SEISMIC EVALUATION OF
EXISTING MULTI STOREY REINFORCED
CONCRETE BUILDING

by

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ABSTRACT

SEISMIC EVALUATION OF EXISTING MULTI STOREY REINFORCED CONCRETE BUILDING

After 1998 Adana-Ceyhan Earthquake, many reinforced concrete buildings damaged and some of them, which moderately damaged needed repairing and strengthening. In this study, one of the moderately damaged six-storey building is selected and analyzed by performing code static (elastic) analysis, code dynamic analysis and capacity spectrum analysis after completing of seismic evaluation of the given structure, the states of the building before and after strengthening are compared by performing inelastic static analysis on three-dimensional model. In both states, the lateral load carrying capacity of the frame system is determined by nonlinear static loading procedure called pushover analysis. Then, the capacity of the structure is compared with earthquake demand in Acceleration Displacement Response Spectra (ADRS) format by Capacity Spectrum Method (CSM) and the performance of the structure is estimated.

The results of the analyses show that the applied static, dynamic and pushover analysis are compatible with each other and rehabilitation procedure applied to the moderately damaged structure in Adana city is satisfactorily effective in response to an earthquake excitation. The added shearwalls increase the lateral stiffness and strength considerably. The deformation capacity of structure is also improved.

ÖZET

ÇOK KATLI BETONARME YAPININ DEPREM ANALİZİNİN YAPILMASI

1998 Adana-Ceyhan depreminden sonra bir çok betonarme yapı hasar gördü, bu binalar arasında orta hasarlı olanların onarım ve güçlendirmeye ihtiyaçları vardı. Bu çalışmada orta hasarlı altı katlı bir betonarme yapı örnek olarak alınmış ve elastik, dinamik ve kapasite spektrumu analizleri ayrı ayrı uygulanarak, binanın sismik özellikleri incelenmiştir. Binanın güçlendirme öncesi ve sonrası durumu, üç boyutlu modelleme üzerinde elastik ötesi statik analiz yapılarak ayrıca incelenmiştir. Binanın her iki durumu için çerçeve sisteminin yanal yük taşıma kapasitesi statik itme analizi yöntemi ile elde edilmiştir. Elde edilen kapasite eğrileri, Kapasite Spektrumu Metodu (CSM) kullanılarak İvme-Deformasyon Tepki Spektrumları (ADRS) formatında deprem spektrumu ile karşılaştırılmış ve yapının performansı belirlenmiştir.

Bu analizin sonuçları seçilen orta hasarlı binaya uygulanabilir onarım ve güçlendirme yönteminin binanın deprem sırasındaki davranışı açısından yeterli derecede etkin olduğu sonucunu göstermiştir. Takviye amacıyla eklenen perde duvarlar binanın yanal ötelenme kapasitesini ve rijitliğini önemli ölçüde artırmıştır. Takviye sonucunda yapının deformasyon kapasitesi de artmıştır.

To my wife, Narmina...

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LIST OF SYMBOLS/ABBREVIATIONS

C_A	Shaking intensity (Acceleration)
C_V	Shaking intensity (Velocity)
N_A	Near source factor (Acceleration)
N_V	Near source factor (Velocity)
$PF_1\phi_{R1}$	Modal participation factors
Rx	Ratio of displacements in x-direction
Ry	Ratio of displacements in y-direction
S_a	Spectral accelerations
S_d	Spectral displacements
S_D	Soil profile
T_j	Elastic period of vibration
V	Base shear
W	Weight of structure
Z	Seismic zone factor
α_1	Effective modal weight ratios
Δ_r	Roof displacement
ADRS	Acceleration displacement response spectrum
CQC	Complete Quadratic Computation
CSM	Capacity spectrum method
DE	Design earthquake
DL	Dead load
EQ	Earthquake load
LL	Live load
SE	Serviceability earthquake
SRSS	Squares of sum of squares

1. INTRODUCTION

1.1. General

Concrete is rather popular as a building material as well as in our country and almost all over the world. For the most part, it serves its functions well; however concrete is inherently brittle and performs poorly during earthquakes if not reinforced properly. The last earthquakes dramatically showed this situation. 1997 code writers revised the design provisions (Published in 1975) for new concrete buildings to provide adequate ductility to resist strong ground shaking. There remain, nonetheless, millions of square meter of nonductile concrete buildings in our country.

The consequences of neglecting this general risk are inevitably catastrophic for some individual buildings. The collapse of single building has the potential for more loss of life than any other catastrophe. The potential defects in these buildings are often not readily apparent. This thesis is focused on the evaluation of these reinforced concrete buildings to investigate their deficiencies during seismic shaking. Depending on the specific characteristics of a particular building, one was selected from an array of alternatives.

Traditional design techniques assume that buildings respond elastically to earthquakes. In reality, large earthquakes can severely damage buildings cause inelastic behavior that dissipates energy. The assumption that buildings remain elastic simplifies the engineer's work but obscures a basic understanding of actual performance. The use of traditional procedures for existing buildings can lead to erroneous conclusions on deficiencies and unnecessarily high retrofit costs.

New analysis procedure described in this thesis, which is known as pushover analysis describes the inelastic behavior of the structural members of a building better and this technique can estimate more accurately the actual behavior of a building during a specific ground motion. This analysis procedure tells how to identify which part of the building will fail first.

1.2. Purpose

The primary purpose of this document is to provide the process of elastic and inelastic analysis to an existing reinforced concrete building and to interpret the obtained results. This thesis is intended to serve as an example reference for the future seismic evaluations of reinforced concrete buildings.

1.3. Object and Scope

This thesis provides a comprehensive methodology and supporting commentary for the seismic evaluation of existing concrete building. For this purpose, elastic analysis, dynamic analysis and pushover analysis are used separately and obtained results are compared in order to define the real behavior and damage zones of the structure under earthquake excitation.

The existing building that had been selected as a model structure for this study is located in Adana-Ceyhan region. It was a six-storey reinforced concrete residential building, which was moderately damaged after Adana-Ceyhan Earthquake. The architectural and structural plans of the building are given in Appendix Figures A.1 and A.2.

1.4. Organization and Contents

This thesis is organized into 8 parts. Part 1 is the introductory part for the given building, its site conditions and gives brief information about Adana-Ceyhan Earthquake. Part 2 provides the guidelines, rules, and assumptions required to develop the analytical model of buildings as three-dimensional system. In Part 3 and 4 load and mass calculations are explained with details. Part 5 presents static analysis (Elastic Analysis) procedure. In Part 6 dynamic analysis is carried out and results of this analysis are given. Part 7 presents the generalized nonlinear static analysis procedure characterized by use of a static pushover analysis method contains acceptability limits for the analysis results and Part 8 provides a detailed discussion of the various analysis results with the principal findings and concluding remarks.

1.5. Uncertainty and Reliability

Uncertainty is a condition associated with essentially all aspects of earthquake related science and engineering of the evaluation of existing buildings. The principle sources of uncertainty lie in the characterization of seismic shaking, the determination of materials properties of existing structural and geotechnical component capacities, and the assignment of the acceptance limits on structural behavior. These uncertainties, for the most part stemming from the lack of and / or the imperfect reliability of the specific supporting data available, affect all analytical methods and procedures applied to the challenge of seismic evaluation.

The performance-based methodology in other words *Pushover Analysis* presented in this thesis cannot and does not eliminate these uncertainties. However through the use of simplified static analysis and dynamic analysis, it provides a more sophisticated and direct approach to address the uncertainties than do traditional linear analysis procedures. As a result, this method is a useful and reliable design tool for assessment of expected building behavior.

1.6. Adana-Ceyhan Earthquake

Adana-Ceyhan earthquake occurred on 27 June 1998 at 16:55 local time (13:55 GMT) having a magnitude $m_b = 5.9$ resp. $M_w = 6.3$ shock southern Turkey [1].

The epicenter is located between the cities of Adana and Ceyhan about 30 km north of the coast of the Mediterranean Sea, which is illustrated in Figure 1.1. The main fault, which is in the northeast direction, can also be seen from this figure. The damage of the reinforced concrete buildings in this Earthquake once again proved the low quality of the reinforced concrete buildings in Turkey as it had happened in the Earthquakes of 1992 Erzincan, 1995 Dinar, 1999 Kocaeli and Düzce where great number of reinforced concrete buildings had been damaged and collapsed.

In this earthquake about 150 people were killed, 1500 were injured and many thousands were made homeless.

Figure 1.1. Adana-Ceyhan region and epicenter of earthquake [1]

Most of the observed damage occurred in traditional rural buildings, but many new multi-story residential buildings and industrial buildings also suffered heavy damage or even collapsed. The maximum intensity of the earthquake was estimated to read IX on the EMS-scale.

1.6.1. Seismological Aspects

The earthquake parameters of the main shock on June 27, 1998 provided by the Earthquake Research Department of Ankara (ERD) indicated a strike - slip earthquake along a 65 degree SE dipping fault plane. The epicenter was located approximately 30 km southeast of Adana at a depth of 23 km. A strong motion acceleration recording of the main shock was made by ERD in the local branch building of the Agricultural Ministry in

Ceyhan located approximately 35 km from the epicenter. As shown in Figure 1.2 the peak horizontal ground acceleration was 274 mg [1].

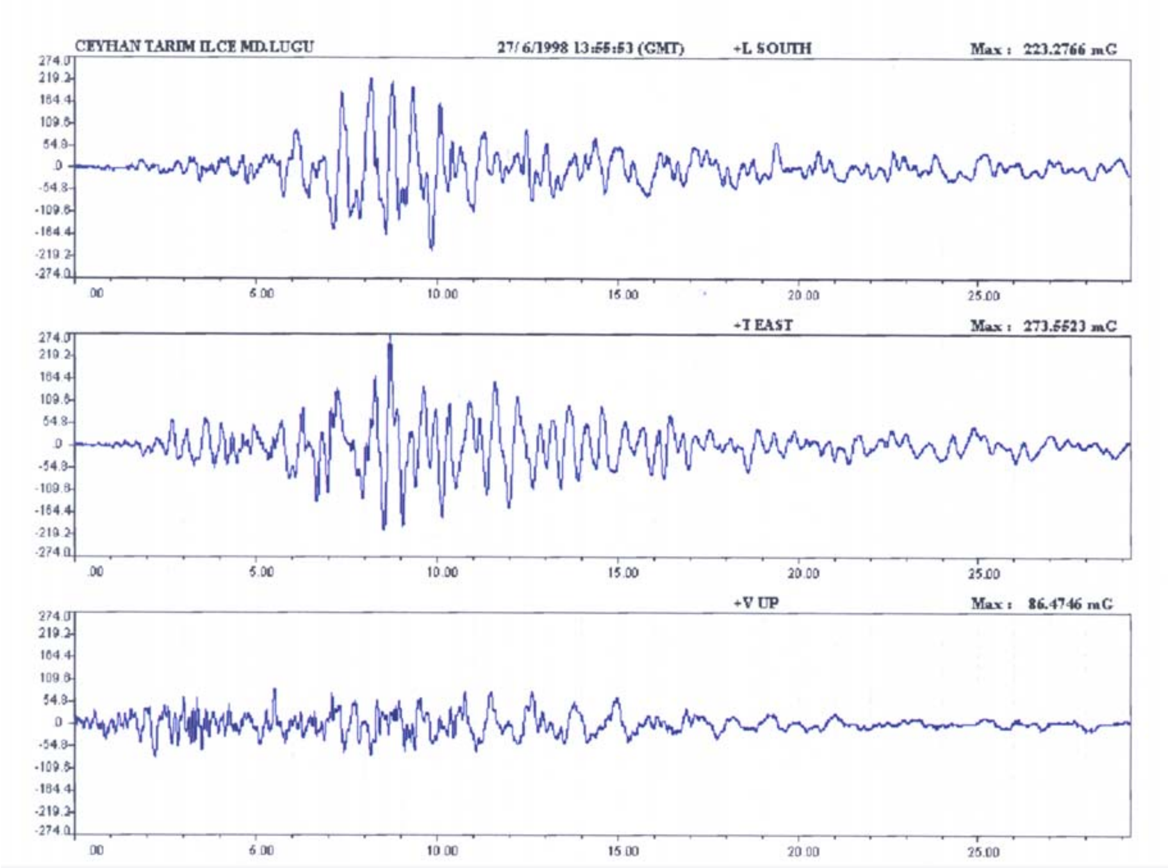


Figure 1.2. Acceleration Velocity and Displacement records of Adana-Ceyhan EQ [1]

1.6.2. Geological Context

The area of Adana city is characterized by a very large alluvial basin with a delta shape, which extends more than 100 km east west and approximately 70 km north south. Most of this basin is filled with quaternary recent Holocene deposits. In the southeast part of the basin, some limestone formations from the Miocene, Oligocene and Eocene Ages are visible at the surface. In the northern part of the basin, between Adana and Ceyhan, outcrops of travertine formations are also visible [1].

1.6.3. Turkish Seismic Code and Adana-Ceyhan Region

The first official code for earthquake resistant design in Turkey, entitled *Specifications for Structures to be built in Disaster Areas*, was published in 1975 (ERD 1975), according to this code Adana and Ceyhan were located in seismic zone 3, the second highest five hazard zone (0 to 4), with a seismic zone coefficient $C_o = 0.08$. Most of the existing modern reinforced concrete buildings in Adana and Ceyhan areas were built after 1975. It is unknown if the 1975 seismic code was systematically used in the design of these buildings. The selected building for this thesis was also built after 1975 and in the category of 1975 earthquake code [1].

In 1997, a new seismic code was introduced in Turkey (ERD 1997) and according to this code the seismic hazard map of the Adana province is given in Figure 1.3 [1]. The most part of the cities of Adana and Ceyhan are now situated in seismic zone II (Zone I is the highest zone) however the selected building is located in seismic zone I.

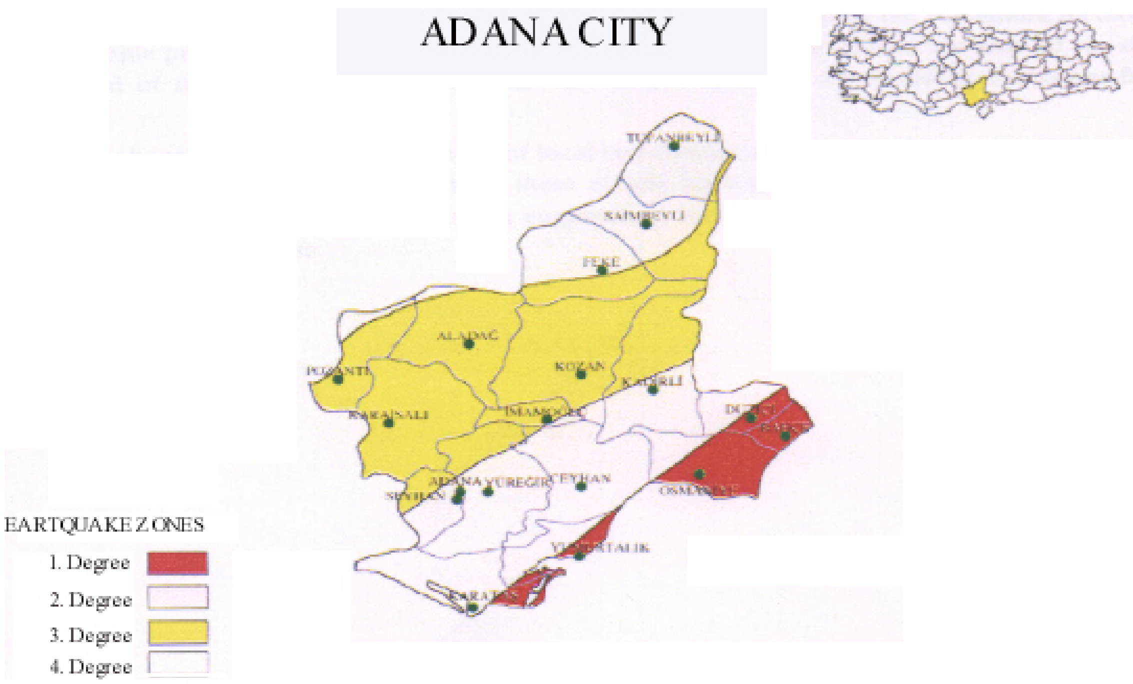


Figure 1.3. Seismic hazard map of the Adana province [1]

1.7. Building Description

Brief information about the building that had been selected as a model structure for this master thesis is given in the following subtopics:

1.7.1. Properties of Model Structure

Location	: Adana-Ceyhan Region
Number of Storey	: 6 Stories (having same storey height)
Storey Height	: 3.00 m
Plan Dimensions	: 20.15 x 14.25
Frame Type	: Reinforced Concrete Elements (No any shear wall)
Usage Purpose	: Residential
Seismic Zone	: Zone 1 (According to Earthquake Code, 1998)
Soil Type	: Z3 (According to Earthquake Code, 1998)
Ductility Level	: Enhanced
Slab Thickness	: 15 cm

1.7.2. Properties of Construction Materials Used in Model Structure

Concrete : BS 20

Fck = 200 kgf/cm

Fcd = 137 kgf/cm

Ec = 25E6 KN/m

Steel : ST III

Fyd = 3650 kgf/cm

Partition Walls : Brick

d = 0.8 ton / m

2. MODELING OF STRUCTURE

2.1. General

Modeling rules represented in this part are intended to guide development of the analytical model used to evaluate an existing building. Analytical building models based on the following rules will be complete and accurate enough to support linear elastic analysis, dynamic analysis and nonlinear static pushover analysis, described in Part 5,6 and 7. Analysis will usually rely on one or more specialized computer programs. Some available programs can directly represent nonlinear load-deformation behavior of individual components. Sap2000 is such a program by which the static, dynamic and pushover analysis processes are carried out.

2.2. Building Considerations

Analytical models for evaluation must represent complete three-dimensional characteristic of building behavior, including mass distribution, strength, stiffness and deformability, through a full range of global and local displacements. SAP2000 Nonlinear program enable us to obtain real behavior of structure by using three-dimensional modeling.

The analytical model of the building should represent all new and existing components that influence the mass, strength, stiffness, and deformability of the structure at or near the expected performance point (Explained with details in Part 7). Elements and components shown not to significantly influence the building assessment need not be modeled.

Behavior of foundation components and effects of soil-structure interaction should be modeled or shown to be insignificant to building assessment. The model of the connection between the columns and foundation will depend on details of the column foundation connection and the rigidity of the soil - foundation system.

According to the information taken from Middle East Technical University, Earthquake Research Center (METU/EERC) about the given building, this structure has no any problems coming from foundation and soil interaction. Therefore the end columns are modeled as fixed supports at the base of the structure.

2.3. Element Models

An element is defined as either a vertical or a horizontal portion of a building that acts to resist lateral and / or vertical load. Common vertical elements in reinforced concrete construction include frames, shear walls and combined frame wall elements. Horizontal elements commonly are reinforced concrete diaphragms. In the proceeding lines the elements used in modeling of the given structure are described with details.

2.3.1. Frame Element

In the given structure, all vertical and lateral loads coming from slabs are transferred to beams firstly then columns. Therefore the building can be modeled as a beam - column frame type of structure.

The analysis model of a beam - column frame elements should represent the strength, stiffness and deformation capacity of beams, columns, beam - column joints, and other components that may be the part of the frame. Beam and column components should be modeled considering flexural and shear rigidities, although the latter may be neglected in many cases. Potential failure of anchorages and splices may require modeling of these aspects as well. Rigid beam-column joints may be assumed.

The analytical model generally can represent a beam-column frame by using line elements with properties concentrated at component centerlines (Given in Figure 2.1).

In some cases the beam and column centerlines will not coincide, in which case a portion of the framing components may not be fully effective to resist lateral loads, and component torsion may result. Where minor eccentricities occur (the centerline of the narrower component falls within the middle third of the adjacent framing component measured transverse to the framing direction), the effect of the eccentricity can be ignored.

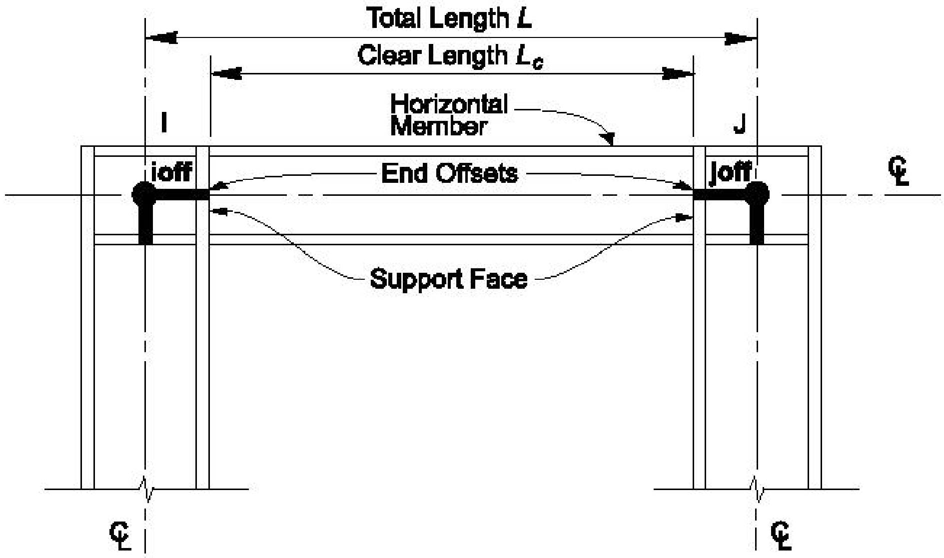


Figure 2.1. Shape of beam-column frame by using line elements

Where larger eccentricities occur, the effect should be represented either by a concentric frame model with reduced effective stiffness, strengths and deformation capacities or by direct modeling of the eccentricity. Where transverse slabs or beams connect beam and column component cross sections do not intersect, but instead beams and columns, the transverse slabs or beams should be modeled directly.

In the modeling of the given structure, some beam and column centerlines do not coincide, the effects of these minor eccentricities are not ignored and rigid link elements are used.

Nonstructural components that interact importantly with the frame should be modeled. Important nonstructural components that should be modeled include partial infill (which may restrict the framing action of the columns) and full - height solid or perforated infill and curtain walls (which may completely interrupt the flexural framing action of a beam-column frame). In general, stairs need not be modeled.

A frame section is a set of material and geometric properties that describe the cross - section of one or more frame elements. Sections are defined independently of the frame elements, and are assigned to the elements. The main sections for beams in the structural plan are 20x40 and 15x70 rectangular sections and the main sections for columns in the structural plan are 60x25, 65x25, 75x25, 80x25, 60x30, 75x30, 40x30 and 85x25 rectangular sections. The unit for all sections is ‘cm’.

2.3.2. Slabs

The slab will act as a diaphragm that determines interaction among different frames. The slab will also act compositely as a beam flange in tension and compression [2].

On the base of this definition slabs are modeled as shell elements. The shell element is a three or four-node formulation that combines separate membrane and plate bending behavior. All slabs in the given structure are reinforced concrete plates having a thickness of standard ‘15 cm’. All these frame and slab elements can be seen from the building model given in Figure 2.2.

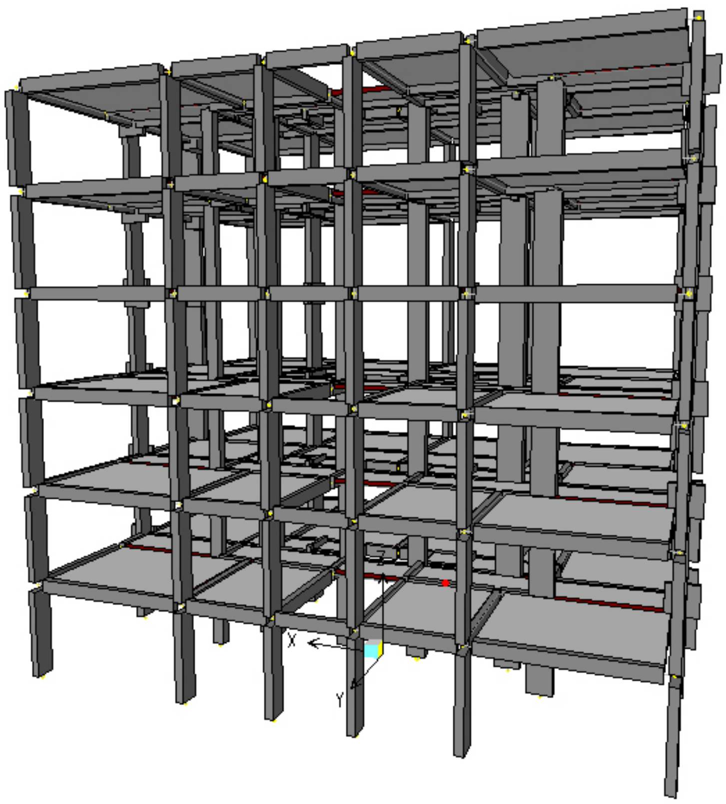


Figure 2.2. 3D-Shape of building model

APPENDIX A : REHABILATED MODERATELY DAMAGED R/C BUILDING

Figure A.1 is the architectural plan of the selecting building which has 6 storeys. By using the architectural plan, the loading calculations are carried out. As it is observe, this structure has no any soft storey condition.

Figure A.2 shows the structural plan of the selecting building. By using the structural plan, the modeling is done in Sap2000. As it is observe from the structural plan, this structure has no any shear walls and some columns shows are distributed unsymetrically.

In Figure A.3, the modeling of the first storey is given. The numeration of joint can be seen from this figure. Especially the location of control joint which is used in pushover analysis can be seen form the figure

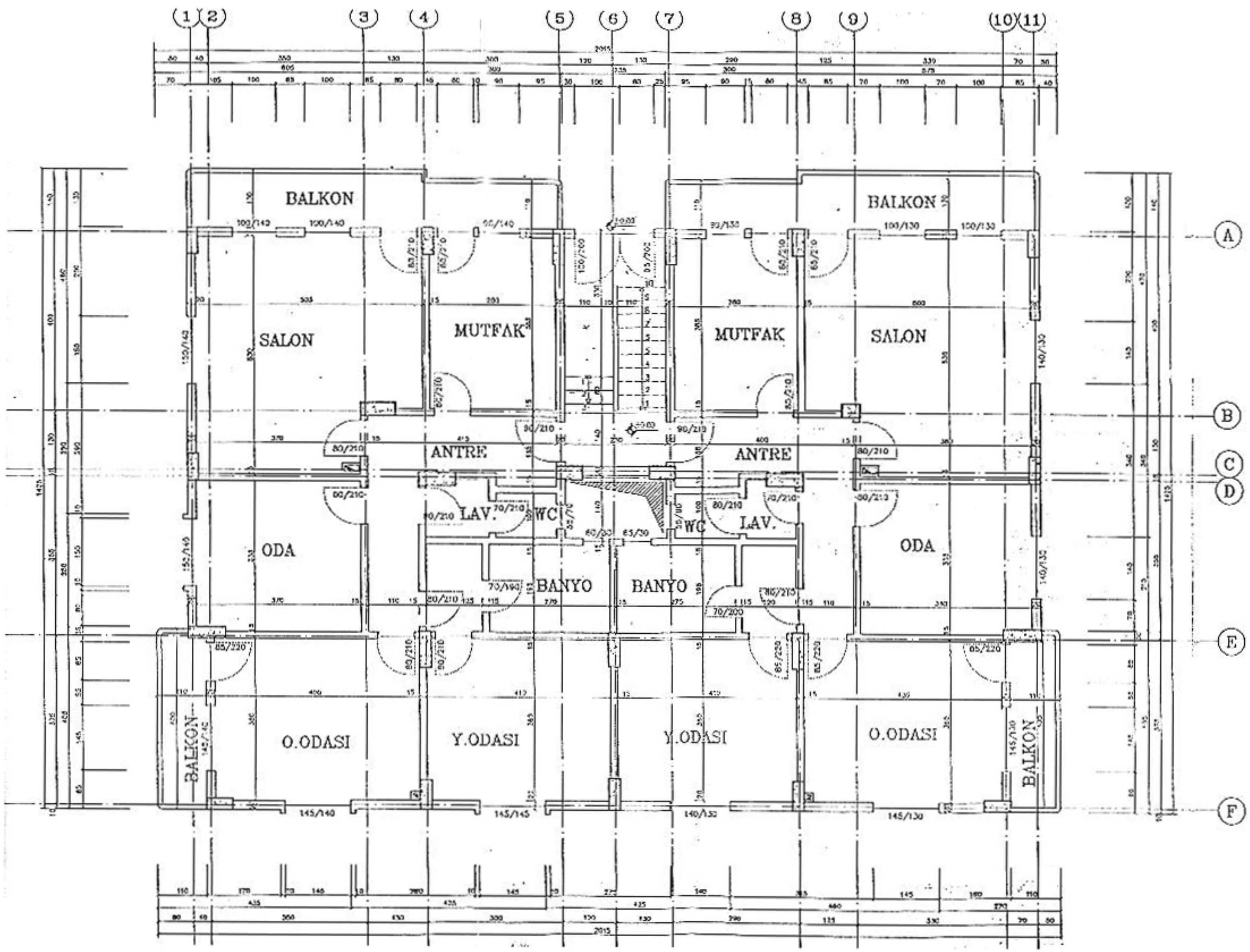


Figure A.1. Architectural Plan

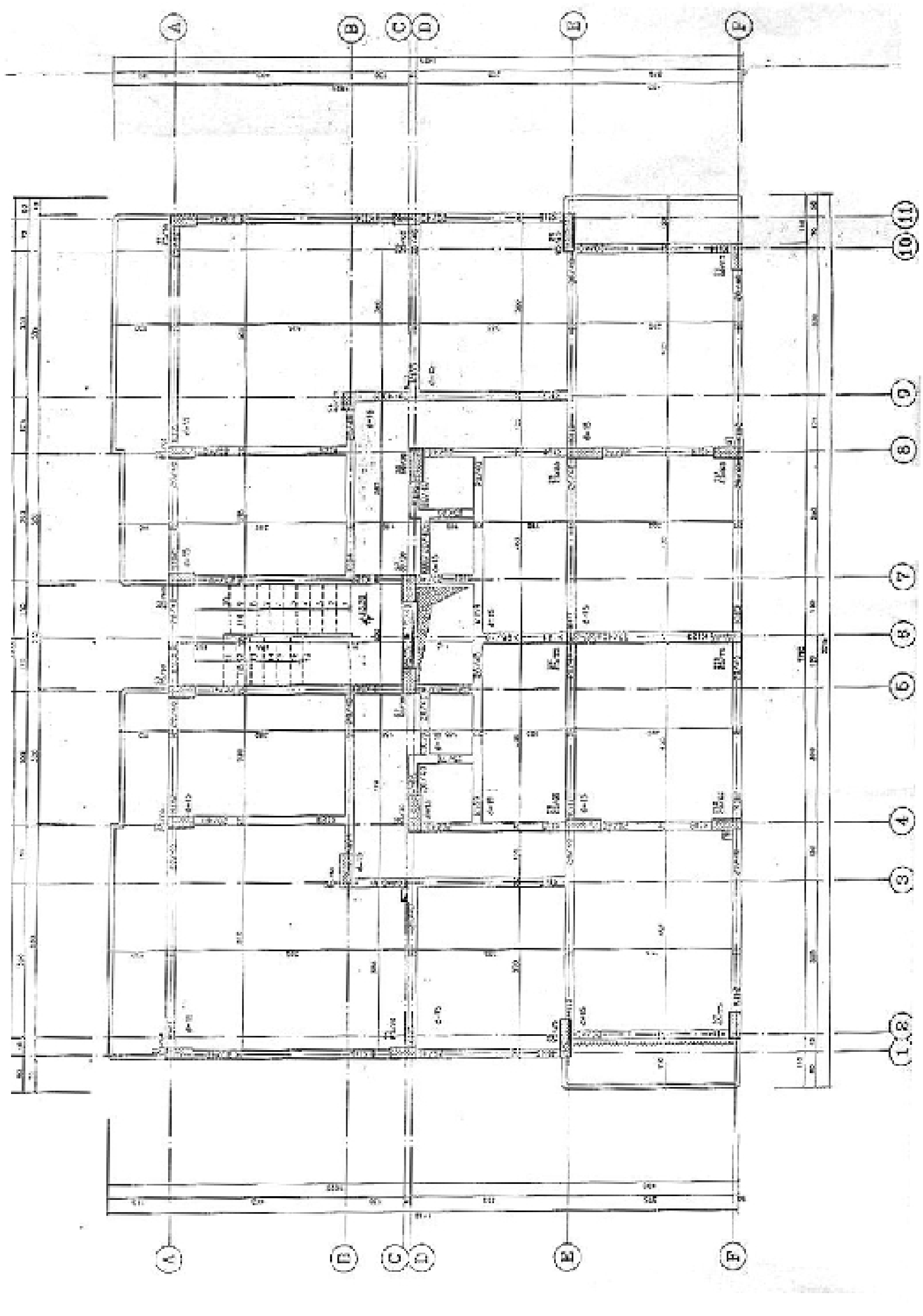


Figure A.2. Structural Plan

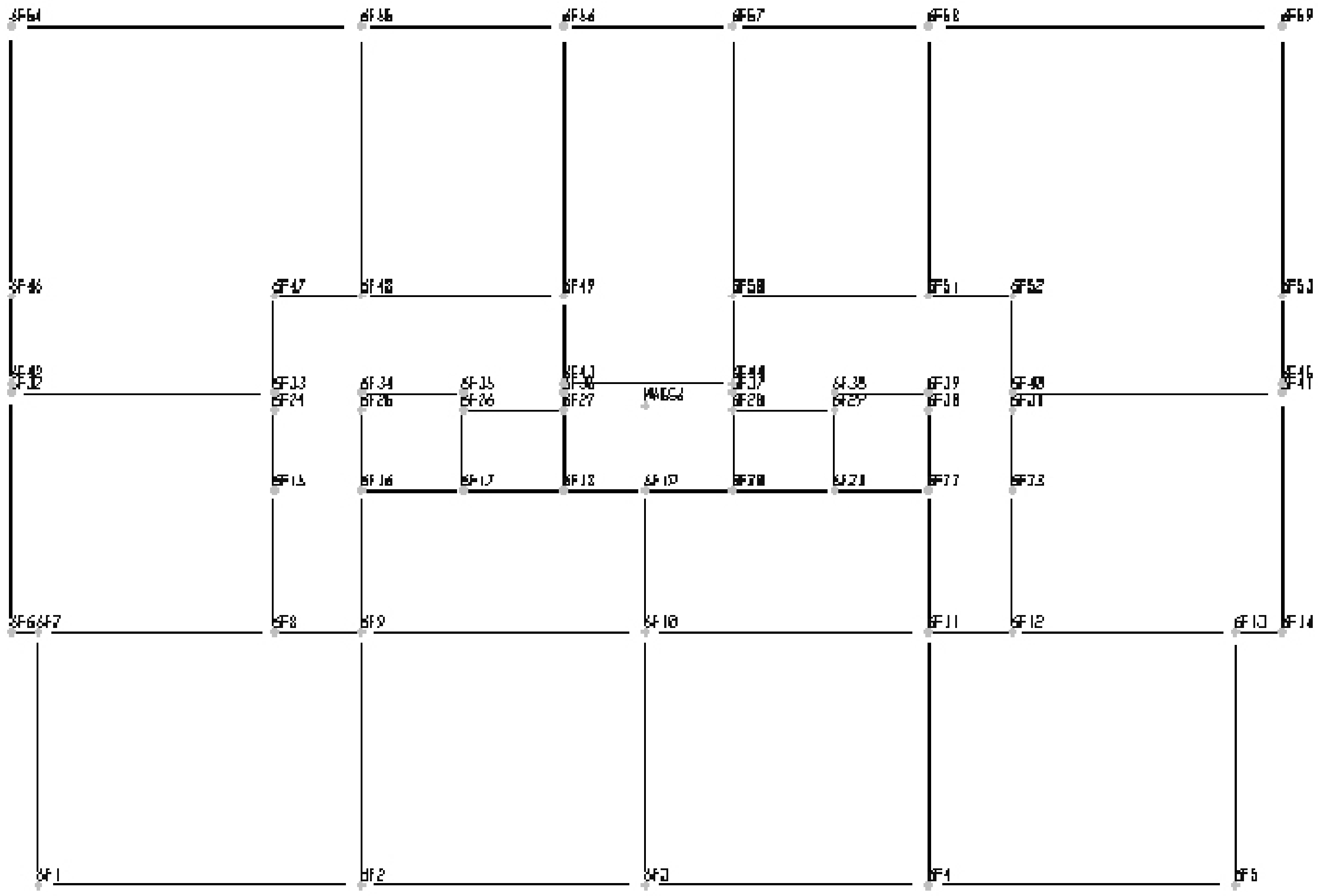


Figure A.3. Modeling Plan in SAP2000

3. LOADING CONDITIONS

3.1. General

The load calculation is carried out according to TS498 and Earthquake Code. On the light of these references, if the given building plans are considered, it can be seen that this structure has no any shear wall. The main structural elements on the plan for dead load calculation are partition walls and reinforced concrete elements such as beams, columns and slabs. This building is used as a residence therefore standard live load should also be included. The calculations for both load cases are given in the following sub topics.

3.2. Dead Load Calculation

For the calculation of the total dead load, the weight of each structural member is calculated separately. The details of weight calculations are as follows:

In order to define the dead load coming from the columns it is necessary firstly to define the net storey height, for that purpose the slab thickness is subtracted from the normal storey height. The storey height calculation is illustrated in Figure 3.1.

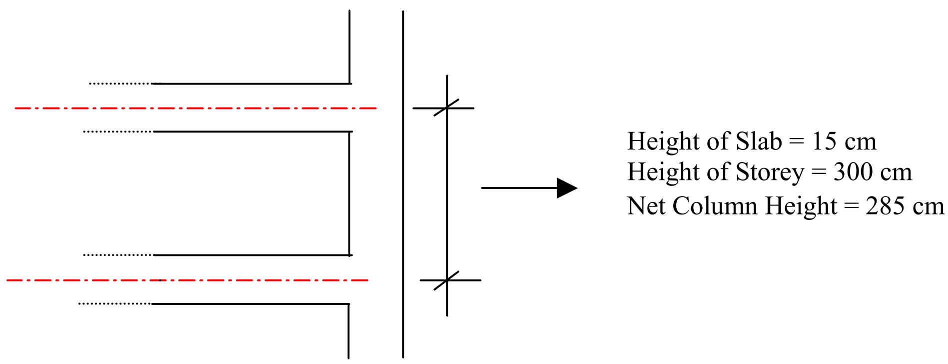


Figure 3.1. Illustration for net storey height

The dimensions and amount of columns in the building for any of the stories are listed in Table 3.1, these values are taken from structural plan.

Table 3.1. Dimensions / amount of columns and weight calculations

AXES	DIMENSIONS (cm)	AMOUNT
F	60/25	4
	65/25	1
	85/25	2
E	80/25	2
	75/25	1
D	25/60	2
	60/30	4
B	75/30	1
	40/30	1
A	25/60	4

TOTAL COLUMN AREA = 3.74 m
TOTAL COLUMN VOLUME = (3.74 x 2.85) = 10.66 m .

WEIGHT OF COLUMNS = 10.66 m x 2.5 ton/m = 26.65 ton For i th floor

The dimensions and amounts of beams in the building for any of the floors is given in Table 3.2.

Table 3.2. Dimensions of beams and weight calculations

AXIS	DIMENSION (cm)	LENGTH (m)
A	40/20	17,70
B	40/20	7,40
D	40/20	17,20
	15/70	4,60
E	40/20	16,70
F	40/20	16,70
1	40/20	7,90
2	40/20	3,50
3	40/20	4,80
4	40/20	9,10
5	40/20	5,00
6	40/20	4,30
7	40/20	5,00
8	40/20	9,10
9	40/20	4,80
10	40/20	3,50
11	40/20	7,90

<u>TOTAL LENGTH</u>

40/20 Beam = 140,60 m
15/70 Beam = 4,60 m

<u>TOTAL VOLUME</u>

140,60 x 0.40 x 0.20 = 11.3 m
4,60 x 0.70 x 0.15 = 0.5 m

<u>TOTAL WEIGHT OF BEAMS</u>

(11.3 + 0.5) x 2.5 = 30 ton For i th floor

The dimensions of slabs in the building for any of the floors can be seen from Architectural plan in Appendix, Figure A.1.

Net SLAB AREA = TOTAL AREA - GAB AREAS

TOTAL AREA = 20.15 x 14.25 = 287 m
GAB AREAS = 2.3 x 1+ 2.3 x 1.3 = 5.29 m

Net SLAB AREA =282 m
TOTAL VOLUME =282 x 0.15 = 42,30 m

WEIGHT OF SLABS = 42,30 m x 2.5 ton/m = 105 ton For i th floor

The thicknesss and lengths of walls with their distribution on the plan for any of the storeys can be seen from Architectural Plan in Appendix, Figure A.1.

OUTSIDE WALL THICKNESS = 20 cm
INSIDE WALL THICKNESS = 10 cm
HEIGHT OF WALL = 285 cm

WALL DENSITY =0.8 ton /m

WEIGHT OF WALLS (37.5 x 0.8 Ton/m) = 30 Ton For i th floor

Total dead load calculations are as follows:

For i th floor 2-3-4-5 FLOORS

TOTAL DEAD LOAD = 26.65 + 105 + 30 + 30 = **192** ton = Wi

For 1th floor

HALF OF COLUMN LOAD + LOAD OF PARTITION WALL + SLAB LOAD

TOTAL DEAD LOAD = 26.65 / 2 + 105 + 30 = **148.3** ton = W1

For 6th floor

HALF OF COL. LOAD + SLAB LOAD + BEAM LOAD

TOTAL DEAD LOAD = 26.65 / 2 + 105 + 30 = **148.3** ton = W6

TOTAL DEAD LOAD OF FLOORS

W1	=	**148,3**	ton
W2	=	**192,0**	ton
W3	=	**192,0**	ton
W4	=	**192,0**	ton
W5	=	**192,0**	ton
W6	=	**148,3**	ton

Metric unit system is used for the analysis part.

Weight : Kgf
Length : cm
Time : sec
Mass : Ton

3.3. Live Load Calculation

The building is a residence therefore the corresponding live load value is taken from TS 498 is equal to 200 kg/m .

The earthquake code states that live load values can be reduced by a specific amount. This concept is explained with details in Part 5. Briefly here, reduction factor according to 5.4.3 is 0.3 for residential buildings and this reduction is taken into live load calculation as follows:

qi = AREA x LL (For Each Floor)
qi = 282 m x 200 kg/m = 56.4 ton

W1 = 148,3 + 0.3 x 56.4 = 165,2 ton **For 1^{th} floor**

Wi = 192 + 0.3 x 56.4 = 209 ton **For i^{th} floor**

W6 = 148,3 ton (No LL on the roof) **For 6^{th} floor**

TOTAL FLOOR WEIGHTS (ton)

W1 = 165.2 ton
W2 = 209.0 ton
W3 = 209.0 ton
W4 = 209.0 ton
W5 = 209.0 ton
W6 = 148.3 ton

TOTAL LOAD OF BUILDING = 1149,5 ton

At the end of the load calculations the distribution of load values for each floor are given in Table 3.3. These values are used in static and dynamic analysis, steps in Part 5 and Part 6.

Table 3.3. Dead load and live load values for each floor

FLOOR	DL		LL	
1	**0,53**	ton / m	**0,2**	ton / m
2	**0,68**	ton / m	**0,2**	ton / m
3	**0,68**	ton / m	**0,2**	ton / m
4	**0,68**	ton / m	**0,2**	ton / m
5	**0,68**	ton / m	**0,2**	ton / m
6	**0,53**	ton / m	**0,0**	ton / m

4. MASS CALCULATION

4.1. General

The mass of the structure is used to compute inertial forces in a dynamic analysis. Normally, the mass is obtained from the elements using the mass density of the material and the volume of the element.

For computational efficiency and solution accuracy, it is better to locate masses at master joints. This means that there is no mass coupling between degrees of freedom at a joint or between different joints. These uncoupled masses are always referred to the local coordinate system of each joint. Mass values along restrained degrees of freedom are ignored.

Inertial forces acting on the joints are related to the accelerations at the joints by a 6x6 matrix of mass values. These forces tend to oppose the accelerations. In a joint local coordinate system, the inertia forces and moments F_1, F_2, F_3, M_1, M_2 and M_3 at a joint are given by:

$$\begin{Bmatrix} F_1 \\ F_2 \\ F_3 \\ M_1 \\ M_2 \\ M_3 \end{Bmatrix} = - \begin{pmatrix} u1 & 0 & 0 & 0 & 0 & 0 \\ & u2 & 0 & 0 & 0 & 0 \\ & & u3 & 0 & 0 & 0 \\ & & & r1 & 0 & 0 \\ & & & & r2 & 0 \\ & & & & & r3 \end{pmatrix} \begin{pmatrix} \ddot{u}_1 \\ \ddot{u}_2 \\ \ddot{u}_3 \\ \ddot{r}_1 \\ \ddot{r}_2 \\ \ddot{r}_3 \end{pmatrix}$$

Where $\ddot{u}_1$, $\ddot{u}_2$, $\ddot{u}_3$, $\ddot{r}_1$, $\ddot{r}_2$, and $\ddot{r}_3$ are the translation and rotation accelerations at the joint, and the terms u1, u2, u3, r1, r2 and r3 are the specified mass values.

Mass values must be given in consistent mass units (W/g) and mass moments of inertia must be in WL /g units. Here W is weight, L is length, and g is the acceleration.

The used mass moment of inertia formula is given in Figure 4.1

Figure 4.1. Mass moment of inertia about vertical axis

Although Sap2000 calculates the masses and location of the master joints, mass values are calculated in Part 3 and location of master joints are calculated below.

4.2. Center of Mass

The floor plan of the structure is almost symmetric with some exceptions about both of X and Y axis. The plan view is divided into three parts for the calculation of the mass center location (i.e. Master Joint Location). The simplified model for this purpose is used and given in Figure 4.2.

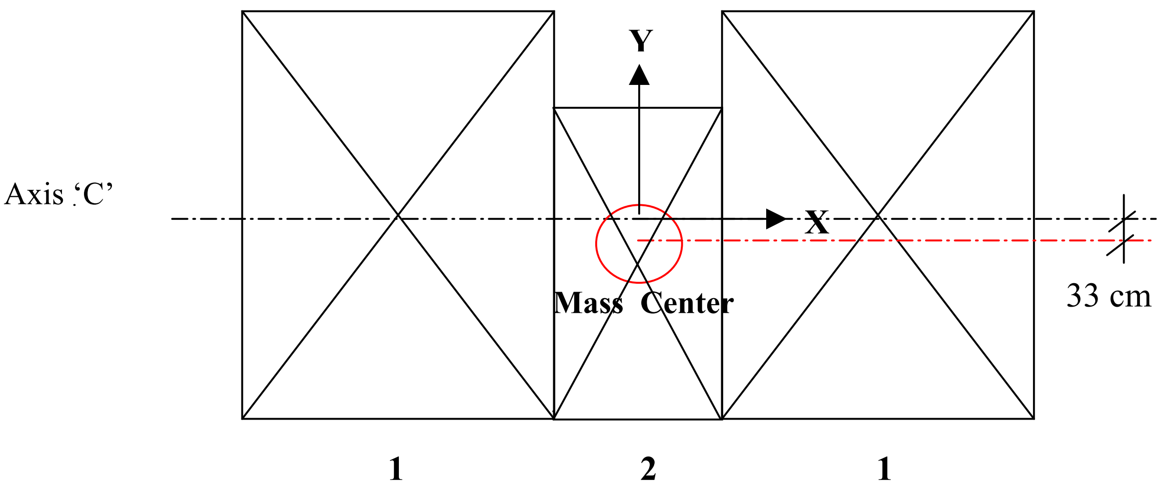


Figure 4.2. Simplified floor plan for mass center determination

M = Total mass of floor

M = m1 +m2 + m3 (From Figure 4.2)

m1 = 0.42 M
m2 = 0.16 M
m3 = 0.42 M

The mass center is found by taking the moment of inertia of simplified areas with respect to the bottom line on Figure 4.3.

4.3. Mass Moment of Inertia

For the calculation of the mass moment of inertia, (4.1) is used; this formulation is taken from Figure 4.1.

$$MMI = 1 / 12 (b + h) \qquad (4.1)$$

MMI = Mass Moment of Inertia

b = Length one side in the X-direction in meters

h = Length one side in the Y-direction in meters

M = Total Mass in tons

d = Transferred Distance in meters

For the transformation of MMI in to another axis is calculated according to (4.2).

$$MMI = MMI + M \times d \qquad (4.2)$$

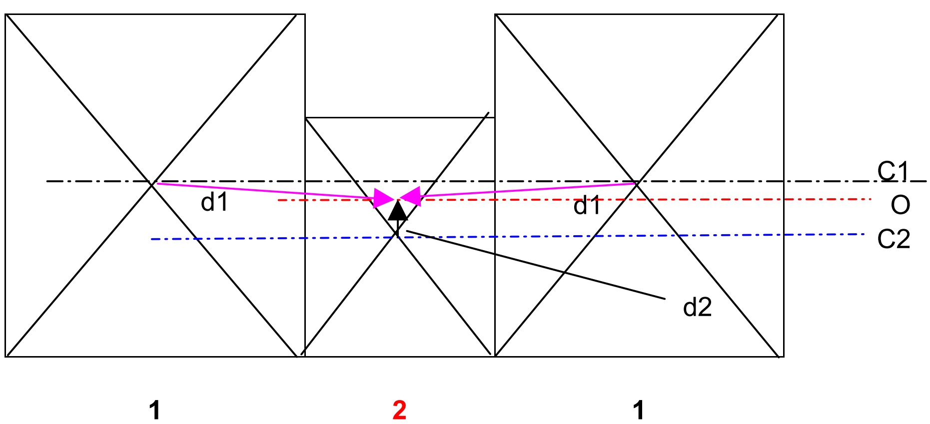


Figure 4.3. Simplified floor plan for MMI calculation

<u>For Section -1- in Figure 4.3</u>

$$MMI = 1/12 \times (14.25 \quad + 8.3 \quad) \times M1$$

$$MMI = 22.61\ M1$$

<u>For Section -2- in Figure 4.3</u>

$$MMI = 1/12 \times (13 \quad +2.5 \quad) \times M2$$

$$MMI = 14.6\ M2$$

<u>For Section -3- in Figure 4.3</u>

$$MMI = 1/12 \times (14.25 \quad + 8.3 \quad) \times M3$$

$$MMI = 22.61\ M3$$

After the each MMI of the three sections is transformed to the mass center axis, the total mass moment of inertia is calculated as follows:

$$56.655 \times M1 + 15.81\ M2 + 56.655 \times M3$$

$$M = M1 + M2 + M3$$

$$47.46\ M + 2.53\ M = 50\ M$$

Here, M = Total mass of the floor.

The mass for each floor is calculated according to above formulation and listed in Table 4.1

Table 4.1. Calculated mass values for each floor

FLOOR	TOTAL LOAD	TOTAL MASS	MMI
1	165,2 ton	16,84 ton/m.sec²	842,0 ton.m/sec²
2	209 ton	21,30 ton/m.sec²	1065,2 ton.m/sec²
3	209 ton	21,30 ton/m.sec²	1065,2 ton.m/sec²
4	209 ton	21,30 ton/m.sec²	1065,2 ton.m/sec²
5	209 ton	21,30 ton/m.sec²	1065,2 ton.m/sec²
6	148,3 ton	15,12 ton/m.sec²	755,9 ton.m/sec²

5. STATIC ANALYSIS

5.1. General

Static, dynamic and pushover analysis are used to determine the response of structure to various loading cases. For static and dynamic loading cases, 1998 earthquake code is used as a basic reference.

The static analysis of a structure involves the solution of the system of linear equations represented by:

$$K\ u = r \tag{5.1}$$

Where K is the stiffness matrix, r is the vector of applied loads, and u is the vector of resulting displacements [3].

For each defined load case, the analysis program automatically creates the load vector r and solves the static displacements u.

In the first step of analysis, the seismic loads are calculated and in the next steps, on the basis of earthquake code, static and dynamic loading are applied to the model structure by using two methods which are described in the proceeding pages.

5.2. Definition of Elastic Seismic Loads

The *Spectral Acceleration Coefficient*, A(T), corresponding to 5% damped elastic *Design Acceleration Spectrum* normalized by the acceleration of gravity, g, is given by (5.2) which shall be considered as the basis for the determination of seismic loads [4].

$$A(T) = A_o\ I\ \ S(T) \tag{5.2}$$

The *Effective Ground Acceleration Coefficient*, A_o, appearing in (5.2) is specified in Table 5.1.

Table 5.1. Effective ground acceleration coefficient [4]

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

The model structure is located in Seismic Zone-1 (Refer to Part 1.7.1) , therefore the corresponding effective ground acceleration A_o value is;

$$A_o = 0.40$$

The *Building Importance Factor*, I, appearing in (5.2) is specified in Table 5.2 [4].

Table 5.2. Building importance factor [4]

Purpose of Occupancy or Type of Building	***Importance Factor* (I)**
1. Buildings to be utilized after the earthquake and buildings containing hazardous materials **a)** Buildings required to be utilized immediately after the earthquake (Hospitals, dispensaries, health wards, fire fighting buildings and facilities, PTT and other telecommunication facilities, transportation stations and terminals, power generation and distribution facilities; governorate, county and municipality administration buildings, first aid and emergency planning stations) **b)** Buildings containing or storing toxic, explosive and flammable materials, etc.	**1.5**
2. Intensively and long-term occupied buildings and buildings preserving valuable goods **a)** Schools, other educational buildings and facilities, dormitories and hostels, military barracks, prisons, etc. **b)** Museums	**1.4**
3. Intensively but short-term occupied buildings Sport facilities, cinema, theatre and concert halls, etc.	**1.2**
4. Other buildings Buildings other than above defined buildings. (**Residential** and office buildings, hotels, building-like industrial structures, etc.)	**1.0**

As it is stated in the introduction part, the model structure is used for residential purposes, according to Table 5.2, the value of building importance factor is as follows;

$$I = 0.40$$

The *Spectrum Coefficient*, S(T), appearing in (5.2) shall be determined by Eqs.(5.3), depending on the local site conditions and the building natural period, T (Fig. 5.1):

$$S(T) = 1 + 1.5\, T / T_A \qquad (0 \leq T \leq T_A) \qquad (5.3a)$$

$$S(T) = 2.5 \qquad (T_A < T \leq T_B) \qquad (5.3b)$$

$$S(T) = 2.5\, (T_B / T)^{0.8} \qquad (T > T_B) \qquad (5.3c)$$

Spectrum Characteristic Periods, T_A and T_B , appearing in (5.3) are specified in Table 5.5, depending on *Local Site Classes* given in Table 5.3 and *Soil groups* which is given in Table 5.4.

As it is defined in Part 1.7.1 soil group for given structure is Group C and corresponding local site class can be taken from Table 5.4.

Table 5.3. Table of local site classes [4]

Local Site Class	***Soil Group according to Table 12.1 and Topmost Layer Thickness* (h_1)**
Z1	Group **(A)** soils Group **(B)** soils with $\mathbf{h_1 \leq 15\ m}$
Z2	Group **(B)** soils with $\mathbf{h_1 > 15\ m}$ Group **(C)** soils with $\mathbf{h_1 \leq 15\ m}$
Z3	Group **(C)** soils with $\mathbf{15\ m < h_1 \leq 50\ m}$ Group **(D)** soils with $\mathbf{h_1 \leq 10\ m}$
Z4	Group **(C)** soils with $\mathbf{h_1 > 50\ m}$ Group **(D)** soils with $\mathbf{h_1 > 10\ m}$

According to Group C soil group and from the Table 5.3, the local site class of the given structure is Z3.

Table 5.4. Table of soil groups in EQ code [4]

Soil Group	***Description of Soil Group***	***Stand. Penetr.* (N/30)**	***Relative Density* (%)**	***Unconf. Compres. Strength* (kPa)**	***Shear Wave Velocity* (m/s)**
(A)	**1.** Massive volcanic rocks, unweathered sound metamorphic rocks, stiff cemented sedimentary rocks **2.** Very dense sand, gravel... **3.** Hard clay, silty lay……..	**—** **> 50** **> 32**	**—** **85–100** **—**	**> 1000** **—** **> 400**	**> 1000** **> 700** **> 700**
(B)	**1.** Soft volcanic rocks such as tuff and agglomerate, weathered cemented sedimentary rocks with planes of discontinuity…… **2.** Dense sand, gravel......... **3.** Very stiff clay, silty clay..	**—** **30–50** **16–32**	**—** **65–85** **—**	**500–1000** **—** **200–400**	**700–1000** **400–700** **300–700**
(C)	**1.** Highly weathered soft metamorphic rocks and cemented sedimentary rocks with planes of discontinuity **2.** Medium dense sand and gravel.....………………… **3.** Stiff clay, silty clay.........	**—** **10–30** **8–16**	**—** **35–65** **—**	**< 500** **—** **100–200**	**400–700** **200–400** **200–300**
(D)	**1.** Soft, deep alluvial layers with high water table....…… **2.** Loose sand..................…. **3.** Soft clay, silty clay....…..	**—** **< 10** **< 8**	**—** **< 35** **—**	**—** **—** **< 100**	**< 200** **< 200** **< 200**

Table 5.5. Table of local site classes corresponds to spectrum periods [4]

Local Site Class acc. to Table 12.2	**T_A *(second)***	**T_B *(second)***
Z1	**0.10**	**0.30**
Z2	**0.15**	**0.40**
Z3	**0.15**	**0.60**
Z4	**0.20**	**0.90**

Thus the corresponding spectrum periods from Table 5.5 in earthquake code are as follows:

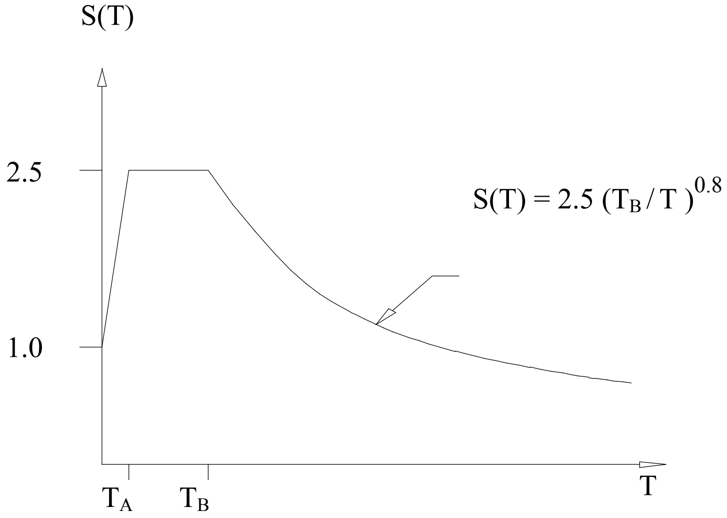


Figure 5.1. Special Design Acceleration Spectra

$$\left.\begin{array}{l} T_A = 0.15 \text{ sec} \\ T_B = 0.60 \text{ sec} \end{array}\right\} \quad \text{For Z3 type of soil.}$$

According to these periods, the obtained spectrum values are as follows:

$$S(T) = 1 + 1.5\, T / T_A \qquad (0 \leq T \leq 0.15)$$

$$S(T) = 2.5 \qquad (T_A < T \leq 0.60)$$

$$S(T) = 2.5\, (T_B / T\,)^{0.8} \qquad (T > 0.60)$$

With the light of these formulations, obtained 'Special Design Acceleration Spectra ' is given in Figure 5.2.

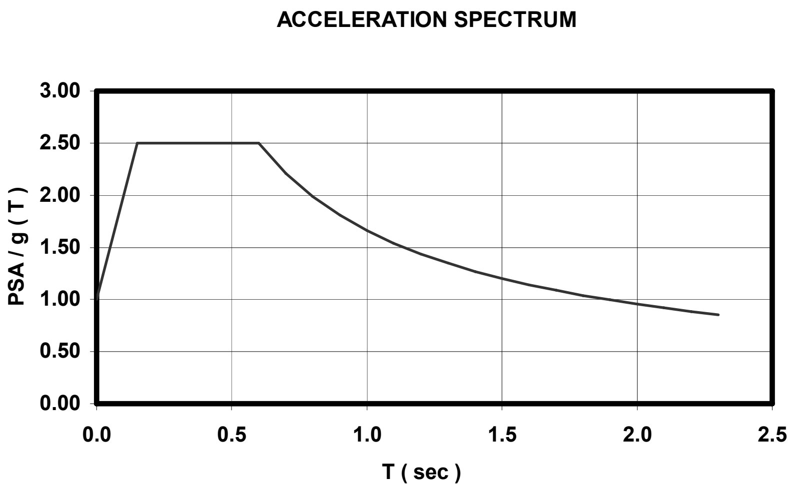


Figure 5.2. Special Design Acceleration Spectra for model structure

When required, elastic acceleration spectrum may be determined through special investigations by considering local seismic and site conditions. However spectral acceleration coefficients corresponding to obtained acceleration spectrum ordinates shall in no case be less than those determined by (5.2) based on relevant characteristic periods TA, TB [4].

5.3. Reduction of Elastic Seismic Loads

Elastic seismic loads to be determined in terms of *spectral acceleration coefficient* defined in equation $A(T) = A_o\ I\ \ S(T)$ shall be divided to below defined *Seismic Load Reduction Factor* to account for the specific nonlinear behavior of the structural system during earthquake.

Seismic Load Reduction Factor, $R_a(T)$, shall be determined by (5.4) here in terms of *Structural Behavior Factor*, R, defined in Table 5.6 below for various structural systems, and the natural vibration period T .

$$R_a(T) = 1.5 + (R - 1.5)\ T / T_A \quad (0 \le T \le T_A) \tag{5.4a}$$

$$R_a(T) = R \qquad (T > T_A) \tag{5.4b}$$

Table 5.6. Table of building structural systems

BUILDING STRUCTURAL SYSTEM	***Systems of Nominal Ductility Level***	***Systems of High Ductility Level***
<u>(1) CAST-IN-SITU REINFORCED CONCRETE BUILDINGS</u>		
(1.1) Buildings in which seismic loads are fully resisted by frames..	**4**	**8**
(1.2) Buildings in which seismic loads are fully resisted by coupled structural walls...	**4**	**7**
(1.3) Buildings in which seismic loads are fully resisted by solid structural walls...	**4**	**6**
(1.4) Buildings in which seismic loads are jointly resisted by frames and solid and/or coupled structural walls...........	**4**	**7**

The model structure has higher ductility level and all seismic loads are carried by cast-in-situ reinforced concrete frames. Therefore the value of corresponding Structural *Reduction Factor* is as follows;

$$R = 8 \text{ (From Table 5.6)}$$

$$R_a(T) = 1.5 + 6.5T / 0.15 \qquad (0 \le T \le 0.15 \text{ sec})$$

$$R_a(T) = 8 \qquad (T > 0.15 \text{ sec})$$

5.4. Selection of Analysis Method

Methods to be used for the seismic analysis of buildings and building-like structures are, *Equivalent Seismic Load Method* given in 5.5, *Mode-Superposition Method* given in 6.5.

Buildings for which *Equivalent Seismic Load Method* given in 5.5 is applicable are summarized in Table 5.7.

Table 5.7. Table of building types for selection of analysis

Seismic Zone	***Type of Building***	***Total Height Limit***
1, 2	Buildings without type **A1** torsional irregularity, or those satisfying the condition $\boldsymbol{\eta_{bi} \le 2.0}$ at every storey	$\boldsymbol{H_N \le 25}$ **m**
1, 2	Buildings without type **A1** torsional irregularity, or those satisfying the condition $\boldsymbol{\eta_{bi} \le 2.0}$ at every storey and at the same time without type **B2** irregularity	$\boldsymbol{H_N \le 60}$ **m**
3, 4	All buildings	$\boldsymbol{H_N \le 75}$ **m**

The model structure has a height of less than 25 m, located in the seismic zone of 1 although it has A1 type of irregularity and the constraints of $\eta_{bi} \le 2.0$ at every storey is satisfied (Explained in Part 5.7). Therefore Equivalent *Seismic Load Method can* be safely applied to the given structure.

5.5. Equivalent Seismic Load Method

Total Equivalent Seismic Load (Total Base Shear), V_t, acting on the entire building in the earthquake direction considered shall be determined by (5.5) [4]. The model view for total base shear is given in Figure 5.4.

$$V_t = W\,A(T_1) / R_a(T_1) \geq 0.10\,A_o\,I\,W \qquad (5.5)$$

Here in (5.5) the first natural vibration period of the building, T_1, shall be calculated below. The first natural vibration period, which is permitted, is calculated by the approximate method given here for buildings with $H_N \leq 25$ m in the first and second seismic zones. This expression is given in (5.6) [4].

$$T_1 \cong T_{1A} = C_t\,H_N^{\,3/4} \qquad (5.6)$$

Since the height of the model structure is 18 m and less than 25 m, (5.6) is used for the determination of the first natural period. Values of C_t in (5.6) are defined below depending on the building structural system:

- The value of C_t shall be calculated in (5.7) for buildings where seismic loads are fully resisted by reinforced concrete structural walls [4].

$$C_t = 0.075 / A_t^{1/2} \leq 0.05 \qquad (5.7)$$

Formulation of equivalent area A_t is given in (5.8) below where the maximum value of (ℓ_{wj}/H_N) shall be taken equal to 0.9 [4].

$$A_t = \sum_{J} A_{wj}\,[0.2 + (\ell_{wj} / H_N)^2] \qquad (5.8)$$

- It shall be $C_t = 0.07$ for buildings whose structural system are composed only of reinforced concrete frames or structural steel eccentric braced frames, $C_t = 0.08$ for buildings made only of steel frames, $C_t = 0.05$ for all other buildings [4].

Since our system contains no any shear walls and structural system is only composed of reinforced concrete frames the value of C_t is equal to 0.07.

Spectral acceleration coefficient formulation and seismic load reduction factor formulations are given below according to (5.2) and (5.4)

$$A(T) = A_o \; I \; S(T) \tag{5.2}$$

$$Ao = 0.40$$
$$I = 1.0$$
$$S(T) = \text{calculated acc. spectrum}$$
$$A(T) = 0.40xS(T)$$

$$R_a(T) = 1.5 + 6.5T / 0.15 \qquad (0 \leq T \leq 0.15 \text{ sec}) \tag{5.4a}$$

$$R_a(T) = 8 \qquad (T > 0.15 \text{ sec}) \tag{5.4b}$$

$$\text{Since } T1 > 0.15 \text{ sec} \Rightarrow \quad R_a(Tr) = 8$$

After defining C_t value the first natural period of the structure is calculated by (5.6) then by using previously obtained acceleration spectra given in Figure 5.3, the corresponding acceleration value to the first natural period is obtained.

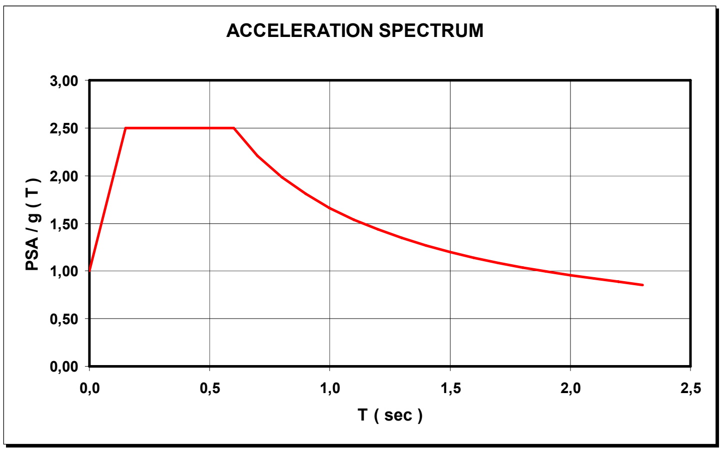


Figure 5.3. Special Design Acceleration Spectra for model structure

Total building weight, W, to be used in (5.5) shall be determined as follows;

$$T_1 \cong T_{1A} = C_t\, H_N^{3/4}$$

$$H_N \leq 25 \text{ m} \quad \text{and } H_N = 18 \text{ m}$$

$$C_t = 0.07$$

$$T_1 \cong T_{1A} = 0.611$$

S (T) = 2.46 According To Spectrum

$$A(T_1) = A_o\, I\; S(T)$$

$$A(T_1) = 0.984$$

$$W = \sum_{i=1}^{N} w_i \qquad (5.9)$$

Storey weights w_i shall be calculated in 5.10

$$w_i = g_i + n\, q_i \qquad (5.10)$$

Live Load Participation Factor, n is given in Table 5.8

Table 5.8. Table of purpose occupancy of building

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

Since the model structure is used for residential purposes the live load participation factor is 0.3 according to Table 5.8 and the corresponding calculation is carried out as follows;

qi = AREA x LL (For Each Floor)
qi = 282 m² x 200 kg/m² = 56.4 ton

W1 = 148,3 + 0.3 x 56.4 = 165,2 ton **For 1th floor**

Wi = 192 + 0.3 x 56.4 = 209 ton **For ith floor**

W6 = 148,3 ton (No LL on the roof) **For 6th floor**

FLOOR	WEIGHTS	
W1 =	**165,2**	ton
W2 =	**209,0**	ton
W3 =	**209,0**	ton
W4 =	**209,0**	ton
W5 =	**209,0**	ton
W6 =	**148,3**	ton

TOTAL LOAD OF BUILDING **= 1150** ton

Total equivalent seismic load determined by (5.5) is expressed in (5.11) as the sum of equivalent seismic loads acting at storey levels.

$$V_t = W\,A(T_1) / R_a(T_1) \geq 0.10\,A_o\,I\,W \qquad (5.5)$$

$$V_t = \Delta F_N + \sum_{i=1}^{N} F_i \qquad (5.11)$$

Since $H_N < 25$ m, *additional equivalent seismic load*, ΔF_N, acting at the N'th storey (top) of the building shall be taken as $\Delta F_N = 0$. Remaining part of the total equivalent seismic load shall be distributed to stories of the building (including N'th storey) in accordance with the equation given below (5.12).

$$F_i = (V_t - \Delta F_N)\,\frac{w_i\,H_i}{\sum_{j=1}^{N} (w_j\,H_j)} \qquad (5.12)$$

$$V_t = W\,A(T_1) / R_a(T_1) \geq 0.10\,A_o\,I\,W \qquad (5.5)$$

W = 1150 ton (Total Weight of Building Including LL) from 5.4.3

$V_t = 1150 \times 0.984 / 8 \geq 0.10 \times 0.40 \times 1.0 \times 1150$

$V_t = 141.45$ ton ≥ 46 ton OK √

Total Equivalent Seismic Load calculations are carried out as follows;*Total Equivalent Seismic Load* is distributed to storey levels as shown in Figure 5.4 by using the (5.12) and the resultant values are tabulated in Table 5.9.

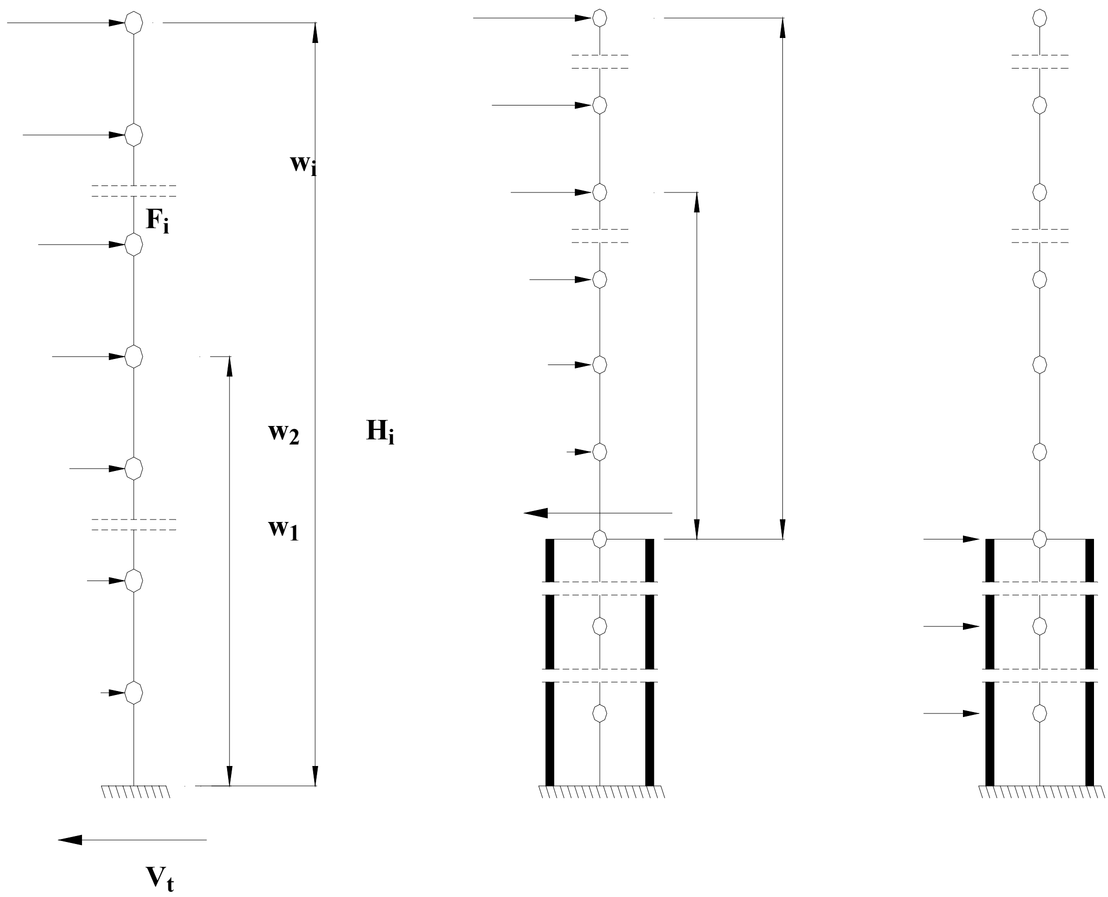


Figure 5.4. Total equivalent seismic load distribution to storey levels

$$H_N = 18 \text{ m} < 25 \text{ m} \Rightarrow \Delta F_N = 0.0$$

$$F_i = (141.45) \frac{w_i H_i}{\sum_{j=1}^{N} (w_j H_j)} \tag{5.12}$$

Table 5.9. Total EQ load values for each floor

FLOOR	Wi (ton)		H (m)	Hi x Wi	EQ LOAD (ton)
1	165,2	Ton	3,0	495,6	5,87 Ton
2	209	Ton	6,0	1254,0	14,85 ton
3	209	Ton	9,0	1881,0	22,28 ton
4	209	Ton	12,0	2508,0	29,70 ton
5	209	Ton	15,0	3135,0	37,13 Ton
6	148,3	Ton	18,0	2669,4	31,62 Ton
TOTAL	1150,0 Ton			11943,0	141,45 Ton

5.6. Analysis Procedure

The details of the modeling are explained in Part 2, dead loads and live loads are applied to shell elements and earthquake loads which are tabulated in Table 5.9 are applied to master joints of model. Load combination used in the analysis is as follows [5];

Load Combination = 1DL + 1LL + 1EQ

In the program stage this combination is called as ‘ COMB1 ‘, by using this load combination static analysis is carried out, the deformed shapes of the static analysis are given below in Figure 5.5and 5.6.

As it is given in deformed shapes, in X and Y directions lateral disturbance are appearing. This is an expected result since structure seems symmetric however not purely symmetric due to unsymmetrical distribution of the some columns.

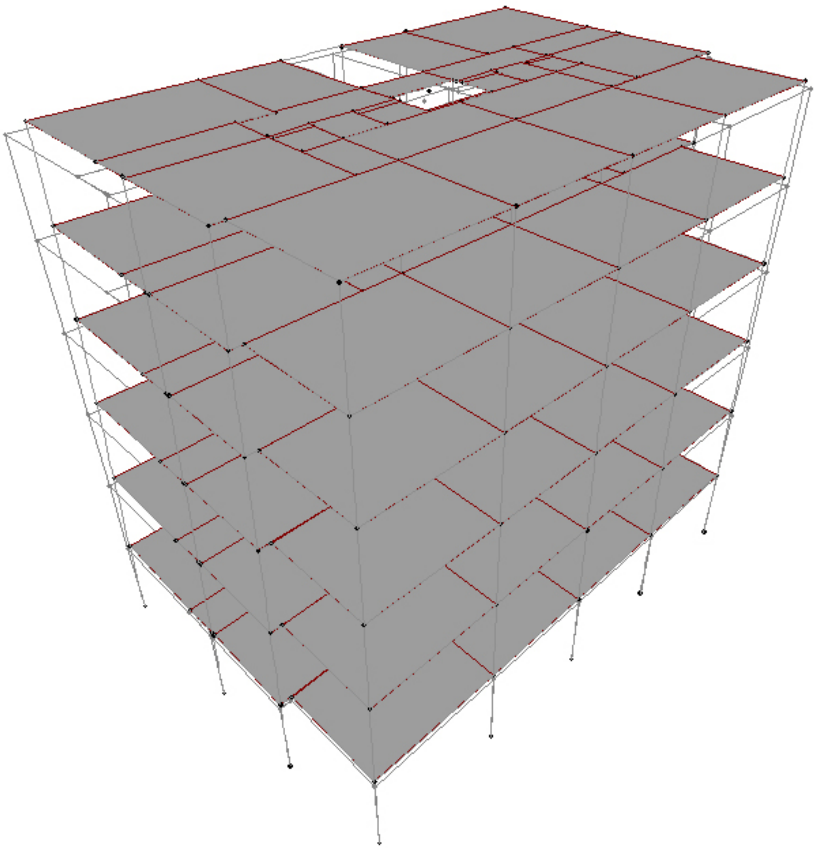


Figure 5.5. Elastic Analysis, 3D deformed Shape

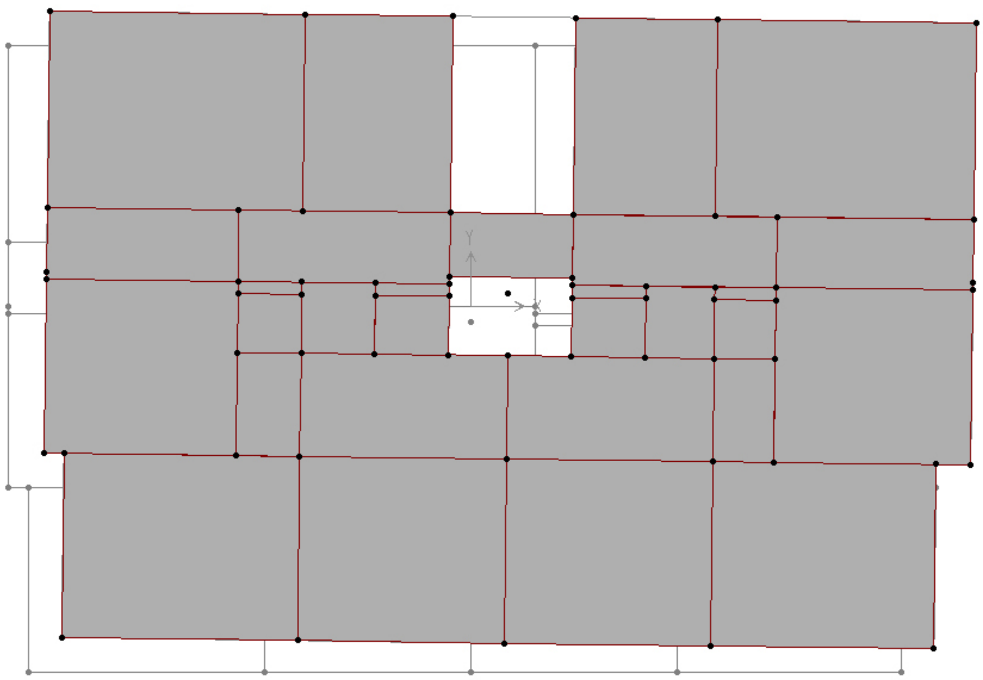


Figure 5.6. Elastic Analysis, Top floor deformed Shape

5.7. Defining of Irregularities in Plan

Since torsion seems critical due to asymmetry in the given building *A1 type* of irregularity is checked below according to Earthquake Code requirements.

The case where *Torsional Irregularity Factor* η_{bi}, 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, is greater than 1.2, which is illustrated in Figure 5.7.

$$[\eta_{bi} = (\Delta_i)_{max} / (\Delta_i)_{ort} > 1.2] \quad (5.12)$$

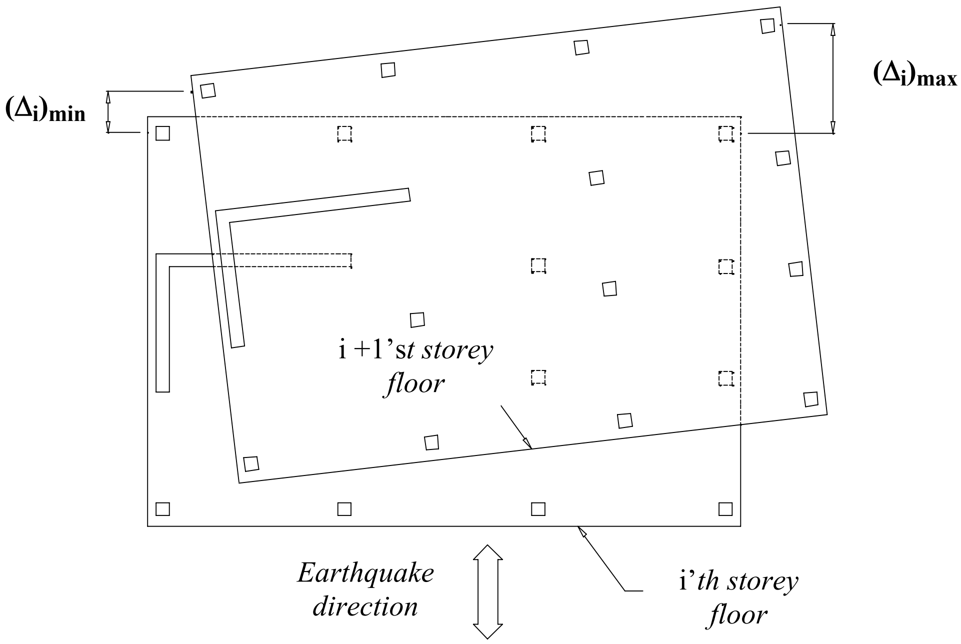


Figure 5.7. Shape of A1 type torsional irregularity

Maximum drift difference happens between 1th and 2th floors. The calculations are given in Table 5.10.

The unit of values is cm and obtained η_{bi} value for Y direction is greater than 1,2 as a result of this calculation, for this building it can be said that it has A1 type of torsional irregularity.

Table 5.10. Maximum drift differences between 1^{th} and 2^{th} floors

JOINT	UX	UY	JOINT	UX	UY	DRIFT DIFF.X	DRIFT DIFF.Y
1F1	0,4085	0,4785	2F1	1,1611	1,2858	0,7526	0,8073
1F2	0,4085	0,4321	2F2	1,1611	1,1663	0,7526	0,7342
1F3	0,4085	0,3916	2F3	1,1611	1,0618	0,7526	0,6702
1F4	0,4085	0,351	2F4	1,1611	0,9573	0,7526	0,6063
1F5	0,4085	0,3071	2F5	1,1611	0,8441	0,7526	0,5370
1F6	0,4448	0,4823	2F6	1,2544	1,2958	0,8096	0,8135
1F7	0,4448	0,4785	2F7	1,2544	1,2858	0,8096	0,8073
1F8	0,4448	0,4447	2F8	1,2544	1,1987	0,8096	0,7540
1F9	0,4448	0,4321	2F9	1,2544	1,1663	0,8096	0,7342
1F10	0,4448	0,3916	2F10	1,2544	1,0618	0,8096	0,6702
1F11	0,4448	0,3510	2F11	1,2544	0,9573	0,8096	0,6063
1F12	0,4448	0,3389	2F12	1,2544	0,9262	0,8096	0,5873
1F13	0,4448	0,3071	2F13	1,2544	0,8441	0,8096	0,5370
1F14	0,4448	0,3003	2F14	1,2544	0,8266	0,8096	0,5263
1F15	0,4650	0,4447	2F15	1,3067	1,1987	0,8417	0,7540
1F16	0,4650	0,4321	2F16	1,3067	1,1663	0,8417	0,7342
1F17	0,4650	0,4176	2F17	1,3067	1,1290	0,8417	0,7114
1F18	0,4650	0,4031	2F18	1,3067	1,0917	0,8417	0,6886
1F19	0,4650	0,3916	2F19	1,3067	1,0618	0,8417	0,6702
1F20	0,4650	0,3790	2F20	1,3067	1,0295	0,8417	0,6505
1F21	0,4650	0,3645	2F21	1,3067	0,9921	0,8417	0,6276
1F22	0,4650	0,3510	2F22	1,3067	0,9573	0,8417	0,6063
1F23	0,4650	0,3389	2F23	1,3067	0,9262	0,8417	0,5873
1F24	0,4766	0,4447	2F24	1,3366	1,1987	0,8600	0,7540
1F25	0,4766	0,4321	2F25	1,3366	1,1663	0,8600	0,7342
1F26	0,4766	0,4176	2F26	1,3366	1,1290	0,8600	0,7114
1F27	0,4766	0,4031	2F27	1,3366	1,0917	0,8600	0,6886
1F28	0,4766	0,3790	2F28	1,3366	1,0295	0,8600	0,6505
1F29	0,4766	0,3645	2F29	1,3366	0,9921	0,8600	0,6276
1F30	0,4766	0,3510	2F30	1,3366	0,9573	0,8600	0,6063
1F31	0,4766	0,3389	2F31	1,3366	0,9262	0,8600	0,5873
1F32	0,4790	0,4823	2F32	1,3428	1,2958	0,8638	0,8135
1F33	0,4790	0,4447	2F33	1,3428	1,1987	0,8638	0,7540
1F34	0,4790	0,4321	2F34	1,3428	1,1663	0,8638	0,7342
1F35	0,4790	0,4176	2F35	1,3428	1,1290	0,8638	0,7114
1F36	0,4790	0,4031	2F36	1,3428	1,0917	0,8638	0,6886
1F37	0,4790	0,3790	2F37	1,3428	1,0295	0,8638	0,6505
1F38	0,4790	0,3645	2F38	1,3428	0,9921	0,8638	0,6276
1F39	0,4790	0,3510	2F39	1,3428	0,9573	0,8638	0,6063
1F40	0,4790	0,3389	2F40	1,3428	0,9262	0,8638	0,5873
1F41	0,4790	0,3003	2F41	1,3428	0,8266	0,8638	0,5263
1F42	0,4805	0,4823	2F42	1,3465	1,2958	0,8660	0,8135
1F43	0,4805	0,4031	2F43	1,3465	1,0917	0,8660	0,6886
1F44	0,4805	0,3790	2F44	1,3465	1,0295	0,8660	0,6505
1F45	0,4805	0,3003	2F45	1,3465	0,8266	0,8660	0,5263

Table 5.10 Continued

1F46	0,4930	0,4823	2F46	1,3789	1,2958	0,8859	0,8135
1F47	0,4930	0,4447	2F47	1,3789	1,1987	0,8859	0,7540
1F48	0,4930	0,4321	2F48	1,3789	1,1663	0,8859	0,7342
1F49	0,4930	0,4031	2F49	1,3789	1,0917	0,8859	0,6886
1F50	0,4930	0,3790	2F50	1,3789	1,0295	0,8859	0,6505
1F51	0,4930	0,3510	2F51	1,3789	0,9573	0,8859	0,6063
1F52	0,4930	0,3389	2F52	1,3789	0,9262	0,8859	0,5873
1F53	0,4930	0,3003	2F53	1,3789	0,8266	0,8859	0,5263
1F54	0,5316	0,4823	2F54	1,4784	1,2958	0,9468	0,8135
1F55	0,5316	0,4321	2F55	1,4784	1,1663	0,9468	0,7342
1F56	0,5316	0,4031	2F56	1,4784	1,0917	0,9468	0,6886
1F57	0,5316	0,3790	2F57	1,4784	1,0295	0,9468	0,6505
1F58	0,5316	0,3510	2F58	1,4784	0,9573	0,9468	0,6063
1F59	0,5316	0,3003	2F59	1,4784	0,8266	0,9468	0,5263

Maximum Drift Difference in X direction =	0,9468	**η_{bi} (-x-) = 1,110**
Average Drift Difference in X direction =	0,8538	
Maximum Drift Difference in Y direction =	0,8135	**η_{bi} (-y-) = 1,217**
Average Drift Difference in Y direction =	0,6702	

6. DYNAMIC ANALYSIS

6.1. General

Dynamic analysis examines the behavior of the structure under dynamic loading coming from ground excitation. In this part, 1998 earthquake code, which is known as 'Specification for Structures to be Built in Disaster Areas', is again used as a basic reference besides other references.

The same model used in static analysis is taken into consideration for the dynamic analysis with same dead load and live load values. These loads are applied to shell elements. 12 modes are selected for the modal analysis and this number of modes is checked in Part 6.4 to satisfy earthquake code requirements. The dynamic analysis applied to given structure mainly including eigenvector analysis and response - spectrum analysis.

6.2. Eigenvector Analysis

Eigenvector analysis determines the undamped free-vibration mode shapes and frequencies of the system. These natural modes provide an excellent insight into the behavior of the structure.

Eigenvector analysis involves the solution of the generalized eigenvalue problem:

$$[\ K - \Omega\ M\ ]\ \phi = 0 \qquad (6.1)$$

where K is the stiffness matix, M is the diagonal mass matrix , Ω is the diagonal matrix of eigenvalues, and ϕ is the matrix of corresponding eigen vectors (ie. Mode Shapes).

Each eigenvalue - eigenvector pair is called a natural Vibration Mode of the structure. The modes are identified by numbers from 1 to n in the order in which the modes

$$T = \frac{1}{f} \tag{6.2}$$

are found.The eigenvalue is the square of the circular frequency , f, and period, T, of the mode are related to ω by:

$$f = \frac{\omega}{2\pi} \tag{6.3}$$

The number of modes actually found, *n*, is limited by:

- The number of mode requested, *n*
- The number of mass degrees of freedom in the model

A mass degree of freedom is any active degree of freedom that possesses translational mass or rotational mass moment of inertia. The mass may have been assigned directly to the joint or may come from connected elements [2]. For the given structure masses are applied to master joints, which are given with details in Part 4.

6.3. Modal Analysis Result

Various properties of the vibration modes can be obtained from dynamic analysis, they are given in the following subtopics:

6.3.1. Periods and Frequencies

The following time - properties are given for each mode:

- Period, *T*, in units of time
- Cyclic frequency, f, in units of cycles per time (This is the inverse of 'T')
- Circular frequency, *w, in* units of radians per time;

$$w = 2\pi f \tag{6.4}$$

At the end of the dynamic analysis, the following periods and frequencies corresponding to 12 modes are obtained, and listed in Table 6.1. The distribution of periods with respect to mode numbers is illustrated in Figure 6.1. As can be seen from both these table and figure, the periods are decreasing regularly in a group of three. This is the result of almost symmetrical distribution of the structural members in the building. Although the 1st period of the structure seems high, this is the result of the non-existence of the shear walls, weak lateral stiffness and asymmetric distribution of some columns on the building. These low lateral stiffness in both X and Y-axis makes structure very weak against lateral deformations.

Table 6.1. The distribution of periods with respect to mode numbers

MODE	PERIOD (TIME)	FREQUENCY (CYC/TIME)	FREQUENCY (RAD/TIME)	EIGENVALUE $(RAD/TIME)^2$
1	0.909349	1.099687	6.909538	47.741722
2	0.786352	1.271696	7.990300	63.844889
3	0.761947	1.312427	8.246220	68.000141
4	0.280428	3.565979	22.405709	502.015783
5	0.241956	4.132990	25.968340	674.354657
6	0.236982	4.219721	26.513289	702.954511
7	0.149690	6.680452	41.974515	1761.860
8	0.128251	7.797194	48.991216	2400.139
9	0.126857	7.882905	49.529753	2453.196
10	0.095690	10.450390	65.661738	4311.464
11	0.081514	12.267833	77.081070	5941.491
12	0.080241	12.462452	78.303896	6131.500

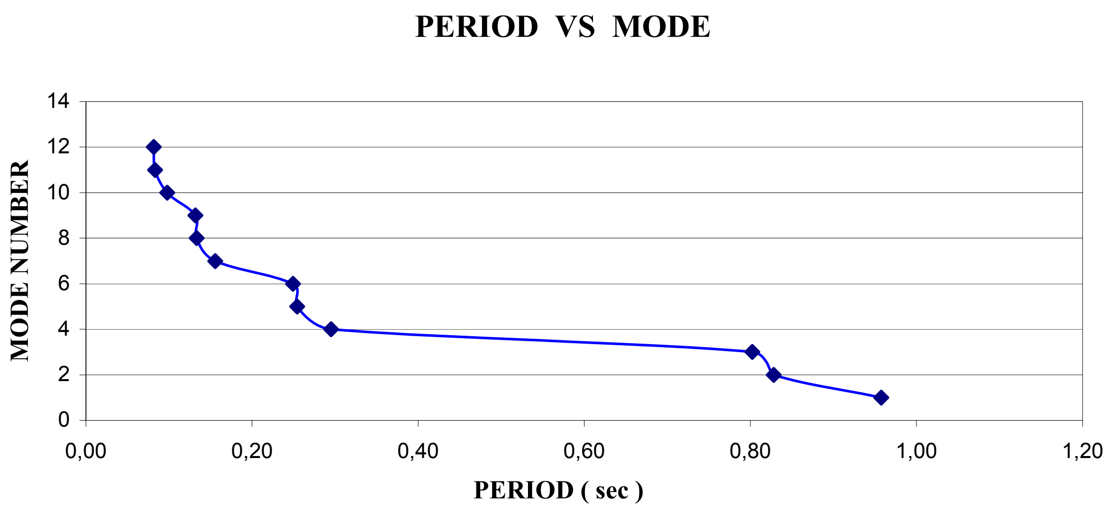


Figure 6.1. The distribution of periods with respect to mode numbers

6.3.2. Participation Factors

The model participation factors are the dot products of the three acceleration loads with the mod shapes. The participation factors for mode 'n' corresponding to acceleration loads in the global X, Y, and Z directions are given by:

$$f_{xn} = \varphi_n^T m_x \tag{6.5}$$

$$f_{yn} = \varphi_n^T m_y \tag{6.6}$$

$$f_{zn} = \varphi_n^T m_z \tag{6.7}$$

Where φ_n is the mode shape and m_x, my and m_z are the unit acceleration loads. These factors are the generalized loads acting on the mode due to each of the acceleration loads. They are referred to the global coordinate system. These values are called ' factors ' because they are related to the mode shape and to unit acceleration. The mode shapes are each normalized, or scaled, with respect to the mass matrix such that:

$$\varphi_n^T M \varphi_n = 1 \tag{6.8}$$

The actual magnitudes and signs of the participation factors are not important. What important is the relative values of three factors for a given mode. The modal participation factors obtained from dynamic analysis are listed in Table 6.2.

Table 6.2. The modal participation factors

MODE	PERIOD	UX	UY	UZ
1	0.909349	-0.885425	0.021743	.000000
2	0.786352	-0.020077	-0.981984	.000000
3	0.761947	0.417258	-0.000562	.000000
4	0.280428	0.300008	-0.007031	.000000
5	0.241956	-0.007680	-0.344394	.000000
6	0.236982	-0.171041	0.003595	.000000
7	0.149690	-0.169182	0.003777	.000000
8	0.128251	0.009098	0.207287	.000000
9	0.126857	-0.128562	0.010155	.000000
10	0.095690	0.110632	-0.002760	.000000
11	0.081514	-0.008476	-0.151222	.000000
12	0.080241	-0.110436	0.009226	.000000

6.3.3. Participating Mass Ratios

The participation mass ratio for a mode provides a measure of how important the mode is for computing the response to the acceleration loads in each of three global directions. The participation mass ratios for mode n corresponding to acceleration loads in the global X, Y, and Z direction are given by:

$$P_{xn} = \frac{(f_{xn})^2}{M_x} \qquad (6.9)$$

$$P_{yn} = \frac{(f_{yn})^2}{M_y} \qquad (6.10)$$

$$P_{zn} = \frac{(f_{zn})^2}{M_z} \qquad (6.11)$$

Where fxn, fyn, and fzn are the participation factors defined in the previous subtopic; and M_x, M_y, and M_z are the total unrestrained masses acting in the X, Y, and Z directions. The participating mass ratios are expressed as percentages.

The cumulative sums of the participating mass ratios for all modes up to mode 'n' can be obtained separately. This measure of how many modes are required to achieve a given level of accuracy for ground acceleration, the requirements for this accuracy is explained in Part 6.4.

If all eigen - modes of the structure are present, the participating mass ratio for each of the three acceleration loads should generally be 100%. However, this may not be the case in the presence of certain types of constraints where symmetry conditions prevent some of the mass from responding to translational accelerations. The participating mass ratios obtained from dynamic analysis are given in Table 6.3.

Table 6.3. The participating mass ratios

MODE	PERIOD	INDIVIDUAL MODE (PERCENT)			CUMULATIVE SUM (PERCENT)		
		UX	UY	UZ	UX	UY	UZ
1	0.909349	**66.8694**	0.0403	0.0000	66.8694	0.0403	0.0000
2	0.786352	0.0344	**82.2494**	0.0000	66.9038	82.2898	0.0000
3	**0.761947**	14.8502	0.0000	**0.0000**	81.7540	82.2898	0.0000
4	0.280428	7.6770	0.0042	0.0000	89.4310	82.2940	0.0000
5	0.241956	0.0050	10.1166	0.0000	89.4360	92.4106	0.0000
6	0.236982	2.4953	0.0011	0.0000	91.9313	92.4117	0.0000
7	0.149690	2.4414	0.0012	0.0000	94.3727	92.4129	0.0000
8	0.128251	0.0071	3.6649	0.0000	94.3798	96.0779	0.0000
9	0.126857	1.4098	0.0088	0.0000	95.7895	96.0867	0.0000
10	0.095690	1.0440	0.0006	0.0000	96.8335	96.0873	0.0000
11	0.081514	0.0061	1.9505	0.0000	96.8396	98.0378	0.0000
12	0.080241	1.0403	0.0073	0.0000	**97.8799**	**98.0451**	0.0000

In Table 6.3, UZ values actually are not zero however they are not calculated by SAP2000 and taken as zero. When the participation factor of first mode is considered, it is appearing that the UX direction is most critical, the building shows totally lateral drift along X direction under dynamic load. In the case of second mode, the building shows totally lateral drift along Y direction and in the third mode although the UZ value seems zero; the dominant behavior of the building is torsion.

6.3.4. Total Unrestrained Mass and Location

The total unrestrained masses, M_x, M_y, and M_z acting in the global X, Y, and Z directions are given in Table 6.4. These masses may differ even if there translational masses assigned to each joint are equal, since the restraints for the three translational degrees of freedom at a joint need not be the same.

Table 6.4. Total unrestrained masses

IN GLOBAL COORDINATES, UNIT = Ton-cm

JOINT	UX	UY	UZ	RX	RY	RZ
MASS1	0.168400	0.168400	.000000	.000000	.000000	84200.000
MASS2	0.213000	0.213000	.000000	.000000	.000000	106520.000
MASS3	0.213000	0.213000	.000000	.000000	.000000	106520.000
MASS4	0.213000	0.213000	.000000	.000000	.000000	106520.000
MASS5	0.213000	0.213000	.000000	.000000	.000000	106520.000
MASS6	0.152000	0.152000	.000000	.000000	.000000	75590.000

6.4. Sufficient Number of Vibration Modes To Be Considered

Sufficient number of vibration modes, n, to be taken into account in the analysis shall be determined to the criterion that the sum of effective participating masses calculated for each mode in each of the given X and Y lateral earthquake directions perpendicular to each other shall in no case be less than 90% of the total building mass. In the earthquake direction considered, all vibration modes with effective participating masses exceeding 5% of the total building mass shall also be taken into account [2].

$$\sum_{r=1}^{n} M_{xr} = \sum_{r=1}^{n} \{[\sum_{i=1}^{N} (m_i \, \Phi_{xir})]^2 / M_r\} \geq 0.90 \sum_{i=1}^{N} m_i \qquad (6.12a)$$

$$\sum_{r=1}^{n} M_{yr} = \sum_{r=1}^{n} \{[\sum_{i=1}^{N} (m_i \, \Phi_{yir})]^2 / M_r\} \geq 0.90 \sum_{i=1}^{N} m_i \qquad (6.12b)$$

The expression of M_r appearing in Eqs.(6.12) is given below for buildings where floors behave as rigid diaphragms:

$$M_r = \sum_{i=1}^{N} (m_i \, \Phi_{xir}^2 + m_i \, \Phi_{yir}^2 + m_{\theta i} \, \Phi_{\theta ir}^2) \qquad (6.13)$$

Eqs.(6.12) and (6.13) are used by SAP2000 and the obtained participating ratios are listed in Table 6.3. As can be seen from Table 6.3. The 12 mode number gives almost 98% of mass participating value and this percentage is satisfies the earthquake code requirements where the limit value for the number of modes in the case of mass participating ratios is given as 90%.

In this method, maximum internal forces and displacements are determined by the statistical combination of maximum contributions obtained from each of the sufficient number of natural vibration modes considered [4].

6.5. Response - Spectrum Analysis

The dynamic equilibrium equations associated with the response of a structure to ground motion are given in (6.14).

$$K\,u(t) + C\,\dot{u}(t) + M\,\ddot{u}(t) = m_x\,\ddot{u}_{gx}(t) + m_y\,\ddot{u}_{gy}(t) + m_z\ddot{u}_{gz}(t) \qquad (6.14)$$

Where K is the stiffness matrix; C is the proportional damping matrix; M is the diagonal mass matrix; u, $\dot{u}$, and $\ddot{u}$ are the relative displacements, velocities, and accelerations with respect to the ground; m_x, m_y, and m_z are the unit acceleration loads; and $\ddot{u}_{gx}$, $\ddot{u}_{gy}$, and $\ddot{u}_{gz}$ are the components of uniform ground acceleration.

Response - spectrum analysis seeks the likely maximum response to these equations rather than the full time history. The earthquake ground acceleration in each direction is given as a digitized response - spectrum curve of pseudo - spectral acceleration response versus period of structure.

Even though accelerations may be specified in three directions, only a single, positive result is produced for each quantity. The response quantities include displacements, forces, and stresses. Each computed result represents a statistical measure of the likely maximum magnitude for that response quantity. The actual response can be expected to vary within a range from this positive value to its negative.

No correspondence between two different response quantities is available. No information is available as to when this extreme value occurs during the seismic loading, or as to what the values of other response quantities are at that time.

6.5.1. Response - Spectrum Curve

The response - spectrum curve for a given direction is defined by digitized points of pseudo - spectral acceleration response versus period of the structure. This curve chosen should reflect the damping that is present in the structure being modeled. The used response spectrum curve, which has a 5% damping, is given in Figure 6.2.

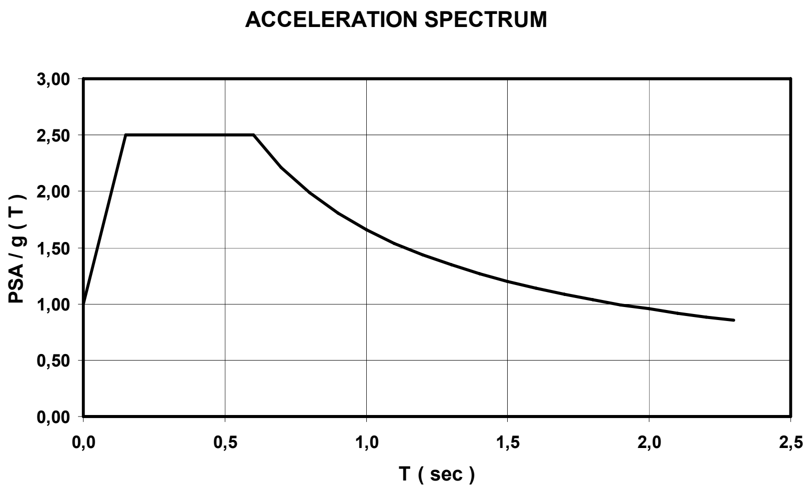


Figure 6.2. Special Design Acceleration Spectra for model structure

6.5.2. Modal Combination

For a given direction of acceleration, the maximum displacements, forces, and stresses are computed throughout the structure for each of the vibration modes. These modal values for a given response quantity are combined to produce a single, positive result for the given direction of acceleration. Rules to be applied for the statistical combination of non-simultaneous maximum contributions of response quantities calculated for each vibration mode, such as the base shear, storey shear, internal force components, displacement and storey drift, are specified in the following provided that they are applied independently for each response quantity:

6.5.2.1. SRSS Method

This method is used in order to combine the modal results by taking the sum of their absolute values .In the cases where natural periods of any two vibration mode with $T_s < T_r$ always satisfy the condition $Ts / T_r < 0.80$, Square *Root of Sum of Squares (SRSS) Rule* may be applied [4].

The SRSS method is checked in Table 6.5 according to earthquake code requirement and as a result, the period ratios are not satisfying 0.80 value, therefore SRSS method is not suitable for superimposing of mode values for the given model.

Table 6.5. Checking of periods for SRSS method

MODE	PERIOD (sec)	Ts / Tr
1	0,909349	
2	0,786352	**0,86**
3	0,761947	**0,97**
4	0,280428	**0,37**
5	0,241956	**0,86**
6	0,236982	**0,98**
7	0,14969	**0,63**
8	0,128251	**0,86**
9	0,126857	**0,99**
10	0,09569	**0,75**
11	0,081514	**0,85**
12	0,080241	**0,98**

T_s / T_r < 0.80

6.5.2.2. CQC Method

Complete Quadratic Combination (CQC) technique described by Wilson, Der Kiureghian and Bayo (1981) is the default method of modal combination.

The CQC method takes into account the statistical coupling between closely spaced modes caused by modal damping. Increasing the modal damping increases the coupling between closely spaced modes. If the damping is zero for all modes, this method degenerates to the SRSS method.

In the cases where the requirements above for the SRSS method for the given condition are not satisfied, *Complete Quadratic Combination (CQC) Rule* shall be applied for the combination of maximum modal contributions.

Since *SRSS* method is not satisfying the requirement of the Earthquake Code for the given building, *Complete Quadratic Combination (CQC) Rule* is used for the superposition of the modes. In the application of this rule, modal damping factors shall be taken as 5% for all modes.

6.6. Lower Limits of Response Quantities

In the case where the ratio of the base shear in the given earthquake direction, V_{tB}, which is obtained through modal combination according to Part 6.5, to the base shear, V_t, obtained by Equivalent *Seismic Load Method* through 5.4 is less than the below given value of $\beta(V_{tB} < \beta V_t)$, all internal force and displacement quantities determined by *Mode Superposition Method* shall be amplified in accordance with (6.14) [4].

$$B_D = (\beta\, V_t / V_{tB})\, B_B \tag{6.14}$$

In the case where at least one of the irregularities of type A1, B2 or B3 exists in a building β=1.00, whereas none of them exists β=0.90 shall be used in (6.14).

β=1.0 for our model due to *A1 Type of Irregularity* according to Part 5.7 therefore;

$$V_{tB} < \beta\, V_t \tag{6.15}$$

V_{tB} = 141.46 ton. Obtained from modal combination, this base shear value is calculated according to support reaction.

V_t = 141.45 ton. Obtained from *Equivalent Seismic Load Method*, the details of calculations are given in Table 5.9.

According to (6.15);

$$V_{tB} < \beta\, V_t$$

$$141.46 \cong 1\text{x}141.45$$

Therefore the requirement of Earthquake Code given in Part 6.6 is satisfied

6.7. Limitation of Storey Drifts

The *storey drift*, Δ_i, of any column or structural wall shall be determined by (6.16) as the difference of displacements between the two consecutive stories.

$$\Delta_i = d_i - d_{i-1} \qquad (6.16)$$

In (6.16) d_i and d_{i-1} represent lateral displacements obtained from the analysis at the ends of any column or structural wall at stories i and i-1.

The maximum value of storey drifts within a storey, $(\Delta_i)_{max}$, calculated by (6.16) for columns and structural walls of the i^{th} storey of a building for each earthquake direction shall satisfy the unfavorable one of the conditions given by (6.17) and (6.18).

$$(\Delta_i)_{max} / h_i \leq 0.0035 \qquad (6.17)$$

$$(\Delta_i)_{max} / h_i \leq 0.02 / R \qquad (6.18)$$

R = 8 According to Part 5.3.1 then

$$(\Delta_i)_{max} / h_i \leq 0.02 / R = 0.0025$$

In the cases where the conditions specified by (6.17) and (6.18) are not satisfied at any storey, the earthquake analysis shall be repeated by increasing the stiffness of the structural system.

The maximum drift difference happens between 1^{th} and 2^{th} floors. The calculations for the limitation of the storey drift are given in Table 6.6.

h_i = 300 cm according to Part 1.7.1 then

As a result of Table 6.6, the conditions specified by (6.17) and (6.18) are satisfied at any storey in the given building.

Table 6.6. Maximum drift differences between 1th and 2th floors

JOINT	u_x	u_y	JOINT	u_x	u_y	Δ_i x	Δ_i y	$\Delta_{i\,x}$ / h_i	$\Delta_{i\,y}$ / h_i
1F1	0,4085	0,4785	2F1	1,1611	1,2858	0,7526	0,8073	0,0025	0,0027
1F2	0,4085	0,4321	2F2	1,1611	1,1663	0,7526	0,7342	0,0025	0,0024
1F3	0,4085	0,3916	2F3	1,1611	1,0618	0,7526	0,6702	0,0025	0,0022
1F4	0,4085	0,351	2F4	1,1611	0,9573	0,7526	0,6063	0,0025	0,0020
1F5	0,4085	0,3071	2F5	1,1611	0,8441	0,7526	0,5370	0,0025	0,0018
1F6	0,4448	0,4823	2F6	1,2544	1,2958	0,8096	0,8135	0,0027	0,0027
1F7	0,4448	0,4785	2F7	1,2544	1,2858	0,8096	0,8073	0,0027	0,0027
1F8	0,4448	0,4447	2F8	1,2544	1,1987	0,8096	0,7540	0,0027	0,0025
1F9	0,4448	0,4321	2F9	1,2544	1,1663	0,8096	0,7342	0,0027	0,0024
1F10	0,4448	0,3916	2F10	1,2544	1,0618	0,8096	0,6702	0,0027	0,0022
1F11	0,4448	0,3510	2F11	1,2544	0,9573	0,8096	0,6063	0,0027	0,0020
1F12	0,4448	0,3389	2F12	1,2544	0,9262	0,8096	0,5873	0,0027	0,0020
1F13	0,4448	0,3071	2F13	1,2544	0,8441	0,8096	0,5370	0,0027	0,0018
1F14	0,4448	0,3003	2F14	1,2544	0,8266	0,8096	0,5263	0,0027	0,0018
1F15	0,4650	0,4447	2F15	1,3067	1,1987	0,8417	0,7540	0,0028	0,0025
1F16	0,4650	0,4321	2F16	1,3067	1,1663	0,8417	0,7342	0,0028	0,0024
1F17	0,4650	0,4176	2F17	1,3067	1,1290	0,8417	0,7114	0,0028	0,0024
1F18	0,4650	0,4031	2F18	1,3067	1,0917	0,8417	0,6886	0,0028	0,0023
1F19	0,4650	0,3916	2F19	1,3067	1,0618	0,8417	0,6702	0,0028	0,0022
1F20	0,4650	0,3790	2F20	1,3067	1,0295	0,8417	0,6505	0,0028	0,0022
1F21	0,4650	0,3645	2F21	1,3067	0,9921	0,8417	0,6276	0,0028	0,0021
1F22	0,4650	0,3510	2F22	1,3067	0,9573	0,8417	0,6063	0,0028	0,0020
1F23	0,4650	0,3389	2F23	1,3067	0,9262	0,8417	0,5873	0,0028	0,0020
1F24	0,4766	0,4447	2F24	1,3366	1,1987	0,8600	0,7540	0,0029	0,0025
1F25	0,4766	0,4321	2F25	1,3366	1,1663	0,8600	0,7342	0,0029	0,0024
1F26	0,4766	0,4176	2F26	1,3366	1,1290	0,8600	0,7114	0,0029	0,0024
1F27	0,4766	0,4031	2F27	1,3366	1,0917	0,8600	0,6886	0,0029	0,0023
1F28	0,4766	0,3790	2F28	1,3366	1,0295	0,8600	0,6505	0,0029	0,0022
1F29	0,4766	0,3645	2F29	1,3366	0,9921	0,8600	0,6276	0,0029	0,0021
1F30	0,4766	0,3510	2F30	1,3366	0,9573	0,8600	0,6063	0,0029	0,0020
1F31	0,4766	0,3389	2F31	1,3366	0,9262	0,8600	0,5873	0,0029	0,0020
1F32	0,4790	0,4823	2F32	1,3428	1,2958	0,8638	0,8135	0,0029	0,0027
1F33	0,4790	0,4447	2F33	1,3428	1,1987	0,8638	0,7540	0,0029	0,0025
1F34	0,4790	0,4321	2F34	1,3428	1,1663	0,8638	0,7342	0,0029	0,0024
1F35	0,4790	0,4176	2F35	1,3428	1,1290	0,8638	0,7114	0,0029	0,0024
1F36	0,4790	0,4031	2F36	1,3428	1,0917	0,8638	0,6886	0,0029	0,0023
1F37	0,4790	0,3790	2F37	1,3428	1,0295	0,8638	0,6505	0,0029	0,0022
1F38	0,4790	0,3645	2F38	1,3428	0,9921	0,8638	0,6276	0,0029	0,0021
1F39	0,4790	0,3510	2F39	1,3428	0,9573	0,8638	0,6063	0,0029	0,0020
1F40	0,4790	0,3389	2F40	1,3428	0,9262	0,8638	0,5873	0,0029	0,0020
1F41	0,4790	0,3003	2F41	1,3428	0,8266	0,8638	0,5263	0,0029	0,0018
1F42	0,4805	0,4823	2F42	1,3465	1,2958	0,8660	0,8135	0,0029	0,0027
1F43	0,4805	0,4031	2F43	1,3465	1,0917	0,8660	0,6886	0,0029	0,0023
1F44	0,4805	0,3790	2F44	1,3465	1,0295	0,8660	0,6505	0,0029	0,0022
1F45	0,4805	0,3003	2F45	1,3465	0,8266	0,8660	0,5263	0,0029	0,0018

Table 6.6. Continued

1F46	0,4930	0,4823	2F46	1,3789	1,2958	0,8859	0,8135	0,0030	0,0027
1F47	0,4930	0,4447	2F47	1,3789	1,1987	0,8859	0,7540	0,0030	0,0025
1F48	0,4930	0,4321	2F48	1,3789	1,1663	0,8859	0,7342	0,0030	0,0024
1F49	0,4930	0,4031	2F49	1,3789	1,0917	0,8859	0,6886	0,0030	0,0023
1F50	0,4930	0,3790	2F50	1,3789	1,0295	0,8859	0,6505	0,0030	0,0022
1F51	0,4930	0,3510	2F51	1,3789	0,9573	0,8859	0,6063	0,0030	0,0020
1F52	0,4930	0,3389	2F52	1,3789	0,9262	0,8859	0,5873	0,0030	0,0020
1F53	0,4930	0,3003	2F53	1,3789	0,8266	0,8859	0,5263	0,0030	0,0018
1F54	0,5316	0,4823	2F54	1,4784	1,2958	0,9468	0,8135	0,0032	0,0027
1F55	0,5316	0,4321	2F55	1,4784	1,1663	0,9468	0,7342	0,0032	0,0024
1F56	0,5316	0,4031	2F56	1,4784	1,0917	0,9468	0,6886	0,0032	0,0023
1F57	0,5316	0,3790	2F57	1,4784	1,0295	0,9468	0,6505	0,0032	0,0022
1F58	0,5316	0,3510	2F58	1,4784	0,9573	0,9468	0,6063	0,0032	0,0020
1F59	0,5316	0,3003	2F59	1,4784	0,8266	0,9468	0,5263	0,0032	0,0018

6.8. Second - Order Effects

In the case where *Second-Order Effect Indicator*, θ_i, satisfies the condition given by (6.19) for the earthquake direction considered at each storey, second-order effects shall be evaluated in accordance with currently enforced specifications of reinforced concrete design [4].

$$\theta_i = \frac{(\Delta_i)_{ort} \sum_{j=i}^{N} w_j}{V_i \, h_i} \leq 0.12 \qquad (6.19)$$

Here $(\Delta_i)_{ort}$ shall be determined in accordance with 6.16 as the average value of storey drifts calculated for i'th storey columns and listed in Table 6.7.

Table 6.7. Average drift between floors

	DRIFT	AVERAGE
STOREY	$(\Delta_i)_{ort}$ X	$(\Delta_i)_{ort}$ Y
0-1	0,4727	0,3915
1-2	0,8538	0,6702
2-3	0,8546	0,6652
3-4	0,7263	0,5611
4-5	0,5328	0,4073
5-6	0,3369	0,2427

$w_{i,}$ Vi and Hi values used in (6.19) are taken from Table 5.9 in Part 5 and $(\Delta_i)_{ort}$ values are taken from Table 6.7. According to these data, the results of (6.19) are listed in Table 6.8 below.

Table 6.8. Calculation for second-order effects

STOREY	$(\Delta_i)_{ort}$ X	$(\Delta_i)_{ort}$ Y	Wi (ton)	Total Wi (ton)	Vi (ton)	hi (m)	θ_i x	θ_i y
0-1	0,4727	0,3915	165,2	165,2	5,87	300	0,044	0,037
1-2	0,8538	0,6702	209,0	374,2	14,85	600	0,036	0,028
2-3	0,8546	0,6652	209,0	583,2	22,28	900	0,025	0,019
3-4	0,7263	0,5611	209,0	792,2	29,70	1200	0,016	0,012
4-5	0,5328	0,4073	209,0	1001,2	37,13	1500	0,010	0,007
5-6	0,3369	0,2427	148,3	1149,5	31,62	1800	0,007	0,005

As a result of Table 6.8, the conditions specified by (6.19) is satisfied for the given building and It is concluded that there is no any problem in the building due to secondary moment effects.

6.9. Dynamic Analysis Results

The deformed shape of the dynamic loading according to first three modes and mode superposition are given below. In Figures 6.3, 6.4, 6.5, 6.6, 6.7, 6.8, 6.9 and 6.10.

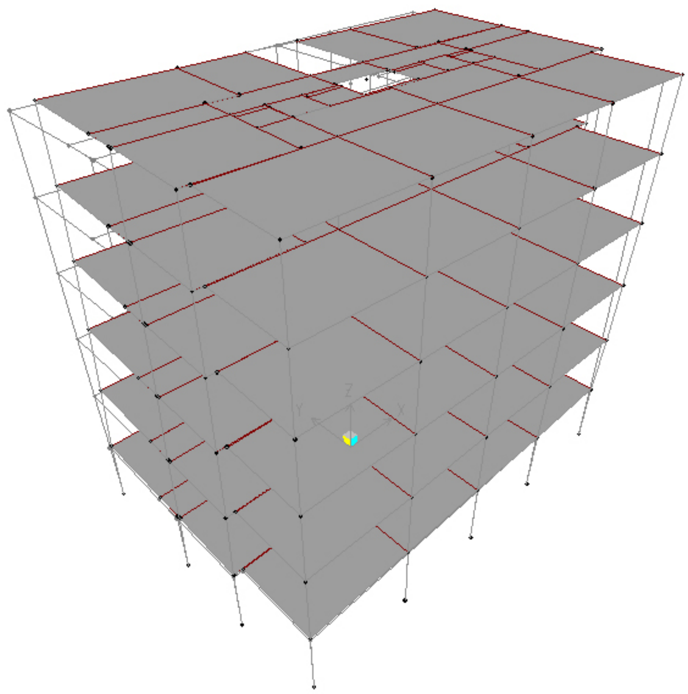


Figure 6.3. Mode 1 (0.909), 3D Deformed Shape

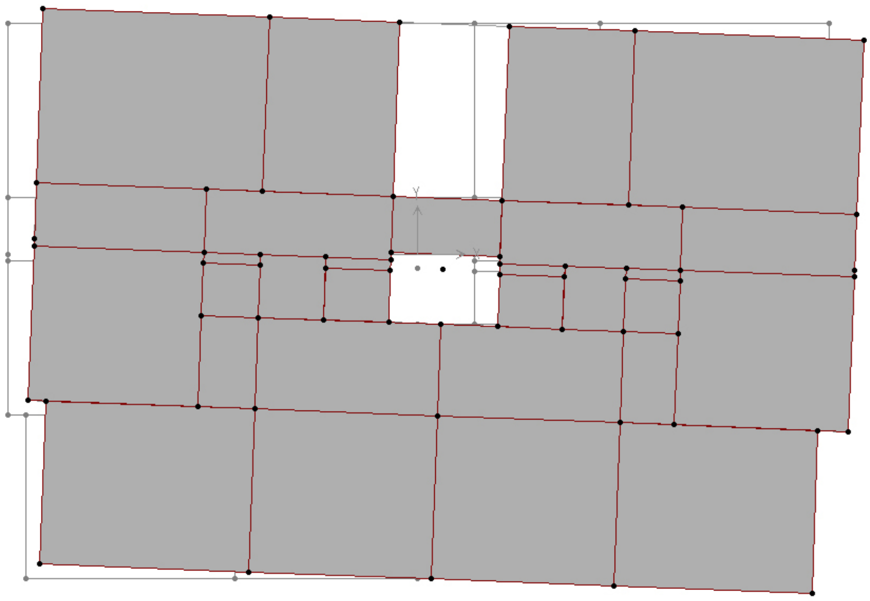


Figure 6.4. Mode 1 (0.909 sec), Top floor deformed Shape

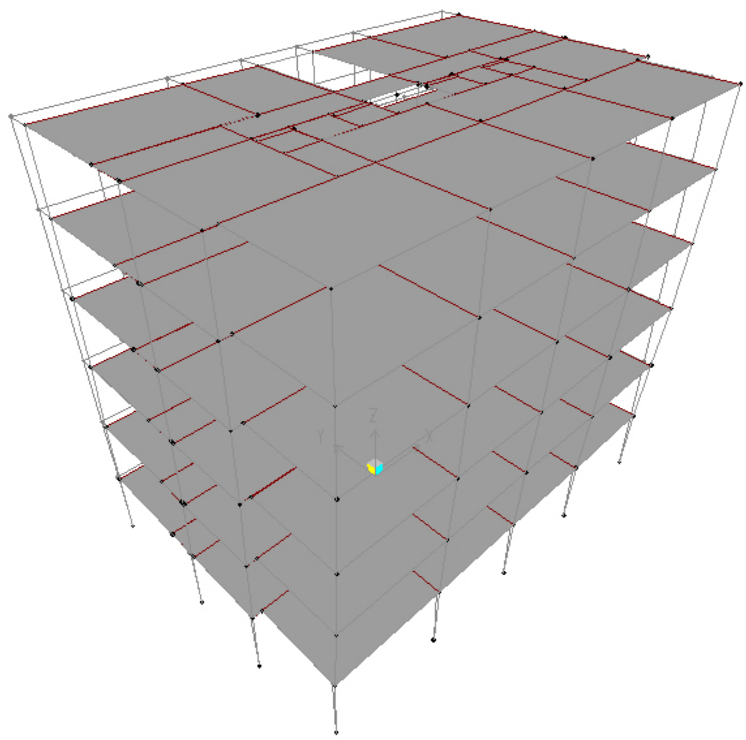


Figure 6.5. Mode 2 (0.786 sec), 3D Deformed Shape

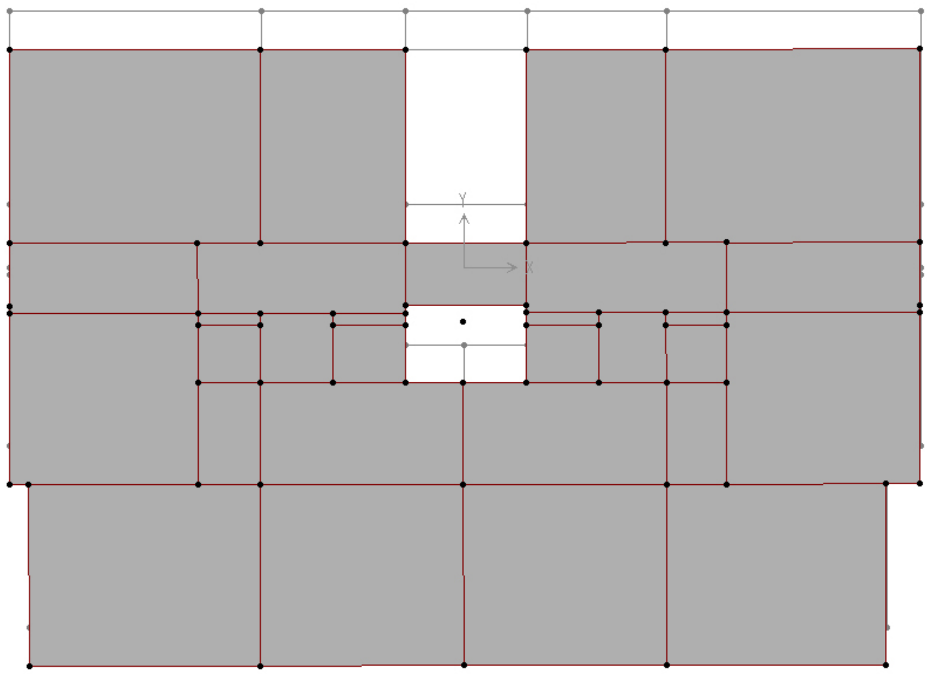


Figure 6.6. Mode 2 (0.786 sec), Top floor deformed Shape

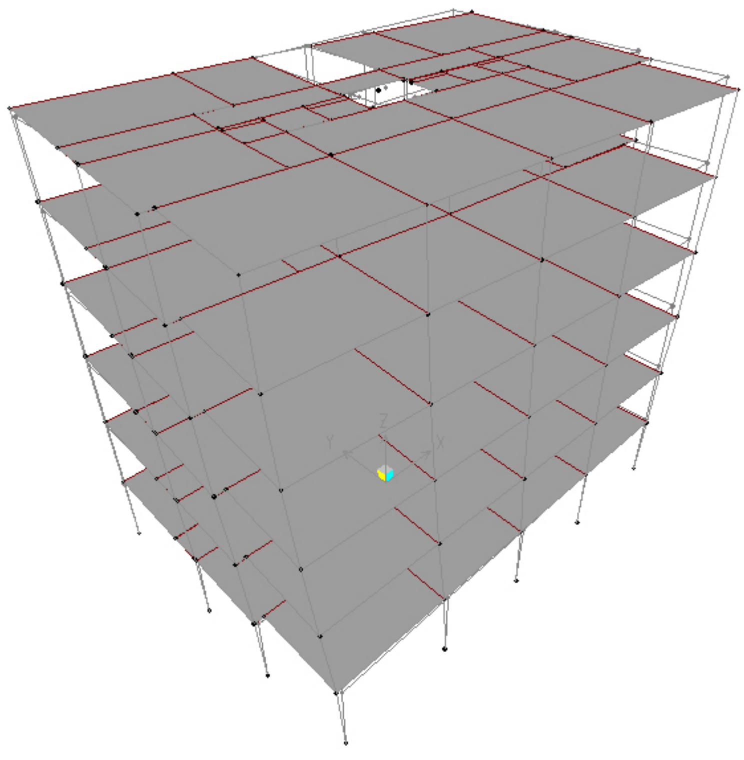


Figure 6.7. Mode 3 (0.762 sec), 3D Deformed Shape

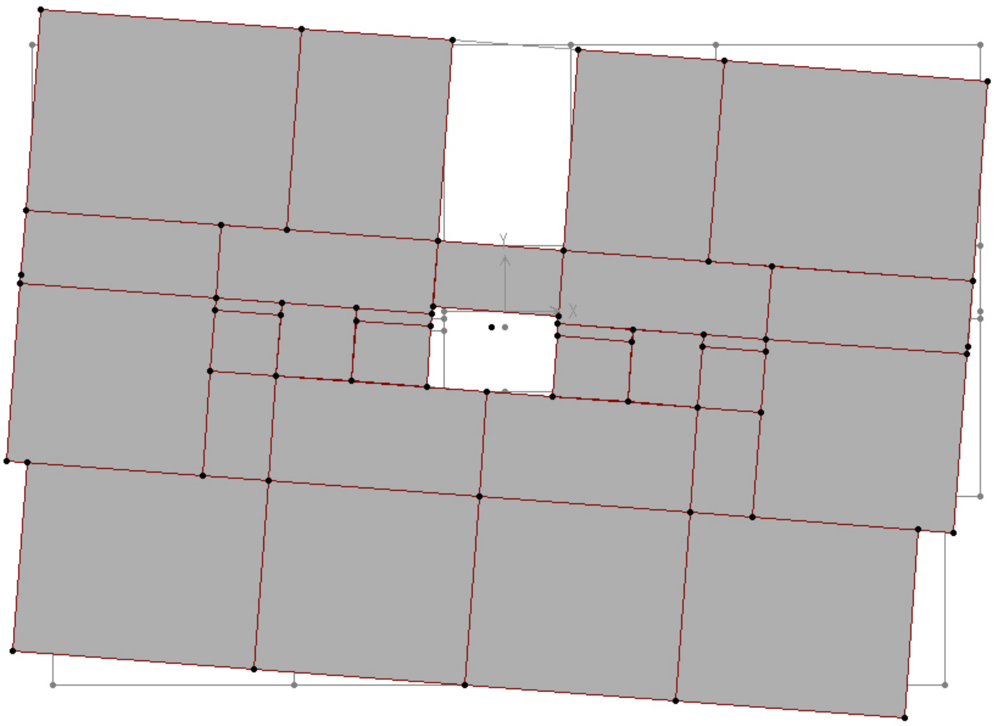


Figure 6.8. Mode 3 (0.762 sec), Top floor deformed Shape

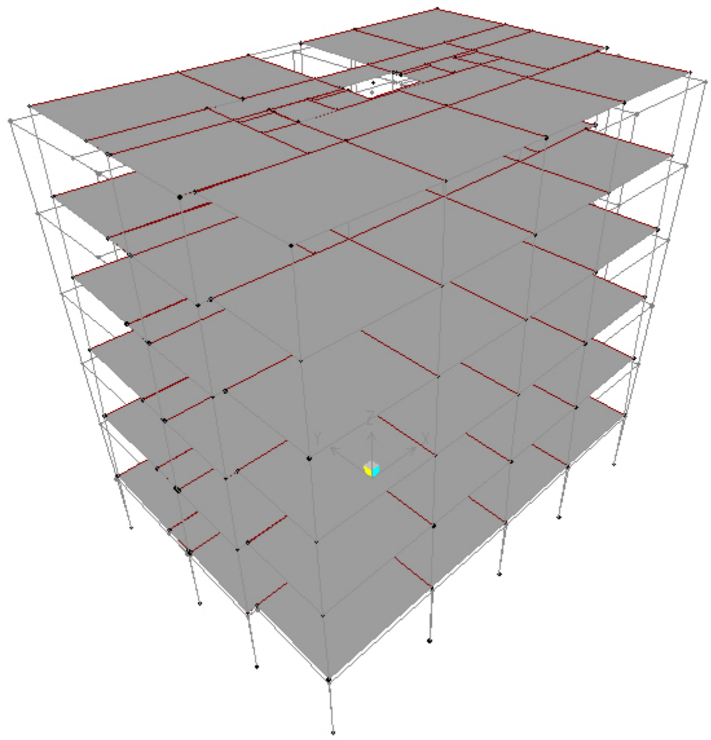


Figure 6.9. Mode Superposition, 3D Deformed Shape

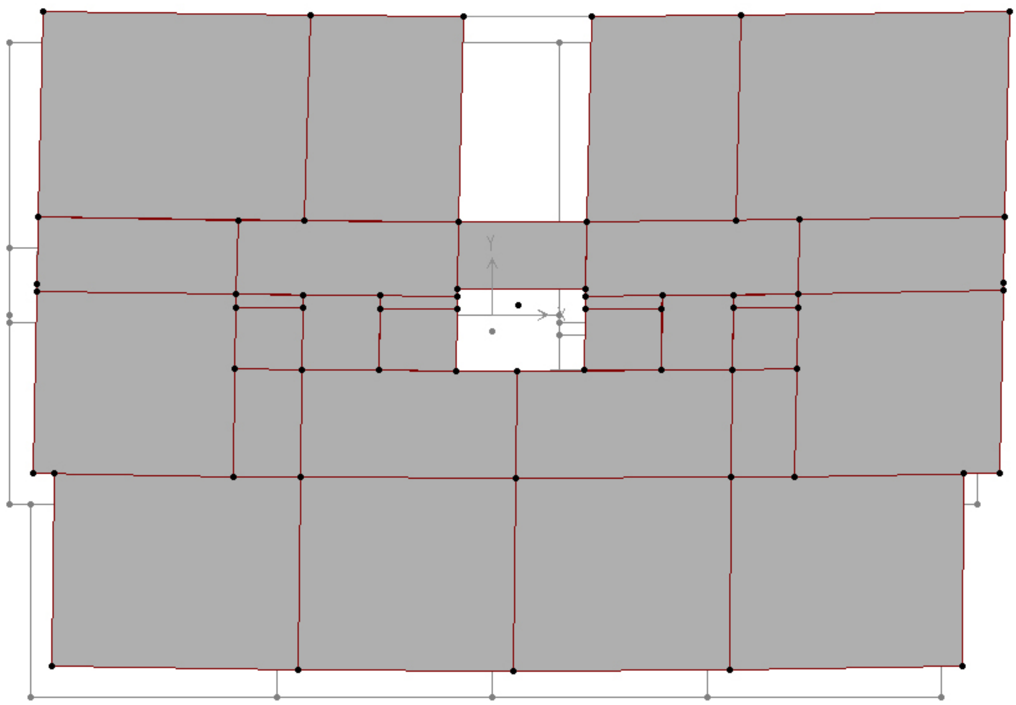


Figure 6.10. Mode Superposition, Top floor deformed Shape

7. CAPACITY SPECTRUM ANALYSIS

7.1. General

This part presents analytical procedures for evaluating the performance of existing reinforced concrete building. Various analysis methods, both elastic (linear) and inelastic (nonlinear), are available for the analysis of structures. Elastic analysis methods available include code static lateral force procedures (Given in Part 5) and code dynamic lateral force procedures (Given in Part 6). The most basic inelastic analysis method is the complete nonlinear time history analysis, which at this time is considered overly complex and impractical for general use. Available simplified nonlinear analysis methods are as follows:

- Nonlinear static analysis procedures include the capacity spectrum method (CSM) that uses the intersection of the capacity (Pushover) curve.
- Reduced response spectrum to estimate maximum displacement.
- Displacement coefficient method that uses pushover analysis and a modified version of the equal displacement approximation to estimate maximum displacement.
- The Secant method; that uses substitute structure and secant stiffness.

This master thesis emphasizes the use of nonlinear static procedures in general and focuses on the capacity spectrum method on the light of the ATC40 documents. The mentioned method provides a particularly rigorous treatment of the reduction of seismic demand for increasing displacement.

Although an elastic analysis gives a good indication of the elastic capacity of structures and indicates where first yielding will occur, it cannot predict failure mechanisms and account for redistribution of forces during progressive yielding. Inelastic analyses procedures help demonstrate how buildings really work by identifying modes of failure and the potential for progressive collapse. The use of inelastic procedures for design and evaluation is an attempt to help engineers in order to better understand how structures will behave when subjected to major earthquakes, where it is assumed that the elastic capacity of the structure will be exceeded. This resolves some of the uncertainties associated with code and elastic procedures.

The capacity spectrum method, a nonlinear static procedure that provides a graphical representation of the global force - displacement capacity curve of the structure (i.e., pushover) and compares it to the response spectra representations of the earthquake demands, is a very useful tool in the evaluation and retrofit design of existing concrete buildings. The method can be used for rapid evaluation of the design of an essential building to verify that code designed structural system will satisfy specified performance goals for major postulated earthquakes. Using this information, the engineer formulates a component model of the building structure. The analysis procedure tells how to identify which part of the building will fail first. As the load and displacement increase, other elements begin to yield and deform inelastically. The resulting graphic '*pushover curve*' is an easy-to-visualize representation of the capacity of the building. The technique explained with detail in this part allow the demand from a specific earthquake or intensity of ground shaking to be correlated with the capacity curve to generate a point on the curve where capacity and demand are equal. This '*performance point*' is an estimate of the actual displacement of the building for the specified ground motion. Using this performance point, it is possible to characterize the associated damage state for the structure and compare it with desired performance objective. This allows the engineer to focus on deficiencies in each building part and address the directly with retrofit measures only where necessary. In short, the procedure gives the engineer a better understanding of the seismic performance characteristics of the building and results in a more effective and cost-efficient retrofit.

7.2. Nonlinearity

Two types of nonlinear behavior can be considered in a nonlinear pushover analysis:

- Material nonlinearity at discrete, user-defined hinges in frame elements. Hinges can be assigned at any number of locations along the length of any frame element. Uncoupled moment, torsion, axial force and shear hinges are available. There is also a coupled P-M2-M3 hinge, which yields based on the interaction of axial force and bending moments at the hinge location.

- Geometric nonlinearity in all elements. P-Delta effects or P-Delta effects plus large displacements can be considered. Large displacement effects consider equilibrium in the deformed configuration and allow for large translations and rotations. The strains within each element are assumed to remain small.

7.3. Capacity Spectrum Method

Two key elements of a performance - based design procedure are demand and capacity. Demand is a representation of the structure's ability to resist the seismic demand. The performance is dependent on the manner that the capacity is able to handle the demand. In other words, the structure must have the capacity to resist the demands of the earthquake such that the performance of the structure is compatible with the objectives of the design.

Simplified nonlinear analysis procedure using pushover method require determination of three primary elements; capacity, demand (displacement) and performance. Each of these elements is briefly discussed in the following subtopics:

7.3.1. Capacity Curve – Pushover

The overall capacity of a structure depends on the strength and deformation capacities of the individual components of the structure. In order to determine capacities beyond the elastic limits, some form of nonlinear analysis, such as the pushover procedure, is required. This procedure is applied by statically loading the structure with realistic gravity loads combined with a set of lateral forces to calculate the roof displacement Δ_r and base shear coefficient 'C_B = V/W that defines first significant yielding of structural elements. The yielding elements are then relaxed to form plastic hinges and incremental lateral loading is applied, this process is continued until the structure becomes unstable or until a predetermined limit is reached. The capacity curve is created by superposition of each increment of displacement and includes tracking displacements at each storey.

There are several levels of sophistication that may be used for the pushover analysis, ranging from applying lateral forces to each storey in proportion to the standard code procedure to applying lateral storey forces as masses times acceleration in proportion to the first mode shape of the elastic model of the structure. For added sophistication, at each increment beyond yielding, the forces may be adjusted to be consistent with the changing deflected shape. It is assumed that the structure can take a number of cycles along the capacity curve and behave in a hysteric manner. The stiffness is assumed to reduce an equivalent global secant modulus measured to the maximum excursion along the capacity curve for each cycle of motion. The Δ_r vs V / W coordinates are converted to spectral displacements (S_d) and spectral accelerations (S_a) by use of modal participation factors ($PF_1\phi_{R1}$) and effective modal weight ratios (α_1) as determined from dynamic characteristics of the fundamental mode of the structure. These values change as the displaced shape changes. An equivalent elastic period of vibration (T_j) at various points along the capacity curve are calculated by use of the secant modulus (i.e., $T_1 = 2\pi (S_{di} / S_{ai} . g)^{1/2}$).

For comparison, both demand and capacity curves are required to be sketched on the same graph in the *Acceleration Displacement Response Spectra* (ADRS) format. The required formulation to convert the capacity curve to ADRS format is given as follows:

$$PF_1 = \left[\frac{\sum_{i=1}^{N} (w_i \phi_i)/g}{\sum_{i=1}^{N} (w_i \phi_i^{2})/g} \right] \tag{7.1}$$

$$\alpha_i = \frac{\left[\sum_{i=1}^{N} (w_i \phi_i)/g \right]^2}{\left[\sum_{i=1}^{N} (w_i)/g \right] \left[\sum_{i=1}^{N} (w_i \phi_i^2)/g \right]} \tag{7.2}$$

$$S_a = \frac{V/W}{\alpha_1} \tag{7.3}$$

$$S_d = \frac{\Delta_{roof}}{PF_1 \phi_{roof,1}} \tag{7.4}$$

Sap2000 uses these formulations and automatically converts the capacity curve to ADRS format.

7.3.2. Demand Curve (Displacement) - Response Spectra

Ground motions during an earthquake produce complex horizontal displacement patterns in structures that may vary with time. Traditional linear analysis methods use lateral forces to represent a design condition. For nonlinear methods it is easier and more direct to use a set of lateral displacements as a design condition. For a given structure and ground motion, the displacement demand is an estimate of maximum expected response of the building to the ground motion. It is represented at various levels of damping. For example, the 5 percent damped response spectrum is generally used to represent the demand when the structure is responding linearly elastic (LERS). Higher damped response spectra are used to represent inelastic response spectra (IRS) to account for hysteretic nonlinear response of the structure.

7.3.3. Performance

Once a capacity curve and demand displacement are defined, a performance check can be done. A performance check verifies that structural and nonstructural components are not damaged beyond the acceptable limits of the performance for the forces and displacements implied by the displacement demand. If the capacity curve can extend through the envelope of the demand curve, the building survives the earthquake. The intersection of the capacity and appropriately damped demand curve represents the inelastic response of the structure.

At the performance point, both capacity and demand spectra are required to correspond to the same ductility level. To achieve this, family of inelastic demand spectra is needed but it is not easy to construct. Instead, family of elastic spectra with equivalent damping values representing different ductility levels is employed.

In representing the inelastic spectra with elastic spectra of equivalent damping values, one important issue is the ductility level of the structure at the ultimate level. If the structural members of the building are of normal ductility class, the energy dissipation by hysteric damping will not be so high. Then the amount of equivalent damping may not be more than some limited value. In ATC40, three building classes (A, B; C) are defined depending on the quality of the structural members and shaking duration. For each class, different maximum limits are given for the equivalent damping to prevent reduction of the elastic spectrum more than allowed.

When the capacity and demand spectra of the same ductility level intersect each other, the ordinates of the performance point, S_d vs S_a give the response of the equivalent SDOF system. Finally they are back substituted using the above transformation formula to obtain the top displacement and the base shear of the MDOF system.

7.4. Pushover Hinges

There are three types of hinge properties used in SAP2000, default hinge properties, user-defined hinge properties and generated hinge properties. Only default hinge properties and user-defined hinge properties can be assigned to frame elements. When these hinge properties (default and user-defined) are assigned to a frame element, the program automatically creates a new generated hinge property for each and every hinge [6]. The built in default hinge properties for concrete members are generally based on ATC 40.

The main reason for the differentiation between defined properties (in this context, defined means both default and user-defined) and generated properties is that typically the hinge properties are section dependent. Thus it is necessary to define a different set of hinge properties for each different frame section type in the model. In order to simplify this process, the concept of default properties is used in SAP2000. When default properties are used, the program combines its built-in default criteria with the defined section properties for each element to generate the final hinge properties. This causes really less work for defining the hinge properties.

The details of default hinge properties are as follows:

- Concrete Axial Hinge

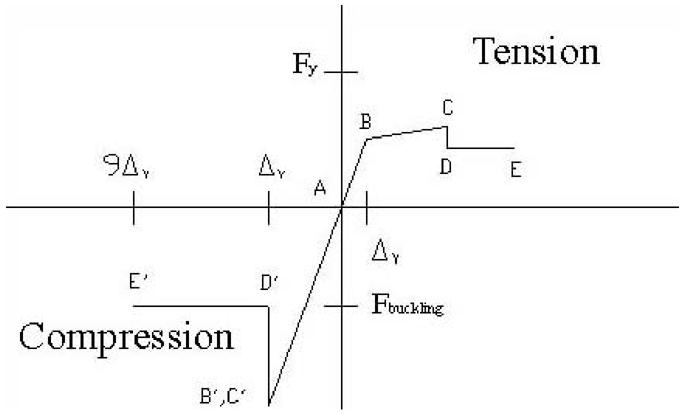


Figure 7.1. Concrete axial hinge

Hinge Features:

- Py = As fy
- Pc = 0.85 Ac f 'c
- Slope between points B and C is taken as 10 % total strain hardening for steel
- Hinge length assumption for Dy is based on the full length
- Point B, C, D and E based on FEMA 273 Table 5.8, Braces in Tension
- Point B' = Pc
- Point E' taken as 9Δy

- Concrete Shear Hinge

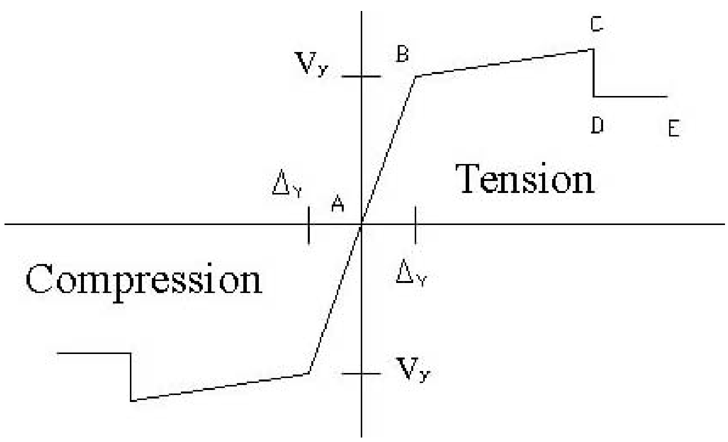


Figure 7.2. Concrete shear hinge

Hinge Features:

- Slope between points B and C is taken as 10 % total strain hardening for steel
- Vy = 2 As (f 'c) + fy Asv d
- Points C, D and E based on ATC40 Table 9.12, Item 2, average of the two rows labeled "Conventional longitudinal reinforcement" and "Conforming transverse reinforcement"

- Concrete Moment Hinge and Concrete P-M-M Hinge

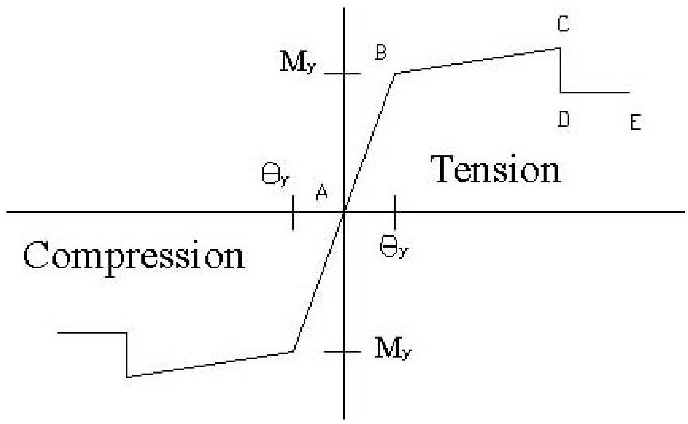


Figure 7.3. Concrete moment and PMM hinge

Hinge Features:

- Slope between points B and C is taken as 10 % total strain hardening for steel
- Qy = 0, since it is not needed
- Points C, D and E based on ATC40 Table 9.6. The four conforming transverse reinforcing rows are averaged
- My based on reinforcement provided, otherwise based on minimum allowable reinforcement
- P-M-M curve taken to be same as the Moment curve in conjunction with the definition of Axial-Moment interaction curves.

For the Columns = Default PMM hinges are assigned due to the existing of both axial forces and moments on the columns. Details of used PMM hinges are given in Figure 7.4.

For the Beams = Default M3 hinges are assigned due to the dominant character of moments on the beams. Details of used M3 hinges are also given in Figure 7.4.

POINT	Force/Yield	Disp/Yield
-E	-8	-0,2
-D	-6	-0,2
-C	-6	-1,25
-B	-1	-1
A	0	0
B	1	1
C	6	1,25
D	6	0,2
E	8	0,2

DEFAULT - PMM & M3 -

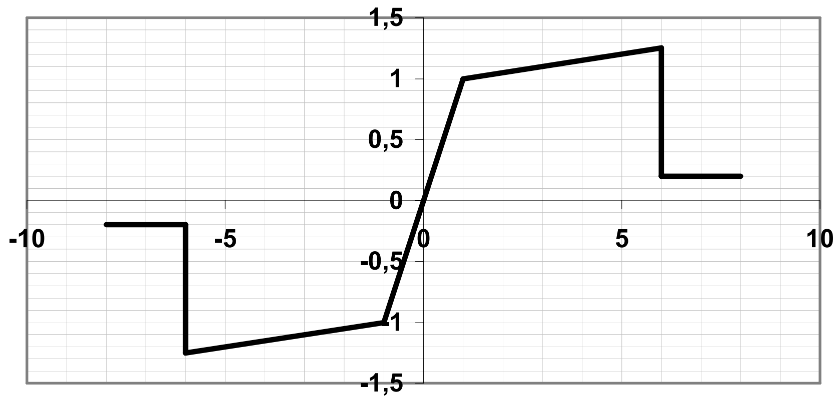


Figure 7.4. Details of used PMM and M3 hinge

7.5. Primary Ground Shaking Criteria

This section specifies the primary ground shaking criteria for the evaluation of the given building. It is necessary to have the accurate values of soil properties and information about site geology, site seismicity and site response, in order to have a correct idea about the demand level of the building under different earthquake excitations.

Primary criteria is including these following sub titles:

- Site Geology and Soil Characteristics
- Site Seismicity Characteristics

- Site Response Spectra

These all criteria are used to define the seismic coefficient C_A and $C_{V,}$ which will be used later for the evaluation of the site response spectra. Since these two values are taken as 0.4 by the program as default values. Herein better to have more accurate values for these coefficients. For the determination of C_A and C_V the procedure explained in ATC40 is applied.

7.5.1. Site Geology and Soil Characteristics

According to Part 1.6.2 and the information taken from METU Earthquake Research Center, the soil profile in the given site can be taken as *Stiff Soil*. The classification of ATC40 on soil profiles is listed in Table 7.1. According to this table the corresponding soil profile for the given site condition is ‘ S_D ‘.

Table 7.1. Site geology and soil characteristics [6]

Soil Profile Type	Soil Profile Name/Generic Description	Average Soil Properties for Top 100 Feet of Soil Profile		
		Shear Wave Velocity, Vs (feet/second)	Standard Penetration Test, N for N_{CH} for cohesionless soil layers (blows/foot)	Undrained Shear Strength, S_u (psf)
S_A	Hard Rock	Vs > 5000	Not Applicable	
S_B	Rock	2500 < Vs < 5000	Not Applicable	
S_C	Very Dense Soil and Soft Rock	1200 < Vs < 2500	N > 50	$s_u > 2000$
S_D	Stiff Soil Profile	600 < Vs < 1200	15 < N < 50	$1000 < s_u < 2000$
S_E	Soft Soil Profile	Vs < 600	N < 15	$s_u < 1000$
S_F	Soil Requiring Site-Specific Evaluation			

7.5.2. Site Seismicity Characteristics

Seismicity characteristics for the site are based on the seismic zone, seismic source type and the proximity of the site to active seismic sources.

Since the given building is within the boundaries of first seismic zone, the corresponding seismic zone factor in ATC40 is equivalent to Z=0.40 which is taken from the Table 7.2 given below.

Table 7.2. Site seismicity characteristics [6]

Zone	1[1]	2A[1]	2B[1]	3	4
Z	0.075	0.15	0.2	0.3	0.40

For the case of seismic source type information, Table 7.3, can be used. On the base of the information taken from Part 1.6 and according to Table 7.3 seismic source type is *‘ B ‘*.

Table 7.3. Seismic zone description [6]

Seismic Source Type	Seismic Source Describtion	Seismic Source Definition	
		Maximum Moment Magnitude, M	Slip Rate (mm/year)
A	Faults that are capable of producing large magnitude events and which have a high rate of seismic activity	M > 7.0	SR > 5
B	All faults other than types A and C	Not Applicable	Not Applicable
C	Faults that are not capable of producing large magnitude events and that have a relative low rate of seismic activity	M > 6.5	SR < 2

The distance from the epicenter of the earthquake to the Adana City is 30 km (Part 1.6) and this value is used for determination of near source factors N_A and N_V according to Table 7.4.

Table 7.4. Pushover hinges

Seismic Source Type	Closest Distance to Known Seismic Source							
	< 2 km		5 km		10 km		15 km	
	N_A	N_V	N_A	N_V	N_A	N_V	N_A	N_V
A	1.5	2.0	1.2	1.6	1.0	1.2	1.0	1.0
B	1.3	1.6	1.0	1.0	1.0	1.0	1.0	1.0
C	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0

$N_A = 1.0$ and $N_V = 1.0$

7.5.3. Site Response Spectra

For the determination of CA and C_V values in order to well define the site response spectra, the serviceability earthquake level and design earthquake level are used. Serviceability earthquake (SE) level is defined probabilistically as the level of ground shaking that has a 50 percent chance of being exceeded in a 50-year period. Design earthquake (DE) level is taken to be a ground motion with a 10 percent chance of being exceeded in 50 years.

Since the given building was damaged due to Adana-Ceyhan Earthquake, these levels of ground shaking can be accepted levels for the calculations.

The seismic coefficient C_A can be calculated according to formulation and table given.

The value of ZEN = ZxExN

Z = 0.40 (Seismic Zone Factor)

E = 0.50 (Serviceability Earthquake Level)

E = 1.00 (Design Earthquake Level)

N_A = 1.00 (Near Source Factor), therefore ZEN = 0.20 (SE)

therefore ZEN = 0.40 (DE)

Table 7.5. Shaking intensity for C_A value [6]

	Shaking Intensity, ZEN					
Soil Profile Type	**= 0.075**	**= 0.15**	**= 0.20**	**= 0.30**	**= 0.40**	**> 0.40**
S_B	0.08	0.15	0.2	0.30	0.40	1.0(ZEN)
S_C	0.09	0.18	0.24	0.33	0.40	1.0(ZEN)
S_D	0.12	0.22	0.28	0.36	0.44	1.1(ZEN)
S_E	0.19	0.30	0.34	0.36	0.36	0.9(ZEN)
S_F	Site - specific geotechnical investigation required to determine C_A					
The value of E used to determine the product. ZEN, should be taken to be equal to 0.5 for the Serviceability Earthquake 1.0 for the Design Earthquake and 1.25 (Zone 4 sites) or 1.5 (Zone 3 sites) for the Maximum Earthquake						

By using Table 7.5, the corresponding C_A value corresponding to soil profile type S_D is equal to 0.28 for SE and 0.44 for DE. According to the given information, when the Table 7.6 is used, the obtained value of the seismic coefficient C_V is equivalent to 0.40 for SE and 0.64 for DE.

Table 7.6. Shaking intensity for Cv value [6]

Shaking Intensity, ZEN						
Soil Profile Type	**= 0.075**	**= 0.15**	**= 0.20**	**= 0.30**	**= 0.40**	**> 0.40**
S_B	0.08	0.15	0.20	0.30	0.40	1.0(ZEN)
S_C	0.13	0.25	0.32	0.45	0.56	1.4(ZEN)
S_D	0.18	0.32	0.40	0.54	0.64	1.6(ZEN)
S_E	0.26	0.50	0.64	0.84	0.96	2.4(ZEN)
S_F	Site - specific geotechnical investigation required to determine C_V					
The value of E used to determine the product. ZEN, should be taken to be equal to 0.5 for the Serviceability Earthquake 1.0 for the Design Earthquake and 1.25 (Zone 4 sites) or 1.5 (Zone 3 sites) for the Maximum Earthquake						

As a result of these calculations, these two coefficients are used to construct site response spectra (Demand Spectra) with different damping values, it means for different earthquake cases.

7.6. Analysis Cases

Pushover analysis can consist of any number of pushover cases. Each pushover case can have a different distribution of load on the structure. A pushover case may start from zero initial conditions, or it may start from the results at the end of any previous pushover case.

In order to complete pushover analysis, it is necessary to define the static pushover analysis cases, for that purpose there are two alternative options in SAP2000:

- Push To Load Level Defined By Pattern*:* This option is used to perform a force-controlled analysis. The pushover typically proceeds to the full load value defined by the sum of all loads (unless it fails to converge at a lower force value). This option is useful for applying gravity load to the structure.

- Push To Displacement Of: This option is used to perform a displacement - controlled analysis. The pushover typically proceeds to the specified displacement in the specified control direction at the specified control joint (unless it fails to converge at a lower displacement value) the specified displacement, specified control joint are all given default values by SAP2000, it is possible to change these default values. The default value for the specified displacement is 0.04 times the Z coordinate at the top of the given structure. The default value for the control joint is a joint located at the top of the structure. The default control direction is U1; other available directions are U2,U3,R1,R2 and R3. These control directions are in global coordinates. The '*Push To Displacement Of*' is useful for applying lateral load to the structure.

Load patterns are used to describe the distribution of force on the structure. There are four different methods of describing of load on the structure for a pushover load case:

- If a uniform acceleration acting in any of the three global directions (X-Y-Z) acc dir X, acc dir Y or acc dir Z is selected, then a uniform acceleration is applied in the appropriate direction, i.e., a lateral force is applied at each joint that is proportional to the mass tributary to that joint.
- If mode shapes have been requested for the analysis, then a MODE load pattern is available. This load pattern applies a lateral force that is proportional to the product of a specified mode shape times the mass tributary to that node.
- Previously defined static load pattern and load combinations can be used in the analysis.
- Combination of any methods described in 1,2 and 3 is also possible.

The load combination for each pushover case is incremental, i.e., it acts in addition to the load already on the structure if starting from a previous pushover case. For the given model seven static pushover cases are defined. The first applies the gravity load to the structure, and the other six apply different distribution of lateral load.

The details of used seven pushover load cases are presented as follows:

- Static Pushover Case: Named '*GRAV*' is used, the details of this load combination is as follows;

GRAV = 1.4 DL + 1.6 LL (Typical Gravitational Load Combination)

  - *Push To Load Level Defined By Pattern* option is used for this case.
  - P-Delta effects are not included.

- Static Pushover Case: Named '*ACCX*' is used, the details of this load combination is as follows;

ACCX = Acc. Dir. X

  - *Push To Displacement Of* option is used for this case. For the application of this option, the default value for the specified displacement is accepted and is given as;

    0.04 x Z coordinate (Height of Building)

    0.04 x (6x3.00 m) = 0.72 m =72 cm

  - Control Direction is changed to U1 (X direction in global direction)
  - This case starts from first pushover case 'GRAV'.
  - P-Delta effects in this load case are included. (Since it is lateral loading)

- Static Pushover Case: Named '*ACCY'* is used, the details of this load combination is as follows;

ACCY = Acc. Dir. Y

  - *Push To Displacement Of* option is again used for this case.
  - Control Direction is changed to U2 which is Y direction in global direction.
  - This case is also starts from first pushover case 'GRAV'.
  - P-Delta effects are again included.

- Static Pushover Case: Named '*MODE2*' is used which is based on the 2^{nd} modal analysis results, the details of this load combination is as follows;

MODE2 = M2

- *Push To Displacement Of* option is used for this case.
- Second mode is taken into calculation as a base deformed shape
- Control Direction is accepted as U2 which is the most critical direction for the given building.
- This case is also starts from first pushover case 'GRAV'.
- P-Delta effects are again included.

- Static Pushover Case: Named '*MODE1*' is used which is based on the 1st modal analysis results, the details of this load combination is as follows;

MODE1 = M1

- *Push To Displacement Of* option is used for this case.
- First mode is taken into calculation as a base deformed shape.
- Control Direction is accepted as U1 which is the most critical direction for Mode-1.
- This case is also starts from first pushover case 'GRAV'.
- P-Delta effects are again included.

- Static Pushover Case: Named 'ACCXY' is used which is based on the elastic analysis results, the details of this load combination is as follows;

ACCXY = Acc.Dir.X + Acc.Dir.Y

- *Push To Displacement Of* option is used for this case.
- Control Direction is again accepted as U1 direction
- This case is also starts from first pushover case 'GRAV'.
- P-Delta effects are again included.

- Static Pushover Case: Named 'EQ' is used which is based on the elastic analysis results, the details of this load combination is as follows;

EQ = 1.0EQ

- *Push To Displacement Of* option is used for this case.
- Control Direction is accepted as U2 direction.
- P-Delta effects are also included.
- This case is also starts from first pushover case 'GRAV'.

7.7. Pushover Analysis Results

As it is mentioned in elastic and dynamic analysis parts, the given building shows torsional behavior. Because of this behavior none of the modes can demonstrate the real behavior of the structure individually. Actually the periods of first three modes prove this approach. The real behavior is the combination of loading along both X and Y directions. Among the pushover analysis cases the most critical and suitable one for the given structure can be taken as ACCXY.

When we consider the ACCXY case, five stages were necessary before a mechanism was performed in the structure. The displacements, corresponding base shears and number of hinges for every step with their levels are presented in Table 7.7.

Table 7.7. The displacements, corresponding base shears and number of hinges for every step

			NUMBER OF HINGES							
Events	**Displacement**	**Base Shear**	**A-B**	**B-IO**	**IO-LS**	**LS-CP**	**CP-C**	**C-D**	**D-E**	**TOTAL**
0	0	0	1168	32	0	0	0	0	0	1200
1	0,153	9,9842	1167	33	0	0	0	0	0	1200
2	5,3346	240,1115	882	311	7	0	0	0	0	1200
3	12,6015	382,4124	709	396	87	8	0	0	0	1200
4	17,2881	440,8413	623	409	123	44	0	1	0	1200
5	15,8324	353,6212	619	412	122	46	0	0	1	1200

The levels of plastic hinge formation given in Table 7.7 as A-B-IO-LS-CP-C-D and E, are illustrated below in Figure 7.5 which is typical force-deformation relationship for model element.

Significant events in the progressive lateral response of the building are annotated on the figures, which are listed in Table 7.8 and more fully described in the same table. As can be seen from the referred figures, the first critical event consists of hinging of columns and beams of 'Axis B' where the staircase is located and the gab due to this stair case in the slab cause weakness in the structural system. The illustrated hinge formations below are taken from first floor and the outside longest axis along the x-direction.

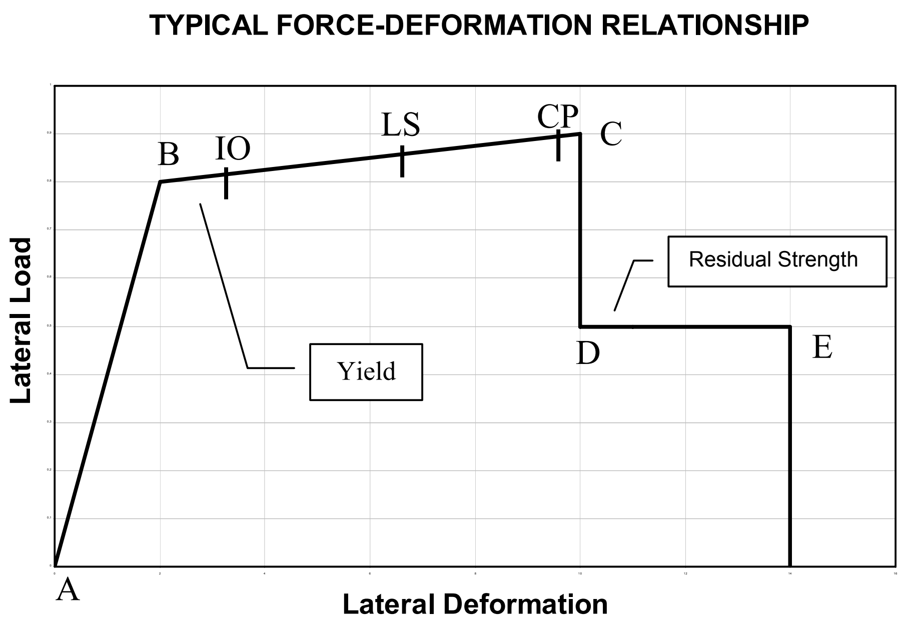


Figure 7.5. Typical force-deformation relationship for model element

Table 7.8. Significant events in the progressive lateral response

LONGITUDIONAL PUSHOVER EVENTS			
Event	Description	Roof Disp. (cm) at control joint (6F54)	Figure No
1	Hinge formation happens only at all column ends at Axis B and stair case beams at the same axis	0,153	Figure 7.6
2	Hinge formation at basement columns and almost all columns and some beams up to 5th storey	6,160	Figure 7.7
3	Hinge formation continuing after 5th storey on beams and columns	14,630	Figure 7.8
4	This step is the continuing of hinge formation on almost every joints	17,288	Figure 7.9
5	This step is also the continuing of hinge formation on almost every joints	15,832	Figure 7.10

As the structural elements of the building yields the center of rigidity of the floors shift westward direction (-x direction in plan). At the 4th stage of the pushover analysis, the enter of rigidity of the 6th floor, shifted 21.5 cm (17.28 cm in X-direction and 14.82 cm in Y-direction) from its original (undamaged) position. This shift of the center of rigidity accentuates the torsional behavior of the building. All these consequences are the results of the weak and unsymmetrical structural system. Hinge development starts at the first pushover events (with small deformations) and continues to the upper stories.

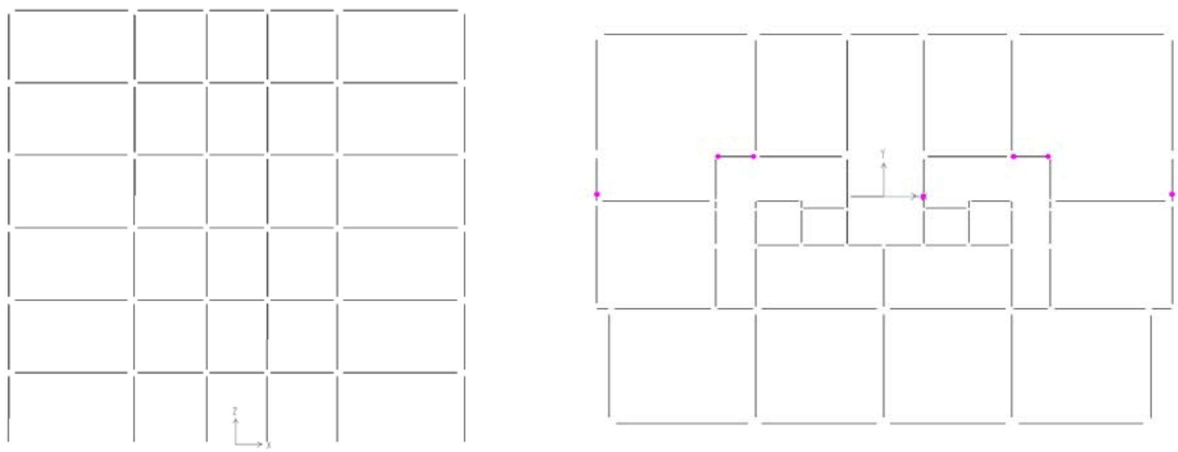


Figure 7.6. 1st stage deformed shapes and hinge formations

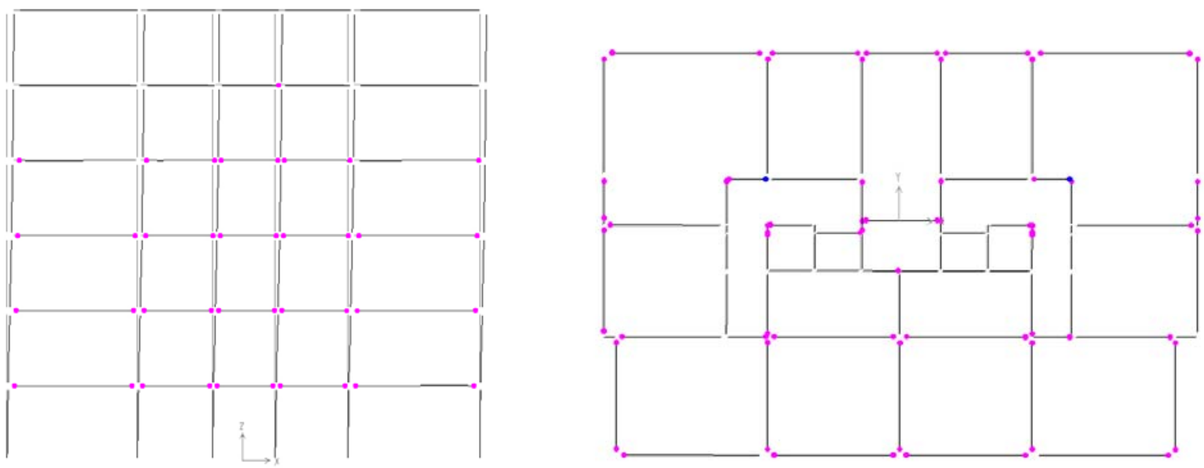


Figure 7.7. 2nd stage deformed shapes and hinge formations

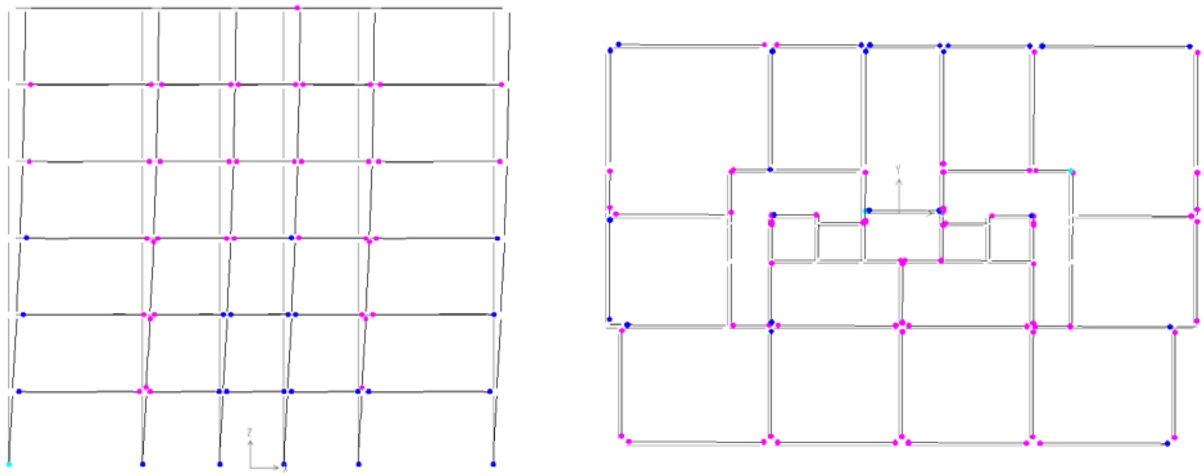


Figure 7.8. 3rd stage deformed shapes and hinge formations

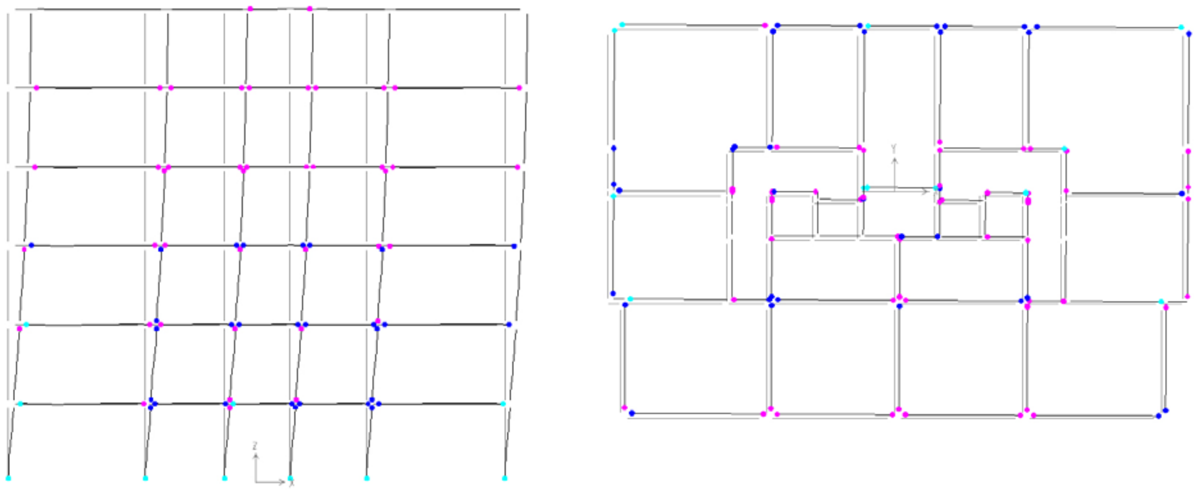


Figure 7.9. 4th stage deformed shapes and hinge formations

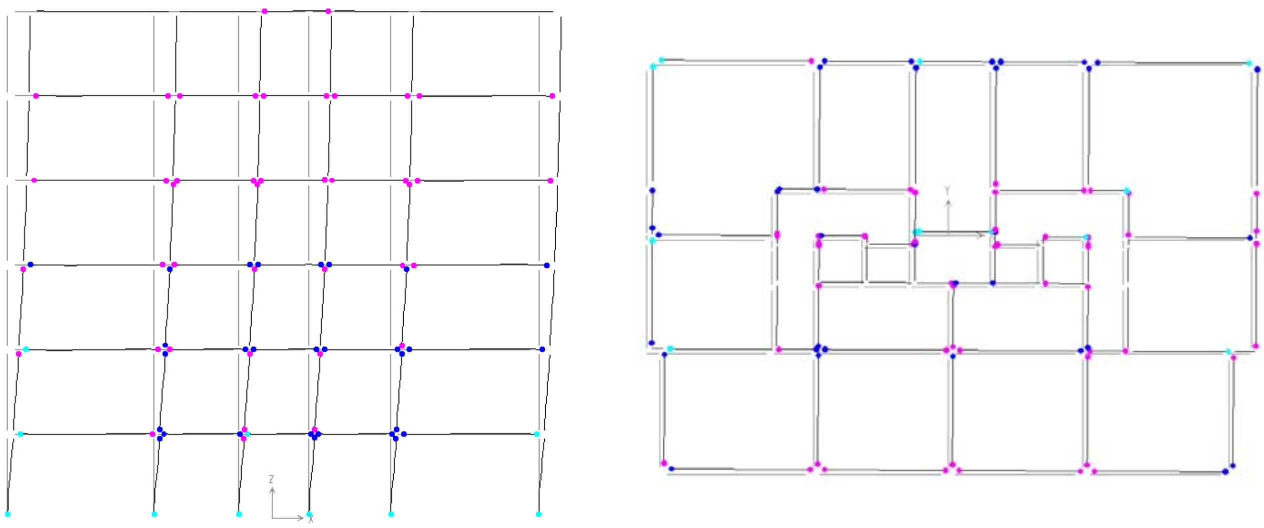


Figure 7.10. 5th stage deformed shapes and hinge formations

7.8. Performance Evaluation with Capacity Spectrum Method

A performance objective specifies the desired seismic performance of the building. Seismic performance is described by designating the maximum allowable damage state (performance level) for an identified seismic hazard. A performance objective may include consideration of damage states for several levels of ground motion.

The basic safety objective selected for this case study is building performance level ‘’ Immediate Occupancy (IO)‘’ at the Serviceability Earthquake (SE) and building performance level ‘’ Life Safety (LS)’’ at the Design Earthquake (DE).

Immediate Occupancy : This corresponds to the most widely used criteria for essential facilities. The building’s spaces and systems are expected to be reasonably usable, but continuity of all services, either primary or backup, is not necessarily provided. Contents may be damaged.

Life Safety : The ‘Life Safety‘’ Building Performance Level is intended to achieve a damage state that presents an extremely low probability of threat to life safety either from structural damage or from falling or tipping of nonstructural building components. User-furnished contents however, are not controlled, and could create falling hazard or secondary hazards, such as chemical releases or fire (Moderately Damaging Level).

To determine whether the given building meets a specified performance objective, response quantities from a nonlinear static analysis are compared with limits for appropriate performance levels.

As a result of the pushover analysis the corresponding capacity curve was developed from the analysis results and is drawn in Figure 7.11 with data presented in Table 7.9. The capacity curve represents the force-displacement characteristics of the lateral force resisting system of the entire structure.

Table 7.9. Pushover analysis results for unstrengthened building

Step	Teff	ßeff	Sd(C)	Sa(C)	Sd(D)	Sa(D)	ALPHA	PF*Ø
0	0,690	0,050	0,020	0,000	6,854	0,580	1,000	1,000
1	0,690	0,050	0,114	0,001	6,854	0,580	0,901	1,343
2	1,021	0,059	3,284	0,127	9,745	0,376	1,646	1,624
3	1,261	0,099	7,706	0,195	10,388	0,263	1,705	1,635
4	1,387	0,113	10,621	0,222	10,975	0,230	1,724	1,628

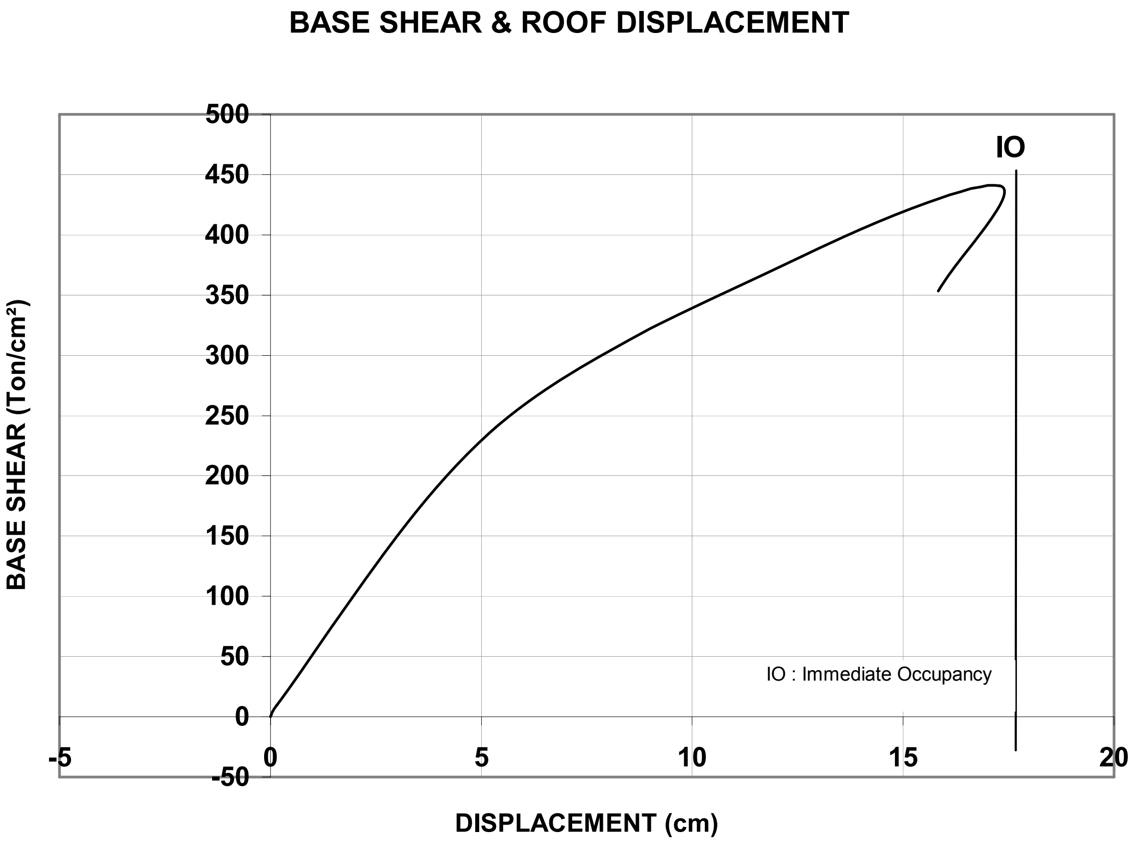


Figure 7.11. Pushover analysis results for unstrengthened building

A default 5 percent damped site response spectrum was developed using the procedure described in Part 7.5. The obtained site response spectra for Serviceability Earthquake and Design Earthquake are presented in Figure 7.12. Structural behavior type is selected as *Type C*. As a result of the pushover analysis the obtained Seismic demand and capacity spectra of the given building are presented in Figure 7.13 for SE and 7.14 for

DE in the ADRS format. The unstrengthened building (as-built stage) possesses an energy dissipation capacity at the ultimate stage equivalent to 10.53% viscous damping (ay=0.127, dy=3.284, ap=0.222, dp=10.621) for which reduced demand spectrum does not intersect with its capacity spectrum. This result verifies that the unstrengthened building is not capable of satisfying the code requirements.

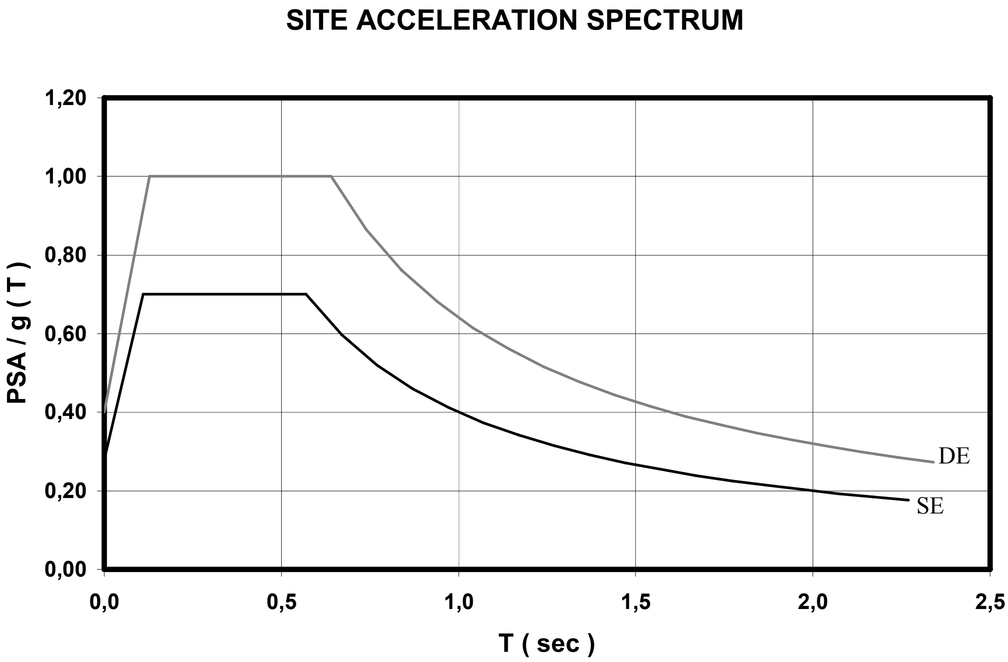


Figure 7.12. Site response spectra for SE and DE

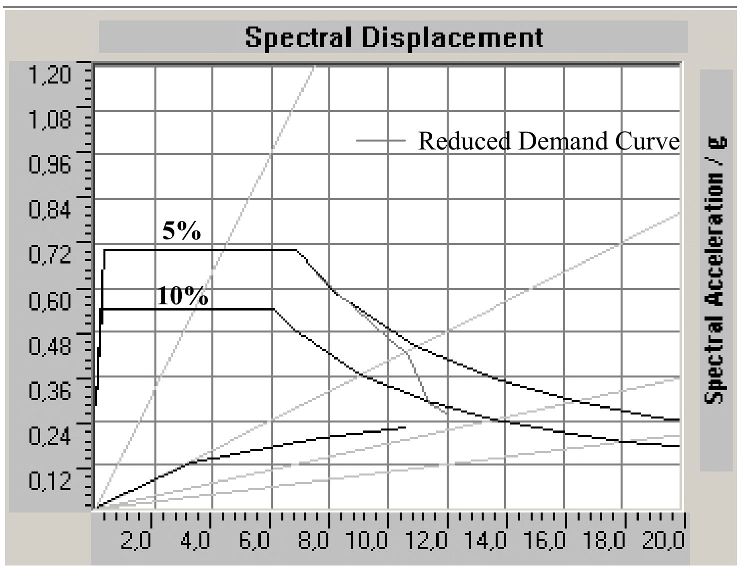


Figure 7.13. Capacity demand curves of unstrengthened building for SE

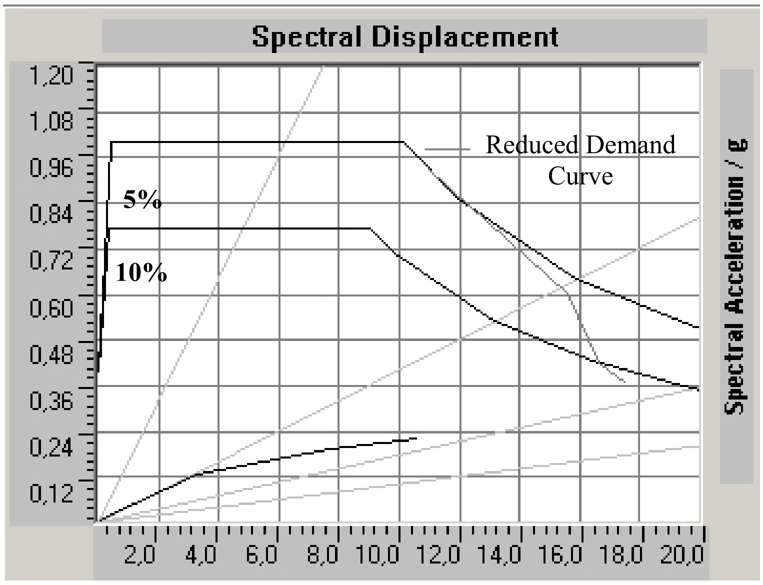


Figure 7.14. Capacity demand curves of unstrengthened building for DE

The building is not reach the performance point, this was the situation for this building during the Adana-Ceyhan earthquake and structure was moderately damaged after main shock. The results of this capacity spectrum method show this real situation.

7.9. Rehabilitation Scheme

The subject building is not capable of achieving the performance point shown in Figure 7.13 and 7.14. This suggests a seismic rehabilitation program, which includes adding of cast-in place reinforced concrete shearwalls to the existing frame system as shown in Figure 7.15. The added shearwalls are indicated on the figure by darker shading. The shearwall area in the strengthened configuration is 0.51% and 0.60% of the base area in the short and long directions of the building, respectively.

The response comparison between unstrengthened and strengthened building models includes the effects of employing different elastic moduli for existing and added shearwall members in the building. The reduction in the modulus of elasticity in the existing members is due to the softening which is the result of the members cracking during Adana-Ceyhan Earthquake. Therefore modulus of elasticity is reduced from 250 ton/cm to 150 ton/cm . Similar reduction is also proposed by Li and Jirsa (1998) [7] in which the calculated response of an instrumented hotel building is compared with the recorded motion of the floors of the building. The resultant capacity curves of both cases are given in Figure 7.16.

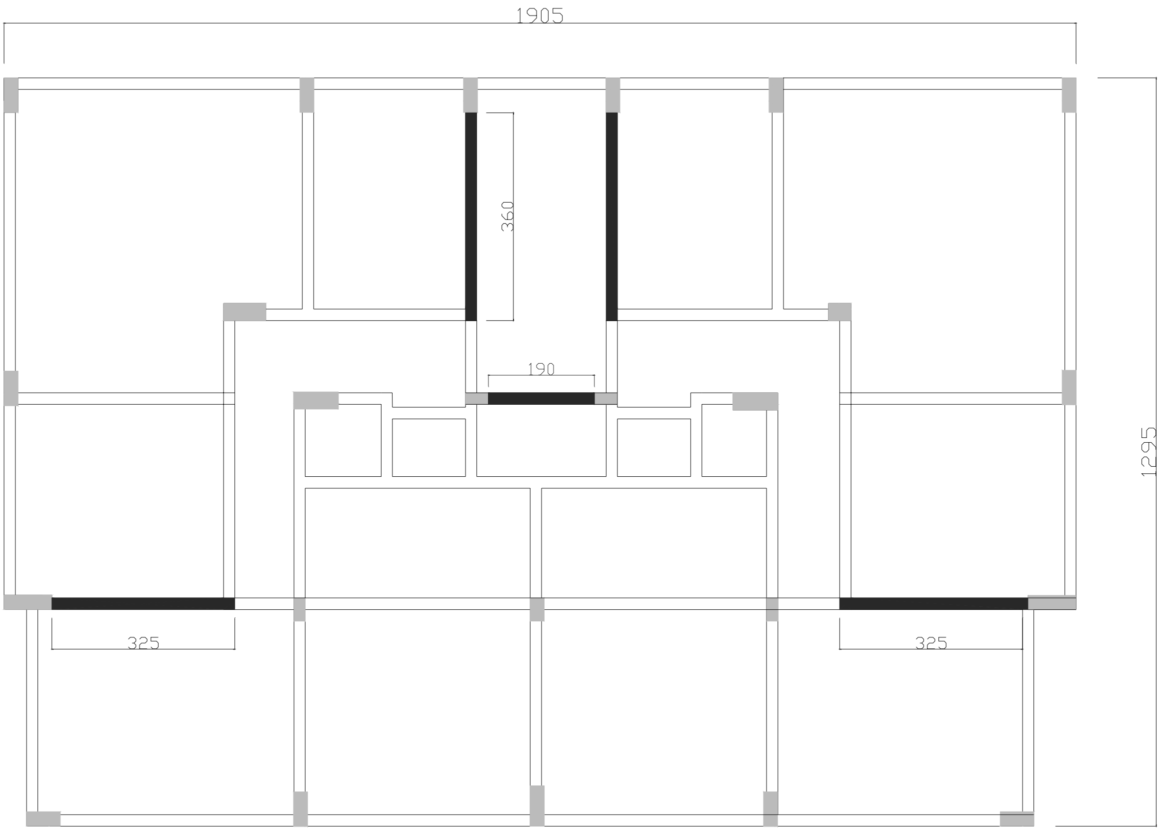


Figure 7.15. Strengthened building structural plan

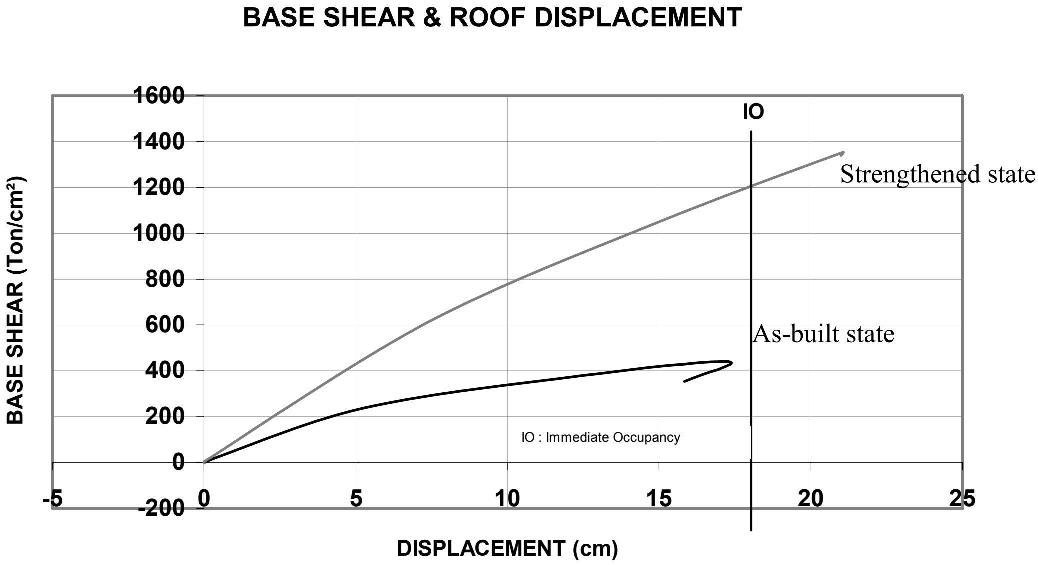


Figure 7.16. Capacity curve of strengthened building

Strengthening of original frame increases the ultimate strength by 3.0 and the ultimate deformation capacity by 1.22, as observed in Figure 7.16. It can be concluded that strengthening of this building result in increasing stiffness, strength and deformability significantly. Accordingly, strengthening walls contribute to enhancing the performance of the building at all performance levels as indicated in ATC40 [6].

For the strengthened building model, five stages were necessary before a mechanism was performed. The hinge formations for each stage are given below in Figures 7.17, 7.18, 7.19, 7.20 and 7.21. and these deformed shapes are taken from first floor and the outside longest axis, along the long direction.

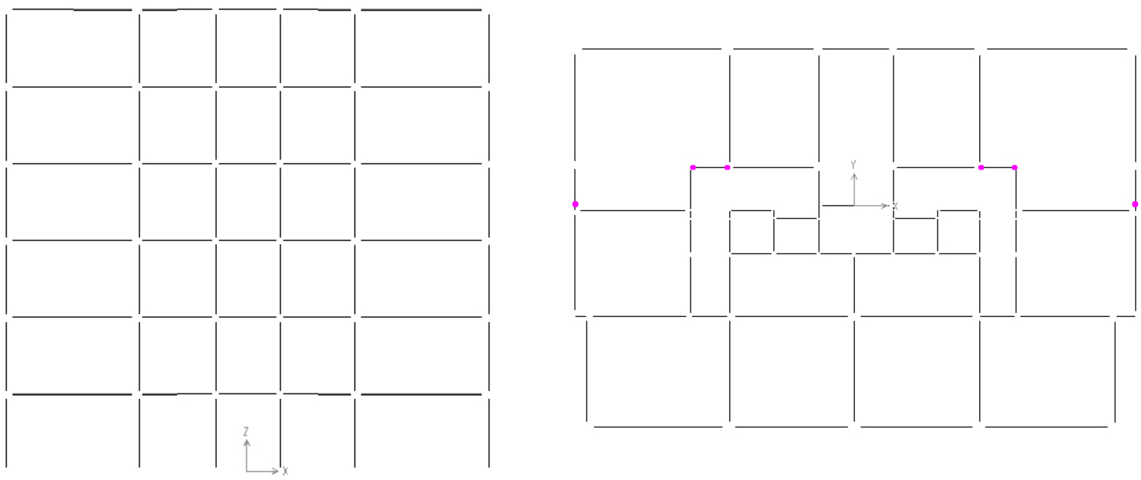


Figure 7.17. 1st stage deformed shapes and hinge formations

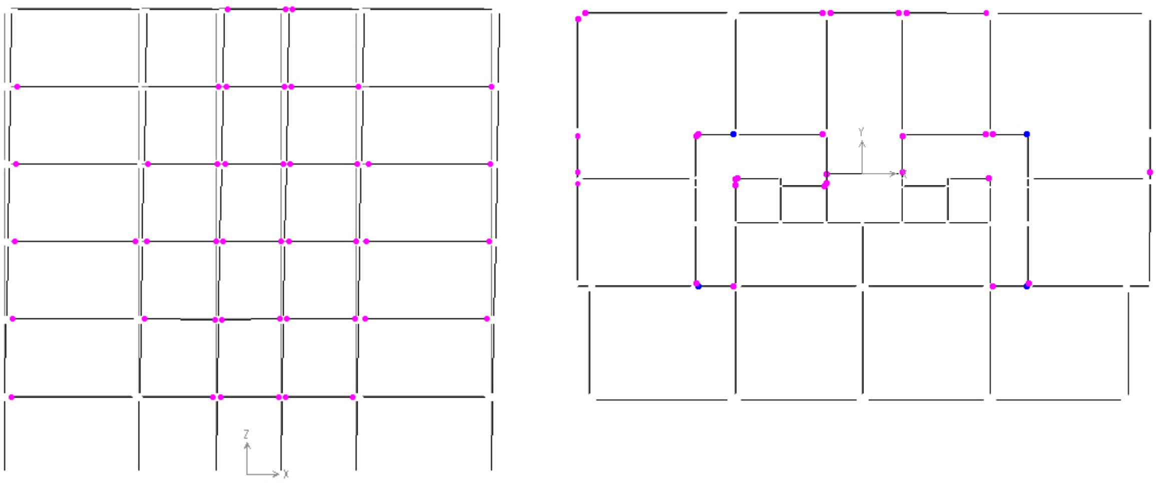


Figure 7.18. 2nd stage deformed shapes and hinge formations

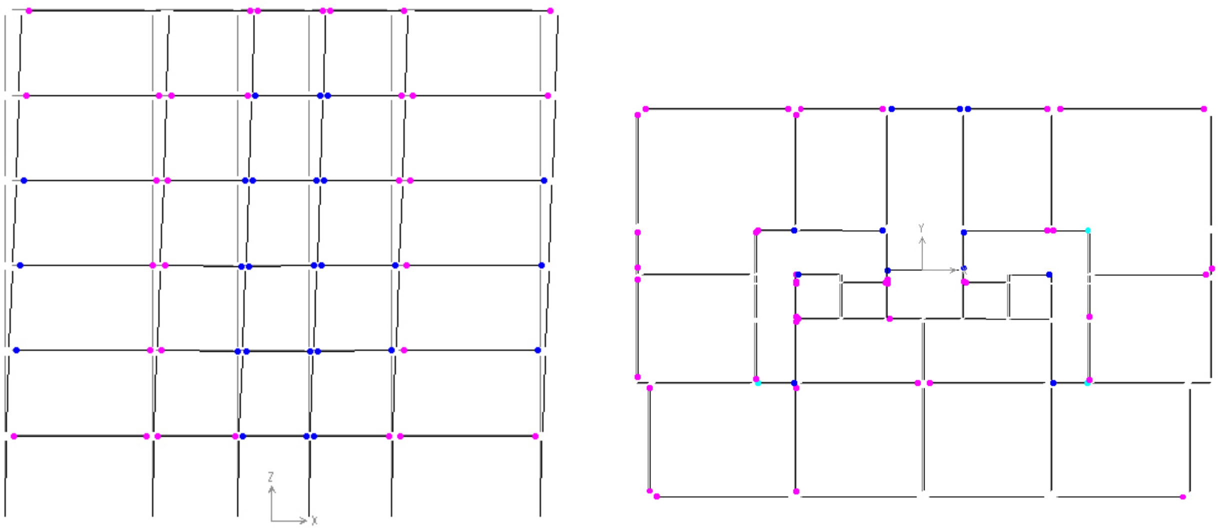


Figure 7.19. 3rd stage deformed shapes and hinge formations

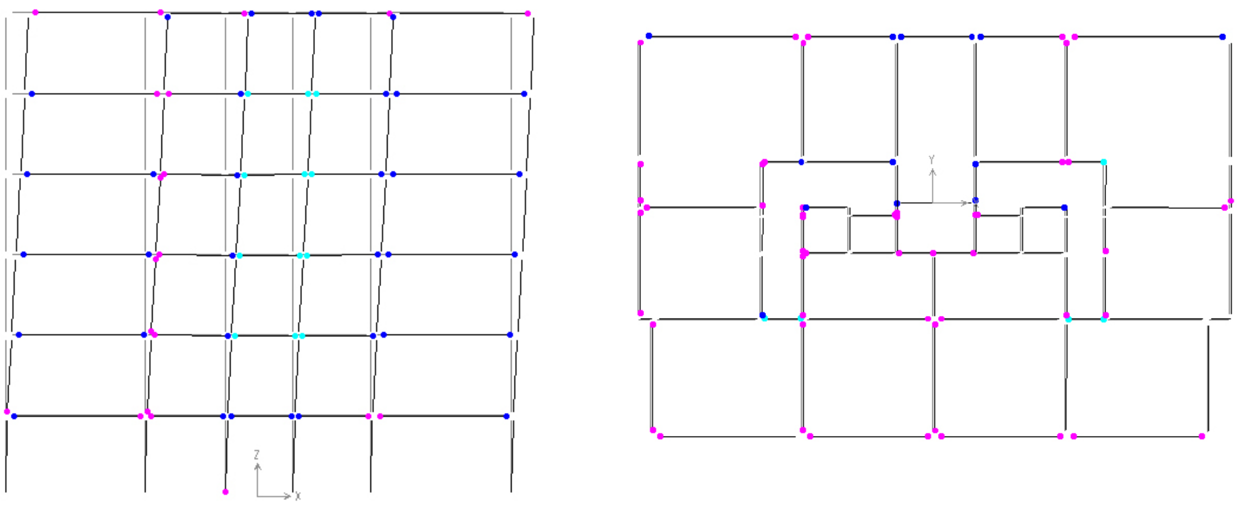
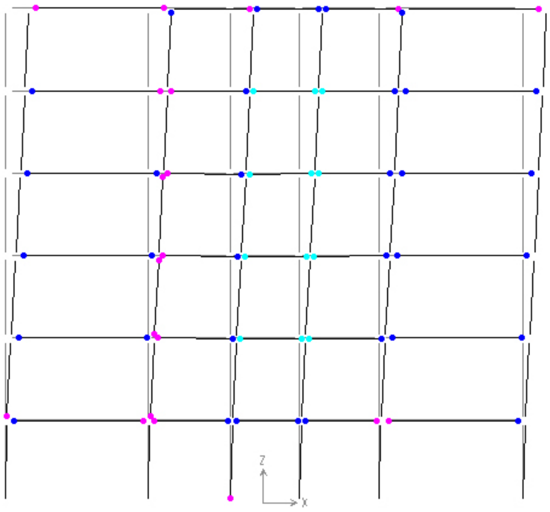


Figure 7.20. 4th stage deformed shapes and hinge formations



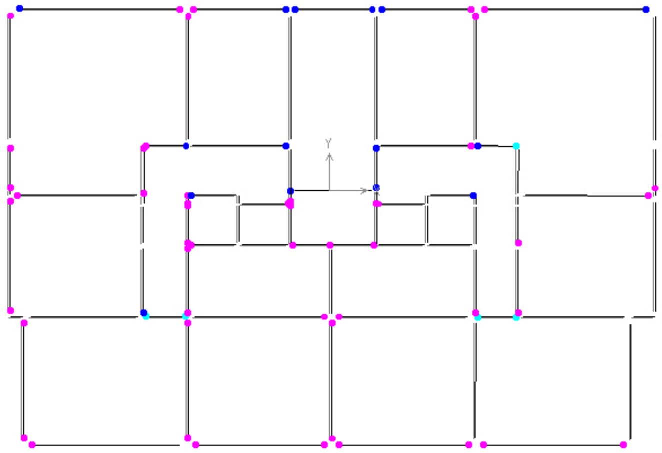


Figure 7.21 5th stage deformed shapes and hinge formations

The unstrengthened building does not provide an acceptable performance with an energy dissipation capacity at the ultimate point equivalent to 10.53% viscous damping. The strengthened building (Type B according to ATC40) provides a performance at an equivalent viscous damping ratio of 7,64% (ay=0.391, dy=3.277, ap=0.616, dp=5.720) and ratio of roof displacement of 0.007 in the case of Serviceability Earthquake. Also it provides a performance at an equivalent viscous damping ratio of 8.22% (ay=0.391, dy=3.277, ap=0.828, dp=8.259) and ratio of roof displacement of 0.012 for Design Earthquake. When these roof displacement ratios are compared with the limits given in ATC40, it can be concluded that the strengthened building satisfies the basic safety objectives selected for this case study, which are ‘’Immediate Occupancy (IO)‘’ performance level at the Serviceability Earthquake (SE) and ‘’Life Safety (LS)’’ performance level at the Design Earthquake (DE).

The corresponding capacity demand curves for the strengthened building for both performance levels are given in ADRS format in Figure 7.22 and 7.23.

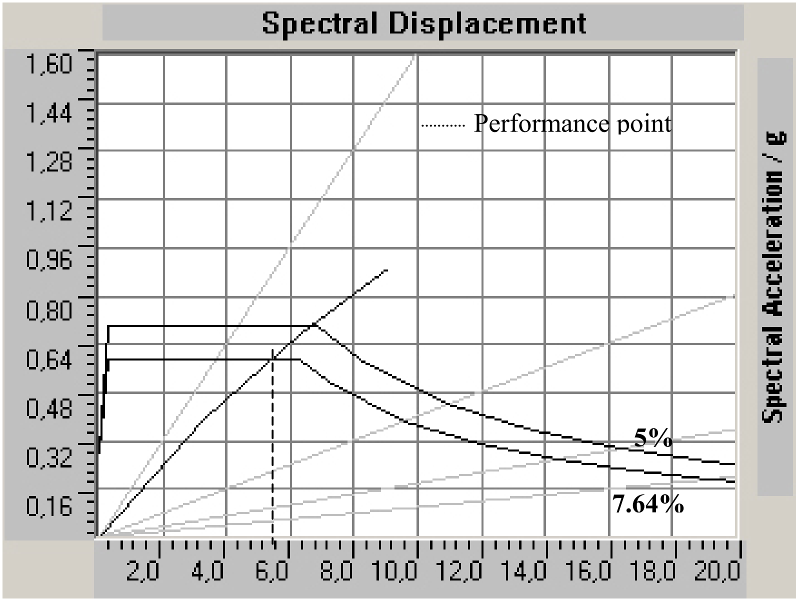


Figure 7.22. Capacity demand curves of strengthened building for SE

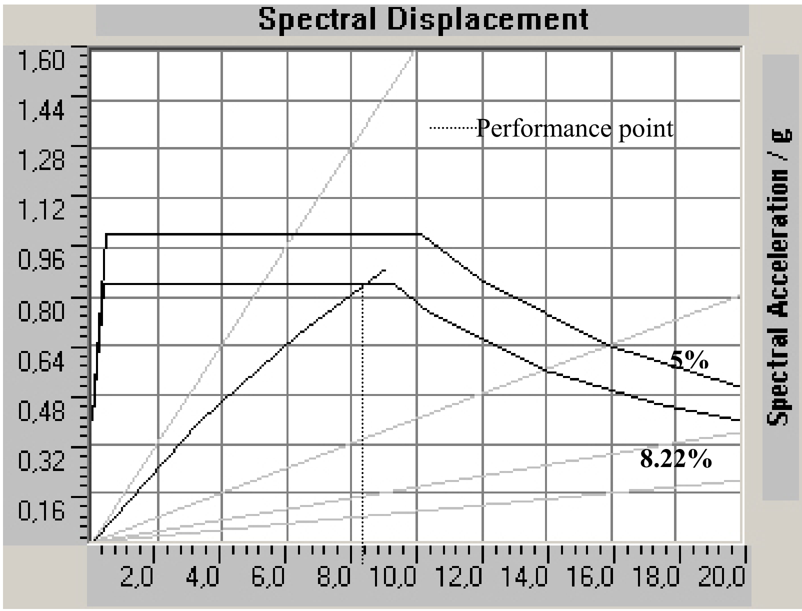


Figure 7.23. Capacity demand curves of strengthened building for DE

7.10. Two Dimensional Pushover Analysis

It could be a great opportunity to check this given structure with two dimensional pushover analysis and make comparison between the obtained results however, the irregularities in the plan with unsymmetrical distribution of structural system which can be seen from the plan view given in Appendix Figure A.2 and torsional characteristic of the system which appear both in elastic and dynamic analysis, do not allow to apply two dimensional pushover analysis.

There are some requirements given in ATC40. In order to apply two dimensional pushover analysis, torsional effects should be sufficiently small such that the maximum displacement at any point on the floor is less than the 120 percent of the displacement at the corresponding center of mass. If the maximum displacement exceeds 120 percent of the displacement at the center of mass, then three-dimensional analysis required.

For the analyzed structure the displacement under COMB1 loading at the top floor center of mass is equal to 3,80 cm in X-direction and 2,94 cm in Y-direction. The maximum displacement at the same floor occurs at joint 6F54 (Location of this joint is presented in Appendix, Figure A.3) with 4,15 cm in X-direction and 3,52 cm in Y-direction. The ratio of displacements is as follows:

$$Rx = 4{,}15 / 3{,}80 \qquad Rx = 1{,}09 < 1{,}20 \therefore \text{OK}$$

$$Ry = 3{,}52 / 2{,}938 \qquad Ry = 1{,}20$$

Therefore, the structure is not satisfying the requirement so that two-dimensional analysis is not a good solution to construct capacity curve.

7.11. Discussion of Results

From the results of the analyses, the following results are drawn:

- The analyses on the unstrengthened frame prove the need for rehabilitation.
- The strengthening scheme applied in this structure improved the system behavior considerably. Not only the stiffness and strength of the structure, but also its deformation capacity was increased.
- Addition of shear walls also modified the mechanism formation at the ultimate stage of the response under pushover loading

- The CSM results showed that the performance of the structure under an excitation in the level of design earthquake of the code is life safety, meaning that the probability of life threatening action is low but structural and non structural damage is possible.
- The added shearwalls should be continuous from base to top. Otherwise, the vibrational characteristics of the stories without shearwalls will be different from the others and they respond like an appendage in this building.
- In the unstrengthened state, performance of the building under the Adana-Ceyhan earthquake excitation is almost in its collapse state on the capacity curve.
- The reduction of the EI values of the existing members before the rehabilitation has no effect on the strength or deformation capacity of the strengthened structure, but it is more reasonable to use reduced EI values for these members since they experienced an earthquake. In the analysis of the structural model with reduced EI values, the added shearwalls dominate behavior even at the low levels of deformation.

8. SUMMARY, CONCLUSIONS AND RECOMMENDATIONS

8.1. Summary

In this study, the effectiveness of the capacity spectrum method applied to the moderately damaged reinforced concrete building in Adana city is evaluated in detail. The unstrengthened and strengthened configuration of the building is modeled for inelastic three-dimensional analysis by using a special analysis program SAP2000 v7.1. After applying code-static and code-dynamic analysis to the given structure, the lateral load carrying capacity of the model is determined by using pushover analysis. Then, the performance of the structural model is determined by Capacity Spectrum Method (CSM). The capacity curve is converted into the Acceleration Displacement Response Spectrum (ADRS) format and graphically compared with the earthquake demand, which is a design spectrum. In these analyses the effects of different lateral loading cases, efficiency of shearwalls and damping effects are discussed.

8.2. Conclusions

The following conclusions can be drawn from the analysis of selected building:

- The Capacity Spectrum Method and the ADRS format have been shown to be a useful tool evaluating existing buildings for seismic performance, verifying designs of construction for performance goals, and correlating observed damage with recorded earthquake motion. The Capacity Spectrum Method can be used as a rapid evaluation procedure to obtain rough estimates for large inventories of buildings or as a detailed procedure appears to be compatible with other approximate inelastic design and evaluation methods.
- The analysis of the existing structure concluded that it could not satisfy the requirements of the assumed level of seismic performance and a seismic rehabilitation scheme consisting of shear walls investigated.

- The stiffness of structures substantially increases by the added shearwalls. The increase in stiffness reduces displacements and story drifts and it is essential in preventing the damage on structural and nonstructural members.
- The existing columns before strengthening which are fail mostly in brittle fail due to improperly confinement. The addition of shearwalls into the system reduces the internal forces on these columns and shear failures are prevented.
- Realistic load distribution should be used for the analysis since the results of the pushover analysis can be very sensitive to the lateral load distribution.
- According to Turkish Earthquake Code [4], in a ductile wall-frame system in which lateral forces are assumed to be resisted only by the shearwall, the minimum shearwall cross sectional area is specified to be 0.002 times the total floor area of the building. This criterion is also used as a starting value of different alternatives in the rehabilitation of the model structure. For a new structure with wall-frame system, even if the lateral loads are assumed to be resisted only by shearwalls, actually the other columns also resist the load in proportion to their stiffness. However, in strengthened structure, material properties of the existing structural members and added shearwalls differ and in the analysis of such a frame, stiffness of the existing members is reduced because they experienced an earthquake. As a result of that, shearwalls in a strengthened frame receive higher proportion of the lateral load than existing frame elements. Therefore, shearwall ratio, which is the shearwall area A_{sw} in one direction divided by total floor area of building A_t, should be greater than 0.002 for a reasonable performance. In Table 8.1, the shearwall ratio in the analyzed direction vs the performance level comparisons of the buildings assessed in this study are given:

Table 8.1. Shearwall ratio vs performance level comparison of the assessed building

	Shearwall Ratio	**Ultimate Strength Level**	**Performance Level**
For SE	0.011	0.616g	Immediate Occupancy
For DE	0.011	0.828g	Life Safety

As seen from Table 8.1, as shear wall ratio is around 0.011, structural performance is at minimum acceptable level. Thus in the retrofit design for moderately damaged reinforced concrete structure, 0.011 should be used as the initial value to determine the total shearwall area required for a acceptable structural performance.

- Although software limitations and other practical considerations preclude assessment of some complex behaviors (e.g. higher mode effects), the nonlinear static pushover procedure is still expected to provide a more complete and more useful picture of expected performance than is linear elastic analysis.

8.3. Future Recommendations

This master thesis presented an application of the code static, code dynamic analysis and seismic evaluation of an existing reinforced concrete building by using three-dimensional static nonlinear analysis procedure. Because two-dimensional analyses do not reflect the torsional effects due to asymmetric structural systems like in the given structure, the use of three-dimensional analysis is strongly recommended.

Small changes in properties or loading can cause large changes in nonlinear response. For this reason, it is extremely important that the analyzer should consider many different loading cases and must be sensitive about the small changes in modeling and describing of the pushover analysis properties.

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