be recorded under this category and used in place of the 'empty column' entry where applicable.

$$\delta_i^{(x)} = \frac{R}{T} \Delta_i^{(x)} \quad \kappa:1 >>> \text{BETONARME YAPI} \quad \lambda \frac{\delta_{i,\max}^{(x)}}{h_i} \leq 0.008 \kappa = 0.008 \quad (\text{Gevrek ve bitişik dolgu duvar}) \quad (4.33-4.34)$$

Tablo: EqX kat ötelemesi

Kat	DD2 Alt yerd.[mm]	DD2 Üst yerd.	Δ	h	δ	$\lambda \delta \cdot h$	Lim	E/K	✓/X
Bod_1	0	0.23969	0.23969	3700	1.3423	0.0019656	0.008	0.024571	✓
Zmn	0.23969	2.5238	2.2841	3700	12.791	0.0018731	0.008	0.23414	✓
Kat_1	2.5238	5.1186	2.5948	3700	14.531	0.0021279	0.008	0.26599	✓

B2:Komşu Katlar Arası Rijitlik Düzensizliği (Yumuşak Kat): En büyük fark "Zmn" ve "Kat_1" arasında olup $\eta_k:0.0021/0.0019=1.1452$ (1 kat bodrum) (Kat eksilme yok)

$$\delta_i^{(x)} = \frac{R}{T} \Delta_i^{(x)} \quad \kappa:1 >>> \text{BETONARME YAPI} \quad \lambda \frac{\delta_{i,\max}^{(x)}}{h_i} \leq 0.008 \kappa = 0.008 \quad (\text{Gevrek ve bitişik dolgu duvar}) \quad (4.33-4.34)$$

Tablo: EqY kat ötelemesi

Kat	DD2 Alt yerd.[mm]	DD2 Üst yerd.	Δ	h	δ	$\lambda \delta \cdot h$	Lim	E/K	✓/X
Bod_1	0	0.19683	0.19683	3700	1.1023	0.0016141	0.008	0.020177	✓
Zmn	0.19683	1.6905	1.4936	3700	8.3644	0.0012249	0.008	0.15311	✓
Kat_1	1.6905	3.2932	1.6027	3700	8.9751	0.0013143	0.008	0.16429	✓

B2:Komşu Katlar Arası Rijitlik Düzensizliği (Yumuşak Kat): En büyük fark "Zmn" ve "Kat_1" arasında olup $\eta_k:0.0013/0.0012=1.0752$ (1 kat bodrum) (Kat eksilme yok)

Tablo 3.6 – Düzensiz Binalar: A1 – Burulma Düzensizliği

Tablo: EqX burulma düzensizliği

Kat	δ_1 [mm]	δ_2	δ_3	δ_4	$\delta_{\max}$	$\delta_{\min}$	δ_{ort}	η_{biX}	A1	4.7.4	Dbi	e%	DyfGen[m]	e
Bod_1	0.24805	0.24805	0.22619	0.22619	0.24805	0.22619	0.23712	1.05	Yok	✓	1	5		
Zmn	2.8115	2.8115	2.0591	2.0591	2.8115	2.0591	2.4353	1.15	Yok	✓	1	5		
Kat_1	5.7954	5.7954	4.0254	4.0254	5.7954	4.0254	4.9104	1.18	Yok	✓	1	5		

Tablo: EqY burulma düzensizliği

Kat	δ_1 [mm]	δ_2	δ_3	δ_4	$\delta_{\max}$	$\delta_{\min}$	δ_{ort}	η_{biY}	A1	4.7.4	Dbi	e%	DyfGen[m]	e
Bod_1	0.203	0.18758	0.18758	0.203	0.203	0.18758	0.19529	1.04	Yok	✓	1	5		
Zmn	1.8953	1.3832	1.3832	1.8953	1.8953	1.3832	1.6393	1.16	Yok	✓	1	5		
Kat_1	3.7697	2.5783	2.5783	3.7697	3.7697	2.5783	3.174	1.19	Yok	✓	1	5		

Figure A.12: Irregularities check from report

Typical irregularities that may occur in branches due to mezzanine floors:

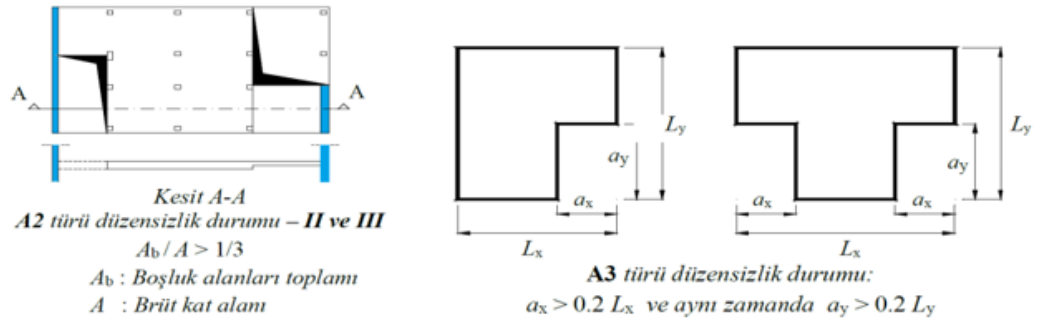


Figure A.13: Typical irregularities (TBEC 2018)



Figure A.14: Photos from structure (irregularities)

E = 2.10 points.

BUILDING CONDITION AT THE TIME OF ASSESSMENT (F)

Explanation: Within Category F, the engineering risk level of buildings is evaluated using both field observations and quantitative analyses performed on an equivalent static system model. This evaluation covers multiple sub-components such as corrosion level, groundwater conditions at foundation level, directional stiffness, column axial stress under axial loads, the spectral multiplier determined through linear analysis, and column E/K ratios.

1. Scoring Based on Observational Findings

First, the observed corrosion level is included in the scoring. For buildings with widespread corrosion, a score of 1.5 is assigned; if corrosion is mild, 1.0; and if corrosion is absent, 0.3. In addition, encountering significant groundwater at the foundation level contributes 1.5 points in view of potential liquefaction risk, whereas for buildings with no groundwater, this contribution is limited to 0.3 points.

Whether the structural stiffness is directional is also assessed. If reinforced concrete shear walls or column inertias are arranged predominantly in one direction and stiffness is provided mainly along a single axis, the building is considered "unidirectionally stiff" and 2.0 points are assigned; otherwise, 0.3 points are assigned. For the sample building in Appendix 1, corrosion is widespread, the foundation is dry, and stiffness is bidirectional. Accordingly, $1.5 + 0.3 + 0.3 = 2.1$ points were calculated from observational findings.

Column Compressive Stresses and Effect-to-Capacity Ratio (E/K): Under this subheading, the value obtained from compressive tests on concrete samples is taken as the design compressive strength in accordance with TS 500, Equation 7.7. In the Appendix 1 example, this value is calculated as $f_{cd} = 20.51 \times 0.9 = 18.45$ MPa. Then, using axial loads determined by the effective tributary-area method, the compressive stresses in two on-site columns are computed as 2.86 MPa and 2.23 MPa. In addition, considering the maximum axial load obtained from the OpenSees-based model, the maximum stress is computed as 11.586 MPa. By normalizing this value with f_{cd} , $E/K = 11.586 / 18.45 = 0.628$ is obtained. This ratio constitutes the final score for this item.

3. Spectral Multiplier for Controlled Damage

In the linear analysis performed in accordance with TBEC 2018, Chapter 15, the building performance under the defined spectral acceleration $S_{ae}(T)$ is evaluated. If

the building directly satisfies the controlled damage performance level, spectrum reduction is not required and the score cell is set to "0". Since the sample building in Appendix 1 meets this target, the score assigned for this item is 0.

4. Reinforced Concrete Column E/K Ratio

Column internal forces (M2, M3, V2, V3) under eight load combinations were computed, and the most unfavorable combinations were evaluated on the moment–axial load interaction diagram. Based on these diagrams, the maximum flexure-governed E/K ratio was calculated as 0.88 and was directly reflected in the scoring.

5. E/K Values Associated with Irregularities

Behavioral irregularities such as relative storey drift, torsional irregularity, and soft-storey response were examined through the equivalent model. For the Appendix 1 case, the maximum soft-storey ratio in the X direction was found to be 1.14 and, by normalizing with the limit value of 2, it was evaluated as $E/K = 0.57$.

6. Stress Score — Final Calculation

E/K values obtained under the four numerical subheadings above:

0.628 (column compressive stress)

0.00 (spectral multiplier)

0.88 (column moment/shear deficiency)

0.57 (irregularity)

0.628 (column compressive stress)

0.00 (spectral multiplier)

0.88 (column moment/shear deficiency)

0.57 (irregularity)

Among these values, the largest one (0.88) was taken as the stress coefficient B_0 , and the stress score was determined as 0.9 using the corresponding coefficient.

$$\text{STRESS SCORE: } \begin{cases} \text{ratio} < 0.8 & \rightarrow & 0 \\ \text{ratio} \geq 0.8 & \rightarrow & 0.5 + (B_0 - 0.8) \times 5 \end{cases}$$

Finally:

Observational score: 2.1

Stress score: 0.9

Total Category F score: $2.1 + 0.9 = 3.0$

This evaluation exemplifies a multi-layer approach in the latest version of RSRA, where engineering-based analysis outputs are integrated with field observations.

Each of the following items is checked and a score is assigned for the observed damage/deficiencies in the building.

Member corrosion condition (none: 0.3; <30%: 1.0; >30%: 1.5✓)



Figure A.15: Spalling and severe reinforcement corrosion

Photo: Spalling and severe reinforcement corrosion were observed in a load-bearing member.

Groundwater emergence from beneath the foundation; liquefiable soil type? (absent: 0.3✓ present: 1.5)

Insufficient stiffness in the building's weak direction (absent: 0.3✓ present: 2.0)

A comparison is made using the largest identifiable span and the column with the smallest cross-sectional dimensions: the ratio of the vertical load (including vertical earthquake effects) to the upper limit defined in TS 500, Section 7.4.1.

If an equivalent static system solution is available for the building, additional indicators are also considered, including the ratio of vertical load to column area and reinforced concrete column E/K ratios.

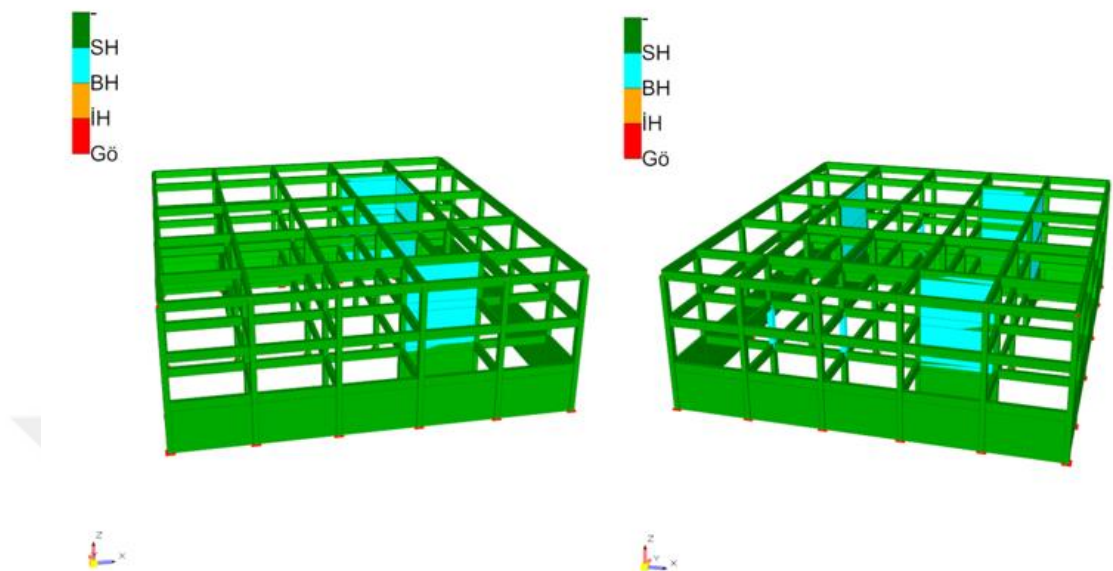


Figure A.16: Performance in the X and Y directions

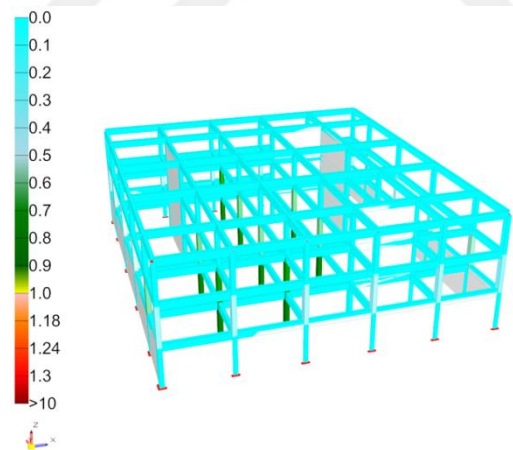


Figure A.17: Building E/K ratios

Max N: 2317.2kN, section A: 2000cm², σ : 11.586N/mm²

Eğilme altında en olumsuz kombinasyon 1.4G+1.6Q, N: -2291.3kN, M2: -6.9295kN.m, M3: 2.1719kN.m, E/K oranı: 0.88 >> ✓

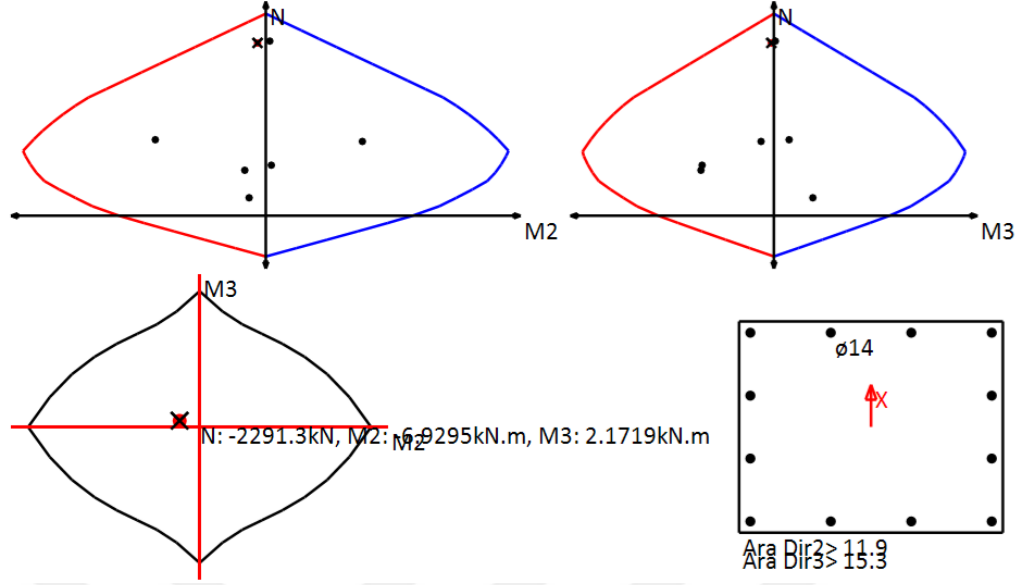
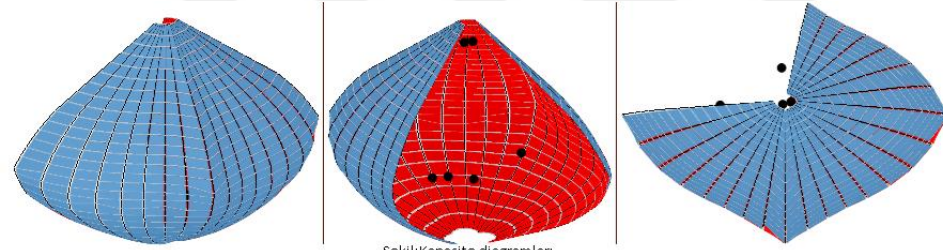


Figure A.18: E/K ratio



En olumsuz kombinasyon V2 is 1.23G+Q+EqX+0.3EqY, N: -603.98kN, V: 32.567kN, E/K oranı: 0.15 >> ✓

Kesme kapasitesi hesabı

f_{cd} : 1.2419N/mm², b: 50cm, h: 40cm, d: 37cm, Nd: 0 için Eğik Çatlama Dayanımı V_{cr} : 149.34kN, kesme k. limiti $V_{d,lim}$: 512.41kN
Etr cap: 8, aralık: 17cm, derinlik: 34cm, kol: 2, f_{yw} : 336N/mm² (Etriyeler h'ye paralel şekilde hesaplandı)

$$\text{Eğik çatlama dayanımı } V_{cr} = 0.65 f_{cd} b_i d \left(1 + \frac{V}{A_c} \frac{N_d}{A_c} \right) : 204.14 \text{ kN} \quad (\text{TS500 8.1})$$

$$\text{Etriye dayanımı } V_i = \frac{A_{sw}}{s} f_{ywd} d = 87.426 \text{ kN} \quad (\text{TS500 8.5})$$

$$\text{Depremsiz durumda toplam dayanım } V_r = 0.8 V_{cr} + V_i = 250.74 \text{ kN} \quad V_{MAX} \leq 0.22 f_{cd} b_i d = 520.72 \text{ kN} \quad (\text{TS500 8.3; 8.7})$$

Tablo: Tasarım kombinasyonu

Name	G	Q	EqX	EqY
G+Q+E1	1.23	1	3.7	1.04
G+Q+E3	1.23	1	3.7	-1.04
G+Q+E5	1.23	1	-3.7	1.04
G+Q+E7	1.23	1	-3.7	-1.04
G+Q+E2	1.23	1	1.11	3.47
G+Q+E4	1.23	1	-1.11	3.47
G+Q+E6	1.23	1	1.11	-3.47
G+Q+E8	1.23	1	-1.11	-3.47

TBDY2018 15.5.1.1 – Binaların deprem hesabında ek dışmerkezlilik göz önüne alınmayacaktır. Toplam eşdeğer deprem yükünün (taban kesme kuvveti) Denk.(4.19) ve Denk.(4.8)'e göre hesabında $R_a = 1$ alınacaktır.

Figure A.19 Capacity check from report

Table A.12: Performance assessment (both direction)

Deprem X

Tablo: Bod_1 katı +X yönü için kesme kuvveti tablosu(Toplam:59177kN)

Mem	V[kN]	%	Perf	Mem	V[kN]	%	Perf	Mem	V[kN]	%	Perf
Col1_0.0_Bod_1	10.255	0.02	SH	Col2_0.1_Bod_1	8.4492	0.01	SH	Col3_0.2_Bod_1	7.5062	0.01	SH
Col4_0.3_Bod_1	6.7471	0.01	SH	Col5_0.4_Bod_1	7.559	0.01	SH	Col6_0.5_Bod_1	7.9464	0.01	SH
Col7_1.0_Bod_1	5.0421	0.01	SH	Col8_1.1_Bod_1	5.0011	0.01	SH	Col9_1.2_Bod_1	4.4946	0.01	SH
Col10_1.3_Bod_1	8.3152	0.01	SH	Col11_1.4_Bod_1	7.6263	0.01	SH	Col12_1.5_Bod_1	2.7809	0	SH
Col13_2.0_Bod_1	4.0124	0.01	SH	Col14_2.1_Bod_1	1.3551	0	SH	Col15_2.2_Bod_1	3.1638	0.01	SH
Col16_2.3_Bod_1	4.0686	0.01	SH	Col17_2.4_Bod_1	3.2037	0.01	SH	Col18_2.5_Bod_1	1.5872	0	SH
Col19_3.0_Bod_1	4.4973	0.01	SH	Col20_3.1_Bod_1	8.4145	0.01	SH	Col21_3.2_Bod_1	3.2482	0.01	SH
Col22_3.3_Bod_1	3.3878	0.01	SH	Col23_3.4_Bod_1	5.602	0.01	SH	Col24_3.5_Bod_1	2.0933	0	SH
Col25_4.0_Bod_1	5.1165	0.01	SH	Col26_4.1_Bod_1	8.0915	0.01	SH	Col27_4.2_Bod_1	4.6442	0.01	SH
Col28_4.3_Bod_1	8.4107	0.01	SH	Col29_4.4_Bod_1	9.9037	0.02	SH	Col30_4.5_Bod_1	2.7507	0	SH
Col31_5.0_Bod_1	10.669	0.02	SH	Col32_5.1_Bod_1	6.9311	0.01	SH	Col33_5.2_Bod_1	7.7416	0.01	SH
Col34_5.3_Bod_1	7.347	0.01	SH	Col35_5.4_Bod_1	7.2034	0.01	SH	Col36_5.5_Bod_1	8.4539	0.01	SH
Wll1_Bod_1	7564.6	12.78	SH	Wll2_Bod_1	5430.5	9.18	BH	Wll3_Bod_1	20.016	0.03	SH
Wll4_Bod_1	22.485	0.04	SH	Wll5_Bod_1	4533.1	7.66	SH	Wll6_Bod_1	4316.8	7.29	SH
Wll7_Bod_1	4859.8	8.21	SH	Wll8_Bod_1	4524.6	7.65	SH	Wll9_Bod_1	4822.4	8.15	SH
Wll10_Bod_1	4499	7.6	SH	Wll11_Bod_1	4815.5	8.14	SH	Wll12_Bod_1	4528	7.65	SH
Wll13_Bod_1	4614.1	7.8	SH	Wll14_Bod_1	4103.6	6.93	SH	Wll15_Bod_1	29.355	0.05	SH
Wll16_Bod_1	24.885	0.04	SH	Wll17_Bod_1	39.95	0.07	SH	Wll18_Bod_1	32.465	0.05	SH
Wll19_Bod_1	35.182	0.06	SH	Wll20_Bod_1	37.876	0.06	SH	Wll21_Bod_1	33.357	0.06	SH
Wll22_Bod_1	30.736	0.05	SH	Wll23_Bod_1	24.335	0.04	SH	Wll24_Bod_1	20.947	0.04	SH

Tablo: +X yönü sonuçları

K	Sınırlı				Belirgin				İleri				Göçme				A&Ü	TOP	Kat
	Kiriş	Kolon	Vkol[kN]	%Vkol	Kiriş	Kolon	Vkol[kN]	%Vkol	Kiriş	Kolon	Vkol[kN]	%Vkol	Kiriş	Kolon	Vkol[kN]	%Vkol			
1	60/60	59/60	53747	90.8	-	1/60	5430.5	9.2	-	-	0	0	-	-	0	0	0	59177	KH
2	60/60	37/40	2077.7	7.6	-	3/40	25247	92.4	-	-	0	0	-	-	0	0	0	27325	KH
3	60/60	37/40	2486.9	14.8	-	3/40	14288	85.2	-	-	0	0	-	-	0	0	0	16775	KH

Adım kesme kuvveti: 59177kN

Hasar tesbit katı:1

Sınırlı hasar bölgesini geçen kolonlar bulunmaktadır, Sınırlı Hasar **X**

Deprem X yönü yapı performansı: **Kontrollü Hasar**

Deprem Y

Tablo: Bod_1 katı +Y yönü için kesme kuvveti tablosu(Toplam:49182kN)

Mem	V[kN]	%	Perf	Mem	V[kN]	%	Perf	Mem	V[kN]	%	Perf
Col1_0.0_Bod_1	8.1987	0.02	SH	Col2_0.1_Bod_1	15.419	0.03	SH	Col3_0.2_Bod_1	8.3697	0.02	SH
Col4_0.3_Bod_1	8.0677	0.02	SH	Col5_0.4_Bod_1	15.803	0.03	SH	Col6_0.5_Bod_1	9.0428	0.02	SH
Col7_1.0_Bod_1	7.3987	0.02	SH	Col8_1.1_Bod_1	23.449	0.05	SH	Col9_1.2_Bod_1	6.1247	0.01	SH
Col10_1.3_Bod_1	9.1347	0.02	SH	Col11_1.4_Bod_1	21.197	0.04	SH	Col12_1.5_Bod_1	5.4812	0.01	SH
Col13_2.0_Bod_1	7.2237	0.01	SH	Col14_2.1_Bod_1	22.869	0.05	SH	Col15_2.2_Bod_1	7.2006	0.01	SH
Col16_2.3_Bod_1	6.9261	0.01	SH	Col17_2.4_Bod_1	23.136	0.05	SH	Col18_2.5_Bod_1	7.4751	0.02	SH
Col19_3.0_Bod_1	7.0046	0.01	SH	Col20_3.1_Bod_1	19.182	0.04	SH	Col21_3.2_Bod_1	6.6145	0.01	SH
Col22_3.3_Bod_1	4.9786	0.01	SH	Col23_3.4_Bod_1	21.027	0.04	SH	Col24_3.5_Bod_1	7.9498	0.02	SH
Col25_4.0_Bod_1	7.5499	0.02	SH	Col26_4.1_Bod_1	20.12	0.04	SH	Col27_4.2_Bod_1	2.8714	0.01	SH
Col28_4.3_Bod_1	6.6963	0.01	SH	Col29_4.4_Bod_1	18.14	0.04	SH	Col30_4.5_Bod_1	6.024	0.01	SH
Col31_5.0_Bod_1	5.8325	0.01	SH	Col32_5.1_Bod_1	12.601	0.03	SH	Col33_5.2_Bod_1	5.2037	0.01	SH
Col34_5.3_Bod_1	5.1011	0.01	SH	Col35_5.4_Bod_1	12.908	0.03	SH	Col36_5.5_Bod_1	6.6216	0.01	SH
Wll1_Bod_1	23.359	0.05	SH	Wll2_Bod_1	23.605	0.05	SH	Wll3_Bod_1	4568.8	9.29	SH
Wll4_Bod_1	3367.5	6.85	SH	Wll5_Bod_1	18.8	0.04	SH	Wll6_Bod_1	14.453	0.03	SH
Wll7_Bod_1	17.511	0.04	SH	Wll8_Bod_1	12.448	0.03	SH	Wll9_Bod_1	16.616	0.03	SH
Wll10_Bod_1	18.593	0.04	SH	Wll11_Bod_1	14.712	0.03	SH	Wll12_Bod_1	13.089	0.03	SH
Wll13_Bod_1	16.131	0.03	SH	Wll14_Bod_1	11.488	0.02	SH	Wll15_Bod_1	2939.9	5.98	SH
Wll16_Bod_1	2880.7	5.86	SH	Wll17_Bod_1	5003.3	10.17	SH	Wll18_Bod_1	4758.9	9.68	SH
Wll19_Bod_1	4964.1	10.09	SH	Wll20_Bod_1	4729.6	9.62	SH	Wll21_Bod_1	4986.7	10.14	SH
Wll22_Bod_1	4771.7	9.7	SH	Wll23_Bod_1	2977.2	6.05	SH	Wll24_Bod_1	2643.7	5.38	SH

Tablo: +Y yönü sonuçları

K	Sınırlı				Belirgin				İleri				Göçme				A&Ü	TOP	Kat
	Kiriş	Kolon	Vkol[kN]	%Vkol	Kiriş	Kolon	Vkol[kN]	%Vkol	Kiriş	Kolon	Vkol[kN]	%Vkol	Kiriş	Kolon	Vkol[kN]	%Vkol			
1	60/60	60/60	49182	100	-	-	0	0	-	-	0	0	-	-	0	0	0	49182	SH
2	60/60	33/40	1633	5.9	-	7/40	26061	94.1	-	-	0	0	-	-	0	0	0	27694	KH
3	60/60	36/40	2083.7	12.1	-	4/40	15085	87.9	-	-	0	0	-	-	0	0	0	17169	KH

Adım kesme kuvveti: 49182kN

Hasar tesbit katı:2

Sınırlı hasar bölgesini geçen kolonlar bulunmaktadır, Sınırlı Hasar **X**

Deprem Y yönü yapı performansı: **Kontrollü Hasar**

Table A.13: Maximum design actions from report

Number of Storeys	3	Slab (cm)	12	Average Storey Height (m)	3,7	Stress limit	18,45
Q(kN/m ²)	3	Load (kN/m ²)	9	G-multiplier	1,4	Q-multiplier	1,6

Kolon/A (cm ²)	b& h (cm)	Mesafe-X	Mesafe-Y	Etkili A(m ²)	N(kN)	Stress & limit (MPa)	Oran
1	70	639	563	35,13	1399,91	2,86	0,155
4900	70	610	562				
2	30	0	314	11,09	468,27	2,23	0,12
2100	70	582	448				
(maximum column compression / column area)					Ek F/A	11,586	0,628
(spectral multiplier for controlled damage)					Ek F/A	1	0,000
(reinforced concrete column E/K ratio)						1. E/K	0,88
(irregularity-related E/K ratio)						2. E/K	0,57
Stress Score	0,9					Maks:	0,88

Maximum design actions for flexure and E/K ratios								
Member	Comb	N[kN]	V2	V3	T[kN.m]	M2	M3	E/K
Col9_1.2_Bod_1	1.4G+1.6Q	-2317,2	0,89872	2,7696	0,000879	3,3179	-1,1534	0.88 ✓
Col99_0.2_Zmn	1.23G+Q-EqX-0.3EqY	-671,26	-32,782	-2,8802	0,23866	4,6	60,279	0.49 ✓
Col104_1.1_Zmn	1.23G+Q-EqY-0.3EqX	-986,25	-6,3615	-46,947	0,18033	80,97	10,831	0.61 ✓
Col132_5.5_Zmn	1.23G+Q+EqX-0.3EqY	-239,26	18,561	7,6406	0,43748	-14,036	-32,62	0.27 ✓
Col104_1.1_Zmn	1.23G+Q-EqY-0.3EqX	-1009	-6,3615	-46,947	0,18033	-92,735	-12,707	0.66 ✓
Col128_5.1_Zmn	1.23G+Q+EqX+0.3EqY	-603,98	32,567	-9,8137	0,28119	-17,696	61,248	0.52 ✓
Col9_1.2_Bod_1	1.4G+1.6Q	-2291,3	0,89872	2,7696	0,000879	-6,9295	2,1719	0.88 ✓

F = 3.00 points.

RESULT: STRUCTURAL INDEX CALCULATION

In the structural index calculation, the building score is obtained by multiplying each penalty-score category.

$$SI=A \times B \times C \times D \times E \times F$$

SI: Structural Index

Structural Index Score of the Building: 385

Priority status	Score range	Sub-categories	
Very low risk	0-200	Existing building material condition (a)	4.815
Low risk	201-350	Structural system type and period (b)	3.000
Lower priority	351-500	Floor diaphragm formation (c)	1.000
Non-urgent	501-700	Seismicity and soil class (d)	4.230
Search for alternatives	701-900	Structural irregularities (e)	2.100
Priority intervention	>900	Building condition at the time of assessment (f)	3.000

Figure A.20: Structural index score final

Genel Bilgiler				Özet			
Metot	Yapı performansı			T1 T2	0.203 c		
Sds Sd1 (DD2)	1.146 0.5			Kütle	28t		
Spektrum Çarpanı				Deprem Kuvveti	8695.9kN 9266		
Ta Tb (DD2)	0.104 0.5			Tepe yerdeğiştirme	0.51186cm 0.3293		
Sds Sd1 (DD3)	0.621 0.2			Görel kat ötelemesi sınırı EK X	0.0021 0.008 0.0		
Rx Ry	5.6 5			Görel kat ötelemesi sınırı EK Y	0.0013 0.008 0.0		
Dx Dy	2.5 2			Yumuşak kat düzensizliği Sınırı EK X	1.136 2 0.5		
Katlar				Yumuşak kat düzensizliği Sınırı EK Y	1.073 2 0.5		
Bodrum				Max Basınç/Alan	11.586N/m²		
Beton f _c E	12,59MPa 25532N/mm² (Ts50)			Kolon eğilme E/K	0.87		
Donatı	Stl-III:336M			Kolon kesme E/K V2	0.15		
Tipik Kolon BxH	50x40cm			Kolon kesme E/K V3	0.18		
Tipik Kiriş BxH	30x70cm			Yapı Hasar Durumu Spekt Çarp	Kontrollü Hasar		
Döşeme Kalınlığı	12cm (Plak)						
Tipik X Aralığı	6.8						
Tipik Y Aralığı	6.8						
Döşeme Birim Ağırlığı	5.5kN/m²						
Hareketli Yük	3.5kN/m²						
Hareketli yük katılım oranı	0.5						
Malzeme Çarpanı	1.0						
Talep Çarpanı	1.0						
Çatlamış kesit	Ev						
Çözücü	Dahili çözücü/Eige						

Figure A.21: General informations about building

APPENDIX-B: VISUAL DOCUMENTATION OF BUILDINGS SUBJECTED TO PERFORMANCE ANALYSIS

Visual documentation is presented for 28 buildings on which nonlinear pushover analysis was conducted within the scope of this thesis. For each building, a field photograph captured from the exterior façade, a three-dimensional model view generated in the ETABS environment, and the corresponding floor plan are provided. These visuals collectively represent both the existing physical condition of the buildings and the behavior of the structural system represented in the analytical environment. Additionally, in each building title, the name of the district/neighborhood is indicated first, followed by the corresponding province. This arrangement enables a joint evaluation of the field photographs and the numerical model results.

Table B.1: Examples of building and RSRA score with multiplier.

NO	BUILDING NAME	RSRA SCORE	SPEC.MULTIPLIER
1	Çarşamba – Samsun	165	1,6
2	Merter – İstanbul	1149	0,23
3	Kırcami – Antalya	517	1,4
4	Mahmutbey – İstanbul	262	1,2
5	Mercan – İstanbul	1262	0,2
6	Serdivan – Sakarya	142	1,2
7	Z. Yatırım (Levent) – İstanbul	1344	0,1
8	Merzifon – Amasya	88	1,2
9	Kumbet – Sivas	410	2
10	Kavak – Samsun	1250	0,3
11	Balmumcu – İstanbul	582.9	1,2
12	Büyükkada – İstanbul	936	0,3
13	Manavgat – Antalya	801	0,5
14	Merter – İstanbul (2nd Building)	381.7	0,5
15	Merkez – Erzincan (13 Şubat)	804.41	0,6
16	Hacıkadi – Zonguldak	397	1,6
17	Tünel Sanat Galerisi – İstanbul	1289	0,6
18	Fatih Bulvarı – Bursa	341	2,4
19	Sarıyer – İstanbul	605	0,5
20	Atatürk Bulvarı – Adıyaman	160	1,4
21	Fındıklı – Rize	882	0,8
22	Serik – Antalya	235	2,4
23	Antalya PGM	15.12	3,2
24	Adana PGM	24.16	3
25	Tirebolu – Giresun	797	1,5
26	Tunceli Merkez	758	0,8
27	Mersin Regional Building	373	1,6
28	Güzelyalı – İstanbul	269	1,2

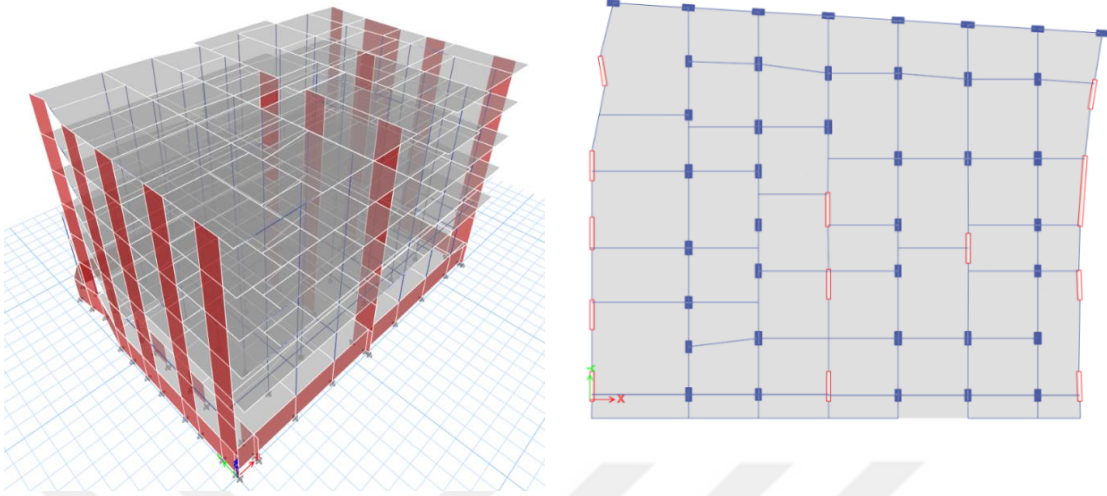


Figure B.1: Çarşamba-Samsun photos

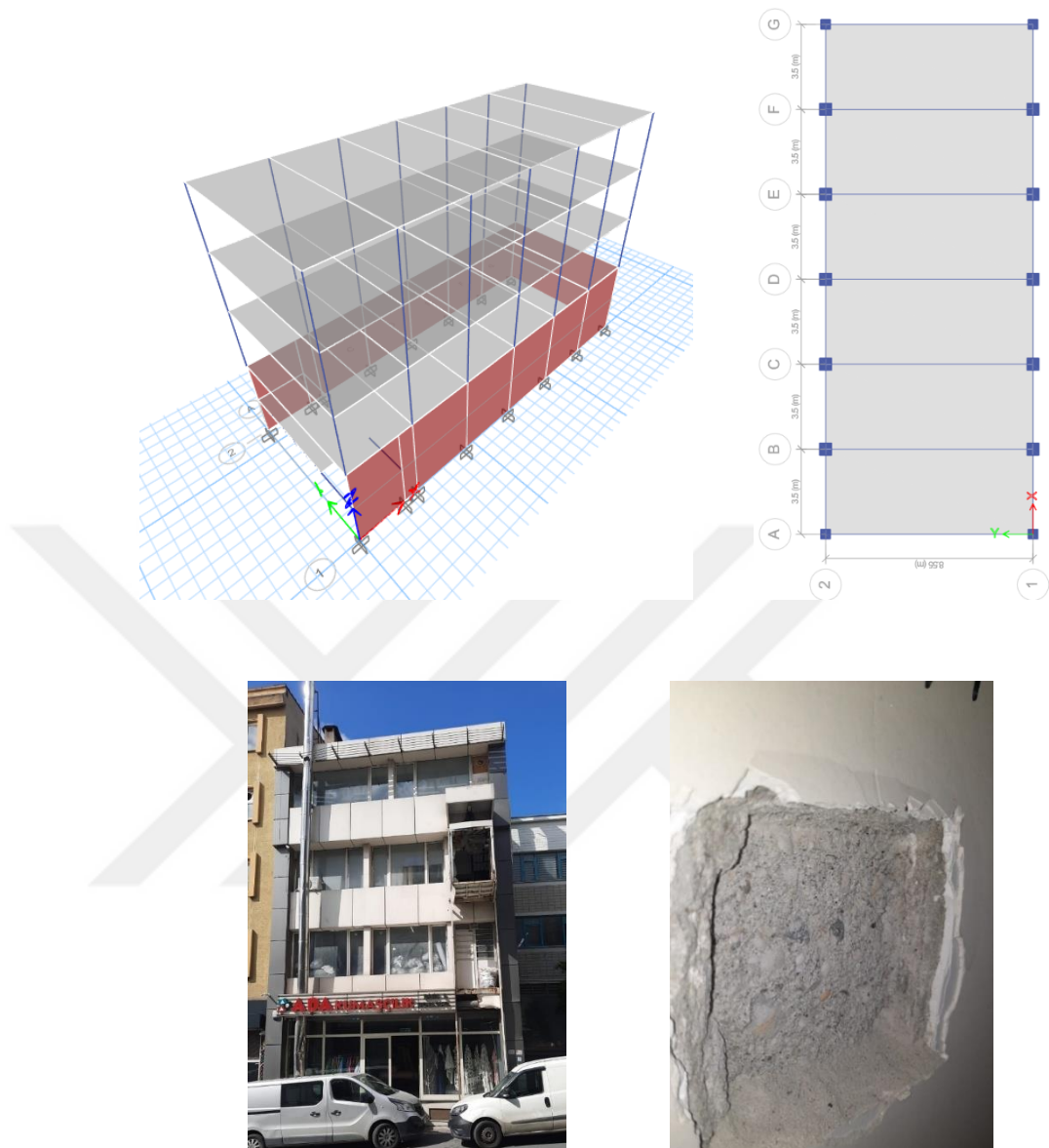


Figure B.2: Merter-İstanbul photos

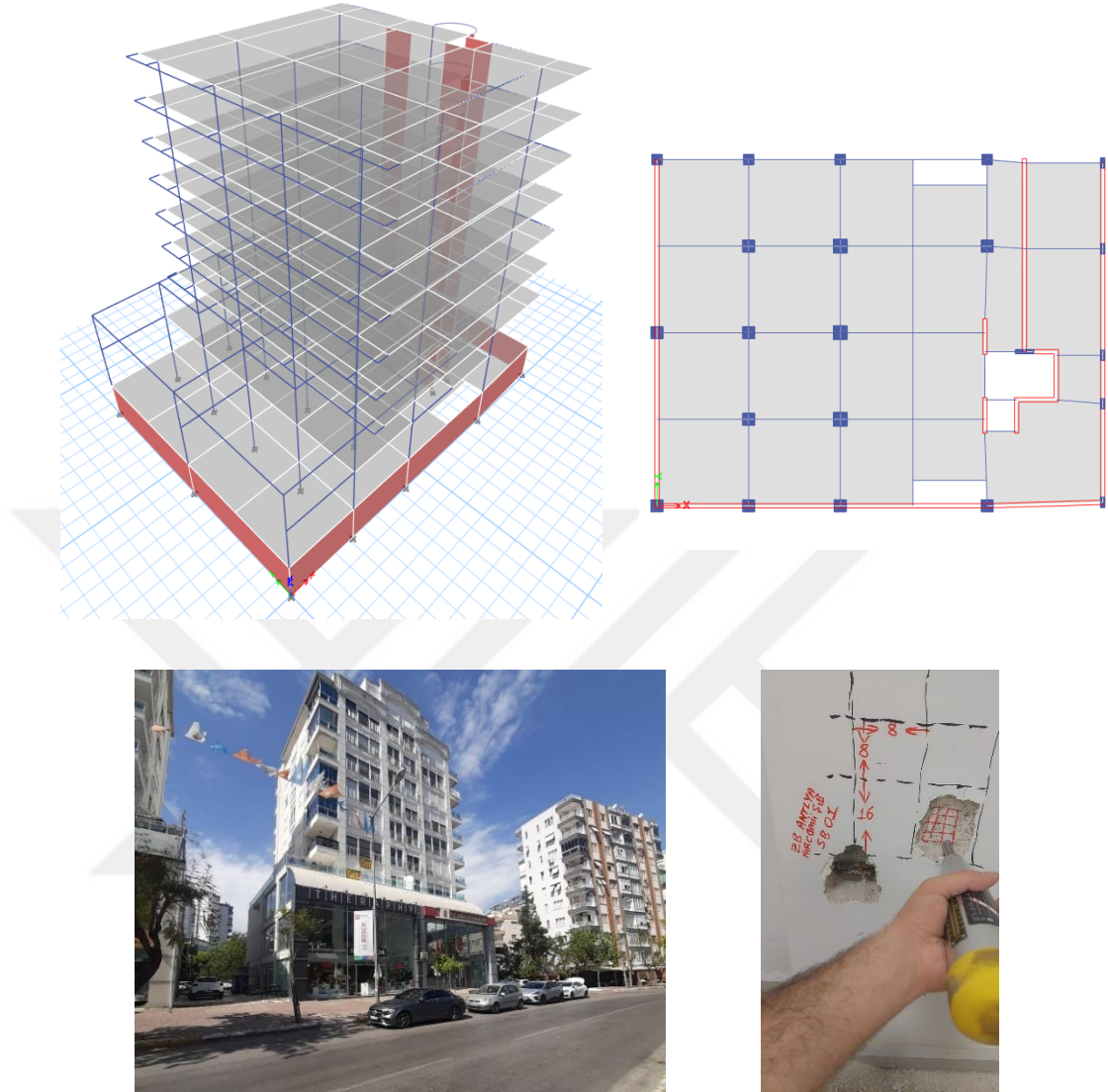


Figure B.3: Kırçami-Antalya photos

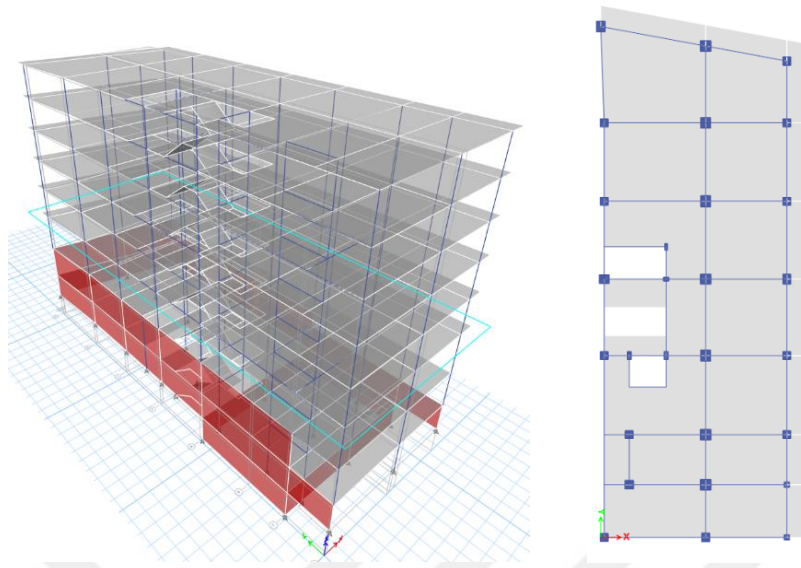


Figure B.4: Mahmutbey-İstanbul photos

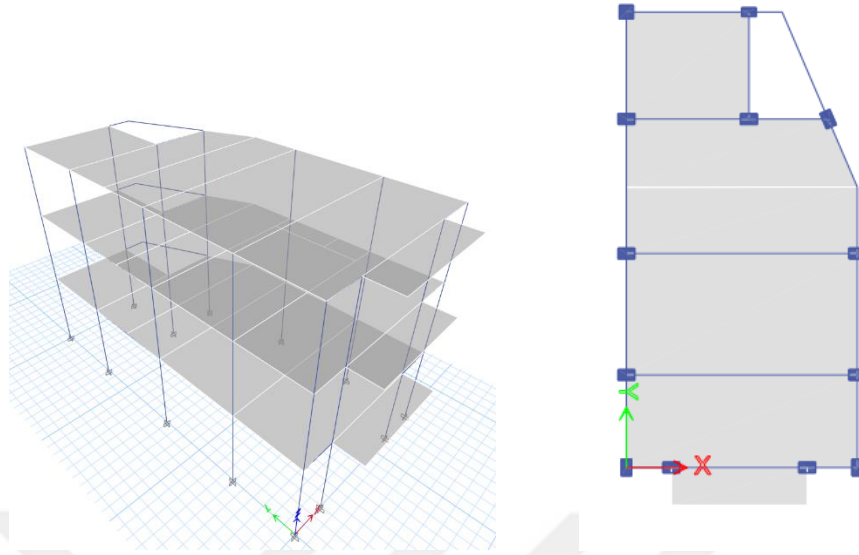


Figure B.5: Mercan-İstanbul photos

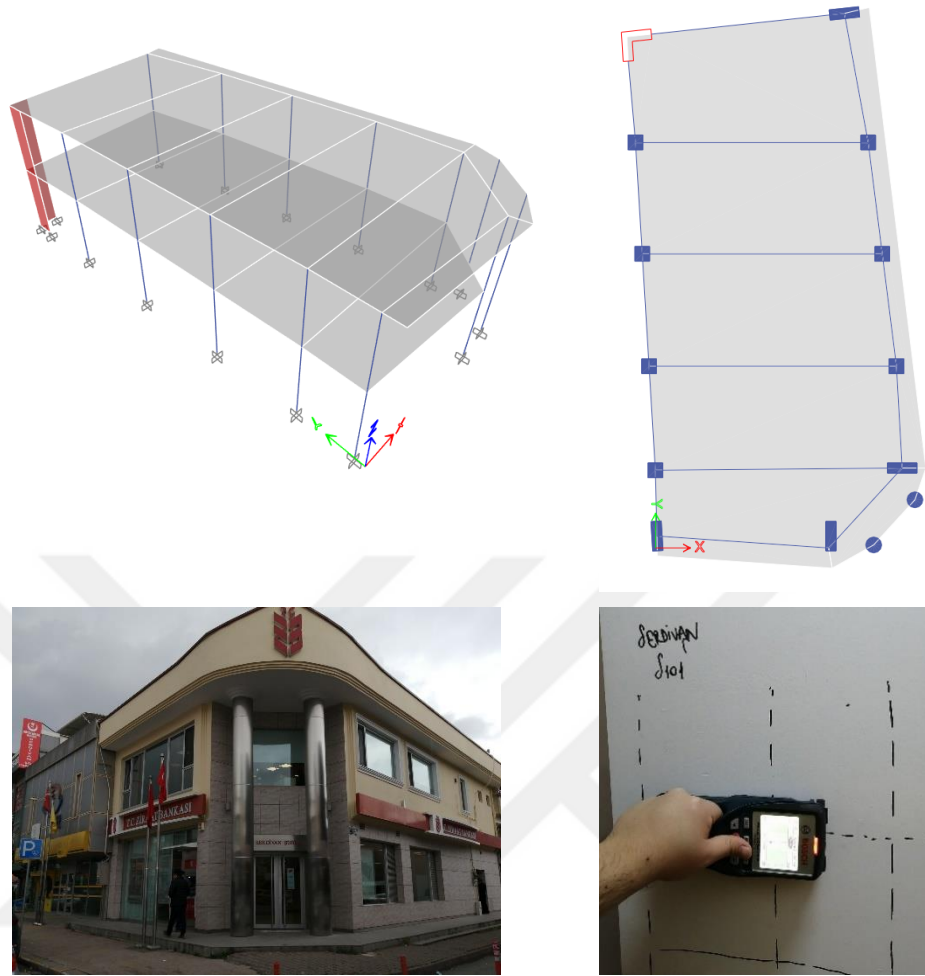


Figure B.6: Serdivan-Sakarya photos

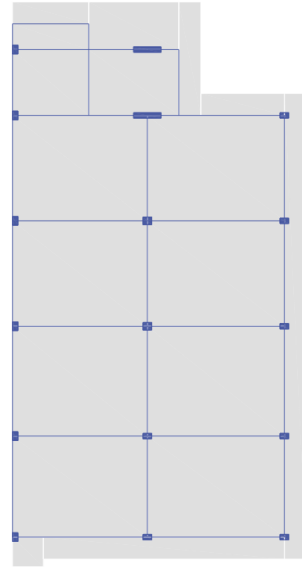
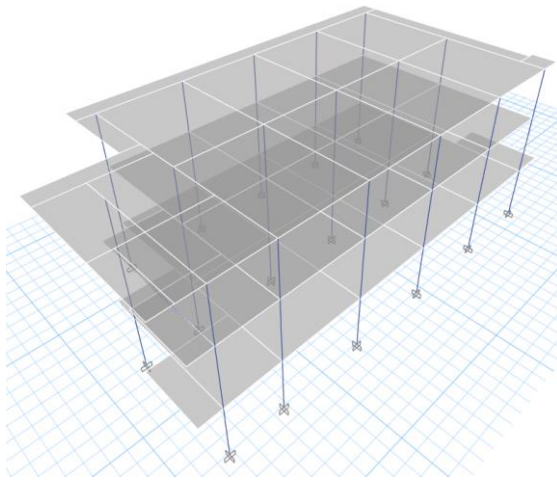


Figure B.7: Z.Yatırım (Levent)-İstanbul photos

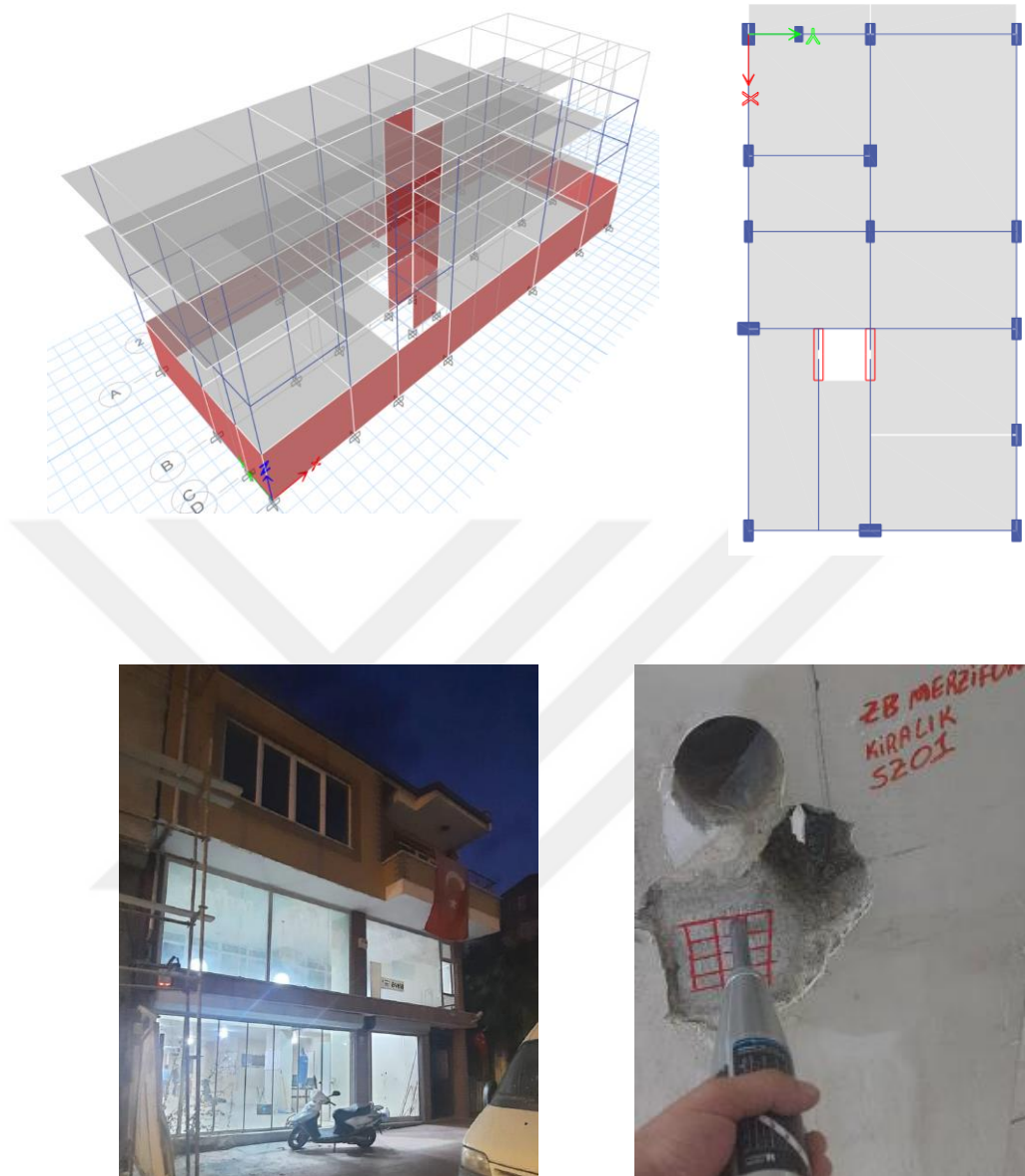


Figure B.8: Merzifon-Amasya photos

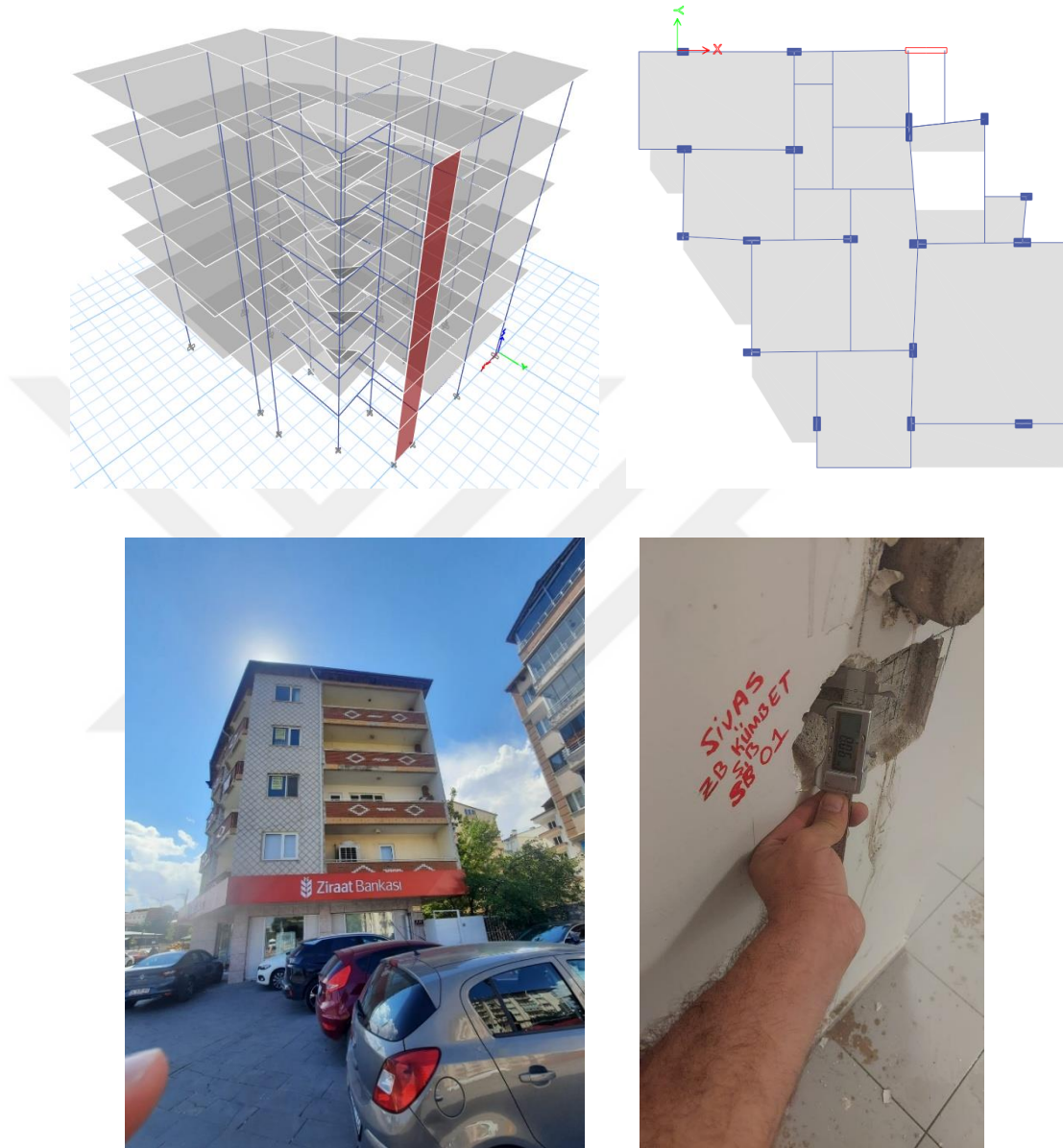


Figure B.9: Kumbet-Sivas photos

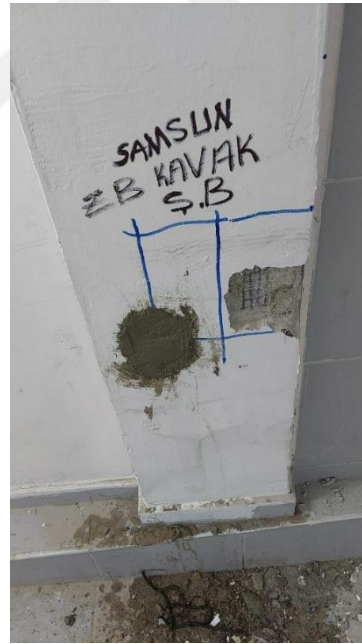
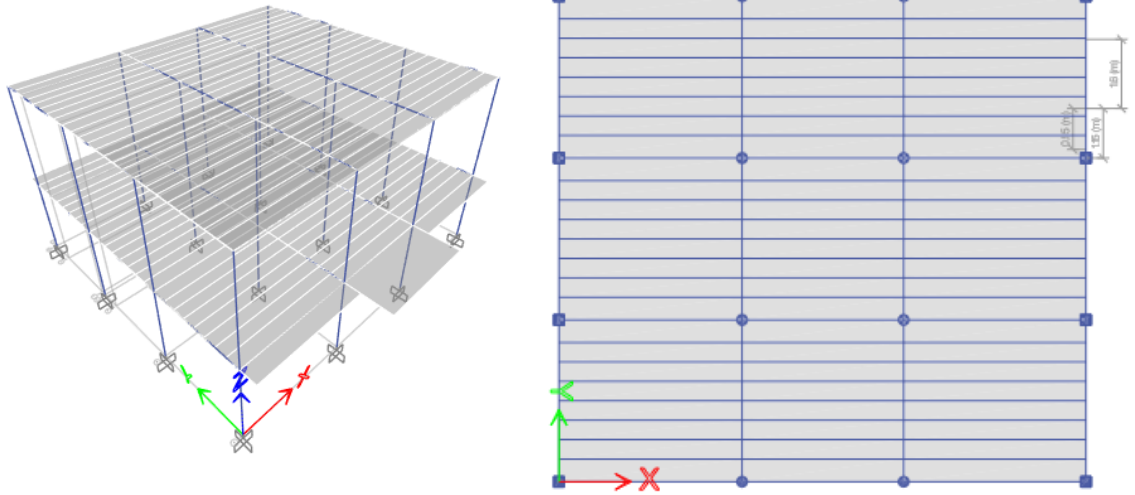


Figure B.10: Kavak-Samsun photos

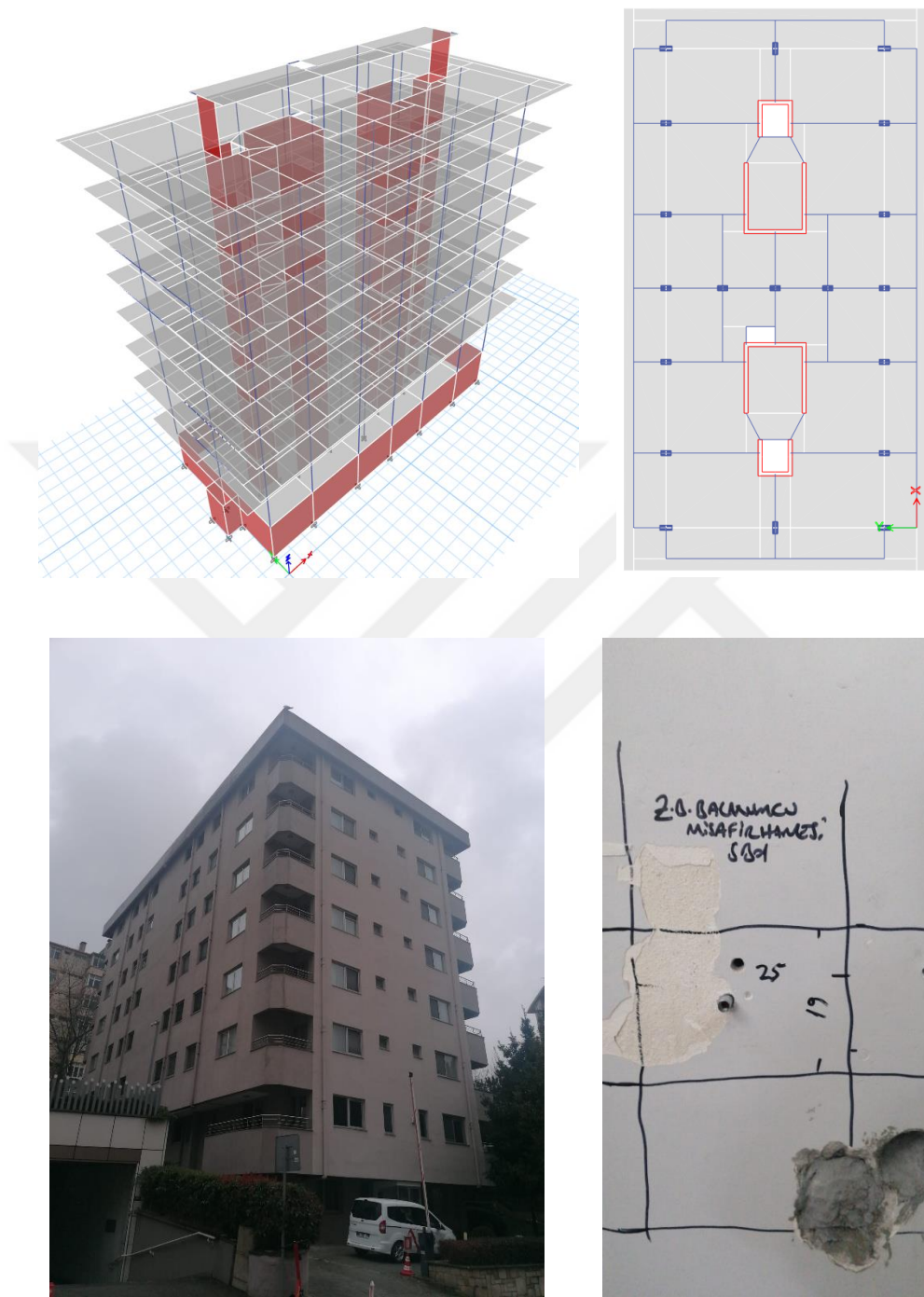


Figure B.11: Balmumcu-İstanbul photos

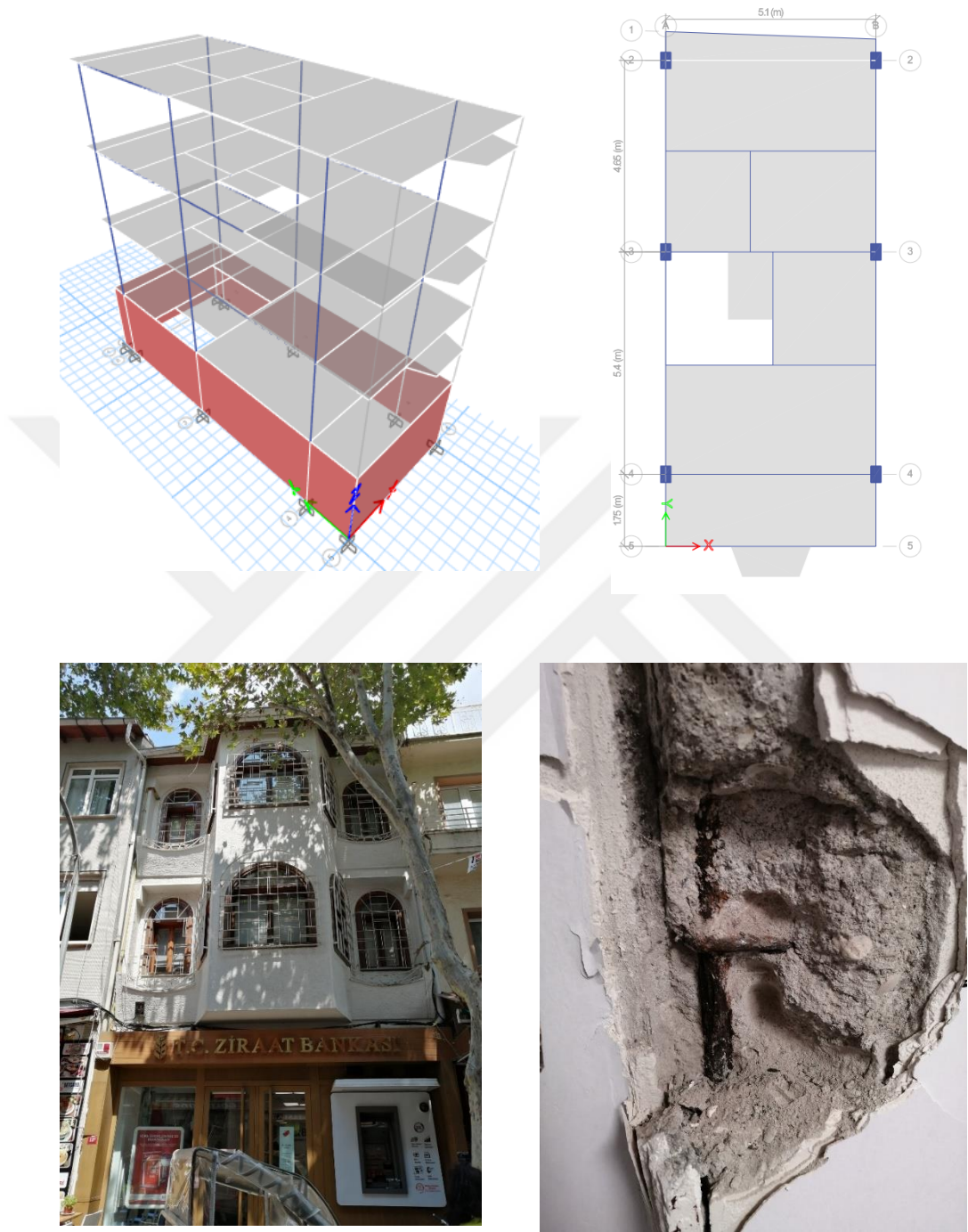


Figure B.12: Büyükada-İstanbul photos

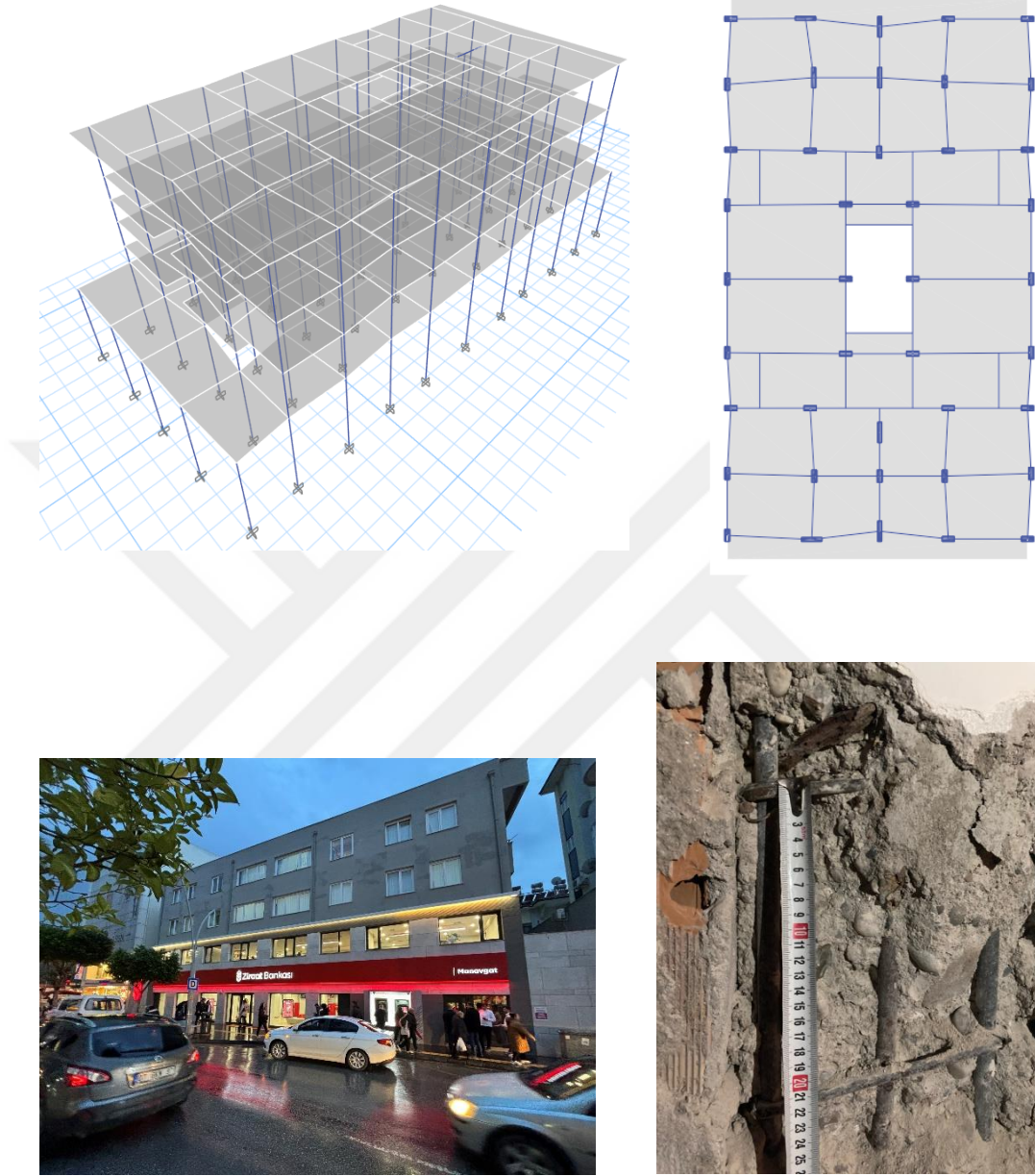


Figure B.13: Manavgat-Antalya photos

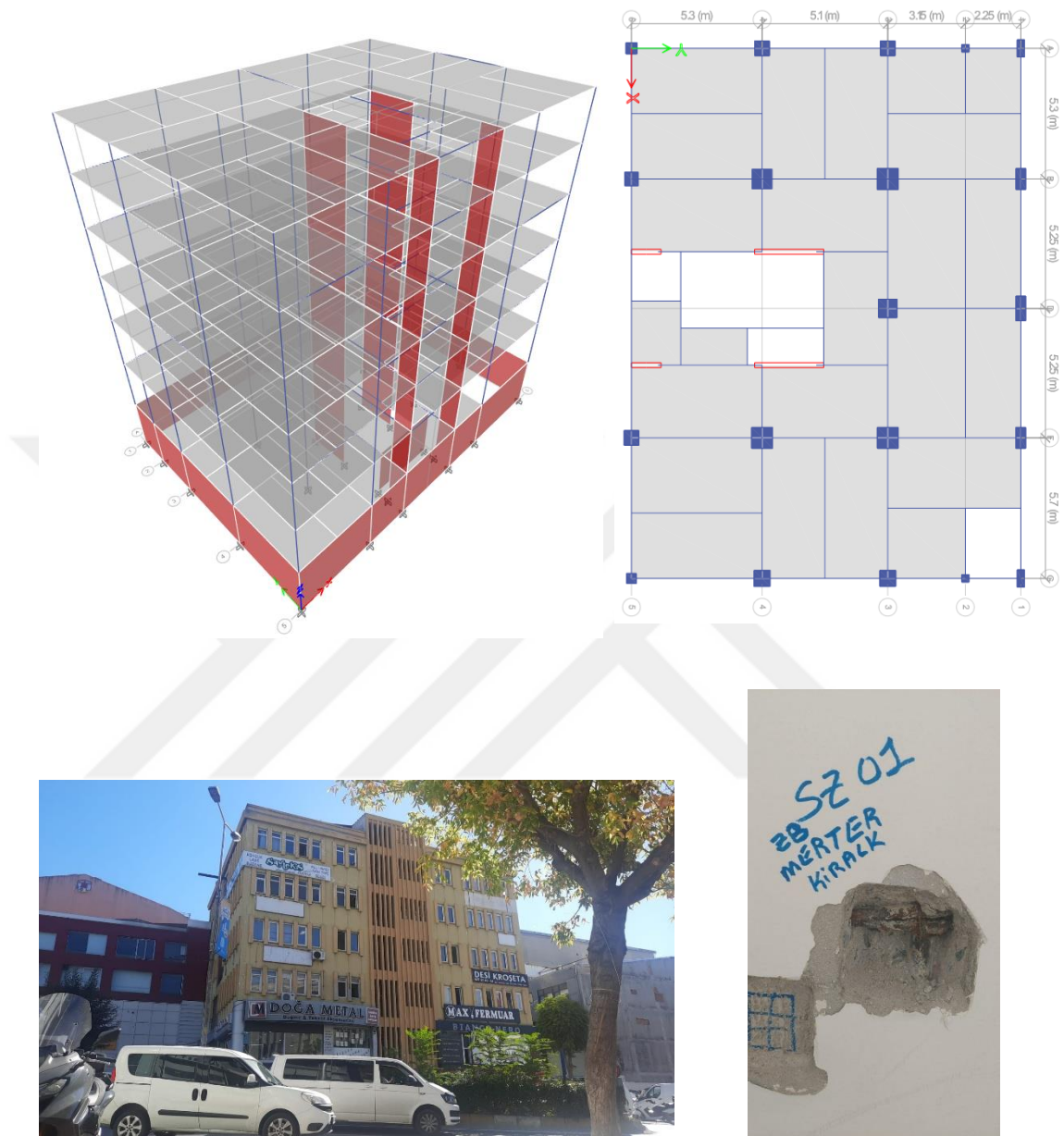


Figure B.14: Merter-İstanbul (2nd building) photos

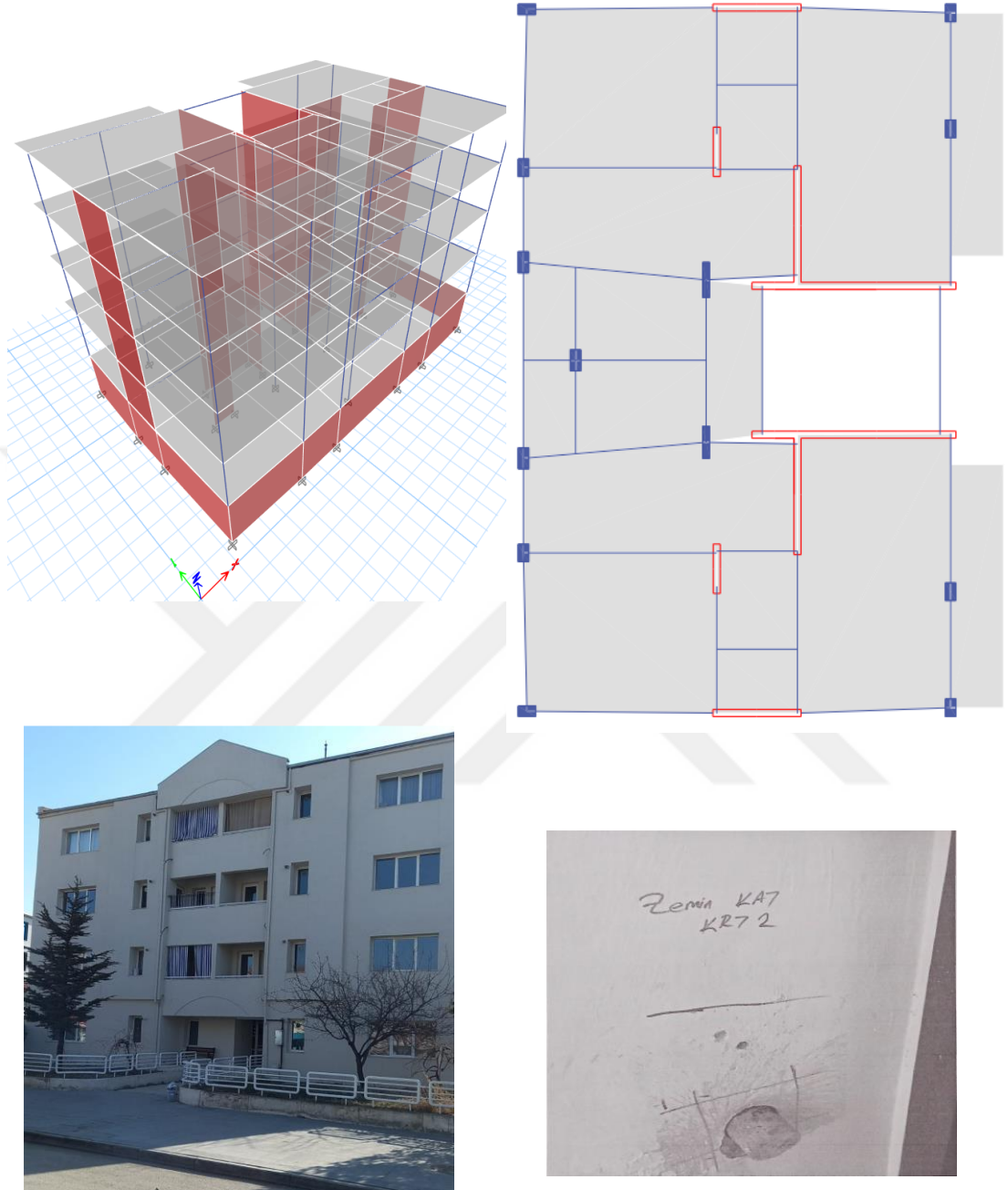


Figure B.15: Merkez-Erzincan (13 Şubat) photos

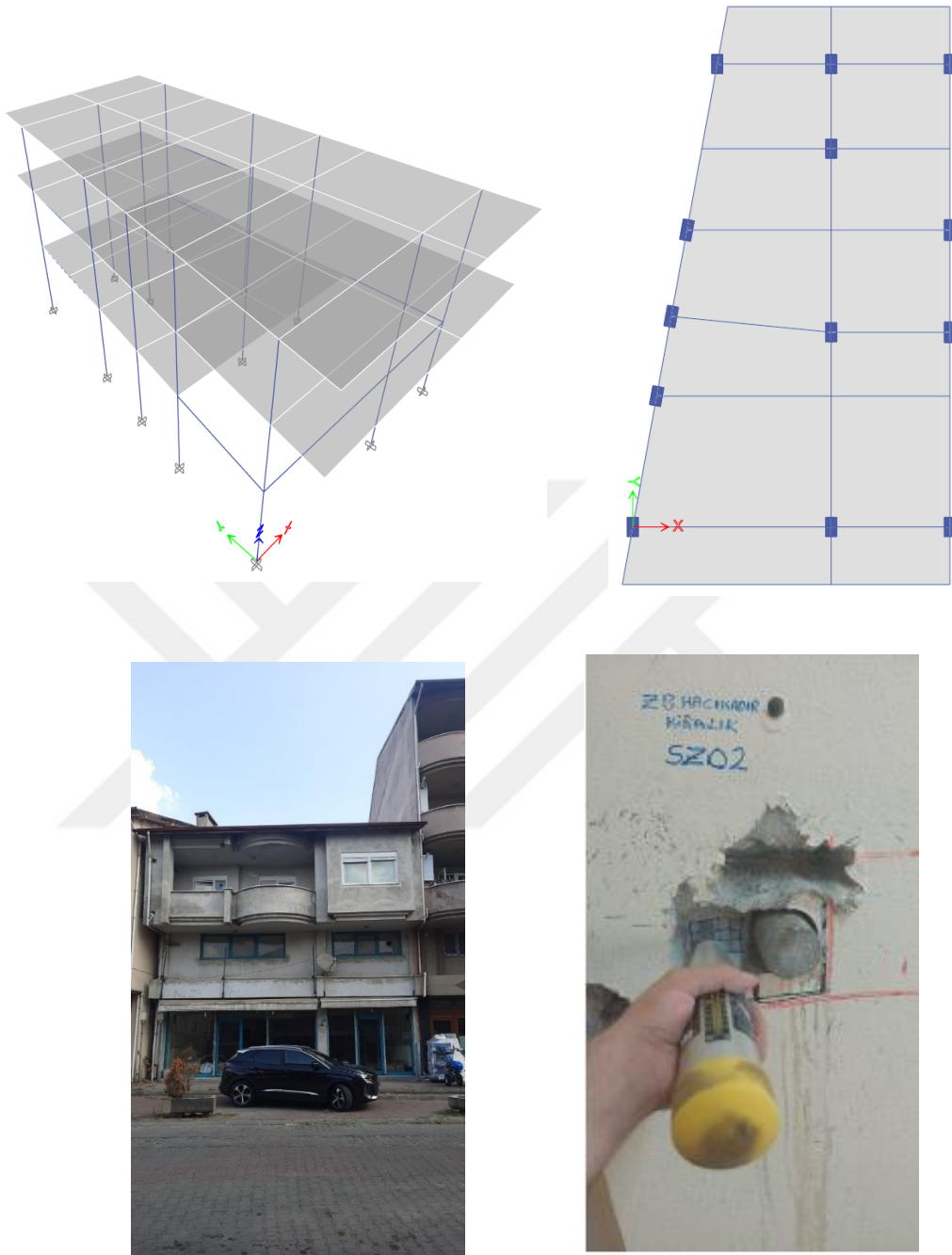


Figure B.16: Hacıkadı-Zonguldak photos

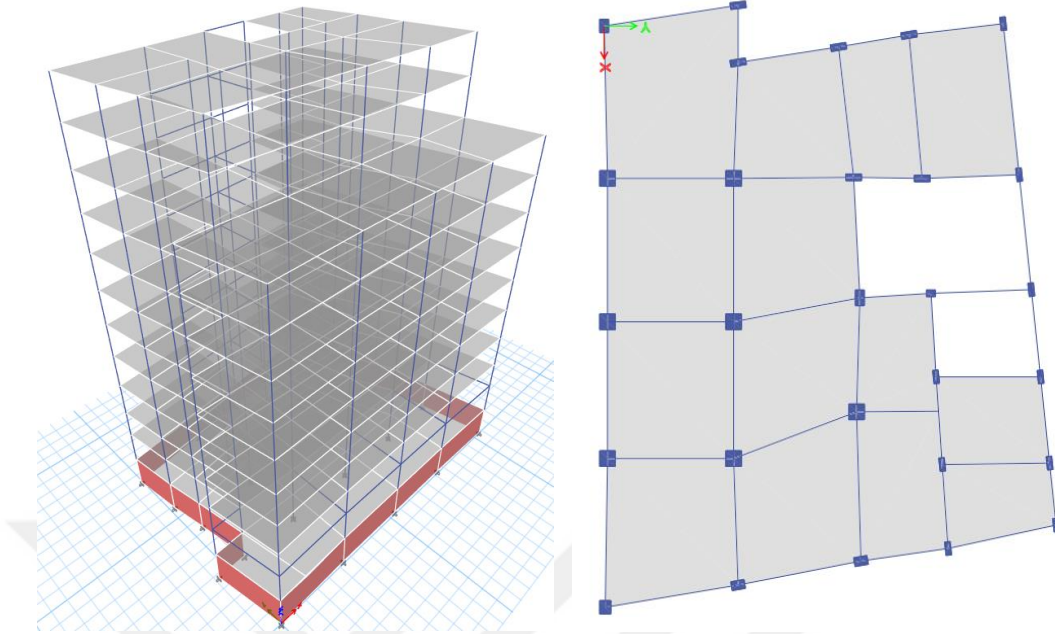


Figure B.17: Tünel Sanat Galerisi-İstanbul photos

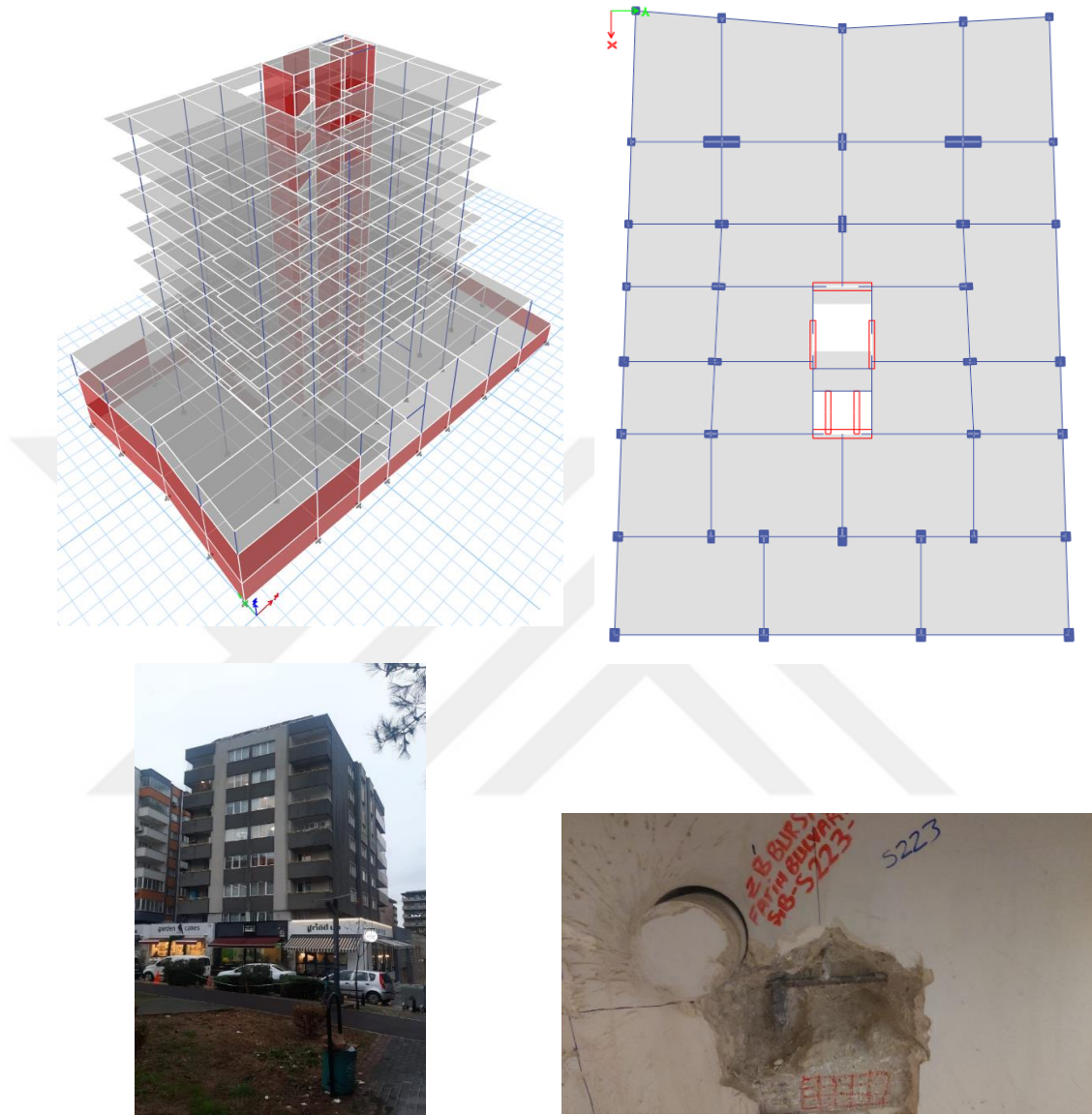


Figure B.18: Fatih Bulvarı-Bursa photos



Figure B.19: Sariyer-İstanbul photos

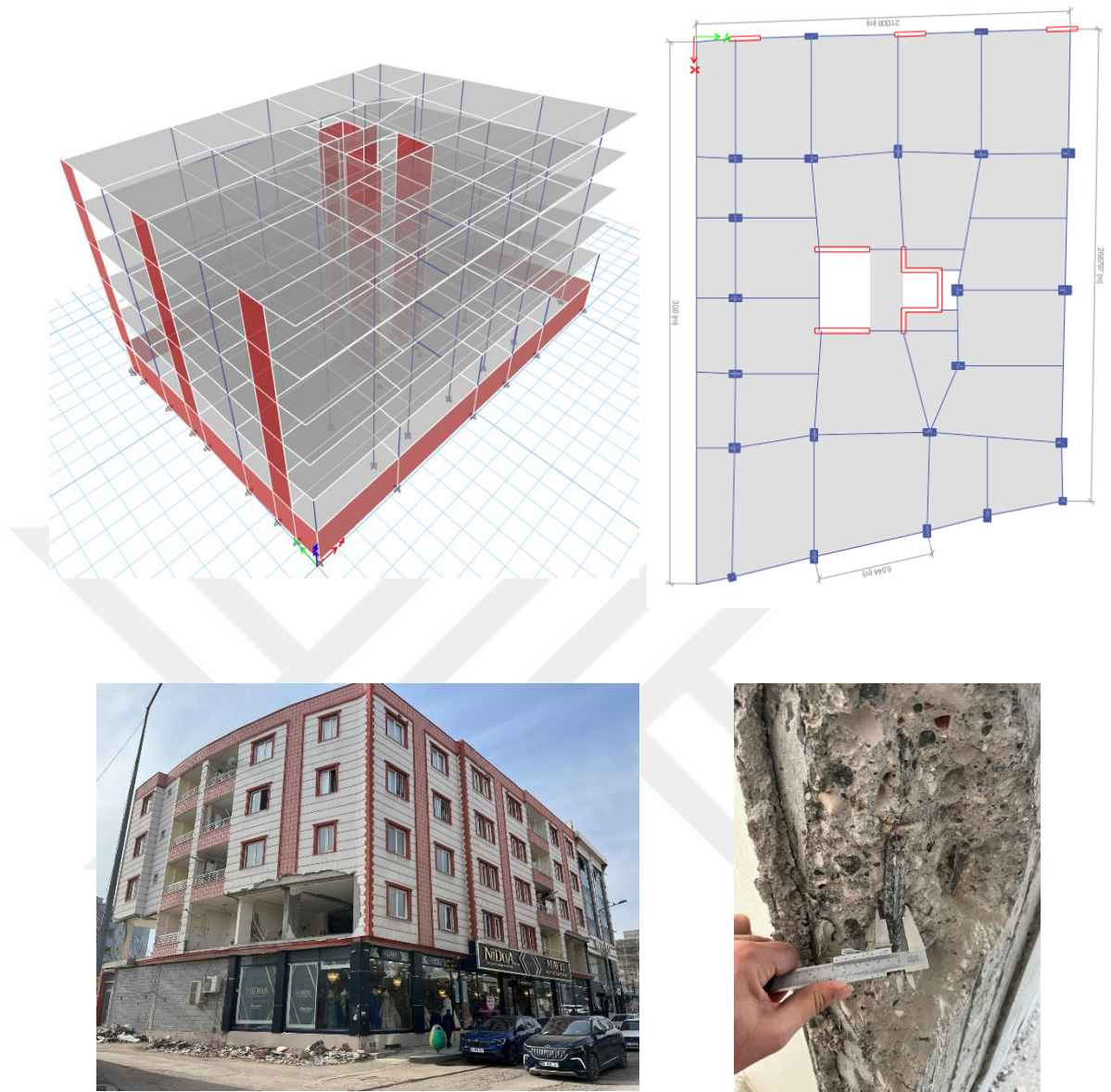


Figure B.20: Atatürk Bulvarı-Adıyaman photos

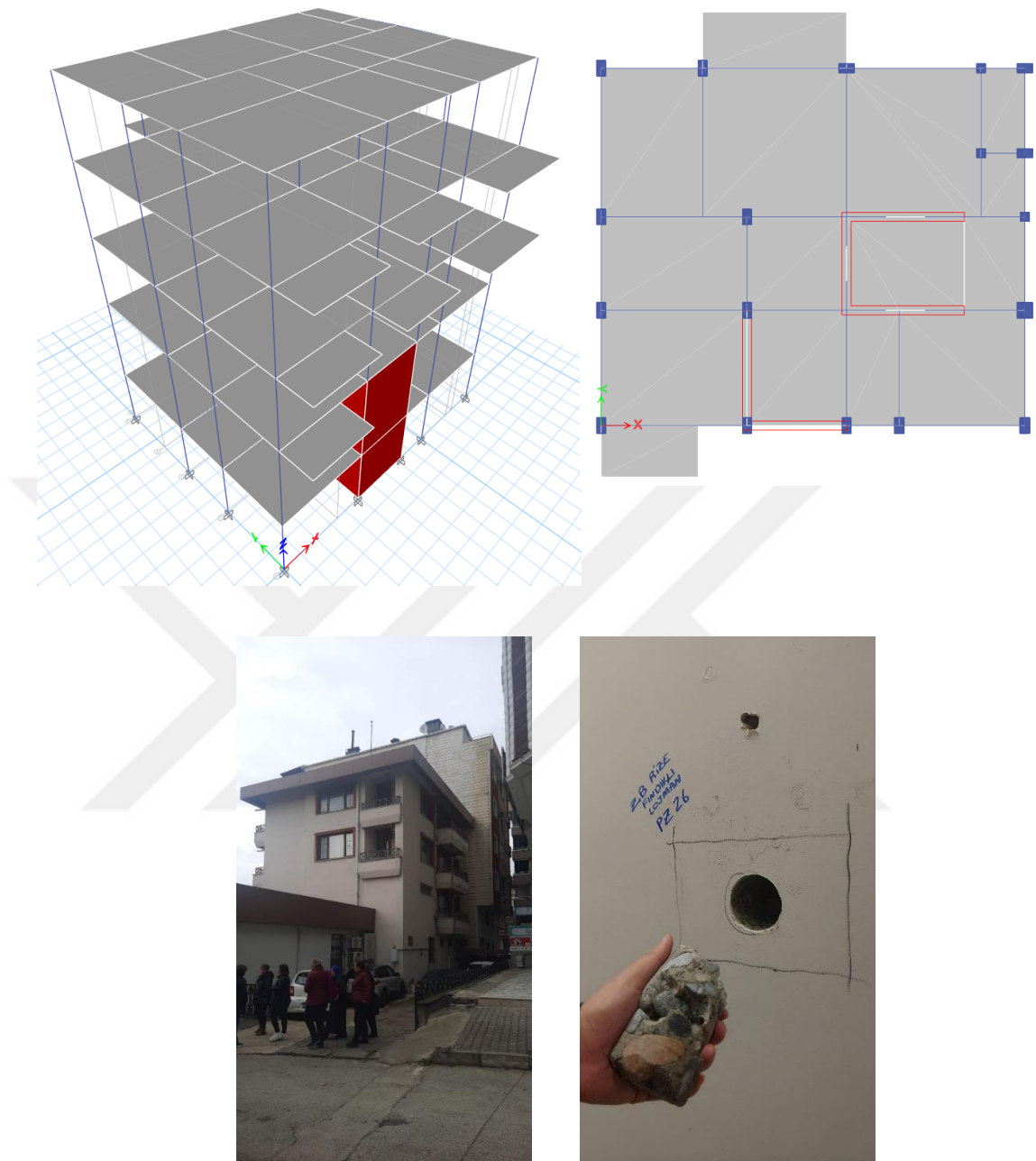


Figure B.21: Fındıklı-Rize photos

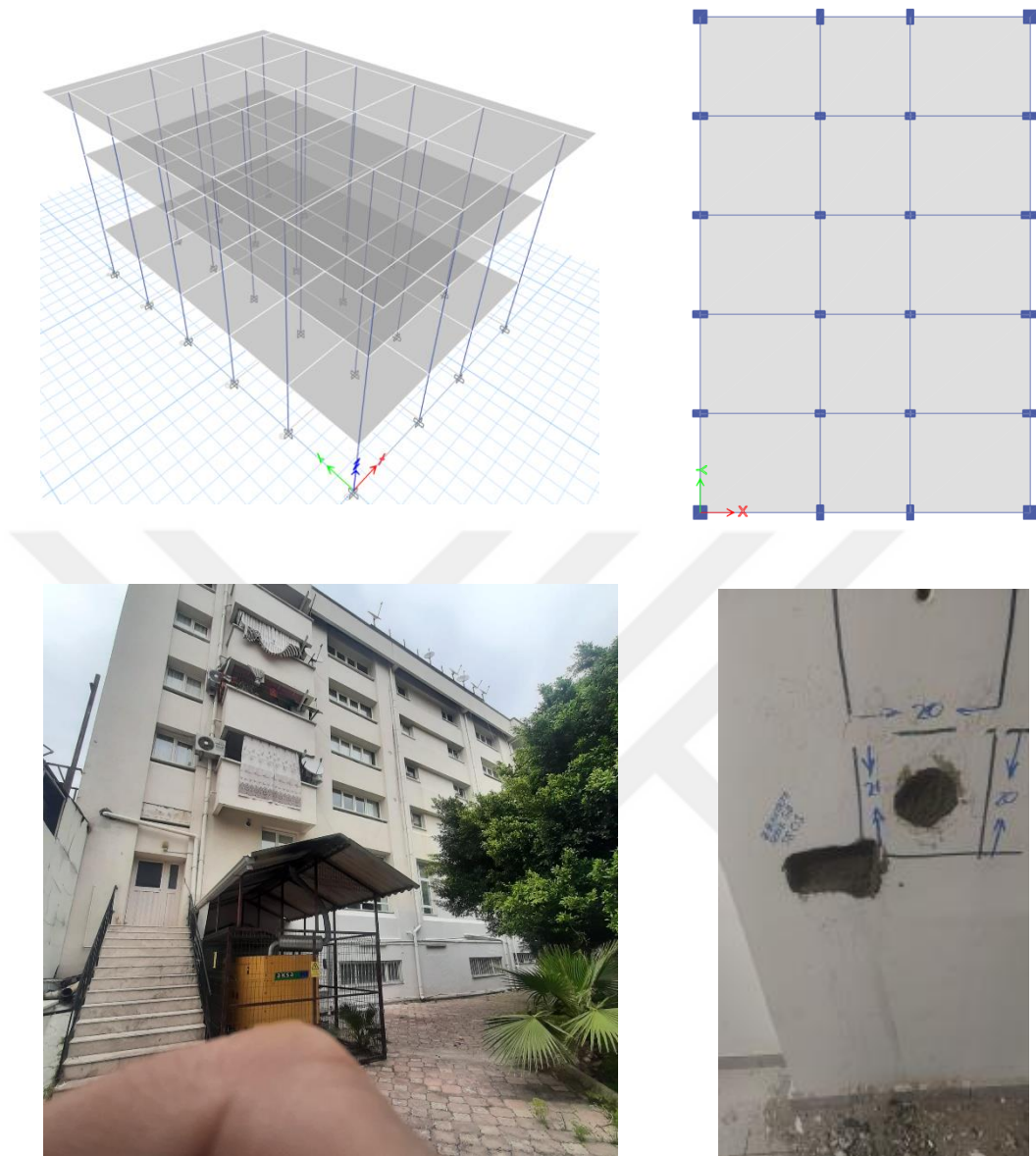


Figure B.22: Serik-Antalya photos

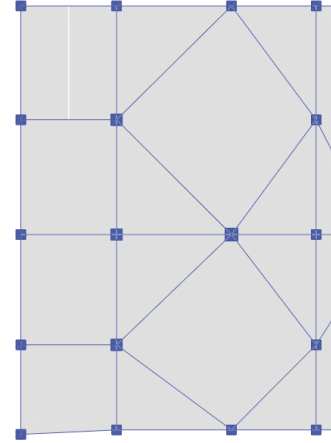
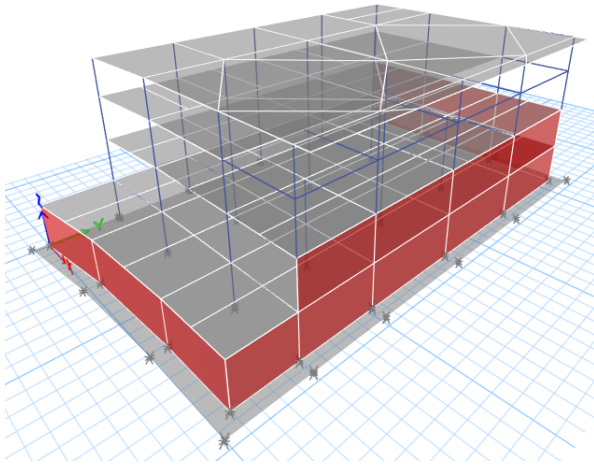


Figure B.23: Antalya PGM photos

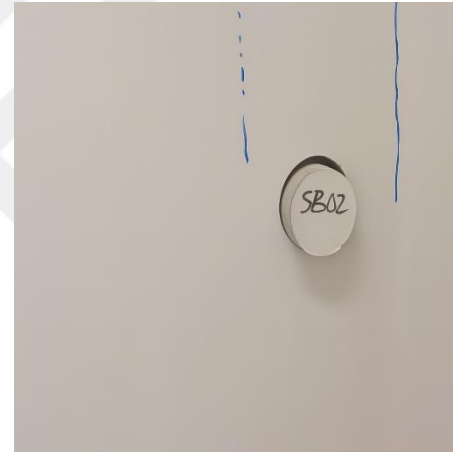
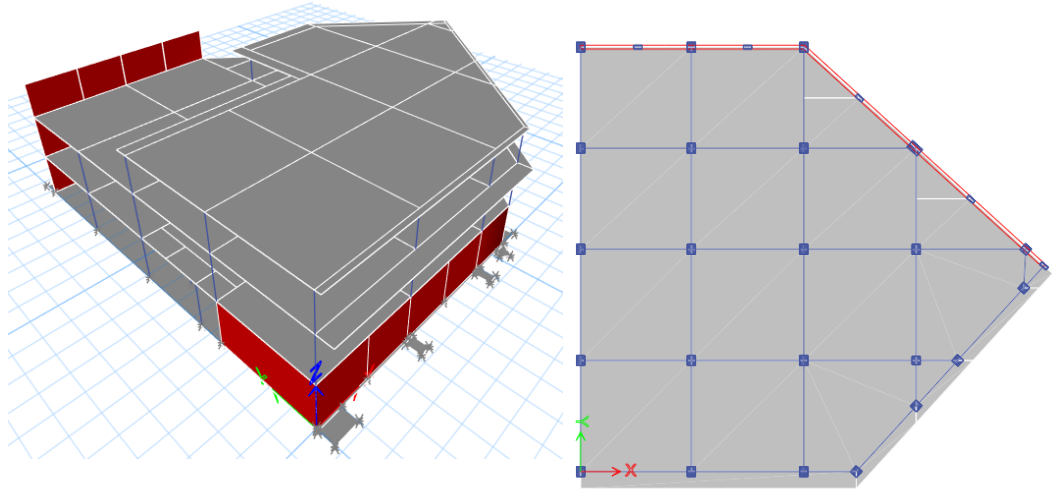


Figure B.24: Adana PGM photos

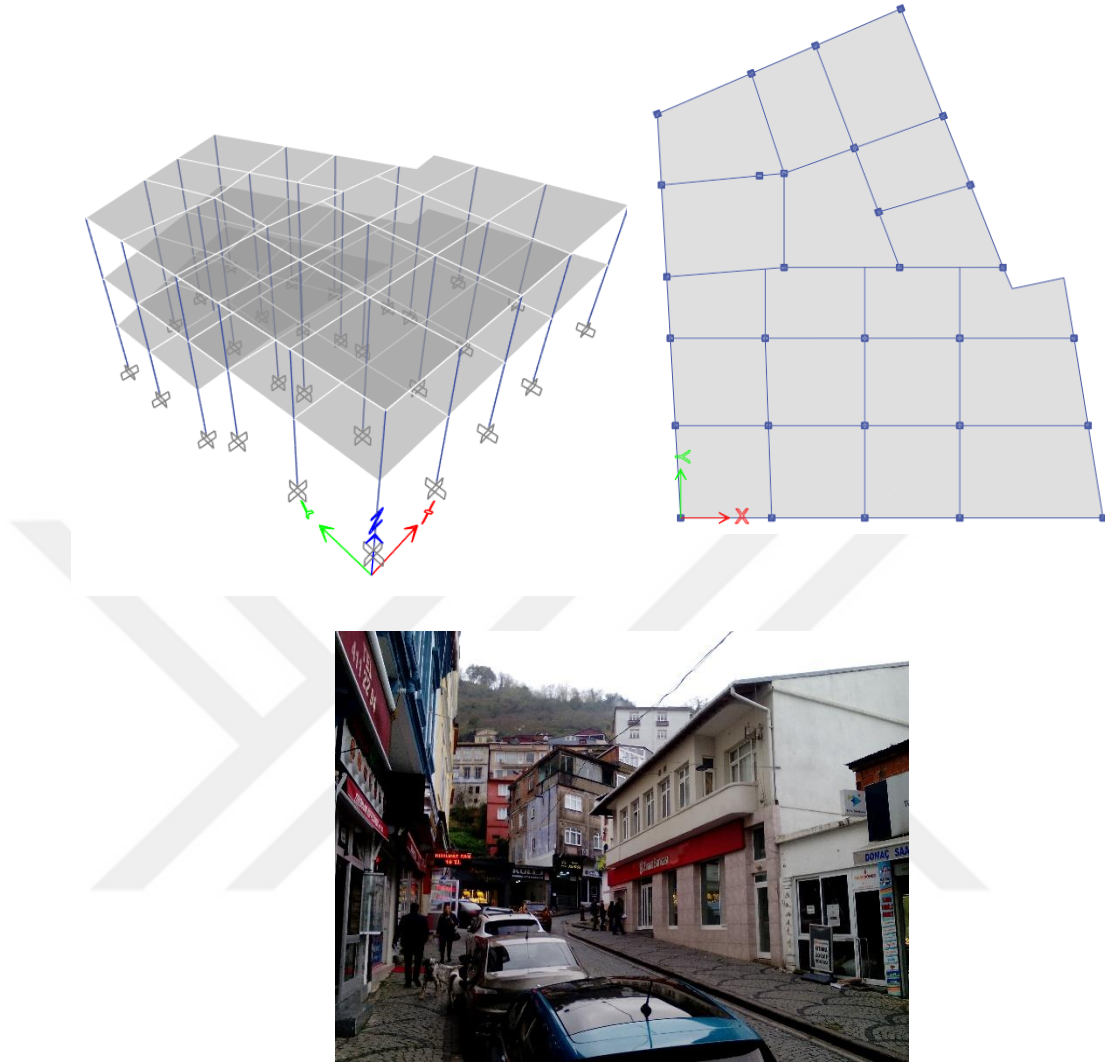


Figure B.25: Tirebolu-Giresun photos

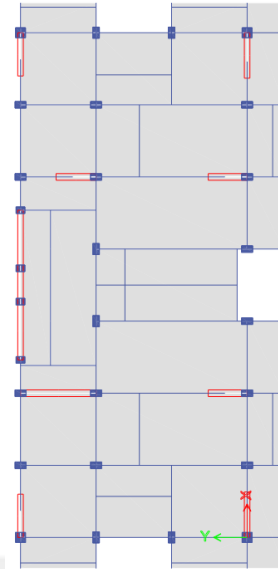
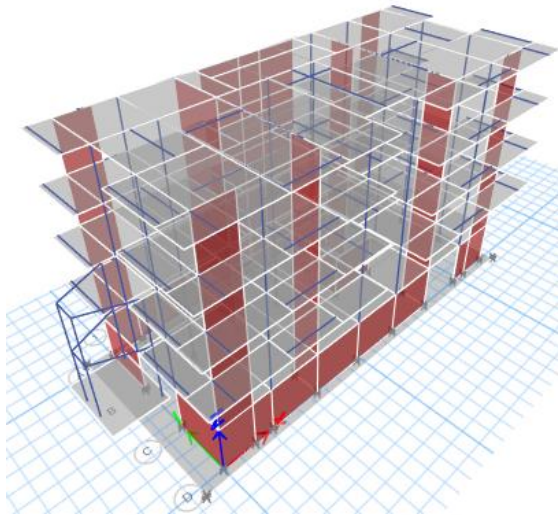


Figure B.26: Tunceli Merkez photos



Figure B.27: Mersin regional building photos

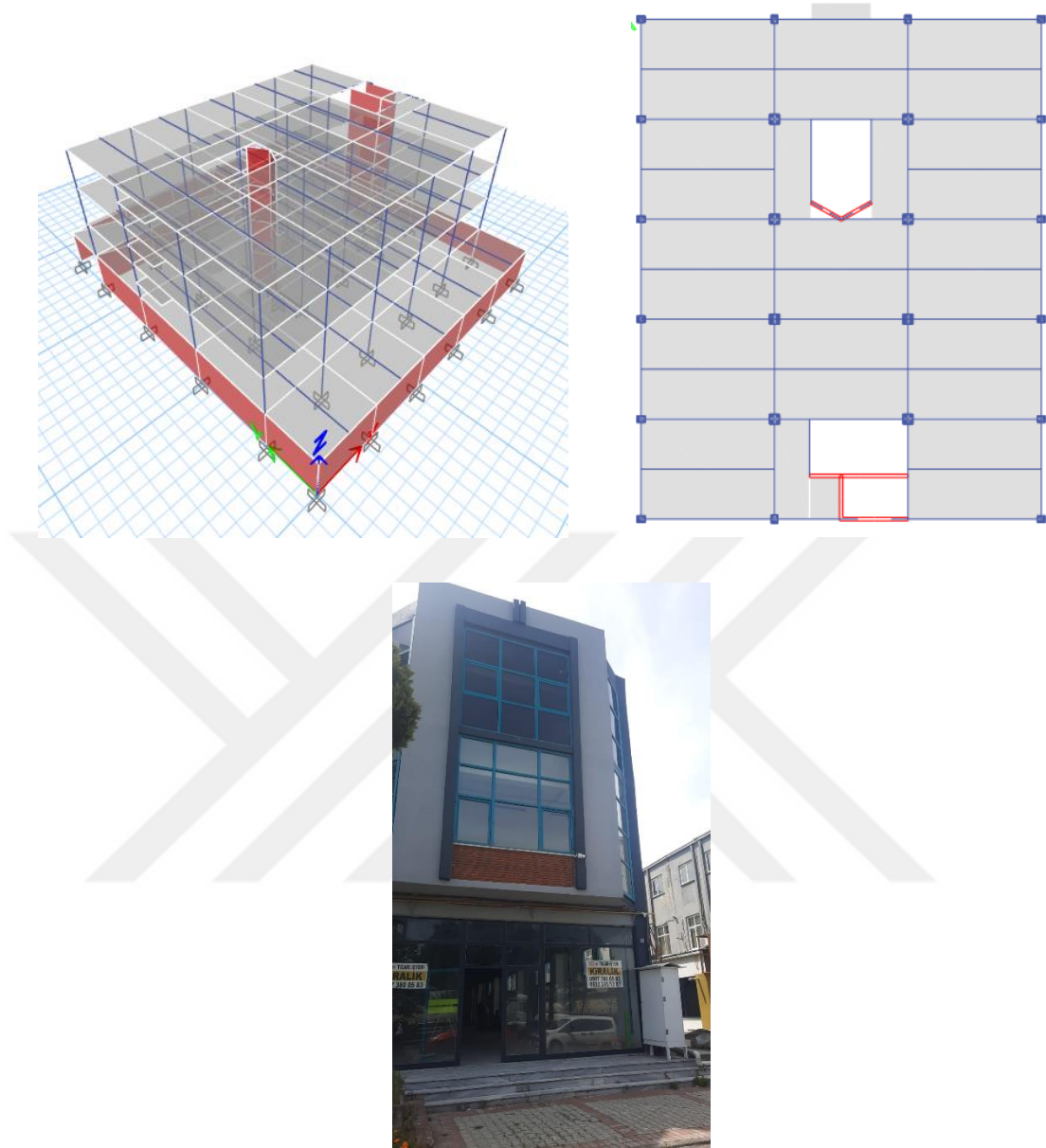


Figure B.28: Güzelyalı-İstanbul photos