COMPARISON OF LINEAR AND NONLINEAR METHODS FOR SEISMIC EVALUATION OF EXISTING REINFORCED CONCRETE BUILDINGS IN THE TURKISH EARTHQUAKE CODE 2007

by

Bahadır Akgül

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to my family

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ABSTRACT

COMPARISON OF LINEAR AND NONLINEAR METHODS FOR SEISMIC EVALUATION OF EXISTING REINFORCED CONCRETE BUILDINGS IN THE TURKISH EARTHQUAKE CODE 2007

Importance of more accurate estimation of the seismic behavior of buildings has been steadily increasing since the 1999 Marmara Earthquake. This increasing trend intensifies the need for a new Earthquake Code in Turkey. Turkish Earthquake Code 2007 has been put in force as of 06.03.2007. Relatively new analyses procedures in this code and developments in technology enable scientists and engineers to predict the seismic behavior of reinforced concrete buildings more realistically. Besides, 73 percent of buildings that collapsed in the 1999 Marmara Earthquake are 1 to 7 story buildings according to the Earthquake Master Plan for Istanbul [Ref. 1]. The purpose of this study is to determine the seismic performance of existing medium-rise (4 to 6 story buildings) reinforced concrete buildings with adequate transverse reinforcement in Turkey using linear and non-linear analysis methods in the Turkish Earthquake Code 2007.

In this study, 4 structural system models which represent the existing medium-rise reinforced concrete buildings in Turkey are chosen and designed in accordance with TEC 75 and TEC 98 regulations. Two of the models are 4 story and the rest are 6 story. Also material properties and seismic zones of the models vary to represent the buildings in different seismic regions with different material characteristics. These models have a symmetric plan as commonly used.

The seismic performances of these models are determined by using linear and non-linear evaluation methods in TEC 2007. In the linear theory, it is assumed that the material is linear-elastic and displacements are very small. Second-order effects are neglected. Therefore, linear methods given in codes are used to obtain the seismic performance of the models. In the non-linear theory, pushover analysis is used. Pushover analysis is a static,

nonlinear analysis procedure in which the magnitude of the lateral earthquake load is incrementally increased until the performance point. Also it is assumed that displacements are not very small in non-linear theory.

Member damage levels calculated by using linear and nonlinear methods of the Turkish Earthquake Code 2007 are compared and evaluated. The basic conclusions obtained from numerical investigations are summarized below;

- The linear method of the Turkish Earthquake Code 2007 gives more conservative member damage levels as compared with those given by the non-linear method.
- Damage levels of structural members obtained from linear and nonlinear methods differ by at most one damage region.
- Members of structural system models designed in accordance with the Turkish Earthquake Code 1998 satisfy adequate seismic performance levels. On the other hand, members of structural system models designed to the Turkish Earthquake Code 1975 are generally in collapse prevention region or collapse region.

ÖZET

MEVCUT BETONARME BİNALARIN DEPREM PERFORMANSLARININ BELİRLENMESİNDE DOĞRUSAL VE DOĞRUSAL OLMAYAN YÖNTEMLERİN KARŞILAŞTIRILMASI

1999 Marmara Depremi' nden beri binaların gerçek sismik davranışlarının daha doğru belirlenmesinin önemi gittikçe artmaktadır. Bu eğilim Türkiye' de yeni bir deprem yönetmeliğinin gerekliliğini ortaya koymuştur. Ardından 2007 Türk Deprem Yönetmeliği 06.03.2007 tarihinden itibaren geçerli olmak üzere yürürlüğe girmiştir. Bu yönetmelikteki nispeten yeni analiz yöntemleri ve teknolojideki gelişmeler bilim adamları ve mühendislere betonarme binaların gerçek sismik davranışlarını tahmin etmede kolaylık sağlamıştır. Bunun yanısıra İstanbul İçin Deprem Master Planı' na göre 1999 Marmara Depremi' nde yıkılan binaların %73'ünü 1-7 katlı betonarme binalar oluşturmaktadır [Ref. 1]. Bu bağlamda yapılan bu çalışmanın amacı ülkemizdeki orta yükseklikteki yeterli kesme donatısına sahip betonarme binaların deprem performanslarının 2007 Türk Deprem Yönetmeliği' ndeki doğrusal ve doğrusal olmayan yöntemlerin kullanılarak belirlenmesidir.

Bu çalışmada Türkiye' deki orta yükseklikteki betonarme binaları temsil etmek üzere 4 taşıyıcı sistem modeli seçilmiş ve 1975, 1998 Türk Deprem Yönetmelikleri' ne göre bu modeller boyutlandırılmıştır. Modellerin ikisi 4 katlı, diğer ikisi 6 katlıdır. Ayrıca seçilen modellerin malzeme özellikleri ve deprem bölgeleri, farklı bölgelerdeki farklı malzeme karakteristiklerindeki binaları temsil etmek üzere farklılık göstermektedir. Tüm bu modellerin kat planları simetrik ve aynıdır.

Bu modellerin deprem performansları 2007 Türk Deprem Yönetmeliği' ndeki doğrusal ve doğrusal olmayan yöntemler kullanılarak belirlenir. Doğrusal teoriye göre hesapta malzemenin doğrusal-elastik ve yerdeğiştirmelerin çok küçük olduğu varsayılmıştır. Bu yüzden yönetmeliklerde yer alan doğrusal hesap yöntemleri kullanılarak

modellerin deprem performansları elde edilir. Doğrusal olmayan hesapta, artımsal itme analizi kullanılmıştır. Artımsal eşdeğer deprem yükü yöntemi birinci titreşim mod şekli ile orantılı olacak şekilde, deprem istem sınırına kadar monotonik olarak adım adım arttırılan eşdeğer deprem yüklerinin etkisi altında doğrusal olmayan itme analizinin yapılmasıdır. Ayrıca doğrusal olmayan hesapta yerdeğiştirmelerin çok küçük olmadığı göz önünde tutulmaktadır.

Son olarak tüm kesitlere ait kesit hasar sınırları doğrusal ve doğrusal olmayan hesap yöntemleri kullanılarak elde edilir ve karşılaştırılır. Sayısal incelemelerden elde edilen sayısal sonuçların başlıcaları aşağıdaki gibidir;

- 2007 Türk Deprem Yönetmeliği' nin doğrusal olmayan hesap yöntemi ile belirlenen kesit hasar bölgeleri daha elverişli sonuçlar vermektedir.
- Doğrusal ve doğrusal olmayan hesap yöntemleri ile belirlenen kesit hasar bölgeleri arasındaki fark bir hasar bölgesi aralığı mertebesindedir.
- 1998 Türk Deprem Yönetmeliği' ne uygun olarak boyutlandırılan yapılarda kesitler yeterli deprem performansına sahip iken 1975 Türk Deprem Yönetmeliği' ne uygun olarak boyutlandırılan yapıların kesitleri genelde *göçmenin önlenmesi* veya *göçme bölgesinde* yer almaktadır.

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LIST OF SYMBOLS/ABBREVIATIONS

$A(T_1)$	:	Spectral acceleration coefficient calculated for T_1 period
A_0	:	Effective ground acceleration coefficient
a_1	:	Modal acceleration corresponding to first mode
$a_1^{(i)}$	:	Modal acceleration corresponding to first mode (in considered earthquake direction) in (i)' th pushover step
a_{y1}	:	Equivalent yield acceleration corresponding to first mode
a	:	Modal displacement
A_c	:	Gross section area of column
A_s	:	Cross sectional area of longitudinal steel reinforcement
b	:	Width of section
b_w	:	Width of beam
C	:	Earthquake coefficient
C_0	:	Seismic zone coefficient
C_{R1}	:	Spectral displacement ratio corresponding to first mode
d	:	Effective height of beam and column, modal displacement
d_1	:	Modal displacement corresponding to first mode
$d_1^{(i)}$	:	Modal displacement corresponding to first mode (in considered earthquake direction) in (i)' th pushover step
$d_1^{(p)}$	:	Modal displacement demand corresponding to first mode
D	:	Width of building along the lateral loads direction
E	:	Modulus of elasticity, earthquake load
E_c	:	Modulus of elasticity corresponding to concrete
EI_0	:	Section stiffness for cracked section
f_{ck}	:	Characteristic compressive strength of concrete
f_{cm}	:	Existing compressive strength of concrete
f_{ctk}	:	Characteristic tensile strength of concrete

f_{ctm}	:	Existing tensile strength of concrete
f_{yk}	:	Characteristic yield strength of reinforcing steel
F	:	Total lateral load
F_c	:	Compressive force of concrete
F_s	:	Tensile force of reinforcing steel under tension
F_i	:	Equivalent earthquake loads acting to the floors
g	:	Dead load
g_i	:	Total dead load at i'th storey of building
G	:	Dead load
G_i	:	Total dead load at i'th storey of building
h	:	Effective height of the section
H_i	:	Height of i'th storey of building measured from the top foundation level
I	:	Moment of inertia of the section, building importance factor
k_1	:	Coefficients related to the locations of the reinforcements at section
k_2	:	Concrete cover coefficient
K	:	Structure type coefficient
l_p	:	Length of plastic hinge
m_x	:	No dimensional bending moment corresponding to design load about X axis
m_i	:	i'th storey mass of building ($m_i = w_i / g$)
$m_{\phi i}$	:	In the case where floors are modeled as rigid diaphragms, mass moment of inertia around vertical axis passing through unshifted mass centre of i'th storey of building
M	:	Bending moment
M_1	:	Modal mass corresponding to first natural vibration mode
M_{cap}	:	Bending moment capacity
M_p	:	Plastic moment capacity of the section
M_{x1}	:	Effective mass corresponding to first mode (in x earthquake direction) defined for linear elastic earthquake direction.

M_r : Modal mass of the r'th natural vibration mode

M_{xr} : Effective participating mass of the r'th natural vibration mode of building in the x earthquake direction considered

M_{yr} : Effective participating mass of the r'th natural vibration mode of building in the y earthquake direction considered

n : No dimensional normal force corresponding to design load, live load participation factor

N : Normal force, total number of stories of building from the foundation level

N_d : Axial compressive force of column under vertical loads

P : Load parameter

P_i : Total live load at i'th storey of building

q : Live load

q_i : Total live load at i'th storey of building

Q : Live load

r : Demand/capacity ratio

R : Structural behavior factor

R_a : Seismic load reduction factor

$R_a(T_1)$: Seismic load reduction factor corresponding to T_1 period

R_{y1} : Strength reduction factor corresponding to the first mode

S : Structure dynamic coefficient

S_a : Spectral acceleration

S_{ae1} : Linear elastic spectral acceleration corresponding to first mode

S_d : Spectral displacement

S_{de1} : Linear elastic spectral displacement corresponding to first mode

S_{di1} : Nonlinear spectral displacement corresponding to first mode

$S(T_1)$: Spectrum coefficient

$T_1^{(1)}$: First natural vibration period of building corresponding to first mode

T : Shear force

T_0	:	Spectrum characteristic period
T_1	:	First natural vibration period of building
T_A, T_B	:	Spectrum characteristic periods
u_n	:	Roof displacement
$u_{xN1}^{(i)}$	:	Roof displacement in (i)’ th pushover step along x direction corresponding to first mode
$u_{xN1}^{(p)}$	:	Roof displacement demand along x direction corresponding to first mode
V	:	Shear force
V_b	:	Base shear force
V_t	:	In the Equivalent Seismic Load Method, total equivalent seismic load acting on the building (base shear) in the earthquake direction considered
$V_{x1}^{(i)}$	:	Base shear force in (i)’ th pushover step along x direction corresponding to first mode
w_i	:	Total weight of (i)’ th story of the building
W	:	Total building weight including live loads multiplied by related participation factors
ΔF_N	:	Additional equivalent seismic load
δ	:	Lateral displacement
ε	:	Unit deformation
ε_c	:	Concrete strain
ε_{cg}	:	Compression strain for confined region of concrete
ε_{cu}	:	Ultimate compression strain of concrete
ε_s	:	Strain for reinforcing steel
ε_{su}	:	Ultimate strain for reinforcing steel
ε_{sy}	:	Yield strain for reinforcing steel
ϕ_p	:	Plastic curvature demand
ϕ_t	:	Total curvature demand
ϕ_u	:	Ultimate curvature demand

ϕ_y : Equivalent yield curvature

Φ_{xN1} : Mode shape of first mode corresponding to N' th story in x direction

Γ_{x1} : Modal contribution coefficient corresponding to first mode in x direction

η_{bi} : Torsional irregularity factor defined at i'th storey of building

λ : Equivalent seismic load reduction factor

θ_p : Plastic rotation demand

μ : Ductility factor

ρ : Geometric reinforcement ratio of tension reinforcement

ρ' : Geometric reinforcement ratio of compression reinforcement

ρ_b : Balanced reinforcement ratio

ρ_s : Volumetric ratio of existing transverse reinforcement steel

ρ_{sm} : Volumetric ratio of transverse reinforcement that is required for design of a new building

$\omega_1^{(1)}$: Angular frequency corresponding to the first mode in first step (i=1) of pushover analysis

ω_B : Angular frequency corresponding to the characteristic period of acceleration spectrum

ATC 40 : Seismic Evaluation and Retrofit of Concrete Buildings

CG : Life Safety Level

SDR : Significant Damage Region

CR : Collapse Region

ETABS : Extended 3d Analysis of Building Systems

GÇ : Collapse Limit (Member Damage Level), Collapse Level

GÖ : Collapse Prevention Level

GV : Visible Damage Limit (Member Damage Level)

HK : Immediate Occupancy Level

MDR	:	Minimum Damage Region
VDR	:	Visible Damage Region
MN	:	Minimum Damage Limit (Member Damage Level)
SSM	:	Structural System Model
SSM-1	:	Structural System Model 1
SSM-2	:	Structural System Model 2
SSM-3	:	Structural System Model 3
SSM-4	:	Structural System Model 4
TS-498	:	Design Loads for Buildings
TS-500	:	Requirements for Design and Construction of Reinforced Concrete Structures
TEC 75	:	Turkish Earthquake Code 1975
TEC 98	:	Turkish Earthquake Code 1998
TEC 2007	:	Turkish Earthquake Code 2007
XTRACT	:	Cross Section Analysis Program for Structural Engineers

1. INTRODUCTION

1.1. General

Anatolia, throughout its history, has experienced countless earthquakes due to its geographical characteristics. As a recent example; the August 17, Marmara Earthquake was a devastating catastrophy and a great human tragedy for Turkey. It is said to have been the largest earthquake to cause heavy damage on an industrialized region since the 1906 San Francisco and 1923 Kwanto, Tokyo-Japan earthquakes [Ref. 2]. Approximately 17,000 fatalities and 44,000 injuries occurred. 100,000 collapsed buildings forced to displace more than 250,000 people. In other words, earthquakes which occur in Turkey cause much more damage, loss of life and loss of property compared with their magnitudes. There also is a striking point that damages are not only present in rural areas but also in densely populated residential areas. This case indicates that the major portion of existing structures in seismic zones do not have sufficient performance against earthquakes.

The reasons for poor seismic performance of buildings, obtained after observations, research and studies performed on damaged buildings are as follows:

a) use of low quality material
b) designs not in accordance with the related codes applicable at the time of design
c) incompliance with basic engineering principles in design and construction

In Turkey, most private, public and government buildings are cast-in place concrete structures designed and constructed before the 1960’s. It is also known that densely populated settlements are located on active fault lines. Therefore, it is necessary to observe the seismic performances of these existing buildings and retrofit them if required under the serious threat of earthquakes.

The reference document for determining the seismic performance of existing buildings is the Turkish Earthquake Code 2007 published in the official gazette dated 06.03.2007. This code has two approaches for determining the seismic performances of buildings namely linear and nonlinear analysis methods.

Linear analysis methods include static lateral force procedures, dynamic lateral force procedures and elastic procedures using demand capacity ratios. Simplified nonlinear analysis methods, referred to as nonlinear static analysis procedures include the capacity spectrum methods that use intersection of the pushover curve and a reduced response spectrum to estimate maximum displacement .

The seismic performances of the significant portion of state and local government buildings have been observed according to these methods given in the code. However, seismic evaluations for the remainder of the buildings also need to be investigated.

1.2. Objective of This Study

The objective of this study is to investigate the seismic performances of medium-rise buildings that represent the significant portion of existing structures in Turkey by using linear and nonlinear methods in the Turkish Earthquake Code 2007. The main tasks of this study are:

- to compare the linear and nonlinear methods given in TEC 2007
- to determine the seismic performances and seismic safety of the medium-rise buildings that constitute the major portion of existing structures in Turkey
- to observe the actual response of these kinds of medium-rise reinforced concrete moment-resisting frames detailed to related codes
- to obtain the information on the ultimate capacities (strength, deformability etc.) of the structures

1.3. Steps of This Study

This research consists of six chapters. The first chapter includes the subject, the scope and objectives of the study.

In the second chapter, performance based design of new structures and seismic performance evaluation methods of existing structures mentioned in the Turkish Earthquake Code 2007 are explained.

In the third chapter, design methodology used to determine the seismic performances of existing structures are given. Steps of case studies are briefly explained.

The fourth chapter is devoted to case studies. Structural system models that represent major portion of existing structures and older buildings in Turkey that are designed in accordance with the Turkish Earthquake Codes 1975 and 1998 regulations. Then, the seismic performances of chosen structural system models are determined according to linear and nonlinear methods given in the Turkish Earthquake Code 2007.

The fifth chapter covers the seismic performance evaluation and comparison of structural system models. Member damage levels and story drifts of all structural system models obtained from linear and nonlinear methods are compared in this chapter.

The sixth chapter includes results obtained from this study. Evaluations and general recommendations also take place in this chapter.

2. PERFORMANCE BASED DESIGN AND EVALUATION

Economic considerations and seismic design philosophy require that buildings be able to resist major earthquakes without collapse but with some structural damage. Especially in recent years, performance based design is improved in United States and Europe for determining the actual seismic performance and seismic safety of the buildings in seismic zones. It is imperative to determine the seismic performances of existing buildings.

In our country, after having the great catastrophy, 1999 Marmara Earthquake, studies on adding a new section to the existing earthquake code about not only determining the seismic performances of the existing buildings but also retrofitting them and revising other sections of the code have begun in 2003. These studies are completed and the Turkish Earthquake Code 2007 is published on 06.03.2007.

Information for determining the seismic safety and seismic performances of medium-rise buildings according to the Turkish Earthquake Code 2007 are as follows [Ref. 5];

2.1. Member Damage Levels and Member Performance Regions of Structural Members

Structural members shall be divided into two parts as “ductile” and “brittle” in determination of member damage levels. Ductile or brittle case of the members depend on that the related structural member reaches its capacity in what failure type.

2.1.1. Member Damage Levels

The member damage level is defined as the post-event conditions of the structural members. Member damage levels for ductile structural members can be divided into three limits. The limits are *Minimum Damage Limit* (MN), *Visible Damage Limit* (GV) and *Collapse Limit* (GÇ).

Minimum damage limit defines the limit that post–elastic behavior has begun at critical section. The limit for post–elastic behavior where critical section provides adequate strength is described as visible damage limit. Collapse limit represents the limit of the behavior before collapsing of the section. It is not allowed to occur post–elastic behavior for brittle structural members.

2.1.2. Member Performance Regions

Structural members whose their member damage levels are less than MN limit shall be considered in *Minimum Damage Region(Minimum Hasar Bölgesi)*, between MN and GV limits shall be considered in *Visible Damage Region(Belirgin Hasar Bölgesi)*, between GV and GÇ limits shall be considered in *Significant Damage Region(İleri Hasar Bölgesi)*. Structural members of whom their member damage levels are greater than GÇ limit define *Collapse Region (Göçme Bölgesi)*.

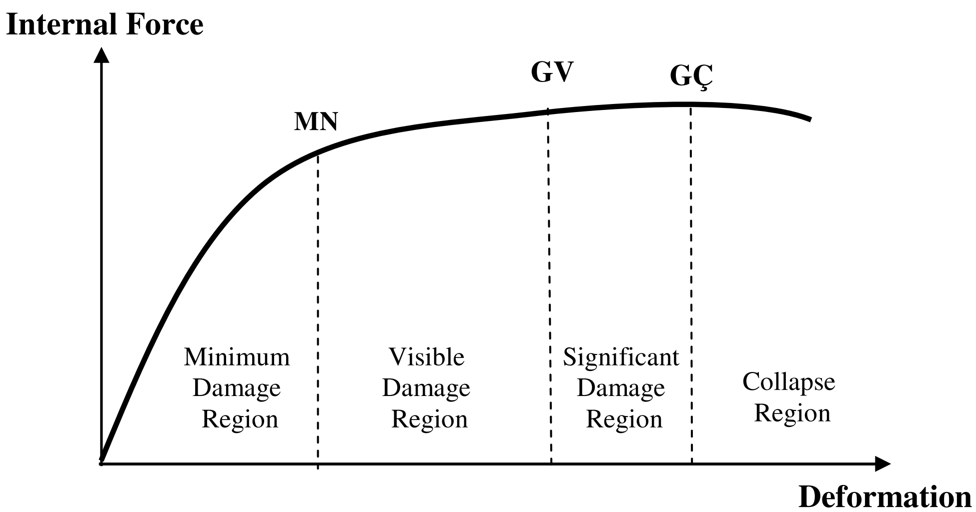


Figure 2.1. Member Damage Levels and Member Performance Regions on Capacity Curve

2.2. Building Performance Levels

Seismic safety of the buildings is related to the damage level possibly to occur in the structure under effect of the seismic load applied. Four building performance levels are defined.

2.2.1. Immediate Occupancy Level (HK)

At any story, as a result of the calculations performed at each seismic load direction, maximum 10 percent of the beams can be in visible damage region, but all other structural members shall be in minimum damage region. In this case, the building can be considered as in *Immediate Occupancy Level* and it is not required to be retrofitted.

2.2.2. Life Safety Level (CG)

At any floor, as a result of the calculations performed at each seismic load direction, maximum 20 percent of the beams and some of the columns can be in significant damage region. But, contribution of the columns in significant damage region must be less than 20 percent of the total shear force carried by columns. All other structural members shall be in minimum damage region or visible damage region. In this case, the building can be considered as in Life Safety Level. To comply with life safety level, the ratio of the shear force carried by the columns whose minimum damage limits are exceeded in both upper and lower end sections at any story to the shear force carried by all columns at the related story ratio must be less than 30%. The ratio of total shear force of the vertical components in significant damage region at roof story to total shear force of the columns at the related story ratio can not be more than 40%. Decision on rehabilitation should be made based on the number and distribution of members that exceed the safety limit.

2.2.3. Collapse Prevention Level (GÖ)

At any floor, as a result of the calculations performed at each seismic load direction, maximum 20 percent of the beams and some of the columns can be in collapse region. But, contribution of the columns in collapse region must be less than 20 percent of the total shear force carried by columns. Columns in the collapse region at any story shall not create a collapse mechanism at that story. All other structural members shall be in minimum damage region, visible damage region or significant damage region. In this case, the building can considered as in *Collapse Prevention Level*. To comply with collapse prevention level, the ratio of the shear force carried by the columns whose minimum damage limits are exceeded in both upper and lower end sections at any story to the shear force carried by all columns at the related story ratio must be less than 30%. The ratio of total shear force of the vertical components in collapse region at roof story to total shear force of the columns at that story ratio can not be more than 40%. Usage of the building in existing situation is undesirable and must be retrofitted. However, the economical efficiency of retrofitting must be evaluated.

2.2.4. Collapse Level (GÇ)

If the building does not provide the conditions of collapse prevention level, it can be considered as in *Collapse Level*. Retrofitting of the building is necessary. But the economical efficiency of retrofitting must be evaluated. The usage of the building in existing condition is not permitted.

2.2.5. Limitations for Story Drifts

Story drift limitations for each seismic direction are given in Table 2.1.

Table 2.1. Story Drift Limitations

Story Drift Ratio	Performance Level		
	MN	GV	GÇ
$(\delta_i)_{max} / h_i$	0. 01	0. 03	0. 04

2.3. Data Collection From The Buildings

2.3.1. The Scope of Data to be Collected From Buildings

2.3.1.1. The dimensions and details of members, data about structural system geometry and material properties to be used in determining the capacity of the structural members of existing buildings and evaluating the seismic performance of them shall be obtained from projects and reports of the buildings, observations and measurements to be made in the building and experiments to be applied to the material samples obtained from the buildings.

2.3.1.2. The processes to be performed in the scope of the obtaining data from buildings are defining the structural system, determination of foundation system, geometric properties and soil properties of the building, determination of the existing damage and changes and / or repairs performed, measurements of dimension of the members, determination of the material properties, controlling of whether all data obtained in field are in compliance with the buildings' project.

2.3.2. Information Levels

Information levels shall be classified as *limited*, *medium* and *detailed information levels* respectively. Information levels obtained affect the choice of the design method to be used in determining the seismic performances of the buildings.

- *Limited Information Level,* structural working drawings of the building do not exist. Structural system properties are determined in the measurements to be taken in the building.

- *Medium Information Level,* if structural working drawings of the building do not exist, more measurements are performed with respect to limited information level. If projects of the building exist, measurements defined in limited information level are performed and the project is verified.

- *Detailed Information Level,* structural working drawings of the building exist. Necessary measurements are taken to confirm the information about the project.

2.3.3. Existing Material Strength

Existing material strength to be used in determining the capacities of the structural members is defined as existing material properties.

2.3.4. Limited Information Level in Reinforced Concrete Buildings

2.3.4.1. *Building Geometry:* The plan of the structural system is prepared by performing a site survey. In case the architectural projects exist, they are used as assistance for the site surveys. The information obtained must include locations, spans, heights and dimensions of all reinforced concrete members and partitions and must be sufficient for modeling of the building. Foundation system is observed through a control pit to be opened at corner of the building. Short columns and similar irregularities in the building are marked on the floor plan. Relation with neighbor buildings (separated, adjacent, joint exists / does not exist) is determined.

2.3.4.2. *Member Details:* Structural working drawings or shop drawings of building do not exist. Details of the reinforced concrete members are assumed to be in compliance with the minimum reinforcement requirements at the time of construction of the building. In order to verify this assumption or in order to determine *the rate of accuracy* of it, reinforcement control is carried out on of 10 percent of the columns and 5 percent of the beams on each story not being less than one, by peeling off the concrete cover shares on two adjacent faces. The peeling off operation shall be carried out on the one third parts in the middle of the span of the length of columns and beams. Surfaces that are peeled off shall be filled later with high–strength repairing plaster.

2.3.4.3. *Material Characteristics:* In each story, at least two concrete samples shall be taken from columns or shear walls in compliance with the conditions determined in TS – 10465. The experiment shall be made and *existing concrete compressive strength* is obtained by using standard deviation and average formulae. Reinforcing steel class shall be determined by visual inspection performed on the surfaces peeled. Characteristic yield strength of the steel in this class shall be taken as *existing steel yield strength*. In this investigation, the members in whose reinforcement steel corrosion is observed shall be marked on the plan and this situation shall be taken into consideration in determining the capacities of the members.

2.3.5. Medium Information Level in Reinforced Concrete Buildings

2.3.5.1. *Building Geometry:* In case structural working drawings of the building exist, the compliance of the existing geometry with the architectural plan and the structural working drawings is checked through measurements to be performed in the building. In case structural working drawings do not exist, the structural system and plan of the building is prepared by site survey. The information obtained shall include the location, span, height and dimensions of all reinforced concrete members. The building geometry information must include the details necessary for the accurate definition of the mass of building. Short columns in the building are marked on the floor plan. The relation of the building with neighbor buildings (separate, adjacent, joint exists / does not exist) is determined. Foundation system is observed through a control pit to be opened at corner of the building.

2.3.5.2. *Member Details:* In case structural working drawings or shop drawings do not exist, the requirements given in 2.3.4.2 apply, but amount of the columns and beams to be reinforcement control will be carried out shall not be less than 20 percent of the columns and 10 percent of the beams, and two each in every story at least. In case structural working drawings or shop drawings exist, the operations indicated in 2.3.4.2 for reinforcement control shall be applied on the reinforced concrete member in the same amount. In case of incompliance between the project and application, the "*reinforcement accuracy coefficient*" indicating the ratio of the existing reinforcement to the reinforcement in the project shall be determined for beams and columns separately. This coefficient, which is used in the determination of the members' capacities, may not be greater than one. Reinforcement ratios shall be determined by applying this coefficient to all other members whose reinforcement determination was not performed.

2.3.5.3. *Material Properties*: In each story, at least three concrete samples and minimum nine samples in the building as one for each 400 m^2 shall be taken from columns or shear walls in compliance with the conditions determined in TS – 10465 and the experiment shall be made. Values (average – standard deviation) obtained from the samples shall be taken as *existing concrete compressive strength* in calculation of the capacities of members. The distribution of the concrete compressive strength in the building shall be controlled with concrete hammer readings arranged with core sample experiment results or similar non-destructing examination tools calibrated with concrete sample test results. Reinforcement steel class shall be determined with visual inspection carried out on the surfaces that are peeled off. The design strength of the reinforcement of this class shall be taken as basis in the member capacity calculations. The members, on the reinforcements of which corrosion is observed, shall be marked on the plan and this condition shall be taken into consideration by an appropriate method in determining the capacities of the members.

2.3.6. Detailed Information Level in Reinforced Concrete Buildings

2.3.6.1. *Building Geometry:* Structural working drawings of the building exist. The compliance of the existing geometry with projects shall be determined through the measurements to be performed in the building. If there are important differences between projects and measurements, the projects are ignored and the building is investigated in compliance with average information level. Short columns and similar irregularities are marked on the floor plan. Relation with neighbor buildings (separate, adjacent, joint exists / does not exist) is determined. Building geometry information must include the details necessary for the accurate definition of the building mass. Foundation system is observed through a control pit to be opened at corner of the building.

2.3.6.2. *Member Details:* Structural working drawings of the building exist. In order to control the compliance of the reinforcement with the project, the operations indicated in 2.3.5.2 above are applied on the reinforced concrete members in the same amount. In case of incompliance between the project and the application, the "*reinforcement accuracy coefficient*" giving the ratio of the existing reinforcement to the reinforcement in the project is determined for beams and columns separately. This coefficient, which is used in the determination of the members' capacities, may not be greater than one. Reinforcement ratios shall be determined by applying this coefficient to all other members whose reinforcement determination was not performed.

2.3.6.3. *Material Properties:* In each story, at least three concrete samples and minimum nine samples in the building as one for each 200 m^2 shall be taken from columns or shear walls in compliance with the conditions determined in TS – 10465 and the experiment shall be made. Values (average – standard deviation) obtained from the samples shall be taken as *existing concrete compressive strength* in calculation of the capacities of members. The distribution of the concrete compressive strength in the building shall be controlled with concrete hammer readings arranged with core sample experiment results or similar non-destructing examination tools related with concrete sample test results. Reinforcement steel class shall be determined with visual inspection in the surfaces peeled off. Experiments shall be made for the steels in each class (S220, S420, etc.) by getting one sample of each; the yield and rupture strengths of the reinforcement shall be determined and their compliance with the project will be checked. If compliant, the *yield strength of the reinforcement* used in the project shall be taken as basis in determining the capacities of the members. In case of incompliance, at least three more samples shall be taken and tested and the minimum yield strength obtained shall be taken as basis in determining the capacities of the members. The members, on the reinforcements of which corrosion is observed, shall be marked on the plan and this condition shall be taken into consideration in determining the capacities of the members.

2.4. Target Performance Levels For Buildings

Three types of ground shaking are defined to be taken into consideration in performance based design and evaluation. These ground shakings are explained by having probability of exceedance in 50 years.

- *Service (Usage) Ground Shaking*: It is defined as ground shaking having a 50% probability of exceedance in 50 years. Return period of this ground shaking is approximately 72 years. The effect of this ground shaking(spectral acceleration) is half of the effect of ground shaking defined below.
- *Design Ground Shaking*: It is defined as ground shaking having a 10% probability of exceedance in 50 years. Return period of this ground shaking is approximately 475 years. This ground shaking is used in the Turkish Earthquake Codes 1998 and 2007.
- *The Biggest Ground Shaking*: It is defined as ground shaking having a 2% probability of exceedance in 50 years. Return period of this ground shaking is approximately 2475 years. The effect of this ground shaking is 1.5 times of the effect of design ground shaking.

Table 2.2. Target Performance Levels for Buildings under Different Ground Shaking Effects

Purpose of Occupancy and Type of Building	**Probability of exceedance**		
	50% in 50 years	**10% in 50 years**	**2% in 50 years**
The buildings to be utilized immediately after the earthquake: Hospitals, health facilities, fire fighting buildings, communication and power facilities, transportation stations, governorate, county and municipality administration buildings, first aid and emergency planning stations	**-**	**HK**	**CG**
Intensively and long-term occupied Buildings: Schools, dormitories, boarding houses, military barracks, prisons, museums, etc.	**-**	**HK**	**CG**
Intensively and short-term occupied Buildings: Cinema, theatre, concert saloons, cultural centers, sport facilities.	**HK**	**CG**	**-**
Buildings containing hazardous materials: Buildings containing or storing toxic, explosive and flammable materials, etc.	**-**	**HK**	**GÖ**
Other buildings: Buildings after than above defined buildings (Residential and office buildings, hotels, tourustic facilities, industrial structures, etc.)	**-**	**CG**	**-**

2.5. Determining the Seismic Performances of Buildings

Two fundamental parameters of performance based design and evaluation are; demand and capacity. Demand represents the ground shaking motion that acts to the structure and capacity represents the behavior of the structure under this earthquake effect. Demand also includes structural response (mass, stiffness, capacity etc.).

Linear and nonlinear analysis methods are used for determining the seismic performances of the existing buildings.

2.5.1. Linear Analysis Methods

2.5.1.1. Principals of Method

Performance limits of sections are obtained by determining the demand / capacity ratios of each section. Some of the main linear analysis methods given in the Turkish Earthquake Code 2007 are *Equivalent Seismic Load Method* and *Mode Superposition Method*.

2.5.1.2. Equivalent Seismic Load Method

Equivalent Seismic Load Method shall be applied to the buildings whose height from basement does not exceed 25 meter and total story number is not more than 8, torsional irregularity (which is defined for any of the two orthogonal earthquake directions as the ratio of the maximum storey drift at any storey to the average storey drift at the same storey in the same direction coefficient calculated without considering additional eccentricities) is $\eta_{bi} < 1.4$. Seismic load reduction factor $R_a = 1$ is taken while calculating total equivalent seismic load and right side of the equation (Eq. 3.10) is multiplied with λ coefficient. λ coefficient is taken 1.0 and 0.85 for two story buildings except of basement and for others respectively.

2.5.1.3. Mode Superposition Method

Seismic load reduction factor $R_a = 1$ is taken while calculating total equivalent seismic load in mode superposition. In determination of member internal forces and capacities in compliance with earthquake side and direction, obtained internal forces directions shall be taken as base in considered mode in this direction.

2.5.1.4. Demand/Capacity Ratio Limits of Reinforced Concrete Members

Demand/capacity ratios (r) are used in determining the member damage levels of ductile members while using linear analysis methods. Demand/capacity ratio limits are given in Table 2.3 – 2.5 for both ductile and brittle members.

Table 2.3. Demand/Capacity Ratios Defining Member Damage Levels for Reinforced Concrete Beams

Ductile Beams			**Performance Limit**		
$\frac{\rho - \rho'}{\rho_b}$	**Confinement**	$\frac{V}{b_w d f_{ctm}}$	**MN**	**GV**	**GÇ**
≤ 0.0	YES	≤ 0.65	3	7	10
≤ 0.0	YES	≥ 1.30	2. 5	5	8
≥ 0.5	YES	≤ 0.65	3	5	7
≥ 0.5	YES	≥ 1.30	2. 5	4	5
≤ 0.0	NO	≤ 0.65	2. 5	4	6
≤ 0.0	NO	≥ 1.30	2	3	5
≥ 0.5	NO	≤ 0.65	2	3	5
≥ 0.5	NO	≥ 1.30	1. 5	2. 5	4

Table 2.4. Demand/Capacity Ratios Defining Member Damage Levels for Reinforced Concrete Columns

Ductile Columns			**Performance Limit**		
$\frac{N_K}{A_c f_{cm}}$	**Confineme nt**	$\frac{V}{b_w d f_{ctm}}$	**MN**	**GV**	**GÇ**
≤ 0.1	YES	≤ 0.65	3	6	8
≤ 0.1	YES	≥ 1.30	2. 5	5	6
≥ 0.4 and ≤ 0.7	YES	≤ 0.65	2	4	6
≥ 0.4 and ≤ 0.7	YES	≥ 1.30	1.5	2.5	3.5
≤ 0.1	NO	≤ 0.65	2	3. 5	5
≤ 0.1	NO	≥ 1.30	1. 5	2. 5	3. 5
≥ 0.4 and ≤ 0.7	NO	≤ 0.65	1. 5	2	3
≥ 0.4 and ≤ 0.7	NO	≥ 1.30	1	1. 5	2
≥ 0.7			1	1	1

Table 2.5. Demand/Capacity Ratios Defining Damage Levels for Reinforced Concrete Shear Walls

Ductile Shear Walls	**Performance Limit**		
Confinement	**MN**	**GV**	**GÇ**
YES	3	6	8
NO	2	4	6

2.5.2. Nonlinear Analysis Methods

2.5.2.1. Principals of Method

The aim of the non–linear analysis methods to be used in determination of structural performances and retrofitting analysis of existing buildings under the effect of the seismic loads, is calculating the plastic rotation demands of ductile behavior for a given earthquake. Then, these demand values are compared with deformation capacities defined in this section. Evaluation of the structural performance is done for the level of member and building.

Some of the main nonlinear analysis methods given in the Turkish Earthquake Code 2007 are *Incremental Equivalent Seismic Load Method, Incremental Mode Superposition Method* and *Analysis Methods in Time Domain*.

- **Incremental Equivalent Seismic Load Method(Pushover Analysis Method):** This method shall be applied to the buildings whose total story number is not more than 8 and torsional irregularity coefficient calculated without considering additional eccentricities is $\eta_{bi} < 1.4$. Besides, in the direction of the considered earthquake , it is necessary that the ratio of effective mass corresponding to the first natural vibration mode calculated on bases of linear behavior to total building mass (except of the mass of the basement floor encircled by shear walls) to be minimum 0.70. In incremental equivalent seismic load method, nonlinear pushover analysis is performed under monotonically increasing equivalent earthquake load until performance point. Performance point is also named as target modal displacement demand. Displacement, plastic deformation, increment in internal forces and related cumulative values are determined at each pushover step. Once the system reaches its performance point, total base reaction and roof displacement values are determined. That is also the last step of pushover analysis.

- **Incremental Mode Superposition Method:** Modal capacity diagrams and modal displacement demands corresponding to all modes considered shall be obtained. On this base; internal force demands, plastic deformations (plastic rotations) and displacements occur on the structural systems shall be calculated.

- **Analysis Methods in Time Domain:** Analysis Method in Time Domain is step by step integration of the movement equation of the system by considering non–linear behavior of the structural system. The displacement, deformation and internal forces occur in the system in the duration of the analysis in each time increase and the maximum equivalent values of them with respect to the seismic demand are calculated.

2.5.2.2. Idealization of Plastic Behavior

In this specification, it is suggested to use "*elastic perfectly plastic hypothesis*" for nonlinear analysis. It is assumed that plastic deformations occur uniformly distributed within the plastic hinge length. In case of simple bending, *length of the plastic deformation region* called *plastic hinge length* (L_p) shall be taken as equal to half of member dimension in bending direction (h).

$$l_p = 0.5h \tag{2.1}$$

It is required that plastic hinges are located in the exact middle of the plastic deformation region theoretically. But in practical operations, following approximate idealizations can be allowed:

- Plastic hinges shall be located at sufficient distance from the column-beam connection region. But, it must be considered that plastic hinges can occur at spans of the beams due to vertical loads.
- In reinforced concrete shear walls, it can be allowed plastic hinges to be assigned in bottom ends of shear walls in each story. U, T, L or box typed shear walls, must be idealized as single shear wall sections. In the case of basement floors of the buildings are encircled by rigid shear walls, plastic hinges of these shear walls going towards the upper floors must be located by starting on basement.

- Yield surfaces of the reinforced concrete members can be modeled as yield lines and yield planes for two dimensional and three dimensional behavior conditions respectively.

Section stiffness for cracked sections shall be used for linear behavior of reinforced concrete members under the effect of bending before yielding. If more certain calculation is not performed, following values shall be used for section stiffness corresponding to cracked sections:

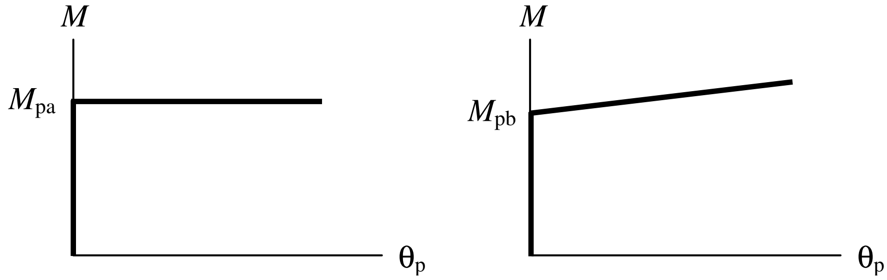
a) For beams : $0.40EI_0$

b) For columns and shear walls : if $\dfrac{N_d}{A_c f_{cm}} \leq 0.10$; $0.40EI_0$

if $\dfrac{N_d}{A_c f_{cm}} \geq 0.40$; $0.80EI_0$

Axial force N_D taking place in the equations above shall be calculated under the vertical loads. Linear interpolation can be made for intermediate values. The following idealizations shall be considered about internal force-plastic deformation equations to be used in pushover analysis model.

i. Strain hardening in internal force-plastic deformation equations strain hardening) can be ignored approximately **(Figure2.2a)**. In this case, in pushover steps after the strain hardening in the sections under the effect of simple or combined bending and axial load, in case of internal forces remain over yield surface, the condition to be perpendicular to the yield plane for vector of plastic deformation shall be considered.
ii. In case of strain hardening is considered **(Figure2.2b)**, in pushover steps after the strain hardening in the sections under the effect of simple or combined bending and axial load, the conditions that internal forces and vector of plastic deformation to be satisfied shall be defined according to a suitable strain hardening model obtained from literature.



(a) **(b)**

Figure 2.2. Bending Moment-Plastic Hinge Rotation Relations

2.5.2.3. Pushover Analysis by Using Incremental Equivalent Seismic Load Method

It is required to be effective mass calculated by considering first natural vibration mode of considered earthquake direction to total building mass shall not be less than 0.70 (Equation 2.3b, Equation 2.3c). In addition, number of stories shall not be more than eight excluding basement. Otherwise Incremental Equivalent Seismic Load Method can not be applied to the structural system.

In *Incremental Equivalent Seismic Load Method*, performance point of building is represented with base shear-roof displacement curve and modal capacity diagram. Roof displacement is the displacement calculated in each pushover step in x earthquake direction considered at center of mass at the top story of the building. Base shear force is the sum of equivalent earthquake loads in each step in x earthquake direction. Structural system is calculated under vertical loads and proportionally increasing earthquake loads to obtain pushover curve until the performance point is reached.

Modal transformation applied to pushover curve and modal capacity diagram having coordinates "modal displacement – modal acceleration" shall be obtained as expressed below:

a) modal acceleration, $a_1^{(i)}$ corresponding to first mode (in considered earthquake direction) in (i)' th pushover step is obtained as following way :

$$a_1^{(i)} = \frac{V_{x1}^{(i)}}{M_{x1}} \tag{2.2}$$

$V_{x1}^{(i)}$: base shear corresponding to first mode (in x earthquake direction) in (i)' th pushover step

M_{x1} : effective mass corresponding to first mode (in x earthquake direction) defined for linear elastic earthquake direction.

b) modal displacement, $d_1^{(i)}$ corresponding to first mode (in considered earthquake direction) in (i)' th pushover step is obtained as following way :

$$d_1^{(i)} = \frac{u_{xN1}^{(i)}}{\Phi_{xN1}\Gamma_{x1}} \tag{2.3}$$

Γ_{x1} : modal contribution coefficient corresponding to first mode in x direction

Φ_{xN1} : mode shape corresponding to N' th story in considered earthquake direction

Γ_{x1} is obtained as following way;

$$\Gamma_{x1} = \frac{L_{x1}}{M_1} \tag{2.3a}$$

M_1 : modal mass corresponding to first natural vibration mode

$$\sum_{n=1}^{Y} M_{xn} = \sum_{n=1}^{Y} \frac{L_{xn}^2}{M_n} \tag{2.3b}$$

$$\sum_{n=1}^{Y} M_{yn} = \sum_{n=1}^{Y} \frac{L_{yn}^2}{M_n} \tag{2.3c}$$

$$\begin{gathered} L_{xn} = \sum_{i=1}^{N} m_i \Phi_{xin}; L_{yn} = \sum_{i=1}^{N} m_i \Phi_{yin} \\ M_n = \sum_{i=1}^{N} \left(m_i \Phi_{xin}^2 + m_i \Phi_{yin}^2 + m_{\phi i} \Phi_{\phi in}^2 \right) \end{gathered} \tag{2.3d}$$

Modal capacity diagram obtained at the end of pushover analysis and elastic response spectrum are taken into consideration together and modal displacement demand of first mode is calculated. Modal displacement demand, $d_1^{(p)}$, is equal to nonlinear spectral displacement S_{di1}.

$$d_1^{(p)} = S_{di1} \tag{2.4}$$

Nonlinear spectral displacement S_{di1} is obtained by using linear elastic spectral displacement, S_{de1} **(Equal Displacement Rule)**. According to "*Equal Displacement Rule*", the maximum inelastic lateral displacement experienced by the structure behaving non-linearly will be equal to the maximum lateral displacement experienced by the structure behaving linear elastically.

$$S_{di1} = C_{R1} S_{de1} \tag{2.5}$$

Linear elastic spectral displacement S_{de1}, is obtained by using linear elastic spectral acceleration, S_{ae1} that is corresponding to the first mode of pushover analysis.

$$S_{de1} = \frac{S_{ae1}}{(\omega_1^{(1)})^2} \tag{2.6}$$

Spectral displacement ratio C_{R1}, in case of initial period $T_1^{(1)}$ is equal to or greater than T_B that is the characteristic period at acceleration spectrum ($T_1^{(1)} \geq T_B$ or $(\omega_1^{(1)})^2 \leq \omega_B^2$)

$$C_{R1} = 1 \tag{2.7}$$

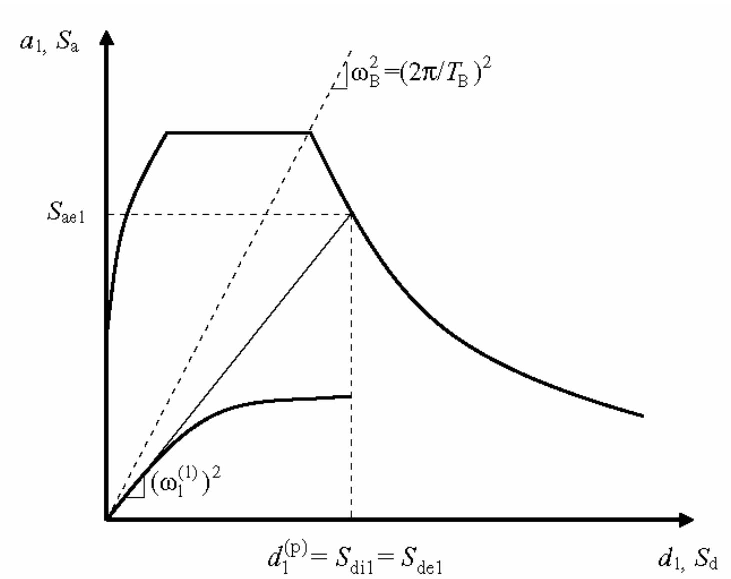


Figure 2.3. Determination of Performance Point ($T_1^{(1)} \geq T_B$) [Ref. 5]

Spectral displacement ratio C_{R1}, in case of initial period $T_1^{(1)}$ is less than T_B that is the characteristic period at acceleration spectrum ($T_1^{(1)} < T_B$ or $(\omega_1^{(1)})^2 > \omega_B^2$), is calculated by successive approximation method in following way.

a. Modal capacity diagram obtained at the end of pushover analysis is converted to a bi-linear diagram. In this diagram, the slope of the beginning line is taken as equivalent to value, $(\omega_1^{(1)})^2$ corresponding to the first mode the angle of line in first step (i=1) of pushover analysis ($T_1^{(1)} = 2\pi / \omega_1^{(1)}$), Figure 2.4.

b. In the first step of successive approximation method it is assumed that $C_{R1} = 1$ and coordinates of equivalent yield point is determined by using equivalent areas rule.

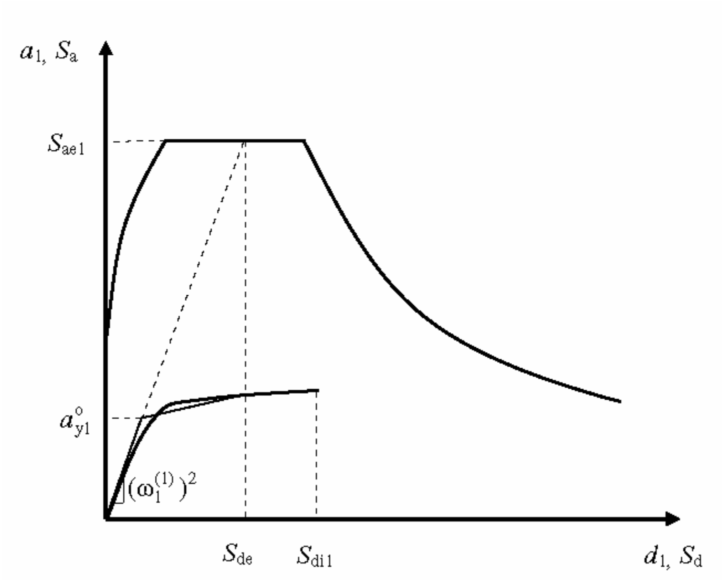


Figure 2.4. Determination of Performance Point ($T_1^{(1)} < T_B$) [Ref. 5]

Later nonlinear spectral displacement S_{di1} is calculated by using Equation (2.5). C_{R1} value is calculated by using Equation (2.8)

$$C_{R1} = \frac{1 + (R_{y1} - 1) T_B / T_1^{(1)}}{R_{y1}} \geq 1 \tag{2.8}$$

R_{y1} in the equation above represents the strength reduction factor corresponding to the first mode.

$$R_{y1} = \frac{S_{ae1}}{a_{y1}} \tag{2.9}$$

c. Coordinates of equivalent yield point is determined again by using equivalent areas rule. a_{y1}, R_{y1} and C_{R1} is calculated again. Successive approximation method is completed when the results of two adjacent steps are approximately same, Figure 2.5.

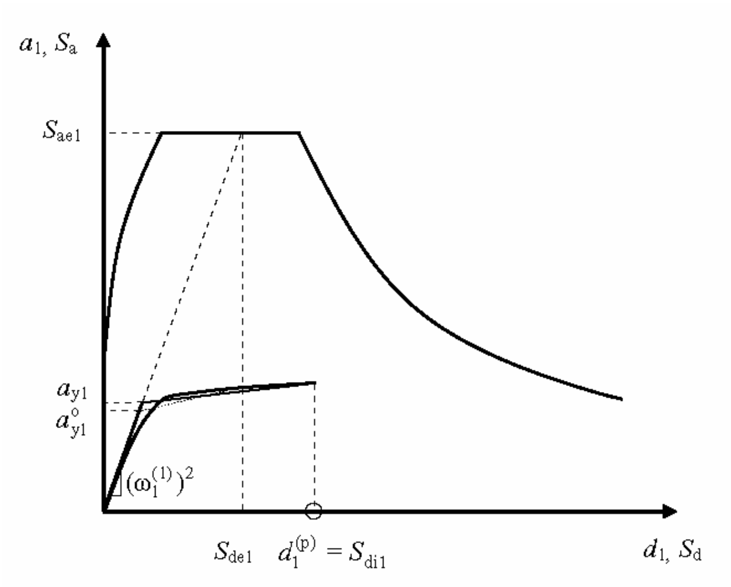


Figure 2.5. Determination of Performance Point ($T_1^{(1)} < T_B$) [Ref. 5]

Modal displacement demand, $d_1^{(p)}$ obtained by using Equation (2.4) is used in Equation (2.3) and roof displacement $u_{xN1}^{(p)}$ along the x earthquake direction is calculated.

$$u_{xN1}^{(p)} = \Phi_{xN1} \Gamma_{x1} d_1^{(p)} \tag{2.10}$$

2.5.2.4. Determination of Strain Demands

Plastic curvature demand in structural systems which are designed according to nonlinear methods is calculated depending on plastic rotation demand θ_p obtained at any section by using the equation below.

$$\phi_p = \frac{\theta_p}{L_p} \tag{2.11}$$

Bilinear bending moment-curvature relation obtained by using concrete and reinforcing steel models defines equivalent yield curvature ϕ_y. Total curvature demand is obtained by adding equivalent yield curvature to plastic curvature demand ϕ_p.

$$\phi_t = \phi_y + \phi_p \tag{2.12}$$

Stress, strain demands of concrete and steel are calculated according to moment-curvature analysis defined above. Seismic demands obtained based on strain demands of concrete and steel are compared with the strain limits given below and member damage levels are determined.

2.5.2.5. Member Damage Strain Capacities of Reinforced Concrete Members

In ductile reinforced concrete structural system members, strain limits depending on their damage levels are defined below.

a. For *Minimum Damage Limit* (MN), the maximum compressive strain demand on concrete (at the extreme fiber) corresponding to that total curvature demand are given in Equation (2.13).

$$(\varepsilon_{cu})_{MN} = 0.0035 \qquad ; \quad (\varepsilon_s)_{MN} = 0.010 \tag{2.13}$$

b. For *Visible Damage Limit* (GV), strain capacities of at the extreme layer of concrete and steel are given in Equation (2.14).

$$(\varepsilon_{cg})_{GV} = 0.0035 + 0.001(\rho_s / \rho_{sm}) \leq 0.0135 \qquad ; \quad (\varepsilon_s)_{GV} = 0.040 \tag{2.14}$$

c. For *Collapse Limit* (GÇ), strain capacities of at the extreme layer of concrete and steel are given in Equation (2.15).

$$(\varepsilon_{cg})_{GC} = 0.004 + 0.014(\rho_s / \rho_{sm}) \leq 0.018 \quad ; \quad (\varepsilon_s)_{GC} = 0.060 \tag{2.15}$$

ρ_s : Volumetric ratio of existing transverse reinforcement steel

ρ_{sm} : Volumetric ratio of transverse reinforcement that is required for design of a new building

In this study, it is assumed that structural members have adequate transverse reinforcement. So the ratio of (ρ_s / ρ_{sm}) is taken as 1.0 in calculations.

3. METHODOLOGY

Linear elastic *Equivalent Seismic Load Method* and nonlinear elastic *Incremental Equivalent Seismic Load Method (Pushover Analysis Method)* in the Turkish Earthquake Code 2007 are performed for determining the seismic performances of existing medium-rise reinforced concrete buildings with adequate transverse reinforcement in this study. Design steps of these methods are as follows;

3.1. Equivalent Seismic Load Method

Equivalent Seismic Load Method design steps performed in this study for determining the seismic performance of existing medium-rise buildings are as follows;

a) Structural system is analyzed under the vertical loads. Bending moments, shear forces and axial forces at all sections are obtained.
b) Bending moment capacities of all beam sections of the system are calculated. In determination of the bending moment capacities, existing compressive strength of the concrete and existing yield strength of reinforcement are used after multiplying with the information level coefficients. Equivalent rectangular compressive stress distribution is used in calculating the moment capacities of beams subject to flexure.

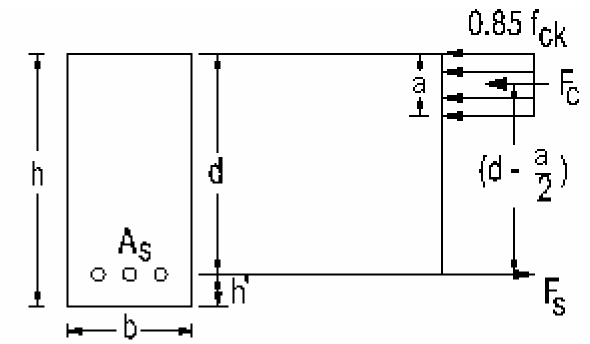


Figure 3.1. Equivalent Rectangular Compressive Stress Distribution

$$F_s = A_s f_{yk} \tag{3.1}$$

$$F_c = 0.85 f_{ck} ba \tag{3.2}$$

$\sum$ X=0 equation gives the result $$a = \frac{A_s f_{yk}}{0.85 f_{ck} b} \tag{3.3}$$

$\sum$ M=0 equation gives the result $$M_{cap} = A_s f_{yk} \left(d - \frac{a}{2} \right) \tag{3.4}$$

In the equations above;

F_s : tensile force at tension reinforcement

F_c : concrete compressive strength

M_{cap} : bending moment capacity of beam section

c) Bending moment capacities of all column sections of the system are calculated. In determination of the bending moment capacities, existing compressive strength of the concrete and existing yield strength of reinforcement are used after multiplying with the information level coefficients. Bending moment capacities of column sections that are subject to combined flexure and axial loads are first analyzed just for the vertical loads.

   Ultimate load design formulae [Ref. 4] can be used for determining the bending moment capacities of rectangular reinforced concrete columns subject to flexure. Characteristic strengths (f_{ck}, f_{yk}) of the materials shall be used at these formulae [Ref. 4].

$$n = \frac{N_d}{bhf_{ck}} \tag{3.5}$$

$$m = \frac{M_{cap}}{bh^2 f_{ck}} \tag{3.6}$$

$$\mu = \frac{A_s}{bh} \frac{f_{yk}}{f_{ck}} \tag{3.7}$$

$$A_s = k_1 k_2 \mu b h \frac{f_{ck}}{f_{yk}} \tag{3.8}$$

1) $n < 0.35$ condition;

$$m = \frac{\mu - 1.44n^2 + 1.29n}{2.49} \tag{3.9a}$$

2) $0.35 \leq n \leq 0.50$ condition;

$$m = \frac{\mu + 0.295}{2.54} \tag{3.9b}$$

3) $n > 0.50$ condition;

$$m = \frac{\mu - 0.305n^2 - 0.22n + 0.49}{2.54} \tag{3.9c}$$

b / h : dimensions of rectangular column

A_s : total longitudinal reinforcement area at section

k_1 / k_2 : coefficients related to the locations of the reinforcements at section and concrete cover coefficient

d) The difference between bending moment capacities of column, beam sections and bending moments calculated just under vertical loads represents the excessive bending moment capacities of the sections.

e) Seismic load reduction factor R_a =1 is taken while calculating total equivalent seismic load.

$$V_t = \frac{WA(T_1)}{R_a(T_1)} \tag{3.10}$$

The system is analyzed again for this total equivalent seismic load only. Bending moments, shear forces, axial forces are obtained at each critical section.

f) By considering the direction of the applied seismic load, demand/capacity ratios (r) are obtained by dividing bending moments calculated just only earthquake loads to excessive bending moment capacities of the section.
g) Obtained demand/capacity ratios (r) are compared with the demand/capacity ratios (r) related to damage levels of the members in the Turkish Earthquake Code 2007. Linear interpolation is performed for determining the intermediate values of (r) coefficients in code. Last, damage levels of each member are determined.
h) Story drifts of the structure obtained at the end of equivalent seismic load method are compared to story drift limitations given in Table 2.1.

3.2. Incremental Equivalent Seismic Load Method

Nonlinear Analysis Procedures mentioned in the Turkish Earthquake Code 2007 are *Incremental Equivalent Seismic Load Method (Pushover Analysis Method), Incremental Mode Superposition Method* and *Analysis Methods in Time Domain*. Pushover Analysis Method is used in this study. Design steps of this method are as follows;

a) Section stiffness for cracked sections are used:

- $0.40EI_0$ is taken for beams
- Axial forces of columns are calculated under vertical loads and section stiffness for cracked sections are obtained by using the equations below;

$$\text{if} \quad \frac{N_d}{A_c f_{cm}} \le 0.10 \qquad : \quad 0.40EI_0$$

$$\text{if} \quad \frac{N_d}{A_c f_{cm}} \ge 0.40 \qquad : \quad 0.80EI_0$$

Linear interpolation is performed for the intermediate values.

b) Probable plastic hinges are assigned at the ends of beams and columns.
c) Structural system model subject to vertical loads and incremental equivalent seismic loads is calculated until such a lateral displacement. Modal participating mass factors, modal participation factors and mode shapes of first mode are obtained at this step.

d) Pushover curve is constructed by obtaining base shear-roof displacement values at each step. The capacity curve is generally constructed to represent the first mode response of the structure based on the assumption that the fundamental mode of vibration is the predominant response of the structure.

e) Pushover curve is converted to a modal capacity diagram which has spectral displacement and spectral acceleration axis by using the equations below.

$$a_1^{(i)} = \frac{V_{x1}^{(i)}}{M_{x1}^{i}} \tag{3.11}$$

$$d_1^{(i)} = \frac{u_{xN1}^{(i)}}{\Phi_{xN1}\Gamma_{x1}} \tag{3.12}$$

f) Modal capacity diagram obtained at the end of pushover analysis and elastic response spectrum are taken into consideration together. Modal displacement demand of first mode is calculated. Modal displacement demand, $d_1^{(p)}$, is equal to nonlinear spectral displacement S_{di1}.

$$d_1^{(p)} = S_{di1} \tag{3.13}$$

g) Nonlinear spectral displacement, S_{di1}, is obtained by using linear elastic spectral displacement, S_{de1} by using the equation below;

$$S_{di1} = C_{R1} S_{de1} \tag{3.14}$$

h) Linear elastic spectral displacement, S_{de1}, is obtained by using linear elastic spectral acceleration, S_{ae1} that is corresponding to the first mode of pushover analysis by using the equation below;

$$S_{de1} = \frac{S_{ae1}}{(\omega_1^{(1)})^2} \tag{3.15}$$

i) Spectral displacement ratio C_{R1}, is determined with respect to the value of initial period, $T_1^{(1)}$ ($T_1^{(1)} = 2\pi / \omega_1^{(1)}$).

- If initial period, $T_1^{(1)}$ is equal to or greater than T_B that is the characteristic period at acceleration spectrum ($T_1^{(1)} \geq T_B$ or $(\omega_1^{(1)})^2 \leq \omega_B^2$), nonlinear elastic spectral displacement S_{di1} is equal to linear elastic spectral displacement S_{de1}. So;

$$C_{R1} = 1 \tag{3.16}$$

- If initial period, $T_1^{(1)}$ is less than T_B ($T_1^{(1)} < T_B$ or $(\omega_1^{(1)})^2 > \omega_B^2$), spectral displacement ratio C_{R1} is calculated by;

$$C_{R1} = \frac{1 + (R_{y1} - 1) T_B / T_1^{(1)}}{R_{y1}} \geq 1 \tag{3.17}$$

R_{y1} in the equation above represents the strength reduction factor corresponding to the first mode.

$$R_{y1} = \frac{S_{ae1}}{a_{y1}} \tag{3.18}$$

j) Roof lateral displacement demand due to the seismic load along X axis;

$$u_{xN1}^{(p)} = \Phi_{xN1} \Gamma_{x1} d_1^{(p)} \tag{3.19}$$

is calculated by using the equation above.

k) Structural system model is analyzed by using this roof displacement demand.

l) Plastic hinge rotations at all critical sections of the structural system are obtained at the end of the pushover analysis.

m) Plastic curvature demands of sections are obtained by dividing plastic hinge rotations to plastic hinge length. ($L_p = 0.5h$)

$$\phi_p = \frac{\theta_p}{L_p} \tag{3.20}$$

n) Equivalent yield curvature ϕ_y is calculated by using Priestley Formula, [Ref. 7].

$$\phi_y = \frac{2.10\varepsilon_{sy}}{h} \tag{3.21}$$

o) Total curvature demand is obtained by adding equivalent yield curvature to plastic curvature demand.

$$\phi_t = \phi_y + \phi_p \tag{3.22}$$

p) For beams; bending moment-curvature diagrams are constructed at each beam section by using concrete and steel models. Steel and concrete strain demands of all beam sections are determined from XTRACT software and these strain demands are compared with the limits given in TEC 2007 to evaluate the damage level of the member.

q) For columns; normal force-total curvature diagrams are constructed in XTRACT software for each column section by using concrete and steel models. Then total curvature demand calculated by using equation (3.22) and obtained normal force is located on the normal force-total curvature diagram to evaluate the damage level of the member. XTRACT software uses Mander Models for confined and unconfined concrete.

r) Story drifts of the structure obtained at the end of pushover analysis are compared to story drift limitations given in Table 2.1.

4. CASE STUDIES

This chapter of thesis includes numerical studies performed for determining the linear, nonlinear behavior and performance levels of reinforced concrete structures under the effect of lateral earthquake forces.

Four reinforced concrete structural system models having 4 and 6 story are analyzed in this part. These models are chosen to represent mid-rise buildings in Turkey. Models are designed according to the Earthquake Codes 1975 and 1998. Later seismic performances of models are determined by using linear and nonlinear analysis methods given in the Turkish Earthquake Code 2007.

4.1. Structural System Models

All structural system models (SSM) are planar moment resisting frames and have two bays with 6 m length. Their story heights are 3 m and distance between frames on plan are 5 m. (SSM-1), (SSM-2) are 4 story and (SSM-3), (SSM-4) are 6 story planar moment resisting frames. Beam and column labels that are valid for all structural system models are given in Figure 4.1a and 4.1b.

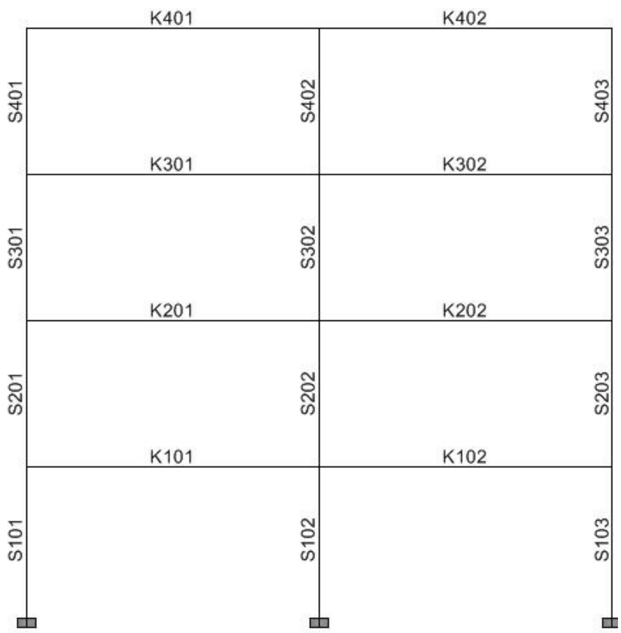


Figure 4.1a. Beam and Column Labels of Structural System Models (4 Story)

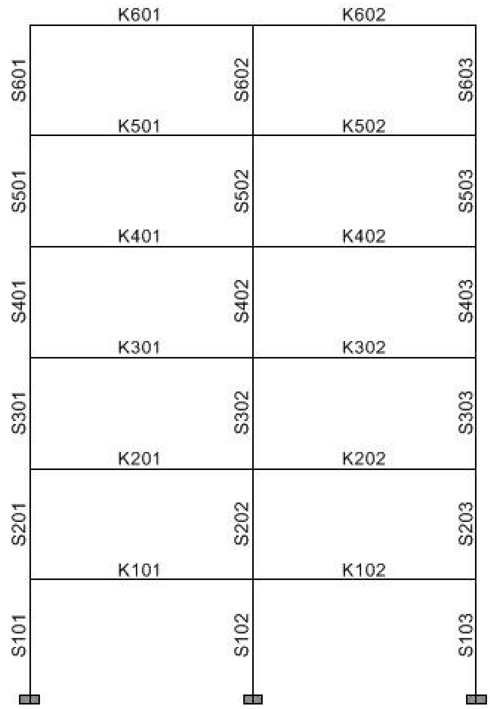


Figure 4.1b. Beam and Column Labels of Structural System Models (6 Story)

4.2. Sizing of Structural System Models

4.2.1. Material Properties

Material properties of concrete and reinforcing steel considered in sizing the models according to TS-500 Code [Ref. 6] are given below.

SSM-1	: Concrete Class C20, Reinforcing Steel Class S420
SSM-2	: Concrete Class C16, Reinforcing Steel Class S220
SSM-3	: Concrete Class C20, Reinforcing Steel Class S220
SSM-4	: Concrete Class C20, Reinforcing Steel Class S220

Table 4.1. Properties of Structural System Models

Structural System Models	EQ Code Used in Design	Material Properties	Story Number
SSM-1	TEC 1998	C20, S420	4 story
SSM-2	TEC 1975	C16, S220	4 story
SSM-3	TEC 1975	C20, S220	6 story
SSM-4	TEC 1998	C20, S220	6 story

4.2.2. Earthquake Characteristics

- Earthquake characteristics of SSM-1 and SSM-4 which are sized and detailed on the basis of the Turkish Earthquake Code 1998 [Ref. 9] are given below.

Building Importance Factor : $I = 1.0$

Live Load Participation Factor : $n = 0.30$

Effective Ground Acceleration Coefficient : $A_0 = 0.40$ (for SSM-1)

$A_0 = 0.30$ (for SSM-4)

Local Site Class : Z2

Spectrum Characteristic Periods : $T_A = 0.15s$, $T_B = 0.40s$

Structural Behavior Factor : $R = 8$

- Earthquake characteristics of SSM-2 and SSM-3 which are sized and detailed on the basis of the Turkish Earthquake Code 1975 [Ref. 10] are given below.

Seismic Zone Coefficient : $C_0 = 0.10$ (for SSM-2)

$C_0 = 0.08$ (for SSM-3)

Live Load Participation Factor : $n = 0.30$

Structure Type Coefficient : $K = 1.0$ (for SSM-2)

$K = 0.80$ (for SSM-3)

Spectrum Characteristic Period : $T_0 = 0.42s$

4.2.3. Load Analysis

Vertical loads determined according to TS 498 [Ref. 8] are given below. They are used in sizing all structural system models.

normal floors, $g = 5.00 \text{ kN/m}^2$

$q = 3.50 \text{ kN/m}^2$ (including infill walls)

roof floors, $g = 5.00 \text{ kN/m}^2$

$q = 2.00 \text{ kN/m}^2$ (including snow load)

4.2.4. Assumptions

Assumptions in sizing the structural system models are as follows:

- Sizing of structural system models are performed according to linear theory.
- Reinforcement design is done by using ultimate load design formulae.
- All joints of planar moment resisting frames are rigid and restraints of the systems are fixed.
- It is assumed that a lateral displacement perpendicular to planar frame is negligible.

4.2.5. Turkish Earthquake Codes Used in Sizing

The bases of earthquake codes used in sizing the structural system models are as follows:

a) Turkish Earthquake Code 1998

Equivalent seismic load method is used in dimensioning SSM-1 and SSM-4. Total Equivalent Seismic Load (base shear), $\mathbf{V_t}$, acting on the entire building in the earthquake direction is,

$$V_t = \frac{WA(T_1)}{R_a(T_1)} \geq 0.10 A_0 I W \qquad (4.1)$$

W : total building weight

T_1 : the first natural vibration period of the building

$A(T_1)$: spectral acceleration coefficient calculated for T_1 period

$R_a(T_1)$: seismic load reduction factor calculated for T_1 period

Total building weight is calculated by using the equation (4.2),

$$W = \sum_{i=1}^{N} w_i = \sum_{i=1}^{N} (g_i + nq_i) \tag{4.2}$$

w_i : story weights

g_i : total dead load at i'th story of building

q_i : total live load at i'th story of building

n : live load participation factor

N : Total number of stories of building from the foundation level (In buildings with rigid peripheral basement walls, total number of stories from the ground floor level)

$n = 0.30$ is taken according to Table 4.2.

Table 4.2. Live Load Participation Factor

Purpose of Occupancy of Building	**n**
Depot, warehouse, etc.	0.80
School, dormitory, sport facility, cinema, theatre, concert hall, carpark, restaurant, shop, etc.	0.60
Residence, office, hotel, hospital, etc.	0.30

Spectral Acceleration Coefficient is calculated by using the equation (4.3)

$$A(T_1) = A_0 I S(T_1) \tag{4.3}$$

A_0 : effective ground acceleration coefficient

I : building importance factor

$S(T_1)$: spectrum coefficient for T_1 period

$I = 1.0$ is chosen for all structural system models. $A_0 = 0.40$, $A_0 = 0.30$ are chosen for SSM-1 and SSM-4 respectively according to Table 4.3.

Table 4.3. Effective Ground Acceleration Coefficient

Seismic Zone	**A_o**
1	0.40
2	0.30
3	0.20
4	0.10

The spectrum coefficient for T_1 period **$S(T_1)$**, shall be determined by using equations (4.4), depending on the local site conditions and the first natural vibration period of the building, T_1 (Fig.4.2):

$$S(T) = 1 + 1.5\frac{T}{T_A} \qquad \text{for } (0 \le T \le T_A) \qquad (4.4a)$$

$$S(T) = 2.5 \qquad \text{for } (T_A < T \le T_B) \qquad (4.4b)$$

$$S(T) = 2.5\left(\frac{T_B}{T}\right)^{0.8} \qquad \text{for } (T > T_B) \qquad (4.4c)$$

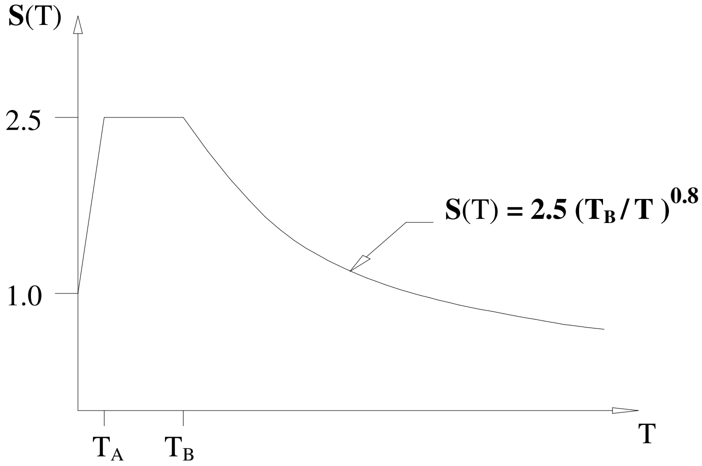


Figure 4.2. Response Spectrum

Spectrum characteristic periods, T_A and T_B, depending on local site classes were given in Section 4.2.2 for Z2 soil type.

Seismic Load Reduction Factor, **$R_a(T)$**, shall be determined by using equations (4.5) in terms of Structural Behavior Factor, **R**, and the natural vibration period **T**.

$$R_a(T) = 1.5 + (R-1.5)\frac{T}{T_A} \qquad \text{for } (0 \le T \le T_A) \qquad (4.5a)$$

$$R_a(T) = R \qquad \text{for } (T > T_A) \qquad (4.5b)$$

The first natural vibration periods (T_1) of the structural system models SSM-1 and SSM-4 are greater than T_A. So seismic load reduction factor,R, is equal to structural behavior factor, R. $R = 8$ is taken for buildings in which seismic loads are fully resisted by frames (systems of high ductility level) according to TEC 98.

Excluding **ΔF_N** , remaining part of the total equivalent seismic load shall be distributed to stories of the building (including **N**'th story) in accordance with equation (4.6).

$$F_i = (V_t - \Delta F_N)\frac{w_i H_i}{\sum_{j=1}^{N}(w_j H_j)} \qquad (4.6)$$

H_i : Height of i'th story of building measured from the top foundation level (In buildings with rigid peripheral basement walls, height of i'th story of building measured from the top of ground floor level) [m]

ΔF_N : additional equivalent seismic load (Fig. 4.3)

Additional equivalent seismic load, **ΔF_N** , acting at the **N**'th story (top) of the building shall be determined by using equation (4.7) depending on the first natural vibration period of the building, **T_1**.

$$\Delta F_N = 0.07 T_1 V_t \le 0.2 V_t \qquad (4.7)$$

Heights of all structural system models are less than 25 m. So **$\Delta F_N = 0$** is taken for **$H_N \le 25$ m** according to TEC 98.

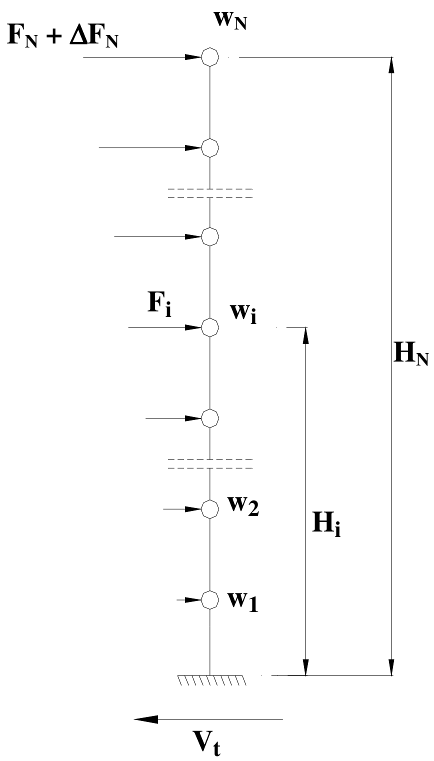


Figure 4.3. Determination of Equivalent Seismic Loads [Ref. 9]

b) Turkish Earthquake Code 1975

Equivalent seismic load method is used in sizing SSM-2 and SSM-3. Total Equivalent Seismic Load (base shear), acting on the entire building in the earthquake direction is,

$$F = CW \tag{4.8}$$

Earthquake coefficient, C, is calculated by using equation (4.9).

$$C = C_0 KSI \tag{4.9}$$

C_0 : seismic zone coefficient
K : structure type coefficient
S : spectrum coefficient
I : building importance factor

Seismic zone coefficients $C_0 = 0.10$, $C_0 = 0.08$ are chosen for SSM-2 and SSM-3 respectively according to Table 13.2 in TEC 75. Also $K = 1.0$, $K = 0.80$ are chosen or SSM-2 and SSM-3 respectively according to Table 13.5 in TEC 75.

Spectrum coefficient is calculated by using equation (4.10),

$$S = \frac{1}{\left|0.8 + T - T_0\right|} \leq 1 \tag{4.10}$$

T : first natural vibration period of structure (sn)

T_0 : spectrum characteristic period

Spectrum characteristic period, T_0 is equal to 0.42 sn for Z2 soil type. On the other hand spectrum coefficient, S takes 1.0 as maximum value. Earthquake coefficient, C, can not be less than $C_0 / 2$ any time.

Total building weight, W is calculated by equation (4.11),

$$W = \sum_{i=1}^{N} w_i = \sum_{i=1}^{N} \left(G_i + nP_i\right) \tag{4.11}$$

G_i : total dead load at i'th story of building

P_i : total live load at i'th story of building

Total equivalent seismic load shall be distributed to stories of the building (including **N**'th story) in accordance with equation (4.12),

$$F_i = (F - F_t) \frac{w_i H_i}{\sum_{j=1}^{N} (w_j H_j)} \tag{4.12}$$

F : total lateral load

w_i : story weights

H_i : height of i'th story of building from the top foundation level
(In buildings with rigid peripheral basement walls, height of i'th story of building measured from the top of ground floor level) [m]

F_t : additional equivalent seismic load

$$F_t = 0.004F\left(\frac{H}{D}\right)^2 \tag{4.13}$$

F_t should verify the conditions below,

- Additional equivalent seismic load, F_t can not be less than $0.15F$
- $F_t = 0$ for $H / D \leq 3$

4.2.6. Load Combinations Used in Sizing

Equivalent seismic loads determined for each floor are applied to the structural system models. Load combinations used in sizing are as follows;

Comb1	:	1.4DL+1.6LL
Comb2	:	1DL+1LL+1EX
Comb3	:	1DL+1LL-1EX
Comb4	:	0.9DL+1EX
Comb5	:	0.9DL-1EX

DL	:	dead load
LL	:	live load
EX	:	earthquake load in X direction

4.2.7. Analysis and Sizing

Seismic loads used in sizing the structural system models according to related Turkish Earthquake Codes are mentioned in previous sections. Structural system models are analyzed under dead loads, live loads and seismic loads. Internal forces obtained from Etabs Analysis Program and load combinations given in section 4.2.6 are used in sizing. Beam, column dimensions and longitudinal, shear reinforcements of sections are determined.

4.3. Performance Evaluation of Structural System Models

Considering the direction of the applied seismic load obtained in accordance with the equation (3.10), demand/capacity ratios (r) are obtained by dividing bending moments calculated just under unreduced earthquake seismic loads to excessive bending moment capacities of the section. Obtained demand/capacity ratios (r) are compared with the demand/capacity ratios (r) related to damage levels of the members in the Turkish Earthquake Code 2007. Linear interpolation is performed for determining the interval values of (r) coefficients in code. Last, damage levels of each member are determined.

Structural system models subject to vertical loads and proportionally increased equivalent seismic loads are calculated until such a lateral displacement by using nonlinear module of Etabs (Extended 3d Analysis of Building Systems). Pushover curve is constructed by obtaining base shear-roof displacement values at each step. Pushover curve is converted to a modal capacity diagram which has spectral displacement and spectral acceleration axis (S_a-S_d). Seismic performance of the structure is determined by using the performance limits obtained for each beam and column sections according to nonlinear analysis method of the Turkish Earthquake Code 2007.

4.4. Detailed Study for SSM-1

Detailed study of sizing the SSM-1 according to the Turkish Earthquake Code 1998 is performed in this section. Then linear and nonlinear analysis methods of the Turkish Earthquake Code 2007 are performed for SSM-1 and seismic performance of model is obtained and evaluated.

4.4.1. Sizing of SSM-1

Concrete class and reinforcing steel class are chosen as C20 (compressive strength of concrete $f_{ck} = 20MPa$) and S420 (yield strength $f_{yk} = 420MPa$) respectively. It is assumed that building is located on first seismic zone and Z2 soil type. Confinement effect is taken into consideration for beams and columns of structural system.

Equivalent Seismic Load Method given in the Turkish Earthquake Code 1998 is used in seismic calculations of the structural system model.

$T_x = 0.5769sn$ (first natural vibration method)

Z2 soil type; $T_A = 0.15sn$, $T_B = 0.40sn$

Spectrum Coefficient for T_1 period,

$S(T_1) = 2.5\left(\dfrac{T_B}{T}\right)^{0.8}$ for $T_X > T_B$. So,

$$S(T_1) = 2.5\left(\frac{0.40}{0.5769}\right)^{0.8}$$

$S(T_1) = 1.865$ is obtained.

Spectral Acceleration Coefficient for T_1 period,

$$A(T_1) = A_0 I S(T_1)$$

$$A_0 = 0.40$$

$$I = 1$$

$$R_a(T_1) = 8$$

$$A(T_1) = 0.40*1*1.865$$

$$A(T_1) = 0.746$$

Total building weight W is,

$$W_{G+0.3Q} = 1758.0kN$$

$$V_t = \frac{WA(T_1)}{R_a(T_1)}$$

$V_t = \dfrac{1758*0.746}{8} = 163.94kN$ is obtained.

$F_i = (V_t - \Delta F_N)\dfrac{w_i H_i}{\sum_{j=1}^{N}(w_j H_j)}$; $\Delta F_N = 0$ is taken for $H_N \leq 25.0m$

Total equivalent seismic load that will be distributed to stories of the building is given in Table (4.4).

Table 4.4. Determination of Seismic Loads Distributed to Stories for SSM-1 (kN)

Story	**H_i(m)**	**W_i(kN)**	**W_i*H_i**	**W_i*H_i/ΣW_i*H_i**	**F_i**
4	12.0	417.00	5004.00	0.384	62.928
3	9.0	444.00	3996.00	0.307	50.252
2	6.0	448.50	2691.00	0.206	33.841
1	3.0	448.50	1345.50	0.103	16.920
		1758.00	13036.50	1.000	163.940
		ΣW_i	**ΣW_i*H_i**	**Σ(W_i*H_i/ΣW_i*H_i)**	**ΣF_i**

Equivalent seismic loads acting on the model are shown in Figure 4.4.

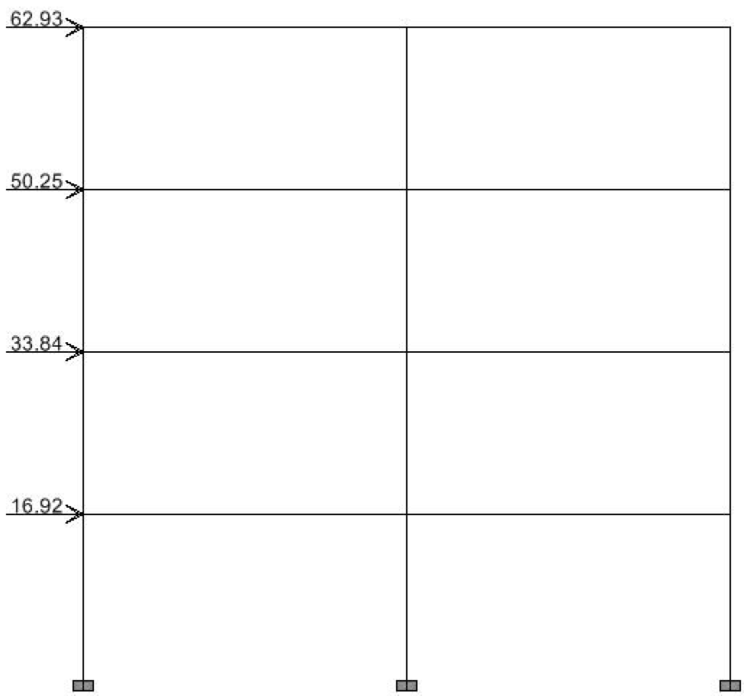


Figure 4.4. Seismic Loads Acting on SSM-1 (kN)

Dead loads acting on the model are shown in Figure 4.5.

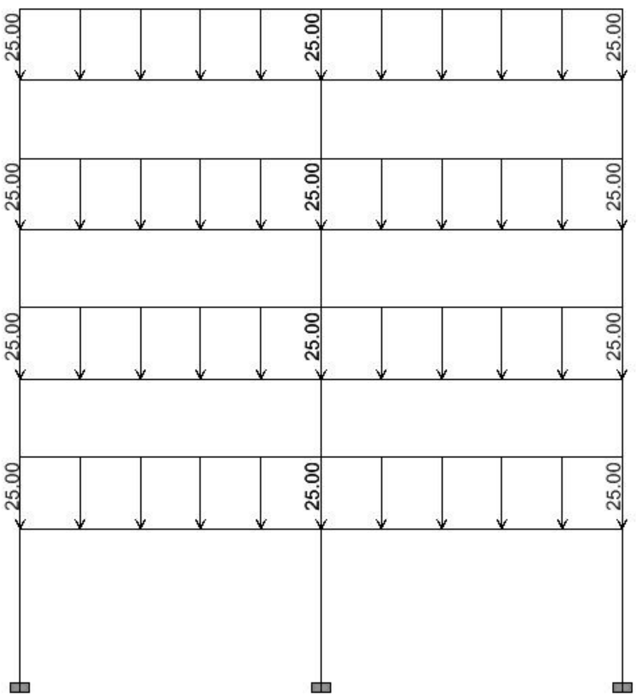


Figure 4.5. Dead Loads Acting on SSM-1 (kN/m)

Live loads acting on the model are shown in Figure 4.6.

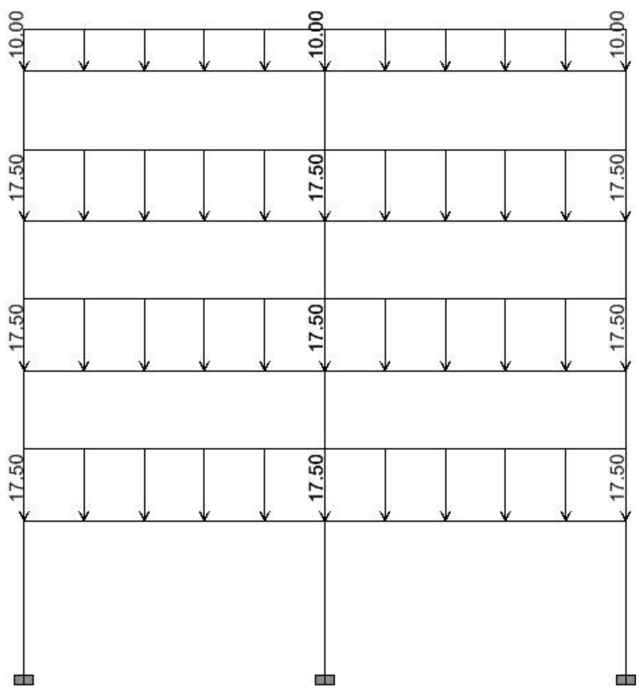


Figure 4.6. Live Loads Acting on SSM-1 (kN/m)

Structural system model is analyzed under load cases described above and sized and detailed according to the related load combinations. Dimensions and longitudinal reinforcements of the sections are given in Table 4.5 and Table 4.6.

Table 4.5. Column Dimensions and Longitudinal Reinforcements for SSM-1

Column	b(cm)	h(cm)	Reinforcement
S101	30	40	8Φ14
S102	30	60	12Φ14
S103	30	40	8Φ14
S201	30	40	8Φ14
S202	30	60	12Φ14
S203	30	40	8Φ14
S301	30	40	8Φ14
S302	30	40	8Φ14
S303	30	40	8Φ14
S401	30	40	8Φ14
S402	30	40	8Φ14
S403	30	40	8Φ14

Table 4.6. Beam Dimensions and Longitudinal Reinforcements for SSM-1

Beam	b/h (cm)	Bottom Reinf.	Top Reinf.	Left Top Reinf.	Right Top Reinf.
K101	30/60	6Φ12	2Φ16	1Φ22	2Φ22
K102	30/60	6Φ12	2Φ16	2Φ22	1Φ22
K201	30/60	6Φ12	3Φ12	2Φ18	2Φ22
K202	30/60	6Φ12	3Φ12	2Φ22	2Φ18
K301	30/60	6Φ12	2Φ14	2Φ18	2Φ20
K302	30/60	6Φ12	2Φ14	2Φ20	2Φ18
K401	30/60	6Φ12	2Φ14	1Φ14	2Φ18
K402	30/60	6Φ12	2Φ14	2Φ18	1Φ14

4.4.2. Determining the Seismic Performance of SSM-1 by Using Linear Method

Design steps performed in determining the seismic performance of sized structural system model by linear method in the Turkish Earthquake Code 2007 are as follows;

a) Structural system is calculated under the vertical loads and bending moments at all sections are obtained Figure 4.7.

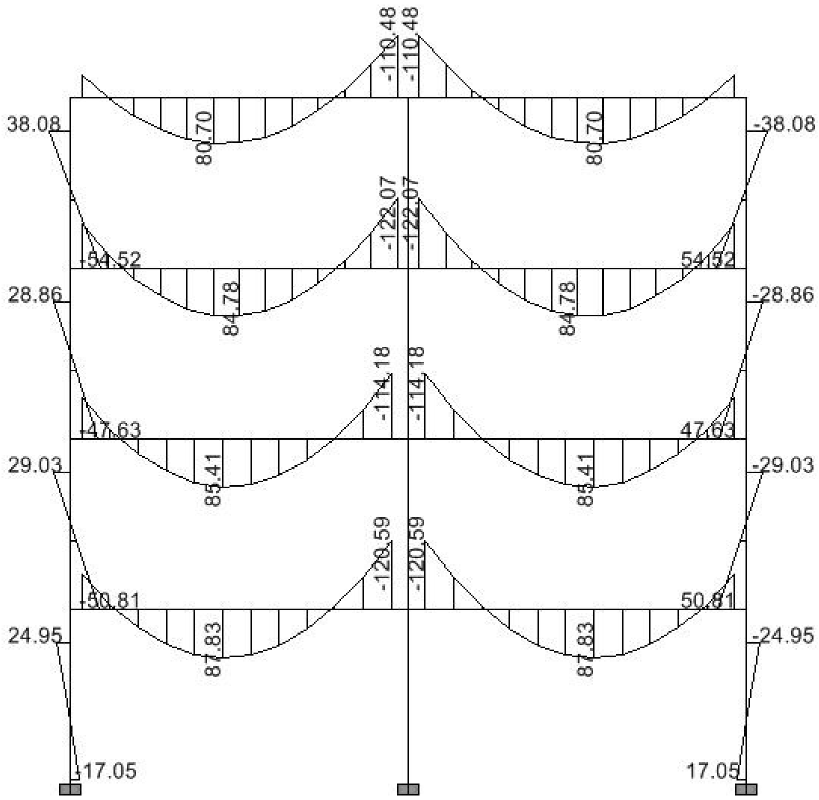


Figure 4.7. M_{G+Q} Diagram of SSM-1 (kNm)

b) Bending moment capacities of all beam sections of the system are calculated by using equation (3.1)-equation (3.4), (Table 4.7).

Table 4.7. Bending Moment Capacities of Beams for SSM-1(kNm)

Beam Edges	**Mcap$^-$**	**Mcap$^+$**
K101 left	174.912	152.938
K101 right	252.411	152.938
K102 left	252.411	152.938
K102 right	174.912	152.938
K201 left	188.744	152.938
K201 right	239.952	152.938
K202 left	239.952	152.938
K202 right	188.744	152.938
K301 left	182.717	152.938
K301 right	206.893	152.938
K302 left	206.893	152.938
K302 right	182.717	152.938
K401 left	105.752	152.938
K401 right	182.172	152.938
K402 left	182.172	152.938
K402 right	105.752	152.938

c) Bending moment capacities of all column sections of the system are calculated by using equation (3.5)-equation (3.9c), (Table 4.8).

d) The difference between bending moment capacities of column, beam sections and bending moments calculated just under vertical loads represents excessive bending moment capacities of the sections.

e) Seismic load reduction factor R_a =1 is taken and total equivalent seismic load is calculated again by using equation (3.10), Table 4.9.

$V_t = \dfrac{1758*0.746}{1} = 1311.468kN$ is obtained.

$$F_i = (V_t - \Delta F_N)\frac{w_i H_i}{\sum_{j=1}^{N}(w_j H_j)} \quad ; \qquad \Delta F_N = 0 \text{ is taken for } H_N \leq 25.0m$$

Table 4.8. Bending Moment Capacities of Columns for SSM-1 (kNm)

Column Edges	m	n	m	Mu
S101 top	0.2535	0.2238	0.1888	266.694
S101 bottom	0.2535	0.2268	0.1896	267.800
S102 top	0.2535	0.3412	0.2113	298.446
S102 bottom	0.2535	0.3443	0.2116	298.971
S103 top	0.2535	0.2238	0.1888	266.694
S103 bottom	0.2535	0.2268	0.1896	267.800
S201 top	0.2535	0.1655	0.1717	242.606
S201 bottom	0.2535	0.1686	0.1727	244.021
S202 top	0.2535	0.2501	0.1952	275.781
S202 bottom	0.2535	0.2532	0.1959	276.761
S203 top	0.2535	0.1655	0.1717	242.606
S203 bottom	0.2535	0.1686	0.1727	244.021
S301 top	0.2535	0.1061	0.1503	212.309
S301 bottom	0.2535	0.1092	0.1515	214.023
S302 top	0.2535	0.2425	0.1935	273.288
S302 bottom	0.2535	0.2456	0.1942	274.301
S303 top	0.2535	0.1061	0.1503	212.309
S303 bottom	0.2535	0.1092	0.1515	214.023
S401 top	0.2535	0.0458	0.1244	175.669
S401 bottom	0.2535	0.0489	0.1258	177.660
S402 top	0.2535	0.112	0.1526	215.585
S402 bottom	0.2535	0.1151	0.1538	217.246
S403 top	0.2535	0.0458	0.1244	175.669
S403 bottom	0.2535	0.0489	0.1258	177.660

Table 4.9. Determination of Seismic Loads Distributed to Stories for SSM-1(kN)

Story	H_i(m)	W_i(kN)	W_i*H_i	$W_i*H_i/\Sigma W_i*H_i$	F_i(kN)
4	12.0	417.00	5004.00	0.384	503.401
3	9.0	444.00	3996.00	0.307	401.996
2	6.0	448.50	2691.00	0.206	270.714
1	3.0	448.50	1345.50	0.103	135.357
		1758.00	13036.50	1.000	1311.468
		ΣW_i	ΣW_i*H_i	$\Sigma(W_i*H_i/\Sigma W_i*H_i)$	ΣF_i

f) The system is calculated again under these earthquake loads only and bending moments at all sections are obtained, Figure 4.8.

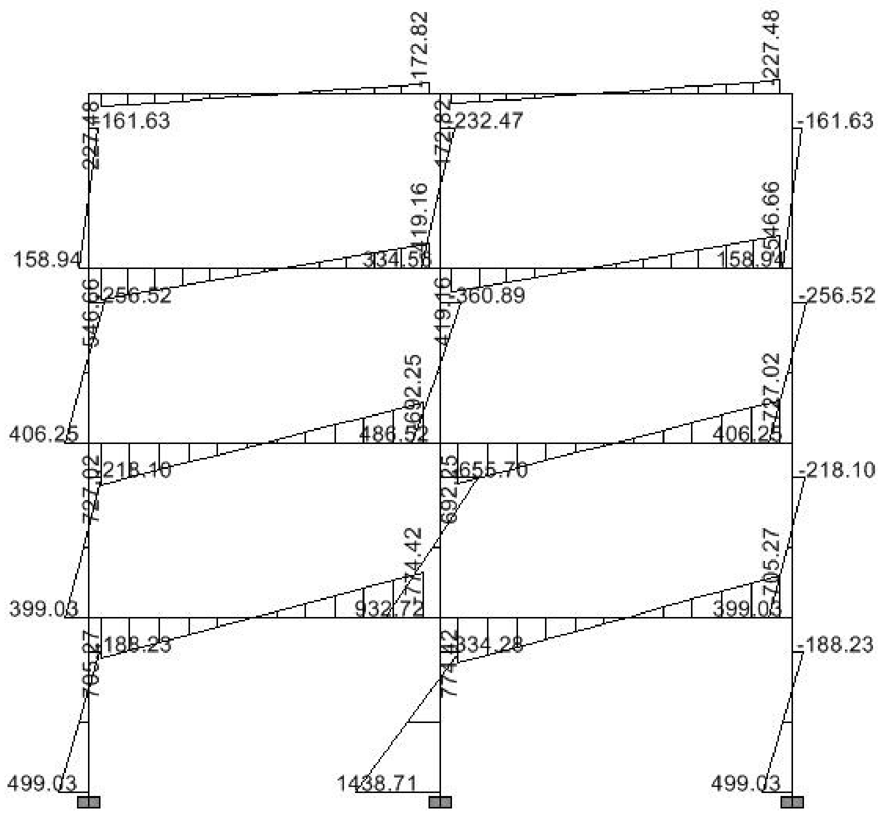


Figure 4.8. M_E Diagram of SSM-1 (kNm)

g) By considering the direction of the applied seismic load, demand/capacity ratios (r) are obtained by dividing bending moments calculated only under earthquake seismic loads to excessive bending moment capacities of the section, Table 4.10 and Table 4.11.

h) Obtained demand/capacity ratios (r) are compared with the demand/capacity ratios (r) related to damage levels of the members in the Turkish Earthquake Code 2007. Linear interpolation is performed for determining the interval values of (r) coefficients in code, Table 4.12 and Table 4.13.

Table 4.10. Demand/Capacity Ratios (r) of Beams for SSM-1(kN)

Beam Edges	**M_{G+Q}**	**M_E**	**M_{cap}^-**	**M_{cap}^+**	**r_{top}**	**r_{bottom}**
K101 left	-61.606	705.535	174.912	152.938	6.227	3.288
K101 right	-120.663	-774.696	252.411	152.938	5.876	2.832
K102 left	-120.663	774.696	252.411	152.938	5.876	2.832
K102 right	-61.606	-705.535	174.912	152.938	6.227	3.288
K201 left	-71.417	727.313	188.744	152.938	6.198	3.241
K201 right	-114.188	-692.488	239.952	152.938	5.506	2.592
K202 left	-114.188	692.488	239.952	152.938	5.506	2.592
K202 right	-71.417	-727.313	188.744	152.938	6.198	3.241
K301 left	-76.910	546.809	182.717	152.938	5.193	2.379
K301 right	-122.135	-419.279	206.893	152.938	4.943	1.524
K302 left	-122.135	419.279	206.893	152.938	4.943	1.524
K302 right	-76.910	-546.809	182.717	152.938	5.193	2.379
K401 left	-40.896	227.592	105.752	152.938	3.503	1.174
K401 right	-110.558	-172.852	182.172	152.938	2.412	0.656
K402 left	-110.558	172.852	182.172	152.938	2.412	0.656
K402 right	-40.896	-227.592	105.752	152.938	3.503	1.174

Table 4.11. Demand/Capacity Ratios (r) of Columns for SSM-1(kN)

Column Edges	**M_{G+Q}**	**M_E**	**M_u**	**r**
S101 top	24.917	-188.254	266.694	0.778
S101 bottom	-17.069	499.231	267.800	1.990
S102 top	0.000	-334.423	298.446	1.120
S102 bottom	0.000	1439.225	298.971	4.814
S103 top	-24.917	-188.254	266.694	0.778
S103 bottom	17.069	499.231	267.800	1.990
S201 top	29.038	-218.174	242.606	1.021
S201 bottom	-50.816	399.169	244.021	2.066
S202 top	0.000	-655.896	275.781	2.378
S202 bottom	0.000	933.029	276.761	3.371
S203 top	-29.038	-218.174	242.606	1.021
S203 bottom	50.816	399.169	244.021	2.066
S301 top	28.841	-256.629	212.309	1.398
S301 bottom	-47.676	406.428	214.023	2.442
S302 top	0.000	-361.008	273.288	1.321
S302 bottom	0.000	486.674	274.301	1.774
S303 top	-28.841	-256.629	212.309	1.398
S303 bottom	47.676	406.428	214.023	2.442
S401 top	38.062	-161.669	175.669	1.175
S401 bottom	-16.481	159.020	177.660	1.291
S402 top	0.000	-232.595	215.585	1.078
S402 bottom	0.000	334.717	217.246	1.540
S403 top	-38.062	-161.669	175.669	1.175
S403 bottom	16.481	159.020	177.660	1.291

Table 4.12. Member Damage Levels of Beams for SSM-1

Beam Edges	$(\rho-\rho')/\rho_b$	Confinement	$V/(b_w.d.f_{ctk})$	r_{top}	r_{MN}	r_{GV}	r_{GC}	Damage Region	r_{bottom}	r_{MN}	r_{GV}	r_{GC}	Damage Region
K101 left	0.0170	YES	0.5977	6.227	3.000	6.993	9.990	VDR	3.288	3.000	7.000	10.000	VDR
K101 right	0.0079	YES	0.6862	5.876	2.972	6.858	9.841	VDR	2.832	3.000	7.000	10.000	MDR
K102 left	0.0079	YES	0.6862	5.876	2.972	6.858	9.841	VDR	2.832	3.000	7.000	10.000	MDR
K102 right	0.0017	YES	0.5977	6.227	3.000	6.993	9.990	VDR	3.288	3.000	7.000	10.000	VDR
K201 left	0.0028	YES	0.6027	6.198	3.000	6.989	9.983	VDR	3.241	3.000	7.000	10.000	VDR
K201 right	0.0069	YES	0.6793	5.506	2.978	6.883	9.869	VDR	2.592	3.000	7.000	10.000	MDR
K202 left	0.0069	YES	0.6793	5.506	2.978	6.883	9.869	VDR	2.592	3.000	7.000	10.000	MDR
K202 right	0.0028	YES	0.6027	6.198	3.000	6.989	9.983	VDR	3.241	3.000	7.000	10.000	VDR
K301 left	0.0023	YES	0.5793	5.193	3.000	6.991	9.986	VDR	2.379	3.000	7.000	10.000	MDR
K301 right	0.0042	YES	0.6449	4.943	3.000	6.983	9.975	VDR	1.524	3.000	7.000	10.000	MDR
K302 left	0.0042	YES	0.6449	4.943	3.000	6.983	9.975	VDR	1.524	3.000	7.000	10.000	MDR
K302 right	0.0023	YES	0.5793	5.193	3.000	6.991	9.986	VDR	2.379	3.000	7.000	10.000	MDR
K401 left	-0.0036	YES	0.4381	3.503	3.000	7.000	10.000	VDR	1.174	3.000	6.990	9.978	MDR
K401 right	0.0023	YES	0.5642	2.412	3.000	6.991	9.986	MDR	0.656	3.000	7.000	10.000	MDR
K402 left	0.0230	YES	0.5642	2.412	3.000	6.991	9.986	MDR	0.656	3.000	7.000	10.000	MDR
K402 right	-0.0036	YES	0.4381	3.503	3.000	7.000	10.000	VDR	1.174	3.000	6.990	9.978	MDR

Table 4.13. Member Damage Levels of Columns for SSM-1

Column Edges	$N/A_c.f_{ck}$	Confinement	$V/(b_w.d.f_{ctk})$	r	r_{MN}	r_{GV}	r_{GC}	Damage Region
S101 top	0.1038	YES	1.0894	0.779	2.433	4.632	6.190	MDR
S101 bottom	0.1008	YES	1.0894	0.991	2.664	5.328	7.328	MDR
S102 top	0.3412	YES	0.8811	1.120	1.887	3.370	5.419	MDR
S102 bottom	0.3443	YES	0.8811	4.814	1.852	3.705	5.705	SDR
S103 top	0.5513	YES	1.0894	0.779	2.000	3.324	5.324	MDR
S103 bottom	0.5544	YES	1.0894	0.991	2.000	4.000	6.000	MDR
S201 top	0.0478	YES	0.8998	1.022	2.808	5.616	7.231	MDR
S201 bottom	0.0447	YES	0.8998	2.066	3.000	6.000	8.000	MDR
S202 top	0.2501	YES	0.8149	2.378	2.145	4.079	6.037	VDR
S202 bottom	0.2532	YES	0.8149	3.371	2.156	4.312	6.312	VDR
S203 top	0.3788	YES	0.8998	1.022	1.788	3.090	5.191	MDR
S203 bottom	0.3819	YES	0.8998	2.066	1.727	3.454	5.454	VDR
S301 top	0.0025	YES	0.7738	1.399	2.905	5.809	7.619	MDR
S301 bottom	0.0055	YES	0.7738	2.443	3.000	6.000	8.000	MDR
S302 top	0.2425	YES	1.2114	1.321	2.109	3.519	5.354	MDR
S302 bottom	0.2456	YES	1.2114	1.774	2.181	4.363	6.363	MDR
S303 top	0.2098	YES	0.7738	1.399	2.272	4.411	6.354	MDR
S303 bottom	0.2129	YES	0.7738	2.443	2.290	4.581	6.581	VDR
S401 top	0.0155	YES	0.5767	1.175	3.000	6.000	8.000	MDR
S401 bottom	0.0185	YES	0.5767	1.291	3.000	6.000	8.000	MDR
S402 top	0.112	YES	0.9575	1.079	2.478	4.780	6.484	MDR
S402 bottom	0.1151	YES	0.9575	1.541	2.616	5.233	7.233	MDR
S403 top	0.0762	YES	0.5767	1.175	3.000	6.000	8.000	MDR
S403 bottom	0.0793	YES	0.5767	1.291	3.000	6.000	8.000	MDR

i) Story drifts of the structure obtained at the end of equivalent seismic load method when R=1 and results are given in Table 4.14.

Table 4.14. Story Drifts for SSM-1

Story	Relative $(\delta_i)_{max}$(m)	h_i(m)	$(\delta_i)_{max}/h_i$
Story1	0.0221	3.0	0.00736
Story2	0.0289	3.0	0.00963
Story3	0.0304	3.0	0.01013
Story4	0.0179	3.0	0.00596

4.4.3. Determining the Seismic Performance of SSM-1 by Using Nonlinear Method

Incremental Equivalent Seismic Load Method (Pushover Analysis Method) given in the Turkish Earthquake Code 2007 is used in determining the seismic performance of SSM-1. Design steps are as follows;

a) Structural system model is calculated under vertical loads (G+Q) and column normal forces are obtained. These values are divided by concrete area at section and existing concrete compressive strength. Results are compared with the limits defined in section 3.2. Section stiffness (for cracked section) for each column is found, Table 4.15. Section stiffness (for cracked section) for beams is taken as $0.40EI_0$, Table 4.16.

Table 4.15. Column Section Stiffness for SSM-1

Column	b(cm)	h(cm)	N_d(kgf)	$N_d/(A_c*f_{ck})$	EI_0
S101	30	40	54430.87	0.2267953	0.569
S102	30	60	123939.47	0.3442763	0.726
S103	30	40	54430.87	0.2267953	0.569
S201	30	40	40468.07	0.168617	0.491
S202	30	60	91140.98	0.2531694	0.604
S203	30	40	40468.07	0.168617	0.491
S301	30	40	26205.18	0.1091883	0.412
S302	30	40	58942.64	0.2455943	0.594
S303	30	40	26205.18	0.1091883	0.412
S401	30	40	11732.70	0.0488863	0.400
S402	30	40	27622.39	0.1150933	0.420
S403	30	40	11732.70	0.0488863	0.400

Table 4.16. Beam Section Stiffness for SSM-1

Beam	b(cm)	h(cm)	EI_0
K101	30	60	0.400
K102	30	60	0.400
K201	30	60	0.400
K202	30	60	0.400
K301	30	60	0.400
K302	30	60	0.400
K401	30	60	0.400
K402	30	60	0.400

b) Probable plastic hinges are assigned at the ends of beams and columns. Plastic hinge labels are shown in Figure 4.9.

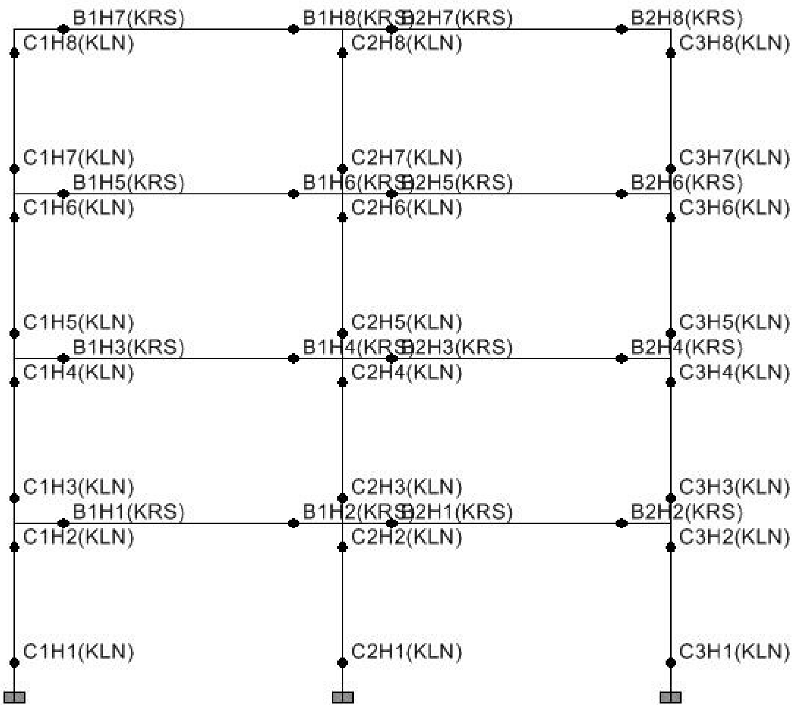


Figure 4.9. Probable Plastic Hinge Labels for SSM-1

c) Structural system model subject to vertical loads and proportionally increased equivalent seismic loads is calculated until such a lateral displacement. Modal participating mass factors, modal participation factors and mode shapes of first mode are obtained at this step, Table 4.17 - Table 4.19.

Table 4.17. Modal Participating Mass Factors(%) for SSM-1

Mode	Period	UX
1	0.827	79.608
2	0.286	12.700
3	0.171	3.858
4	0.111	3.833

Table 4.18. Modal Participation Factors(%) for SSM-1

Mode	Period	UX
1	0.827	-11.837
2	0.286	-4.728
3	0.171	-2.606
4	0.111	-2.597

Table 4.19. Mode Shapes for SSM-1

Story	Mode	UX
Story1	1	0.0206
Story2	1	0.0525
Story3	1	0.0876
Story4	1	0.1092

d) Base shear-roof displacement values at each step are given in Table 4.20. Pushover curve constructed by using these values is also given in Figure 4.10.

Table 4.20. Base Shear-Roof Displacement Values for SSM-1

Step	Displacement(m)	Base Shear(kN)
0	0.000	0.000
1	0.023	144.173
2	0.029	174.752
3	0.055	244.891
4	0.082	288.100
5	0.090	293.037
6	0.159	305.181
7	0.175	307.737
8	0.223	307.858

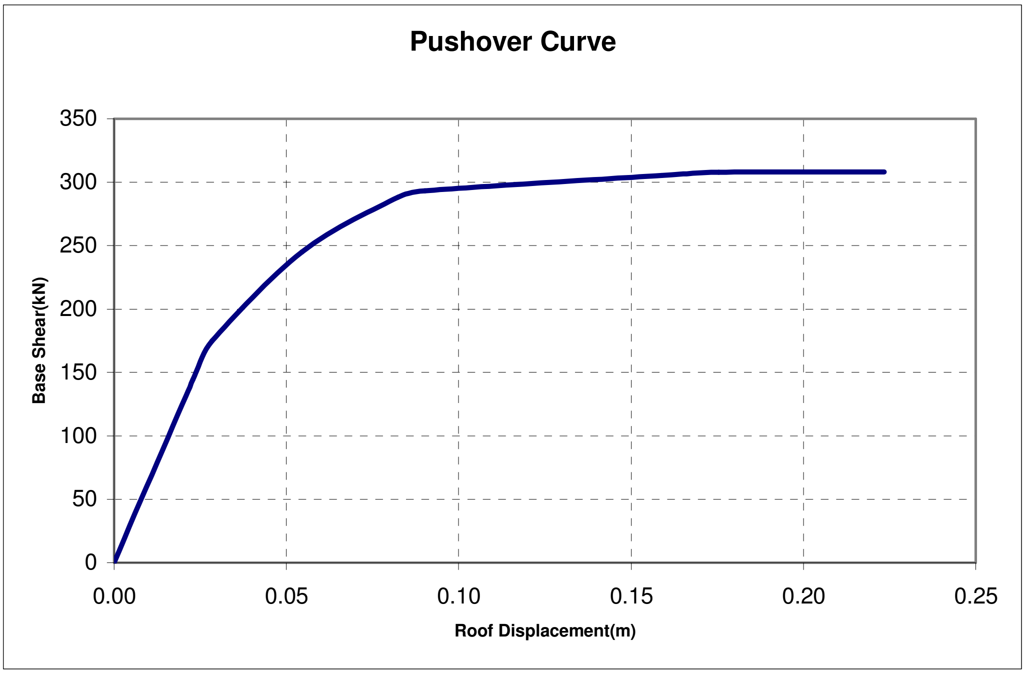


Figure 4.10. Pushover Curve for SSM-1

e) Pushover curve is converted to a modal capacity diagram (Figure 4.11) which has spectral displacement and spectral acceleration axis by using Table 4.21.

Table 4.21. Spectral Displacement – Spectral Acceleration Values for SSM-1

Step	Displacement (m)	BaseForce (kN)	a_1	d_1
0	0.000	0.000	0.000	0.000
1	0.023	144.173	1.009	0.018
2	0.029	174.752	1.223	0.022
3	0.055	244.891	1.714	0.042
4	0.082	288.100	2.017	0.063
5	0.090	293.037	2.051	0.069
6	0.159	305.181	2.136	0.122
7	0.175	307.737	2.154	0.134
8	0.223	307.858	2.155	0.171

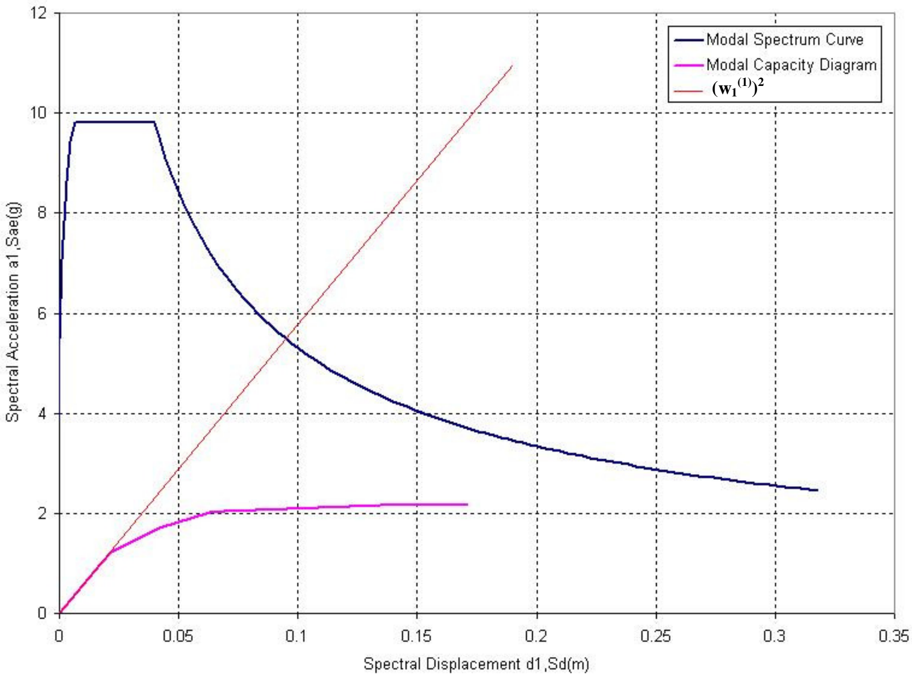


Figure 4.11. Modal Capacity and Modal Spectrum Curves for SSM-1

f,g,h,i) Modal contribution coefficient corresponding to first mode is obtained by using Table 4.22.

Table 4.22. Determining Modal Contribution Coefficient Corresponding To First Mode for SSM-1

Story	W_i(kN)	Mode	m_i	Φ_{i1}	$m_i\Phi_{i1}$	L_{x1}	$m_i\Phi_{i1}^2$	M_1	M_{x1}	Ratio	Γ_{x1}
4	417.00	1	42.508	0.1092	4.643	11.953	0.507	1.000	142.866	79.72%	11.953
3	444.00	1	45.260	0.0876	3.964		0.347				
2	448.50	1	45.719	0.0525	2.402		0.126				
1	448.50	1	45.719	0.0206	0.944		0.020				

S_{ae1}, S_{de1} and S_{di1} is calculated by using Equations (3.13 – 3.16) by knowing that the first mode period for SSM-1 is $T = 0.8267 sn$.

Table 4.23. Determining S_{ae1}, S_{de1} and S_{di1}

Ao	T_1	T_B	S_{ae1}	S_{de1}	C_{R1}	$S_{di1} = d_1^{(p)}$
0.4	0.8267	0.4000	5.4883	0.0950	1	0.0950

j) Roof lateral displacement demand due to the seismic load along the X axis is calculated by using Equation (3.19), Table 4.24.

Table 4.24. Determining Roof Lateral Displacement Demand

Γ_{x1}	Φ_{xN1}	M_{x1}	$d_1^{(p)}$	u_{xN1}(m)
11.953	0.1092	142.866	0.0950	0.1240

k) Structural system is pushed until the determined displacement demand. Plastic hinges occur at the end of this pushover analysis. Related plastic hinges are shown in Figure 4.12.

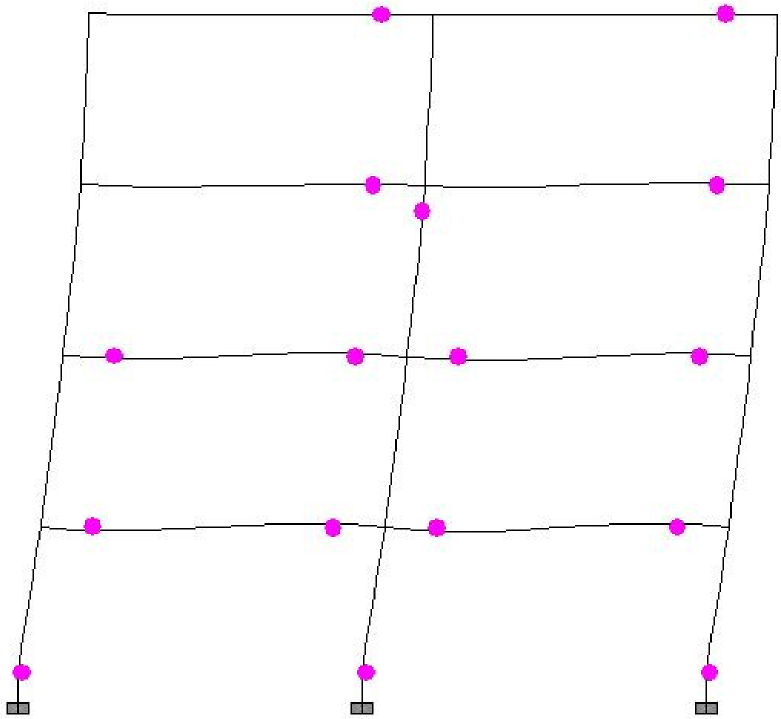


Figure 4.12. Plastic Hinges for SSM-1

l) Plastic hinge rotations at all critical sections of the structural system are obtained at the end of pushover analysis.

m) Plastic curvature demands of sections are obtained by using Equation (3.20).

n) Equivalent yield curvature ϕ_y is calculated by using Equation (3.21).

o) Curvature demands for beams mentioned in steps (l-n) are given in Table 4.25.

Table 4.25. Determining Total Curvature Demands of Beams for SSM-1

Story	Beam	Hinge	θ_p	ϕ_p	ϕ_y	ϕ_t	ε_{su}
STORY1	B1	B1H1	0.006450	0.021500	0.007350	0.028850	0.017000
STORY1	B1	B1H2	-0.012660	0.042200	0.007350	0.049550	0.024400
STORY1	B2	B2H1	0.005820	0.019400	0.007350	0.026750	0.012870
STORY1	B2	B2H2	-0.015050	0.050167	0.007350	0.057517	0.030000
STORY2	B1	B1H3	0.004020	0.013400	0.007350	0.020750	0.010680
STORY2	B1	B1H4	-0.011010	0.036700	0.007350	0.044050	0.022360
STORY2	B2	B2H3	0.004010	0.013367	0.007350	0.020717	0.007580
STORY2	B2	B2H4	-0.012400	0.041333	0.007350	0.048683	0.023100
STORY3	B1	B1H5	0.000000	0.000000	0.007350	0.007350	0.003540
STORY3	B1	B1H6	-0.005100	0.017000	0.007350	0.024350	0.012290
STORY3	B2	B2H5	0.000000	0.000000	0.007350	0.007350	0.003550
STORY3	B2	B2H6	-0.006680	0.022267	0.007350	0.029617	0.013440
STORY4	B1	B1H7	0.000000	0.000000	0.007350	0.007350	0.003530
STORY4	B1	B1H8	-0.000400	0.001333	0.007350	0.008683	0.004400
STORY4	B2	B2H7	0.000000	0.000000	0.007350	0.007350	0.003560
STORY4	B2	B2H8	-0.000970	0.003233	0.007350	0.010583	0.004500

p) Steel and concrete strain demands of all beam sections are found by using concrete and steel models in XTRACT models (Fig. 4.13). In determining the member damage level of the beam sections based on the total curvature demands, numerical examples show that steel strain demands govern. However both steel and concrete strain demands for total curvature of sections are determined and compared with the limits in TEC 2007. The results for ductile behavior of the system are given in Table 4.26.

Table 4.26. Member Damage Levels of Beams for SSM-1

Story	Beam	Hinge	ε_{su}	Damage Region	ε_{cu}	Damage Region	Result
STORY1	B1	B1H1	0.017000	VDR	0.001492	MDR	VDR
STORY1	B1	B1H2	0.024400	VDR	0.001970	MDR	VDR
STORY1	B2	B2H1	0.012870	VDR	0.001088	MDR	VDR
STORY1	B2	B2H2	0.030000	VDR	0.002650	MDR	VDR
STORY2	B1	B1H3	0.010680	VDR	0.001117	MDR	VDR
STORY2	B1	B1H4	0.022360	VDR	0.001750	MDR	VDR
STORY2	B2	B2H3	0.007580	MDR	0.001450	MDR	MDR
STORY2	B2	B2H4	0.023100	VDR	0.001873	MDR	VDR
STORY3	B1	B1H5	0.003540	MDR	0.000633	MDR	MDR
STORY3	B1	B1H6	0.012290	VDR	0.001183	MDR	VDR
STORY3	B2	B2H5	0.003550	MDR	0.000734	MDR	MDR
STORY3	B2	B2H6	0.013440	VDR	0.001531	MDR	VDR
STORY4	B1	B1H7	0.003530	MDR	0.000678	MDR	MDR
STORY4	B1	B1H8	0.004400	MDR	0.006921	MDR	MDR
STORY4	B2	B2H7	0.003560	MDR	0.000734	MDR	MDR
STORY4	B2	B2H8	0.004500	MDR	0.000578	MDR	MDR

q) Curvature demands for columns mentioned in steps (l-n) are given in Table 4.27.

Table 4.27. Determining Total Curvature Demands of Columns for SSM-1

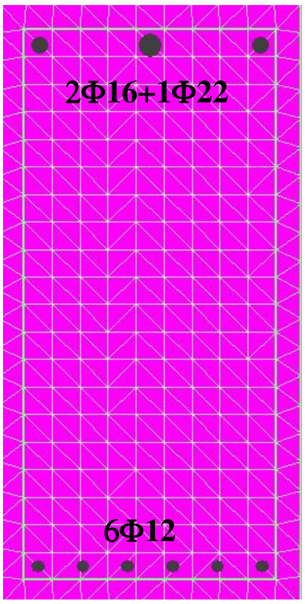
Story	Beam	Hinge	P	θ_p	ϕ_p	ϕ_y	ϕ_t
STORY1	C1	C1H1	-363.32	0.00991	0.04955	0.011025	0.060575
STORY1	C1	C1H2	-356.12	0	0	0.011025	0.011025
STORY1	C2	C2H1	-1197.22	0.01146	0.0382	0.007350	0.045550
STORY1	C2	C2H2	-1186.42	0	0	0.007350	0.007350
STORY1	C3	C3H1	-722.46	0.01012	0.0506	0.011025	0.061625
STORY1	C3	C3H2	-715.26	0	0	0.011025	0.011025
STORY2	C1	C1H3	-279.87	0	0	0.011025	0.011025
STORY2	C1	C1H4	-272.67	0	0	0.011025	0.011025
STORY2	C2	C2H3	-885.27	0	0	0.007350	0.007350
STORY2	C2	C2H4	-874.47	0	0	0.007350	0.007350
STORY2	C3	C3H3	-522.37	0	0	0.011025	0.011025
STORY2	C3	C3H4	-515.17	0	0	0.011025	0.011025
STORY3	C1	C1H5	-194.61	0	0	0.011025	0.011025
STORY3	C1	C1H6	-187.41	0	0	0.011025	0.011025
STORY3	C2	C2H5	-577.25	0	0	0.011025	0.011025
STORY3	C2	C2H6	-570.05	0.00111	0.00555	0.011025	0.016575
STORY3	C3	C3H5	-320.14	0	0	0.011025	0.011025
STORY3	C3	C3H6	-312.94	0	0	0.011025	0.011025
STORY4	C1	C1H7	-93.75	0	0	0.011025	0.011025
STORY4	C1	C1H8	-86.55	0	0	0.011025	0.011025
STORY4	C2	C2H7	-273.62	0	0	0.011025	0.011025
STORY4	C2	C2H8	-266.42	0	0	0.011025	0.011025
STORY4	C3	C3H7	-133.63	0	0	0.011025	0.011025
STORY4	C3	C3H8	-126.43	0	0	0.011025	0.011025

For columns; normal force-total curvature diagrams are constructed in XTRACT software for each column section by using concrete and steel models (Fig.4.13). Then total curvature demand calculated by using equation (3.22) and obtained normal force is located on the normal force-total curvature diagram. Member damage levels of column sections are determined, Table 4.28.

Normal force – total curvature diagrams are constructed in XTRACT software for each column type. Diagrams for S102 (bottom&top), S301 (bottom), S401 (bottom) and their member damage levels are given in Figure 4.14 - 4.16.

Table 4.28. Member Damage Levels of Columns for SSM-1

Story	Beam	Hinge	P	θ_p	ϕ_p	ϕ_y	ϕ_t	Damage Region
STORY1	C1	C1H1	-363.32	0.00991	0.04955	0.011025	0.060575	VDR
STORY1	C1	C1H2	-356.12	0	0	0.011025	0.011025	MDR
STORY1	C2	C2H1	-1197.22	0.01146	0.0382	0.007350	0.045550	VDR
STORY1	C2	C2H2	-1186.42	0	0	0.007350	0.007350	MDR
STORY1	C3	C3H1	-722.46	0.01012	0.0506	0.011025	0.061625	VDR
STORY1	C3	C3H2	-715.26	0	0	0.011025	0.011025	MDR
STORY2	C1	C1H3	-279.87	0	0	0.011025	0.011025	MDR
STORY2	C1	C1H4	-272.67	0	0	0.011025	0.011025	MDR
STORY2	C2	C2H3	-885.27	0	0	0.007350	0.007350	MDR
STORY2	C2	C2H4	-874.47	0	0	0.007350	0.007350	MDR
STORY2	C3	C3H3	-522.37	0	0	0.011025	0.011025	MDR
STORY2	C3	C3H4	-515.17	0	0	0.011025	0.011025	MDR
STORY3	C1	C1H5	-194.61	0	0	0.011025	0.011025	MDR
STORY3	C1	C1H6	-187.41	0	0	0.011025	0.011025	MDR
STORY3	C2	C2H5	-577.25	0	0	0.011025	0.011025	MDR
STORY3	C2	C2H6	-570.05	0.00111	0.00555	0.011025	0.016575	MDR
STORY3	C3	C3H5	-320.14	0	0	0.011025	0.011025	MDR
STORY3	C3	C3H6	-312.94	0	0	0.011025	0.011025	MDR
STORY4	C1	C1H7	-93.75	0	0	0.011025	0.011025	MDR
STORY4	C1	C1H8	-86.55	0	0	0.011025	0.011025	MDR
STORY4	C2	C2H7	-273.62	0	0	0.011025	0.011025	MDR
STORY4	C2	C2H8	-266.42	0	0	0.011025	0.011025	MDR
STORY4	C3	C3H7	-133.63	0	0	0.011025	0.011025	MDR
STORY4	C3	C3H8	-126.43	0	0	0.011025	0.011025	MDR



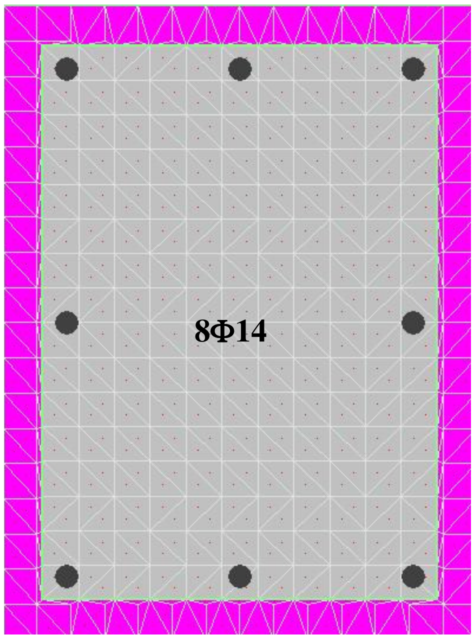


Figure 4.13. B1H1 Beam End Section and S101 Column Xtract Models

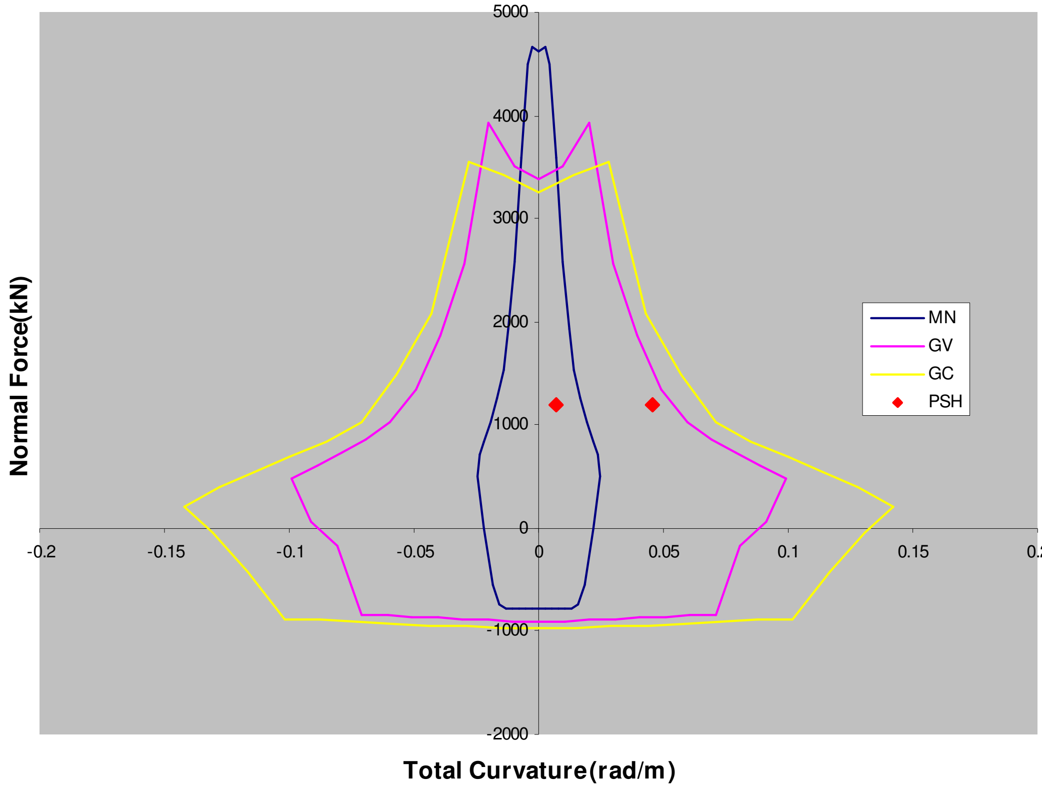


Figure 4.14. S102 Column (bottom&top) Member Damage Level for SSM-1

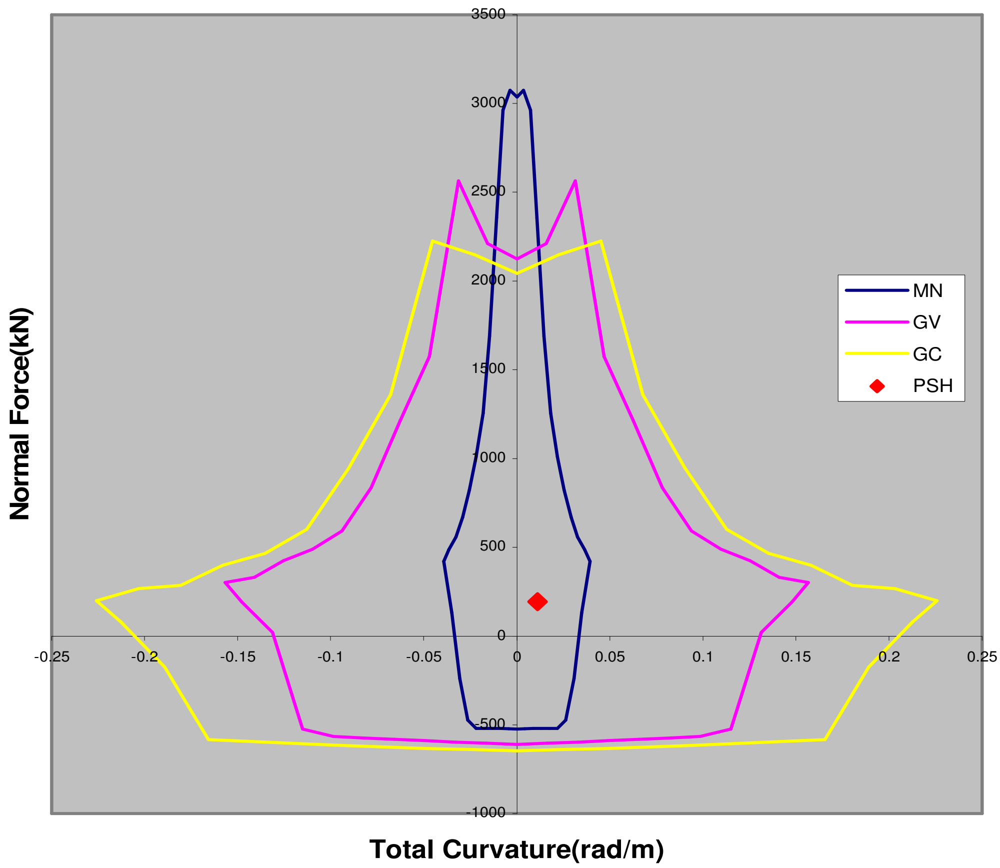


Figure 4.15. S301 Column (Bottom) Member Damage Level for SSM-1

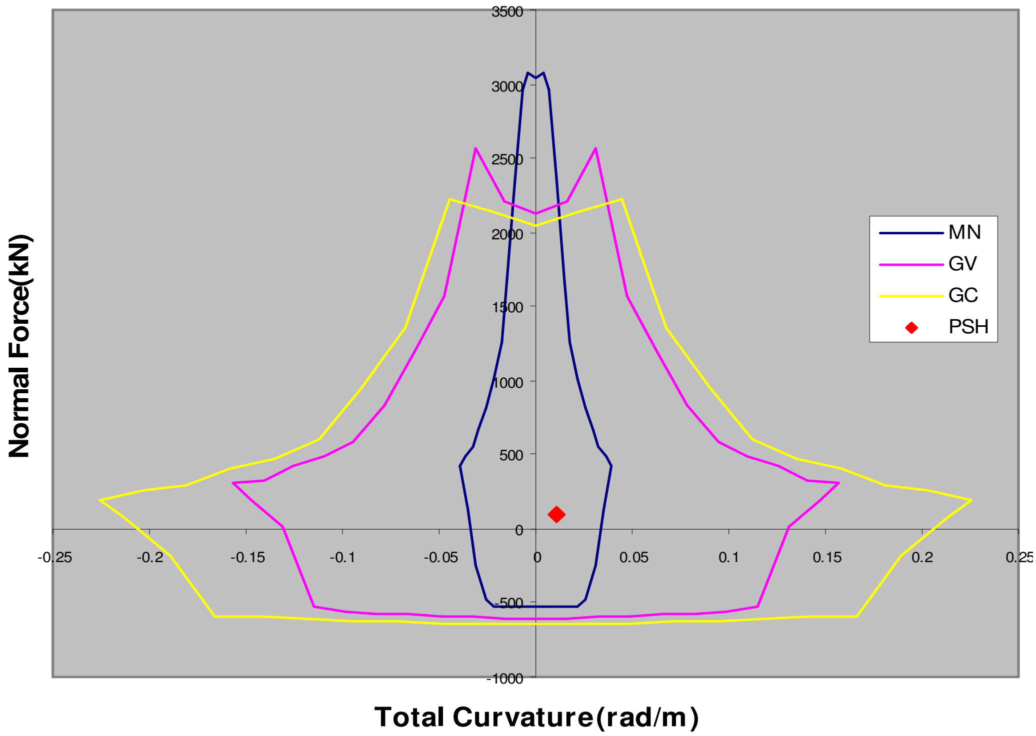


Figure 4.16. S401 Column (Bottom) Member Damage Level for SSM-1

r) Story drifts of the structure obtained at the end of pushover analysis are given in Table 4.29.

Table 4.29. Story Drifts For SSM-1

Story	Relative $(\delta_i)_{max}$(m)	h_i(m)	$(\delta_i)_{max}/h_i$
Story1	0.005	3.0	0.00167
Story2	0.0075	3.0	0.00250
Story3	0.0083	3.0	0.00277
Story4	0.0052	3.0	0.00173

5. COMPARISONS

In this chapter, seismic performance evaluation and comparison of structural system models are given.

5.1. Performance Evaluation and Comparison for SSM-1

SSM-1 which is chosen for numerical simulations in this study is plain moment resisting frame and has two bays with 6 m length. Its story height is 3 m and distance between frames on plan is 5 m. It is 4 story building.

Concrete Class is C20, Reinforcing Steel Class is S420. SSM-1 is sized and detailed on the basis of the Turkish Earthquake Code 1998. Structural behavior factor (R) is taken as 8 (eight) and it is assumed that model behaves in high ductility level.

Dimensions and longitudinal reinforcements of the sections are given in Table 5.1 and Table 5.2. Pushover curve and occurred plastic hinges in nonlinear method are given in Figure 5.1 and Figure 5.2 respectively.

Table 5.1. Column Dimensions and Longitudinal Reinforcements for SSM-1

Column	b(cm)	h(cm)	Reinforcement
S101	30	40	8Φ14
S102	30	60	12Φ14
S103	30	40	8Φ14
S201	30	40	8Φ14
S202	30	60	12Φ14
S203	30	40	8Φ14
S301	30	40	8Φ14
S302	30	40	8Φ14
S303	30	40	8Φ14
S401	30	40	8Φ14
S402	30	40	8Φ14
S403	30	40	8Φ14

Table 5.2. Beam Dimensions and Longitudinal Reinforcements for SSM-1

Beam	b/h (cm)	Bottom Reinf.	Top Reinf.	Left Top Reinf.	Right Top Reinf.
K101	30/60	6Φ12	2Φ16	1Φ22	2Φ22
K102	30/60	6Φ12	2Φ16	2Φ22	1Φ22
K201	30/60	6Φ12	3Φ12	2Φ18	2Φ22
K202	30/60	6Φ12	3Φ12	2Φ22	2Φ18
K301	30/60	6Φ12	2Φ14	2Φ18	2Φ20
K302	30/60	6Φ12	2Φ14	2Φ20	2Φ18
K401	30/60	6Φ12	2Φ14	1Φ14	2Φ18
K402	30/60	6Φ12	2Φ14	2Φ18	1Φ14

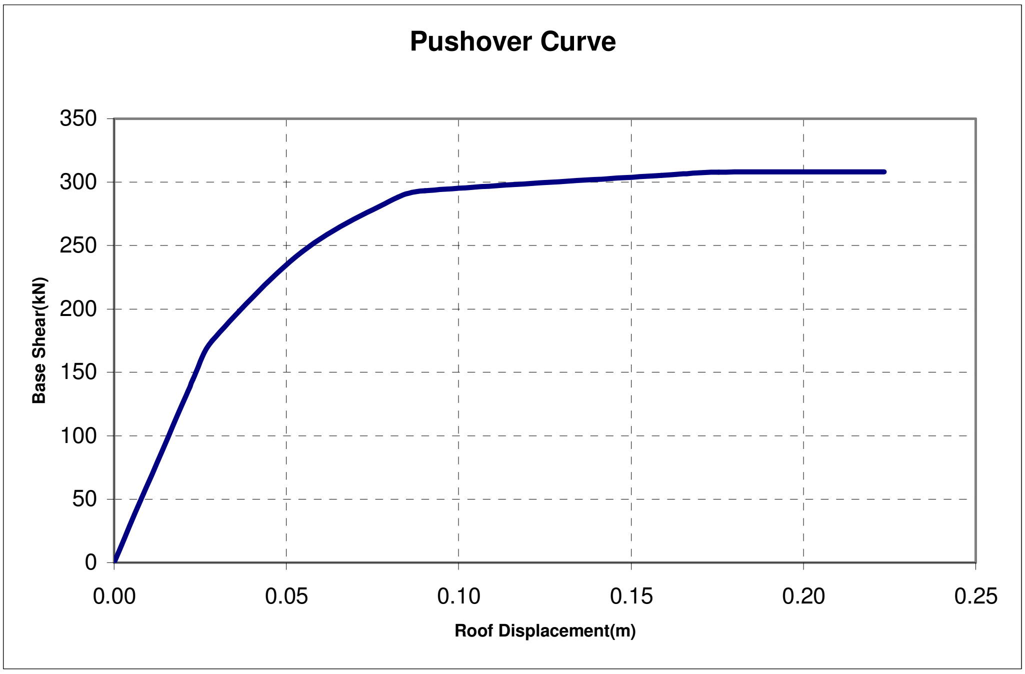


Figure 5.1. Pushover Curve for SSM-1

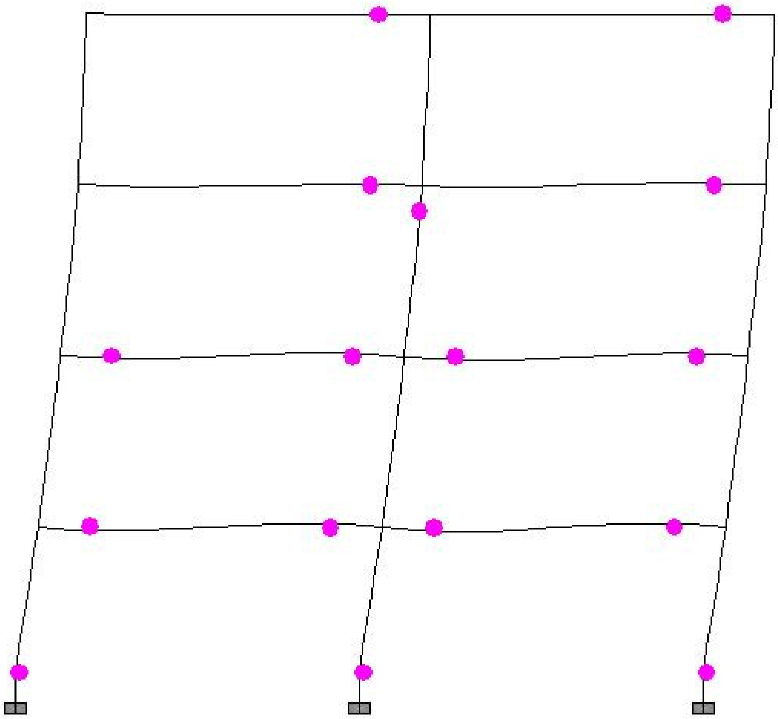


Figure 5.2. Plastic Hinges for SSM-1

5.1.1. Comparison for Story Drifts in SSM-1

Comparison for story drifts in SSM-1 is given in Table 5.3.

Table 5.3. Comparison of Story Drifts for SSM-1

Story	Linear Method $(\delta_i)_{max}$(m)	Nonlinear Method $(\delta_i)_{max}$(m)	Linear Method $(\delta_i)_{max}/h_i$	Nonlinear Method $(\delta_i)_{max}/h_i$
Story1	0.0221	0.005	0.00736	0.00167
Story2	0.0289	0.0075	0.00963	0.00250
Story3	0.0304	0.0083	0.01013	0.00277
Story4	0.0179	0.0052	0.00596	0.00173

5.1.2. Comparison of Member Seismic Performances in SSM-1

Member damage levels of beam, column edges of SSM-1 obtained by performing linear - nonlinear analysis methods mentioned in TEC 2007 are given in Table 5.4, Table 5.5. Confined concrete model is used for beams and columns.

Table 5.4. Comparison of Member Damage Levels of Beams for SSM-1

Story	Beam	Hinge	Linear Method	Nonlinear Method	Check
STORY1	B1	B1H1	VDR	VDR	✓
STORY1	B1	B1H2	MDR	VDR	**X**
STORY1	B2	B2H1	MDR	VDR	**X**
STORY1	B2	B2H2	VDR	VDR	✓
STORY2	B1	B1H3	VDR	VDR	✓
STORY2	B1	B1H4	MDR	VDR	**X**
STORY2	B2	B2H3	MDR	MDR	✓
STORY2	B2	B2H4	VDR	VDR	✓
STORY3	B1	B1H5	MDR	MDR	✓
STORY3	B1	B1H6	MDR	VDR	**X**
STORY3	B2	B2H5	MDR	MDR	✓
STORY3	B2	B2H6	MDR	VDR	**X**
STORY4	B1	B1H7	MDR	MDR	✓
STORY4	B1	B1H8	MDR	MDR	✓
STORY4	B2	B2H7	MDR	MDR	✓
STORY4	B2	B2H8	MDR	MDR	✓

Table 5.5. Comparison of Member Damage Levels of Columns for SSM-1

Story	Beam	Hinge	Linear Method	Nonlinear Method	Check
STORY1	C1	C1H1	MDR	VDR	**X**
STORY1	C1	C1H2	MDR	MDR	✓
STORY1	C2	C2H1	SDR	VDR	**X**
STORY1	C2	C2H2	MDR	MDR	✓
STORY1	C3	C3H1	MDR	VDR	**X**
STORY1	C3	C3H2	MDR	MDR	✓
STORY2	C1	C1H3	MDR	MDR	✓
STORY2	C1	C1H4	MDR	MDR	✓
STORY2	C2	C2H3	VDR	MDR	**X**
STORY2	C2	C2H4	VDR	MDR	**X**
STORY2	C3	C3H3	VDR	MDR	**X**
STORY2	C3	C3H4	MDR	MDR	✓
STORY3	C1	C1H5	MDR	MDR	✓
STORY3	C1	C1H6	MDR	MDR	✓
STORY3	C2	C2H5	MDR	MDR	✓
STORY3	C2	C2H6	MDR	MDR	✓
STORY3	C3	C3H5	VDR	MDR	**X**
STORY3	C3	C3H6	MDR	MDR	✓
STORY4	C1	C1H7	MDR	MDR	✓
STORY4	C1	C1H8	MDR	MDR	✓
STORY4	C2	C2H7	MDR	MDR	✓
STORY4	C2	C2H8	MDR	MDR	✓
STORY4	C3	C3H7	MDR	MDR	✓
STORY4	C3	C3H8	MDR	MDR	✓

5.2. Performance Evaluation and Comparison for SSM-2

SSM-2 which is chosen for numerical simulations in this study is plain moment resisting frame and has two bays with 6 m length. Its story height is 3 m and distance between frames on plan is 5 m. It is 4 story building.

Concrete Class is C16, Reinforcing Steel Class is S220. SSM-2 is sized and detailed on the basis of the Turkish Earthquake Code 1975. Structure type coefficient (K) is taken as 1.0.

Dimensions and longitudinal reinforcements of the sections are given in Table 5.6 and Table 5.7 Pushover curve and occurred plastic hinges in nonlinear method are given in Figure 5.3 and Figure 5.4 respectively.

Table 5.6. Column Dimensions and Longitudinal Reinforcements for SSM-2

Column	b(cm)	h(cm)	Reinforcement
S101	30	40	8Φ20
S102	30	60	12Φ22
S103	30	40	8Φ20
S201	30	40	8Φ22
S202	30	60	12Φ14
S203	30	40	8Φ22
S301	30	40	8Φ20
S302	30	40	8Φ14
S303	30	40	8Φ20
S401	30	40	8Φ20
S402	30	40	8Φ14
S403	30	40	8Φ20

Table 5.7. Beam Dimensions and Longitudinal Reinforcements for SSM-2

Beam	b/h (cm)	Bottom Reinf.	Top Reinf.	Left Top Reinf.	Right Top Reinf.
K101	30/50	4Φ22	5Φ14	3Φ24	4Φ26
K102	30/50	4Φ22	5Φ14	4Φ26	3Φ24
K201	30/50	4Φ22	3Φ18	3Φ26	4Φ26
K202	30/50	4Φ22	3Φ18	4Φ26	3Φ26
K301	30/50	4Φ22	6Φ12	3Φ24	4Φ24
K302	30/50	4Φ22	6Φ12	4Φ24	3Φ24
K401	30/40	5Φ20	3Φ18	2Φ22	5Φ22
K402	30/40	5Φ20	3Φ18	5Φ22	2Φ22

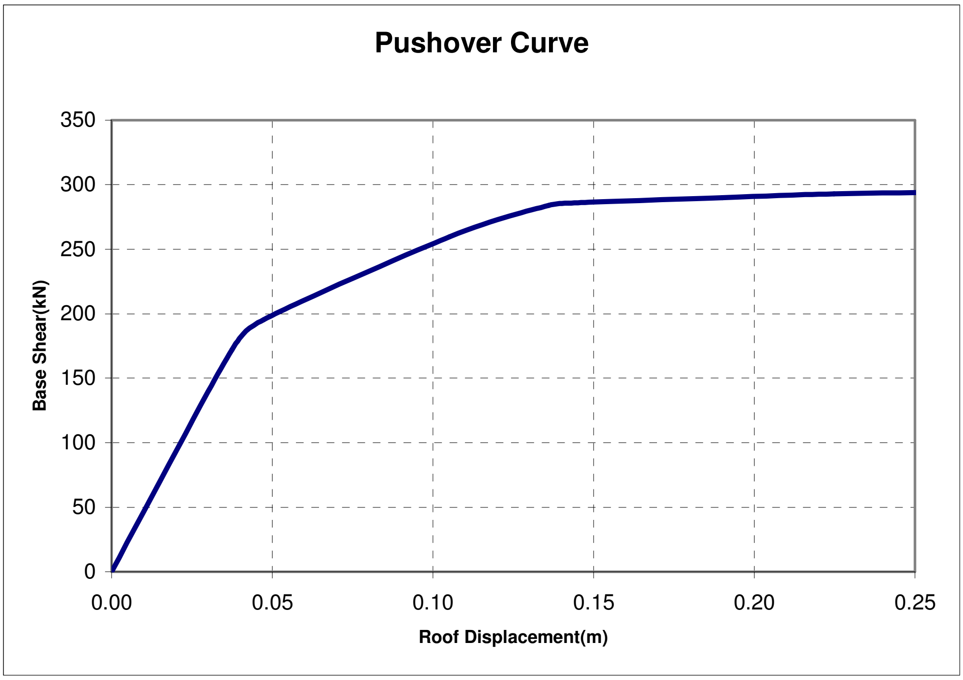


Figure 5.3. Pushover Curve for SSM-2

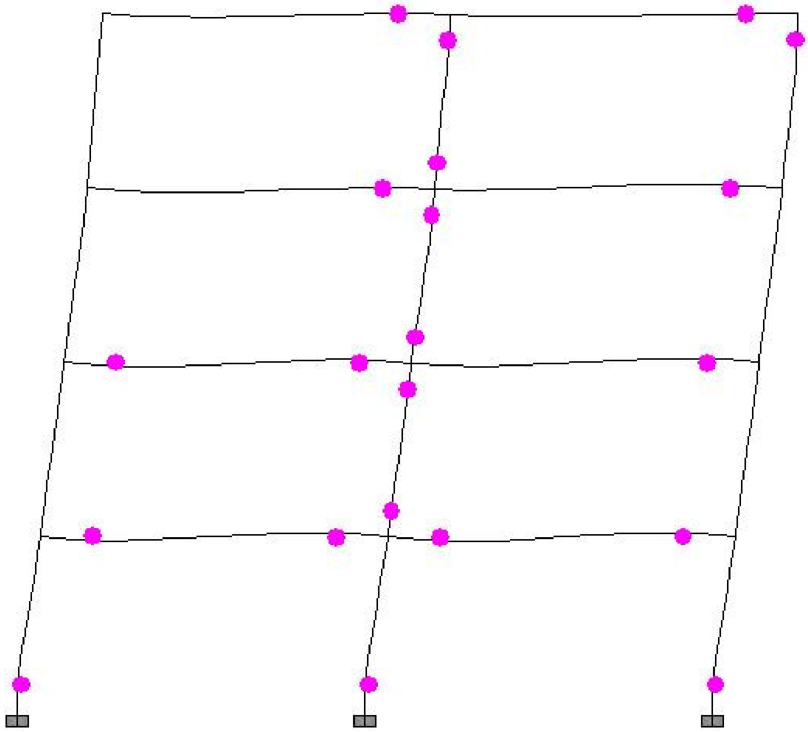


Figure 5.4. Plastic Hinges for SSM-2

5.2.1. Comparison for Story Drifts in SSM-2

Comparison for story drifts in SSM-2 is given in Table 5.8.

Table 5.8. Comparison of Story Drifts for SSM-2

Story	Linear Method $(\delta_i)_{max}$(m)	Nonlinear Method $(\delta_i)_{max}$(m)	Linear Method $(\delta_i)_{max}/h_i$	Nonlinear Method $(\delta_i)_{max}/h_i$
Story1	0.0243	0.0064	0.00810	0.00213
Story2	0.0353	0.0109	0.01176	0.00363
Story3	0.0359	0.0115	0.01196	0.00383
Story4	0.0242	0.0086	0.00806	0.00286

5.2.2. Comparison of Member Seismic Performances in SSM-2

Confined and unconfined concrete models are used for beams, columns in SSM-2a and SSM-2b respectively. Member damage levels of beam, column edges of SSM-2a and SSM-2b obtained by performing linear - nonlinear analysis methods mentioned in TEC 2007 are given.

5.2.2.1. Comparison of Member Seismic Performances in SSM-2a

Confined concrete model is used for beams and columns in SSM-2a. Member damage levels of beam, column edges of SSM-2a obtained by performing linear – nonlinear analysis methods mentioned in TEC 2007 are given in Table 5.9, Table 5.10.

Table 5.9. Comparison of Member Damage Levels of Beams for SSM-2a

Story	Beam	Hinge	Linear Method	Nonlinear Method	Check
STORY1	B1	B1H1	VDR	MDR	X
STORY1	B1	B1H2	VDR	MDR	X
STORY1	B2	B2H1	VDR	MDR	X
STORY1	B2	B2H2	VDR	VDR	✓
STORY2	B1	B1H3	VDR	MDR	X
STORY2	B1	B1H4	VDR	VDR	✓
STORY2	B2	B2H3	VDR	MDR	X
STORY2	B2	B2H4	VDR	VDR	✓
STORY3	B1	B1H5	VDR	MDR	X
STORY3	B1	B1H6	VDR	VDR	✓
STORY3	B2	B2H5	VDR	MDR	X
STORY3	B2	B2H6	VDR	VDR	✓
STORY4	B1	B1H7	VDR	MDR	X
STORY4	B1	B1H8	MDR	MDR	✓
STORY4	B2	B2H7	MDR	MDR	✓
STORY4	B2	B2H8	VDR	MDR	X

Table 5.10. Comparison of Member Damage Levels of Columns for SSM-2a

Story	**Beam**	**Hinge**	**Linear Method**	**Nonlinear Method**	**Check**
STORY1	C1	C1H1	MDR	VDR	**X**
STORY1	C1	C1H2	MDR	MDR	✓
STORY1	C2	C2H1	SDR	VDR	**X**
STORY1	C2	C2H2	MDR	MDR	✓
STORY1	C3	C3H1	MDR	VDR	**X**
STORY1	C3	C3H2	MDR	MDR	✓
STORY2	C1	C1H3	MDR	MDR	✓
STORY2	C1	C1H4	MDR	MDR	✓
STORY2	C2	C2H3	SDR	MDR	**X**
STORY2	C2	C2H4	VDR	MDR	**X**
STORY2	C3	C3H3	MDR	MDR	✓
STORY2	C3	C3H4	MDR	MDR	✓
STORY3	C1	C1H5	MDR	MDR	✓
STORY3	C1	C1H6	MDR	MDR	✓
STORY3	C2	C2H5	VDR	MDR	**X**
STORY3	C2	C2H6	MDR	VDR	**X**
STORY3	C3	C3H5	MDR	MDR	✓
STORY3	C3	C3H6	MDR	MDR	✓
STORY4	C1	C1H7	MDR	MDR	✓
STORY4	C1	C1H8	MDR	MDR	✓
STORY4	C2	C2H7	MDR	MDR	✓
STORY4	C2	C2H8	MDR	MDR	✓
STORY4	C3	C3H7	MDR	MDR	✓
STORY4	C3	C3H8	MDR	MDR	✓

5.2.2.2. Comparison of Member Seismic Performances in SSM-2b

Unconfined concrete model is used for beams and columns in SSM-2b. Member damage levels of beam, column edges of SSM-2b obtained by performing linear - nonlinear analysis methods mentioned in TEC 2007 are given in Table 5.11, Table 5.12.

Table 5.11. Comparison of Member Damage Levels of Beams for SSM-2b

Story	Beam	Hinge	Linear Method	Nonlinear Method	Check
STORY1	B1	B1H1	SDR	MDR	X
STORY1	B1	B1H2	SDR	MDR	X
STORY1	B2	B2H1	SDR	MDR	X
STORY1	B2	B2H2	SDR	VDR	X
STORY2	B1	B1H3	SDR	MDR	X
STORY2	B1	B1H4	SDR	VDR	X
STORY2	B2	B2H3	SDR	MDR	X
STORY2	B2	B2H4	SDR	VDR	X
STORY3	B1	B1H5	SDR	MDR	X
STORY3	B1	B1H6	SDR	VDR	X
STORY3	B2	B2H5	SDR	MDR	X
STORY3	B2	B2H6	SDR	VDR	X
STORY4	B1	B1H7	VDR	MDR	X
STORY4	B1	B1H8	MDR	MDR	✓
STORY4	B2	B2H7	MDR	MDR	✓
STORY4	B2	B2H8	VDR	MDR	X

Table 5.12. Comparison of Member Damage Levels of Columns for SSM-2b

Story	Beam	Hinge	Linear Method	Nonlinear Method	Check
STORY1	C1	C1H1	MDR	VDR	X
STORY1	C1	C1H2	MDR	MDR	✓
STORY1	C2	C2H1	CR	CR	✓
STORY1	C2	C2H2	MDR	MDR	✓
STORY1	C3	C3H1	VDR	CR	X
STORY1	C3	C3H2	MDR	MDR	✓
STORY2	C1	C1H3	MDR	MDR	✓
STORY2	C1	C1H4	MDR	MDR	✓
STORY2	C2	C2H3	CR	MDR	X
STORY2	C2	C2H4	CR	MDR	X
STORY2	C3	C3H3	VDR	MDR	X
STORY2	C3	C3H4	MDR	MDR	✓
STORY3	C1	C1H5	MDR	MDR	✓
STORY3	C1	C1H6	MDR	MDR	✓
STORY3	C2	C2H5	SDR	MDR	X
STORY3	C2	C2H6	SDR	CR	X
STORY3	C3	C3H5	VDR	MDR	X
STORY3	C3	C3H6	MDR	MDR	✓
STORY4	C1	C1H7	MDR	MDR	✓
STORY4	C1	C1H8	MDR	MDR	✓
STORY4	C2	C2H7	VDR	MDR	X
STORY4	C2	C2H8	VDR	MDR	X
STORY4	C3	C3H7	MDR	MDR	✓
STORY4	C3	C3H8	MDR	MDR	✓

5.3. Performance Evaluation and Comparison for SSM-3

SSM-3 which is chosen for numerical simulations in this study is plain moment resisting frame and has two bays with 6 m length. Its story height is 3 m and distance between frames on plan is 5 m. It is 6 story building.

Concrete Class is C20, Reinforcing Steel Class is S220. SSM-3 is sized and detailed on the basis of the Turkish Earthquake Code 1975. Structure type coefficient (K) is taken as 0.80.

Dimensions and longitudinal reinforcements of the sections are given in Table 5.13 and Table 5.14. Pushover curve and occurred plastic hinges in nonlinear method are given in Figure 5.5 and Figure 5.6 respectively.

Table 5.13. Column Dimensions and Longitudinal Reinforcements for SSM-3

Column	b(cm)	h(cm)	Reinforcement
S101	30	50	10Φ14
S102	30	70	12Φ26
S103	30	50	10Φ14
S201	30	50	10Φ14
S202	30	70	12Φ16
S203	30	50	10Φ14
S301	30	50	10Φ14
S302	30	70	12Φ16
S303	30	50	10Φ14
S401	30	40	10Φ16
S402	30	50	10Φ14
S403	30	40	10Φ16
S501	30	40	10Φ16
S502	30	50	10Φ14
S503	30	40	10Φ16
S601	30	40	10Φ18
S602	30	50	10Φ14
S603	30	40	10Φ18

Table 5.14. Beam Dimensions and Longitudinal Reinforcements for SSM-3

Beam	b/h (cm)	Bottom Reinf.	Top Reinf.	Left Top Reinf.	Right Top Reinf.
K101	30/60	4Φ20	4Φ14	4Φ18	3Φ24
K102	30/60	4Φ20	4Φ14	3Φ24	4Φ18
K201	30/60	4Φ20	4Φ14	4Φ20	3Φ24
K202	30/60	4Φ20	4Φ14	3Φ24	4Φ20
K301	30/60	4Φ20	4Φ14	3Φ22	4Φ20
K302	30/60	4Φ20	4Φ14	4Φ20	3Φ22
K401	30/50	7Φ16	6Φ12	4Φ20	5Φ20
K402	30/50	7Φ16	6Φ12	5Φ20	4Φ20
K501	30/50	7Φ16	5Φ12	3Φ22	3Φ24
K502	30/50	7Φ16	5Φ12	3Φ24	3Φ22
K601	30/50	4Φ20	5Φ12	3Φ14	3Φ22
K602	30/50	4Φ20	5Φ12	3Φ22	3Φ14

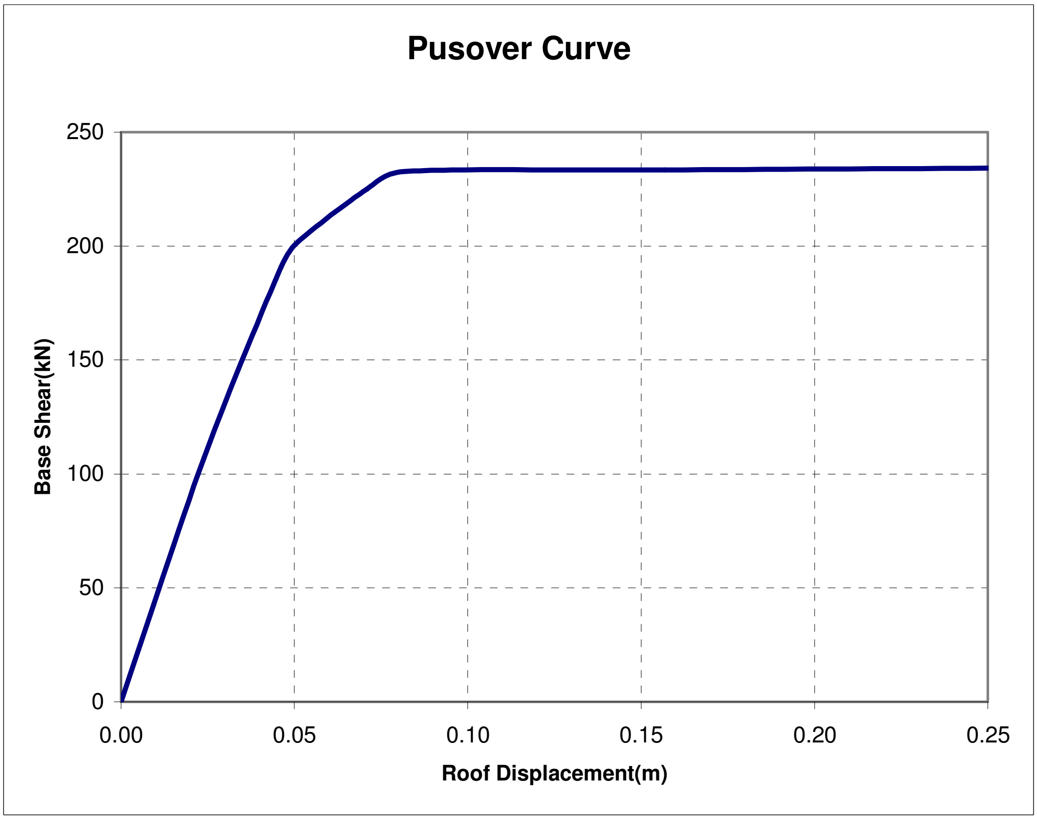


Figure 5.5. Pushover Curve for SSM-3

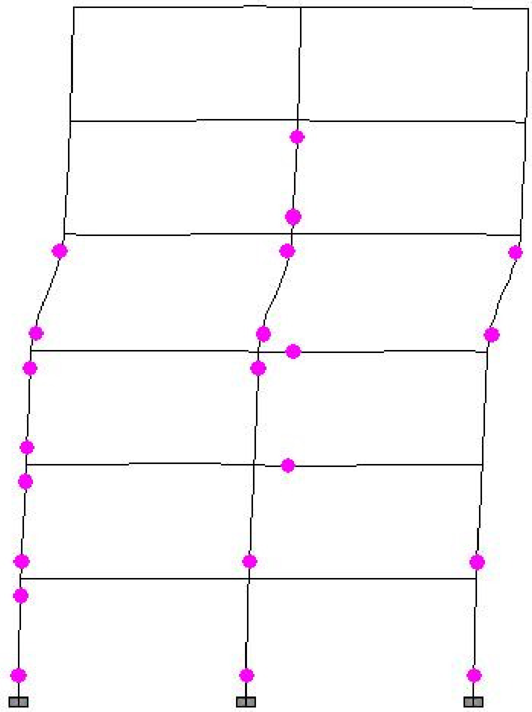


Figure 5.6. Plastic Hinges for SSM-3

5.3.1. Comparison for Story Drifts in SSM-3

Comparison for story drifts in SSM-3 is given in Table 5.15.

Table 5.15. Comparison of Story Drifts for SSM-3

Story	Linear Method $(\delta_i)_{max}$(m)	Nonlinear Method $(\delta_i)_{max}$(m)	Linear Method $(\delta_i)_{max}/h_i$	Nonlinear Method $(\delta_i)_{max}/h_i$
Story1	0.0143	0.0036	0.00476	0.00120
Story2	0.0219	0.0064	0.00730	0.002133
Story3	0.0215	0.0067	0.00716	0.00223
Story4	0.0273	0.0084	0.00910	0.00280
Story5	0.0232	0.0076	0.00773	0.00253
Story6	0.0136	0.0046	0.00453	0.00153

5.3.2. Comparison of Member Seismic Performances in SSM-3

Member damage levels of beam, column edges of SSM-3 obtained by performing linear - nonlinear analysis methods mentioned in TEC 2007 are given in Table 5.16, Table 5.17. Confined concrete model is used for beams and columns.

Table 5.16. Comparison of Member Damage Levels of Beams for SSM-3

Story	**Beam**	**Hinge**	**Linear Method**	**Nonlinear Method**	**Check**
STORY1	B1	B1H1	VDR	MDR	**X**
STORY1	B1	B1H2	VDR	MDR	**X**
STORY1	B2	B2H1	VDR	MDR	**X**
STORY1	B2	B2H2	VDR	MDR	**X**
STORY2	B1	B1H3	VDR	MDR	**X**
STORY2	B1	B1H4	VDR	MDR	**X**
STORY2	B2	B2H3	VDR	MDR	**X**
STORY2	B2	B2H4	VDR	MDR	**X**
STORY3	B1	B1H5	VDR	MDR	**X**
STORY3	B1	B1H6	VDR	MDR	**X**
STORY3	B2	B2H5	VDR	MDR	**X**
STORY3	B2	B2H6	VDR	MDR	**X**
STORY4	B1	B1H7	VDR	MDR	**X**
STORY4	B1	B1H8	VDR	MDR	**X**
STORY4	B2	B2H7	VDR	MDR	**X**
STORY4	B2	B2H8	VDR	MDR	**X**
STORY5	B1	B1H9	VDR	MDR	**X**
STORY5	B1	B1H10	VDR	MDR	**X**
STORY5	B2	B2H9	VDR	MDR	**X**
STORY5	B2	B2H10	VDR	MDR	**X**
STORY6	B1	B1H11	VDR	MDR	**X**
STORY6	B1	B1H12	MDR	MDR	✓
STORY6	B2	B2H11	MDR	MDR	✓
STORY6	B2	B2H12	VDR	MDR	**X**

Table 5.17. Comparison of Member Damage Levels of Columns for SSM-3

Story	Beam	Hinge	Linear Method	Nonlinear Method	Check
STORY1	C1	C1H1	MDR	MDR	✓
STORY1	C1	C1H2	MDR	MDR	✓
STORY1	C2	C2H1	VDR	MDR	**X**
STORY1	C2	C2H2	MDR	MDR	✓
STORY1	C3	C3H1	MDR	MDR	✓
STORY1	C3	C3H2	MDR	MDR	✓
STORY2	C1	C1H3	MDR	MDR	✓
STORY2	C1	C1H4	MDR	MDR	✓
STORY2	C2	C2H3	VDR	MDR	**X**
STORY2	C2	C2H4	MDR	MDR	✓
STORY2	C3	C3H3	MDR	MDR	✓
STORY2	C3	C3H4	MDR	MDR	✓
STORY3	C1	C1H5	MDR	MDR	✓
STORY3	C1	C1H6	MDR	MDR	✓
STORY3	C2	C2H5	MDR	MDR	✓
STORY3	C2	C2H6	MDR	MDR	✓
STORY3	C3	C3H5	MDR	MDR	✓
STORY3	C3	C3H6	MDR	MDR	✓
STORY4	C1	C1H7	MDR	SDR	**X**
STORY4	C1	C1H8	MDR	VDR	**X**
STORY4	C2	C2H7	MDR	SDR	**X**
STORY4	C2	C2H8	MDR	SDR	**X**
STORY4	C3	C3H7	MDR	SDR	**X**
STORY4	C3	C3H8	MDR	VDR	**X**
STORY5	C1	C1H9	MDR	MDR	✓
STORY5	C1	C1H10	MDR	MDR	✓
STORY5	C2	C2H9	MDR	MDR	✓
STORY5	C2	C2H10	MDR	MDR	✓
STORY5	C3	C3H9	MDR	MDR	✓
STORY5	C3	C3H10	MDR	MDR	✓
STORY6	C1	C1H11	MDR	MDR	✓
STORY6	C1	C1H12	MDR	MDR	✓
STORY6	C2	C2H11	MDR	MDR	✓
STORY6	C2	C2H12	MDR	MDR	✓
STORY6	C3	C3H11	MDR	MDR	✓
STORY6	C3	C3H12	MDR	MDR	✓

5.4. Performance Evaluation and Comparison for SSM-4

SSM-4 which is chosen for numerical simulations in this study is plain moment resisting frame and has two bays with 6 m length. Its story height is 3 m and distance between frames on plan is 5 m. It is 6story building.

Concrete Class is C20, Reinforcing Steel Class is S220. SSM-3 is sized and detailed on the basis of the Turkish Earthquake Code 1998. Structural behavior factor (R) is taken as 8 (eight) and it is assumed that model behaves in high ductility level.

Dimensions and longitudinal reinforcements of the sections are given in Table 5.18 and Table 5.19. Pushover curve and occurred plastic hinges in nonlinear method are given in Figure 5.7 and Figure 5.8 respectively.

Table 5.18. Column Dimensions and Longitudinal Reinforcements for SSM-4

Column	b(cm)	h(cm)	Reinforcement
S101	40	50	8Φ18
S102	40	70	12Φ18
S103	40	50	8Φ18
S201	40	50	8Φ18
S202	40	70	12Φ18
S203	40	50	8Φ18
S301	40	50	8Φ18
S302	40	70	12Φ18
S303	40	50	8Φ18
S401	40	40	8Φ16
S402	40	60	12Φ16
S403	40	40	8Φ16
S501	40	40	8Φ16
S502	40	60	12Φ16
S503	40	40	8Φ16
S601	40	40	8Φ20
S602	40	60	12Φ16
S603	40	40	8Φ20

Table 5.19. Beam Dimensions and Longitudinal Reinforcements for SSM-4

Beam	b/h (cm)	Bottom Reinf.	Top Reinf.	Left Top Reinf.	Right Top Reinf.
K101	30/60	4Φ20	5Φ12	4Φ18	4Φ20
K102	30/60	4Φ20	5Φ12	4Φ20	4Φ18
K201	30/60	4Φ20	5Φ12	4Φ20	5Φ18
K202	30/60	4Φ20	5Φ12	5Φ18	4Φ20
K301	30/60	4Φ20	5Φ12	3Φ22	4Φ20
K302	30/60	4Φ20	5Φ12	4Φ20	3Φ22
K401	30/50	7Φ16	4Φ14	4Φ20	3Φ24
K402	30/50	7Φ16	4Φ14	3Φ24	4Φ20
K501	30/50	3Φ24	5Φ12	4Φ20	5Φ18
K502	30/50	3Φ24	5Φ12	5Φ18	4Φ20
K601	30/50	3Φ24	5Φ12	3Φ18	5Φ18
K602	30/50	3Φ24	5Φ12	5Φ18	3Φ18

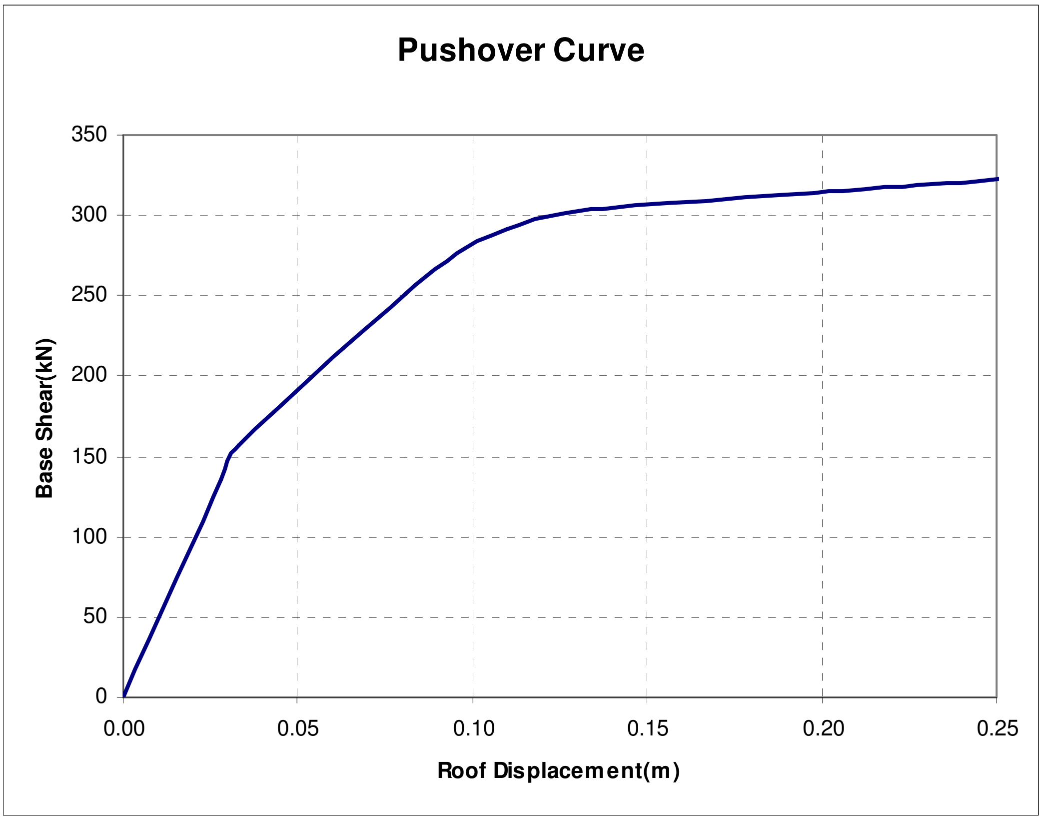


Figure 5.7. Pushover Curve for SSM-4

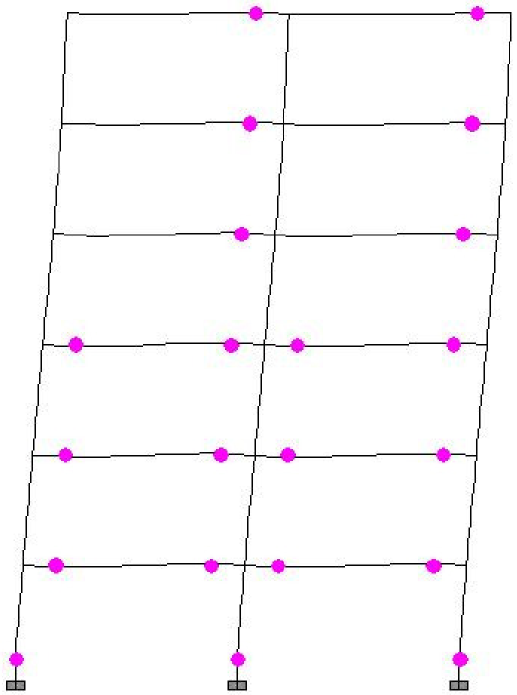


Figure 5.8. Plastic Hinges for SSM-4

5.4.1. Comparison for Story Drifts in SSM-4

Comparison for story drifts in SSM-4 is given in Table 5.20.

Table 5.20. Comparison of Story Drifts for SSM-4

Story	Linear Method $(\delta_i)_{max}$(m)	Nonlinear Method $(\delta_i)_{max}$(m)	Linear Method $(\delta_i)_{max}/h_i$	Nonlinear Method $(\delta_i)_{max}/h_i$
Story1	0.0130	0.0031	0.00433	0.00103
Story2	0.0212	0.0056	0.00707	0.00187
Story3	0.0213	0.0060	0.00710	0.00200
Story4	0.0239	0.0069	0.00797	0.00230
Story5	0.0209	0.0062	0.00697	0.00207
Story6	0.0138	0.0041	0.00460	0.00137

5.4.2. Comparison of Member Seismic Performances in SSM-4

Member damage levels of beam, column edges of SSM-4 obtained by performing linear - nonlinear analysis methods mentioned in TEC 2007 are given in Table 5.21, Table 5.22. Confined concrete model is used for beams and columns.

Table 5.21. Comparison of Member Damage Levels of Beams for SSM-4

Story	**Beam**	**Hinge**	**Linear Method**	**Nonlinear Method**	**Check**
STORY1	B1	B1H1	VDR	MDR	**X**
STORY1	B1	B1H2	VDR	VDR	✓
STORY1	B2	B2H1	VDR	MDR	**X**
STORY1	B2	B2H2	VDR	VDR	✓
STORY2	B1	B1H3	VDR	MDR	**X**
STORY2	B1	B1H4	VDR	VDR	✓
STORY2	B2	B2H3	VDR	MDR	**X**
STORY2	B2	B2H4	VDR	VDR	✓
STORY3	B1	B1H5	VDR	MDR	**X**
STORY3	B1	B1H6	VDR	VDR	✓
STORY3	B2	B2H5	VDR	MDR	**X**
STORY3	B2	B2H6	VDR	VDR	✓
STORY4	B1	B1H7	VDR	MDR	**X**
STORY4	B1	B1H8	VDR	VDR	✓
STORY4	B2	B2H7	VDR	MDR	**X**
STORY4	B2	B2H8	VDR	VDR	✓
STORY5	B1	B1H9	VDR	MDR	**X**
STORY5	B1	B1H10	VDR	VDR	✓
STORY5	B2	B2H9	VDR	MDR	**X**
STORY5	B2	B2H10	VDR	VDR	✓
STORY6	B1	B1H11	MDR	MDR	✓
STORY6	B1	B1H12	MDR	MDR	✓
STORY6	B2	B2H11	MDR	MDR	✓
STORY6	B2	B2H12	MDR	MDR	✓

Table 5.22. Comparison of Member Damage Levels of Columns for SSM-4

Story	Beam	Hinge	Linear Method	Nonlinear Method	Check
STORY1	C1	C1H1	MDR	MDR	✓
STORY1	C1	C1H2	MDR	MDR	✓
STORY1	C2	C2H1	VDR	VDR	✓
STORY1	C2	C2H2	MDR	MDR	✓
STORY1	C3	C3H1	MDR	MDR	✓
STORY1	C3	C3H2	MDR	MDR	✓
STORY2	C1	C1H3	MDR	MDR	✓
STORY2	C1	C1H4	MDR	MDR	✓
STORY2	C2	C2H3	VDR	MDR	X
STORY2	C2	C2H4	MDR	MDR	✓
STORY2	C3	C3H3	MDR	MDR	✓
STORY2	C3	C3H4	MDR	MDR	✓
STORY3	C1	C1H5	MDR	MDR	✓
STORY3	C1	C1H6	MDR	MDR	✓
STORY3	C2	C2H5	MDR	MDR	✓
STORY3	C2	C2H6	MDR	MDR	✓
STORY3	C3	C3H5	MDR	MDR	✓
STORY3	C3	C3H6	MDR	MDR	✓
STORY4	C1	C1H7	MDR	MDR	✓
STORY4	C1	C1H8	MDR	MDR	✓
STORY4	C2	C2H7	MDR	MDR	✓
STORY4	C2	C2H8	MDR	MDR	✓
STORY4	C3	C3H7	MDR	MDR	✓
STORY4	C3	C3H8	MDR	MDR	✓
STORY5	C1	C1H9	MDR	MDR	✓
STORY5	C1	C1H10	MDR	MDR	✓
STORY5	C2	C2H9	MDR	MDR	✓
STORY5	C2	C2H10	MDR	MDR	✓
STORY5	C3	C3H9	MDR	MDR	✓
STORY5	C3	C3H10	MDR	MDR	✓
STORY6	C1	C1H11	MDR	MDR	✓
STORY6	C1	C1H12	MDR	MDR	✓
STORY6	C2	C2H11	MDR	MDR	✓
STORY6	C2	C2H12	MDR	MDR	✓
STORY6	C3	C3H11	MDR	MDR	✓
STORY6	C3	C3H12	MDR	MDR	✓

6. CONCLUSIONS AND RECOMMENDATIONS

6.1. Evaluations of Numerical Studies

In this study, numerical studies are performed choosing four reinforced concrete structural system models having 4 and 6 storys with adequate transverse reinforcement. The aim for this selection is to represent medium-rise buildings in Turkey. Linear and nonlinear analysis methods given in the Turkish Earthquake Code 2007 are used to determine the seismic performances of members. Results obtained from calculations are compared and evaluated.

The followings are results obtained from this study;

1. The linear method of the Turkish Earthquake Code 2007 generally gives more conservative member damage levels as compared with those given by the nonlinear method as expected. However nonlinear method gives more conservative member damage levels 5.21 percent in beams and 9.72 percent in columns at numerical studies.
2. Reinforcing steel strain governs in determining seismic performances of members where adequate lateral reinforcement is provided at plastic regions.
3. Damage levels of structural members obtained from linear and nonlinear methods typically differ by at most one damage region.
4. Members of structural system models designed in accordance with the Turkish Earthquake Code 1998 are generally in minimum damage or visible damage regions. Whereas, members of structural system models designed in accordance with the Turkish Earthquake Code 1975 are generally in significant damage or collapse regions.
5. The results obtained from SSM-2 also showed that older buildings with low material quality are more susceptible to damage under seismic loads.
6. The results obtained from SSM-3 may indicate the truth that major portion of collapsed buildings in the Marmara Earthquake are older and are low-rise to medium-rise buildings. This case stems from two reasons; cross sections at

medium-rise buildings differ at approximately the mid height of the structure due to economical reasons and there is no strong column-weak beam requirement in the Turkish Earthquake Codes before 1998. Therefore, collapse mechanism occurs at mid height of the older structures which is undesirable for structural engineers.

7. Member performance regions obtained from SSM-1 and SSM-4 by using linear and nonlinear analysis methods are similar. This case indicates that medium-rise structures (including 4 or 6 stories) that are designed according to the Turkish Earthquake Code 1998 have adequate seismic performance.
8. Story drift ratios obtained by using linear method are higher than story drifts obtained by using nonlinear method.
9. Studies show that the seismic performance of structures with adequate transverse reinforcement designed according to the Turkish Earthquake Code 1998 is generally adequate.

6.2. Recommendations

To determine the seismic performances of structures, both linear and nonlinear methods shall be applied. Unfortunately, linear methods are preferred in determining seismic performances due to its simplicity in structural engineering practice. So, nonlinear methods mentioned in the Turkish Earthquake Code 2007 can also be simplified to enlarge its usage.

In this study, two dimensional frame models were chosen to represent medium-rise buildings in Turkey. This study can be made more detailed by using space frame models. By this way building performance levels of buildings can be obtained for repair and rehabilitation decisions.

This study investigates the expected levels of damage in members where adequate transverse reinforcement is assumed to exist. Additional studies should be performed to assess the damage/performance levels, based on comparison of concrete strain demands and capacities, in members where the amount of transverse reinforcement provided is inadequate.

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